



**Ain Shams University** 

# **Steel Design note**

**Structural Engineering Department-CHEP** 

**Steel Structures Project** 

**Proj N08- Industrial Planet** 

Class of 2023

Prepared by

ST: Mohamed Hassan Ali Bezawy

Mail:1902242@eng.asu.edu.eg

Senior - Level (2)

2022/2023

# Contents

CHAPTER I	IV
Loads	IV
Loads frame	V
1-Frame	V
2-Mezzanine	v
Wind Load	V
Wind load (+X)	VI
Wind load(-X)	VI
Wind Load [+y]	VII
Wind Load [-y]	VII
Max. Straining Action (SAP2000)	VII
Axial Force	VII
Shear Force	VIII
Bending Moment	VIII
CHAPTER II	9
Cold Formed	9
Purlin	10
Loads	10
Straining Action	10
Choice of Section	11
Code Limit for Slender Sections	11
The Effective Parts of Sections	11
Check Bending	13
Check Shear	13
Check Deflection	13
CHAPTER III	12
Dosign of Main System	13

## CHAPTER I

## Loads

Chapter I Loads

#### Loads frame

#### 1-Frame

O.W Portal Frame, by SAP200

O.W of Sheets =  $8 Kg/m^2$ 

Installations =  $20 Kg/m^2$ 

O.W Purlin =  $5 Kg/m^2$ 

Dead load on frame (S = 6m) = [5 + 8 + 20] \* 6 = 198 Kg / m = 0.198 ton / m

 $W_{L.L\,Frame} = 60 - 66.67 * tan(2.86) = 56.67 kg/m^2 = 0.05667 ton/m^2$ 

Live load on frame (S=6m) =  $0.05667 * 6 = 0.34 \text{ t/m}' \rightarrow Lr - live$  inaccessible roof

#### 2-Mezzanine

$$O.\,W_{R.C\,Slab}\,=\gamma_c*t_s=2500*0.14m=350Kg/m^2$$

 $F.C = 150 \, Kg/m^2$ 

O.W Sec Beams = 30 Kg/m

O.W Walls = 150 Kg/m

Then

$$W_{D,L} = (250+150)*2 + 30 = 830 \text{ kg/m} = 0.83 \text{ t/m}$$

$$W_{(1.Lmezz)} = 250 Kg / m^2 * 2 = 0.5 ton / m$$

#### Wind Load

$$W_{wind} = Ce * q * k (kg / m^2) \pm Ci * q * k \rightarrow (q = 68 kg / m^2),$$
  
where k  $(0 \rightarrow 5) = 1.15$ , k  $(5 \rightarrow 15) = 1.4$ 

Chapter I Loads

## Wind load (+X)

 $W_1 = 0.37 \text{ t/m}$ 

 $W_2 = 0.29 \text{ t/m}$ 

 $W_3 = 0.33 \text{ t/m}$ 

 $W_4 = 0.204 \text{ t/m}$ 

 $W_5 = 0.46 \text{ t/m}$ 

 $W_6 = 0.235 \text{ t/m}$ 

## Wind load(-X)

 $W_1 = 0.38 \text{ t/m}$ 

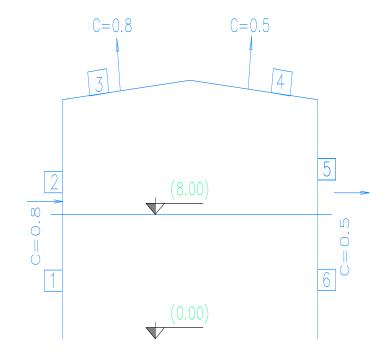
 $W_2 = 0.63 \text{ t/m}$ 

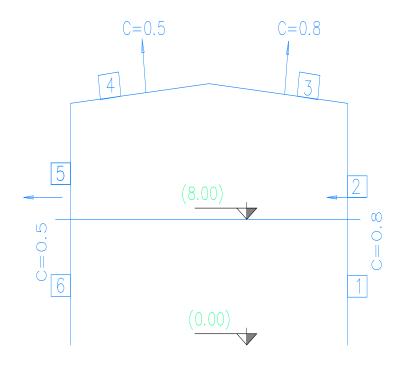
 $w_3 = 0.33 \text{ t/m}$ 

 $w_4 = 0.204 \text{ t/m}$ 

 $w_5 = 0.114 \text{ t/m}$ 

 $w_6 = 0.235 \text{ t/m}$ 





Chapter I Loads

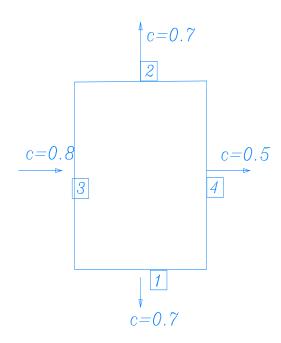
#### Wind Load [+y]

$$W_{_{1}} = 0.33 \text{ t/m}$$
,  $w'_{_{1}} = 0.57 \text{ t/m}$ 

$$W_2 = 0.33 \text{ t/m}$$
,  $w'_2 = 0.47 \text{ t/m}$ 

$$W_3 = 0.38 \text{ t/m}$$
,  $w'_3 = 0.29 \text{ t/m}$ 

$$W_4 = 0.235 \text{ t/m}$$
 ,  $w'_4 = 0.46 \text{ t/m}$ 



### Wind Load [-y]

$$W_{_2} = 0.33 \text{ t/m}$$
 ,  $w'_{_2} = 0.228 \text{ t/m}$ 

$$W_2 = 0.33 \text{ t/m}$$
 ,  $w'_2 = 0.228 \text{ t/m}$ 

$$W_{_{3}}=\,0.235\,\mathrm{t/m}$$
 ,  $w'_{_{3}}=0.114\,\mathrm{t/m}$ 

$$W_{\scriptscriptstyle 4}$$
 = 0.38 t/m , w'  $_{\scriptscriptstyle 4}$  = 0.63 t/m

# 0.7 1 c=0.8 3 c=0.8

## Max. Straining Action (SAP2000)

**Axial Force** 

hapter I	Load	S

Shear Force

Bending Moment

## CHAPTER II

# Cold Formed

#### Purlin

$$S = 6 m$$

$$a = 2 m$$

Slope = 
$$1/20$$
  $\alpha = 2.86^{\circ}$ 

$$\alpha = 2.86^{\circ}$$

Loads

$$W_{-c} = 10 \text{ Kg/m}^2$$
, O.W = 5 Kg/m<sup>2</sup>

$$W_{D.L} = 10 * 2/\cos(2.86) + 2*5 = 30 \text{ Kg/m}$$

$$W_{D.Lx}$$
= 30 \* cos (2.86) = 29.96 Kg / m`

$$W_{D.Ly} = 30 * sin (2.86) = 1.5 Kg / m$$

$$W_{L,L} = (60 - 66.6 * 1/20) *2 = 113.34 \text{ Kg}/\text{m}$$

$$W_{L,L,x} = 113.34 * cos (2.86) = 113.198 Kg/m$$

$$W_{L,L,y} = 113.34 * sin (2.86) = 5.65 Kg / m$$

$$W_x = 29.96 *1.2 + 1.6 * 113.198 = 217.068 \text{ Kg/m}/1000 = 0.217 \text{ t/m}$$

$$W_y = 1.5 *1.2 + 1.6 * 5.65 = 10.84 \text{ Kg/m}/1000 = 0.011 \text{ t/m}$$

Straining Action

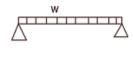
$$M_x = \frac{0.217 \times 6^2}{8} = 0.976 \ t.m$$

$$M_y = \frac{0.011*3^2}{8} = 0.012 \ t.m$$
 -----(Use one Tie rod)

$$Q_{-x} = \frac{0.217*6}{2} = 0.651 \ ton$$

$$Q_{-y} = \frac{0.011*3}{2} = 0.0165 \ ton$$





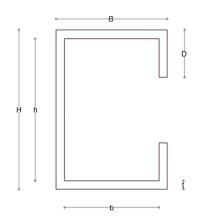


Choice of Section

$$F_{bc} = \frac{Mx}{Sx} + \frac{My}{Sy} = 0.85 * F_y = 2.04 ton/cm^2$$

Assume 
$$--- \triangleright s_y = \frac{sx}{6} ---- \triangleright Sx = 6 * Sy$$

$$Sx = \frac{0.976 * 100 + 6 * 0.012 * 100}{2.04} = 72.35 \text{ cm}^3$$



Code Limit for Slender Sections

#### For Web

$$h = H - 2r - 2t = 180 - 2*8 - 2*4 = 156 mm$$

$$h/t = 156 / 4 = 39 < 200 ...... OK$$

#### For Partially Stiff Flange

$$b = 80 - 2*8 - 2*4 = 56 \text{ mm}$$

$$b/t = 14 < 40 \dots OK$$

#### **For Lip**

$$d = D - r - t = 25 - 8 - 4 = 20 \text{ mm}$$

#### So ... Section Satisfies Code Limits

The Effective Parts of Sections

#### **Flange**

Partially Stiff Flange is Compression.... So ... $\psi = 1$ 

b= 56 m .... 
$$S = 1.28 \sqrt{\frac{E}{F_y}} = 1.28 \sqrt{\frac{2100}{2.4}} = 37.863$$

$$S/3 = 12.62$$
,  $b/t = 56/4 = 14$ ,  $S/3 < b/t < S$ 

$$I_{-a} = 399* \left(\frac{\frac{b}{t}}{s} - 0.33\right)^{3} * t^{4} = 399* \left(\frac{14}{37.86} - 0.33\right)^{3} * 0.4^{4}$$

$$I_{-a} = 6.43 * 10^{-4} \text{ cm}^4$$

$$I_{-S} = \frac{t*d^3}{12} = \frac{0.4*1.3^3}{12} = 0.073 \ cm^4$$

$$K\alpha = \left(4.82 - 5 * \frac{D}{b}\right) \sqrt{\frac{IS}{Ia}} + 0.43 = 47.03 > 5.25-5(D/b)$$

So, take 
$$5.25 - 5(25 / 56) = 3$$

$$\lambda_{\text{-p}} = \frac{\frac{b}{t}}{44} * \sqrt{\frac{fy}{K\alpha}} = 0.28$$

$$\rho = \frac{\lambda p - 0.15 - 0.05 * \Psi}{\lambda p^2} = 1.02 > 1$$

#### so, Flange is Fully Effective

#### Stiff Web

 $\Psi$  = -1 is subjected to M, K $\alpha$  = 23.9, b=h= 156 mm

$$\lambda - p = \frac{\frac{b}{t}}{44} * \sqrt{\frac{fy}{K\alpha}} = 0.28$$

$$p = \frac{0.28 - 0.15 - 0.05 * \Psi}{\lambda n^2} = 2.3 > 1$$

So, Web is Fully effective

#### Lip

#### **Un Stiff Lip**

$$\Psi$$
 = 1, K $\alpha$  = 0.43, d= 13 mm

$$d/t = 3.25 < \frac{16.9}{\sqrt{Fy}} = 10.9$$

#### So, Lip is Fully effective

#### So, Section is Fully effective

Check Bending

$$F_{bc} = \frac{Mx}{Sx} + \frac{My}{Sy} = 0.976 * \frac{100}{78.4} + 0.012 * \frac{100}{20.3} = 1.3 \text{ t/cm}^2$$

$$F_{-b-c} < 0.75 * .85 * F-y = 1.53 t/cm^2 .... Ok$$

Check Shear

$$h_{-w}/t_{-w} = 180 / 4 = 45 < \frac{112}{\sqrt{2.4}} = 72.3$$

$$V_{-all} = 0.85 * 0.6 * F-y * Aw$$

$$V_{-all} = 0.85 * 0.6 * 2.4 * 18 * 0.4 = 8.81 ton > Q-y = 0.0165 -ton$$

o.k. safe

**Check Deflection** 

S-act = 
$$\frac{5}{385} * \frac{WLL*L^4*10^{-5}}{E*Ix}$$

S-act = 
$$\frac{5}{385} * \frac{113.34 *600^4 *10^{-5}}{2100 *705.6} = 1.28 \ cm$$

$$S-all = 600 / 300 = 2 cm > S-act$$

o.k. safe

So, Use Purlin with Lipped Channel (180 \*80 \* 24 \* 4 \* 8 mm)

#### **CHAPTER III**

## Design of Main System

Chapter IV STAIR

L.L = 0.4 t/m2. D.L = o.w (Chequered plate) + 0.02 (finishing) =

$$\gamma_{steel} * t_{plate} + 0.02 = 7.85 * 0.01 + 0.02 = 0.0985 \text{ t/m}^2$$

W(tot) = 1.2 \* 0.0985 + 1.6 \* 0.4 = 0.7582 t/m2., W(t/m') = 0.7582 \* 0.3 = 0.22746 t/m'.

M(Ult)= 0.22746 \* 2.5^2/8= 0.178 t.m.

 $\Delta = 550 / 200 = 2.78 \text{ cm} > 2.43 \text{ cm} \text{ (safe)}$ 

S0, chequered plate is safe.

Design of Beams: for B1

 $W_{D.L} = own$  of upn 220 + D.L = 29.4/1000 + 0.0985 \* 2.5/2 = 0.153 t/m.

 $W_{LL} = 0.4 * 2.5 / 2 = 0.5 \text{ t/m}$ . from sap2000:

Mult= 4.4 t.m, Qult= 2.3 ton, delta (U3) = 24.32 mm. case 1.2 DL+1.6LL

#### **Check bending:**

Lb= 
$$\frac{2500}{3}$$
 = 833 mm ,  $L_p = \frac{80*2.3}{\sqrt{2.4}}$  = 96 cm , so Lb < Lp (case A)

Mn=Mp= Fy\*Zp= 2.4\* 1.13 \* 245=6.64 m.t,  $\phi Mn = 0.85*6.64 = 5.64 m.t$ 

So, 4.4 < 5.64 Mult capacity --- so upn 220 is ok

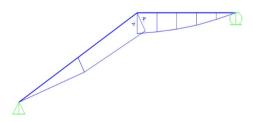
#### **Check shear:**

h/tw = 18.89 < 112/sqrt(fy).

<u>Qult Capacity</u> = 0.85 \* (0.6\*3.7\*(17.9\*0.8)\*58.8 = 33.3 ton > 2.3 ton.

#### **Check deflection:**

 $\Delta = 550 / 200 = 2.78 \text{ cm} > 2.43 \text{ cm} \text{ (safe)}$ 



MEM. B	BRACING	Length(m)	Vu(tonf)	Pu(tonf)	M(t.m)	LBx(m)	Lby(m)	Connection type	D	edge dist.	Pitch dist	L-take	N1	N2	LOAD CASE	FINAL IPE
	SEC-B-261	6	8.1432	, ,	3.16E-15			SHEAR	1.6	30	60	cm 240	4	3	1.2D+1.6LL+0.5Lr	220
	SEC-B-36		7.5456		3.55E-15			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	220
	SEC-B-276		3.5875		4.27E-15			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	220
	SEC-B-181		5.9564		4.55E-15			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	220
	SEC-B-299		5.5167		4.81E-15			SHEAR SHEAR	1.6	30	60	120 120	2	2	1.2D+1.6LL+0.5Lr	220
	SEC-B-71 SEC-B-179		3.6729 -3.5E-08		8.88E-15 5.94E-12			SHEAR	1.6	30 30	60 60	120	2	1	1.2D+1.6LL+0.5Lr 1.2D+1.6LL+0.5Lr	220 220
	SEC-B-47		-3.1E-08		6.98E-08			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-59		-1.9E-07		1.13E-07			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-48		1.9E-07		1.31E-06			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	220
نموذج ك.ث (1)	SEC-B-104		6.7082		9.33E-06			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
(1) (-9-	SEC-B-46		0.000185		0.00014			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-145		4.3823		0.00038	-		SHEAR SHEAR	1.6	30	60 60	120 120	2	2	1.2D+1.6LL+0.5Lr	220 220
	SEC-B-140 SEC-B-119		0.0116 0.1083		0.02207 0.16376			SHEAR	1.6	30 30	60	120	2	1	1.2D+1.6LL+0.5Lr 1.2D+1.6LL+0.5Lr	220
	SEC-B-32		2.8389		3.26523			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-33		2.9718		3.39976			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-34		2.9767		3.40222			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-46		3.3118		3.6896			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-47		3.5302		3.88773			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-48 SEC-B-177-4		3.5386 1.646		3.89242 4.61663	-		SHEAR SHEAR	1.6	30 30	60 60	120 120	2	1	1.2D+1.6LL+0.5Lr 1.2D+1.6LL+0.5Lr	220
	SEC-B-36		3.4703		8.74635			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	330
	SEC-B-179		1.6079		12.8733			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	330
نموذج ك.ث (2)	SEC-B-86		4.411		13.76461			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	330
	SEC-B-169		7.88E-05		-9.41827			SHEAR	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	330
	Main-B-23-3				-5.11255			BOLTED	1.6	54	84	57.55	5	5		330
Ţ	Main-B-38-4				-5.58683	ļ	ļ	BOLTED	1.6	54	84	57.55	5	5		330
Ļ	Main-B-18-2	<b> </b>	2 0205	0.7070	-6.43621	<b> </b>	<b> </b>	BOLTED	1.6	54	84	57.55	5	5	<del>                                     </del>	330
ŀ	Main-B-85-1 Main-B-77-1	<b> </b>	-2.8295 -2.8451	-9.7879 -11.0266	-3.37607 -5.44985	1	1	BOLTED BOLTED	1.6	54 54	84 84	57.55 57.55	5	5	<del> </del>	330 330
F	Main-B-24-1		2.0431	-11.8051	-6.12008	1	1	BOLTED	1.6	54	84	57.55	5	5		330
نموذج ك.ر (1)	Main-B-33-1		-6.2235	-7.2958	-6.29068			BOLTED	1.6	54	84	57.55	5	5		330
نمودج ك.ر (1)	Main-B-78-1		-3.0695	-12.0172	-6.6586			BOLTED	1.6	54	84	57.55	5	5		330
Ļ	Main-B-11-1		-12.8405		-7.03825			BOLTED	1.6	54	84	57.55	5	5		330
-	Main-B-4-1 Main-B-4-1		-13.3485		-7.6233 -12.3159			BOLTED BOLTED	1.6 2.7	54 54	84 84	57.55 57.55	5	5 5		330 330
F	Main-B-53-1		-6.6253		-7.87244			BOLTED	2.7	54	84	57.55	5	5		330
F	Main-B-10-1				-8.16157			BOLTED	2.7	54	84	57.55	5	5		330
	Main-B-55-4		-13.7585		8			BOLTED	2.7	54	84	57.55	5	5		330
	Main-B-3-1		-14.7649		-13.8719			BOLTED/FILLET WELD	2.7	54	84	57.55	5	5		450
Ļ	Main-B-18-1		11		13.92064			BOLTED/FILLET WELL	2.7	54	84	57.55	5	5		450
-	Main-B-6-1 Main-B-8-1		-10.5385		-15.9015 -18.0098			BOLTED/FILLET WELL	2.7	54 54	84 84	57.55 57.55	5	5 5		450 450
نموذج ك.ر (2)	Main-B-20-1		-17.2963 -12.4118		-18.0098 -17.8531			BOLTED BOLTED	2.7	54	84	57.55	5	5		450 450
(=/3 @-3	Main-B-25-4		-12.4110		-18.1829			BOLTED	2.7	54	84	57.55	5	5		450
	Main-B-55-4				-19.876			BOLTED	2.7	54	84	57.55	5	5		450
	Main-B-39-1		-30.0492		-18.8106			BOLTED	2.7	54	84		5	5		IPE 450
	Main-B-37-3		42.0200		-18.9974			BOLTED	2.7	54	84	72.45	5	5		IPE 450
-	Main-B-48-1 Main-B-12-1		-12.9388 -28.7206		-21.5345 -26.2375			BOLTED BOLTED	2.7	54 54	94	73.45 73.45	6	6	FULL PN. GROVE WLD	IPE 500
نموذج ك.ر (3)	Main-B-39-1		-30.097		-34.1155			BOLTED	2.7	34		73.45	6	6	FULL PN. GROVE WLD	IPE 500
(0/3/2 (2/3)	Main-B-39-9				-39.2354			BOLTED	2.7	54	94	73.45	6	6	FULL PN. GROVE WLD	IPE 500
	Main-B-40-1		-32.2508		-41.8943			BOLTED	2.7	54	94	73.45	6	6	FULL PN. GROVE WLD	IPE 500
	Main-Col-12		6.45	-2.72	5.79			BOLTED	2.7							IPE 600
Ļ	Main-Col-20-8		-10.1411	-2.3367	-4.5388			BOLTED	2.7							IPE 600
ŀ	Main-Col-20 Main-Col-18-8	-	7.75 -19.3122	-15.9 -10.623	-9.51866	<del>                                     </del>	<b> </b>	BOLTED BOLTED	2.7	-	<b> </b>	-		-	<b>_</b>	IPE 600 IPE 600
نموذج ع.ر (1)	Main-Col-21		7.12	-10.623	16.34	<del>                                     </del>	<b> </b>	BOLTED	2.7	1					<del>                                     </del>	IPE 600
سودی ے۔ر ۱۔۱	Main-Col-8		7.12	-43	-11.61	1	1	BOLTED	2.7	1					<b>†</b>	IPE 600
j	Main-Col-69		10.5	-37	27.06			BOLTED	2.7							IPE 600
	Main-Col-29-5		-18.8442	-7.9958	-12.044			BOLTED	2.7							IPE 600
	Main-Col-30		10.4	-21	13.7			BOLTED	2.7							IPE 600
Į.	Main-Col-16		-14.1 4.61	-104	23.4	<del>                                     </del>	<del>                                     </del>	BOLTED	2.7	<u> </u>					<b>-</b>	HEA 800
نموذج ع.ر(2)	Main-Col-25 Main-Col-26		4.61 5.8	-109.21 -116.47	27.24 31.71	<b>-</b>	<b> </b>	BOLTED BOLTED	2.7	-					-	HEA 800 HEA 800
ŀ	Main-Col-27	<b> </b>	12.42	-116.47	13.85	1	1	BOLTED	2.7	t	1			1	<b>-</b>	HEA 800
	Main-Col-23		6.86	-75.9	15.87	1	1	BOLTED	2.7							HEA 600
نموذج ع.ر(3)	Main-Col-14		-12.7	-77.7	11.56			BOLTED	2.7							HEA 600
سودي ١٠١٥٠٠	Main-Col-15		-13.16	-85.24	22.75	l		BOLTED	2.7	ļ						HEA 600
	Main-Col-24		-3.49	-74.79	29.19	<b> </b>		BOLTED	2.7	ļ					ļ	HEA 600
Ļ	Main-Col-28 Main-Col-17	<b> </b>	5.84	-65.96	10.75	<b> </b>	<b> </b>	BOLTED BOLTED	2.7	<b> </b>					<del>                                     </del>	HEA 500
ŀ	Main-Col-18	<b> </b>	-7.45 -11.2	-56 -41.3	-11.49	1	1	BOLTED	2.7	1	1				<del> </del>	HEA 500 HEA 500
ŀ	Main-Col-19	<b> </b>	-7.79	-41.3	32.7	1	1	BOLTED	2.7	t	1			1	<b>-</b>	HEA 500
(4) 5	Main-Col-67		7.38	-45.9	38.5			BOLTED	2.7						1	HEA 500
نموذج ع.ر(4)	Main-Col-68		14.23	-56	16.3	<u></u>		BOLTED	2.7							HEA 500
Ţ	Main-Col-70		13.95	-50.7	13.36			BOLTED	2.7							HEA 500
	Main-Col-71		9.2	-50.7	22.34			BOLTED	2.7							HEA 500
Ļ	Main-Col-22	ļ	7.4	-50.7	41.68	<b> </b>	<b> </b>	BOLTED	2.7		<b></b>	<b>.</b>			ļ	HEA 500
(1) 4 4 -: :	Main-Col-13 Cant-M-Beam-1-1		3.11	-54	5.017	<b> </b>	<b> </b>	BOLTED	2.7	F.4	0.4	70 45		-		HEA 500
نموذج ك.ك (1)	Califiliate Degill-1-1	Ī	-13.1136		-25.1995	1	<del>                                     </del>	BOLTED	2.7	54 54	94 94	73.45 73.45	6	6	<del>                                     </del>	450 450
Г			-13,1136		-25,1995											
-	Cant-M-Beam-2-1 Cant-M-Beam-3-1		-13.1136 -13.2536		-25.1995 -15.7973			BOLTED  BOLTED/FILLET WELD	2.7	54	94	73.45	6	6		450
	Cant-M-Beam-2-1															
	Cant-M-Beam-2-1 Cant-M-Beam-3-1		-13.2536		-15.7973			BOLTED/FILLET WELL	2.7	54	94	73.45	6	6		450

## Project

Steel F	37		
F <sub>y</sub> =	2.4	t/cm <sup>2</sup>	
F <sub>u</sub> =	3.7	t/cm <sup>2</sup>	

Chosen sec	60	)x6	<b>Double Angle</b>
t <sub>G.PL</sub> =	1	cm	Bolted
Bolt Dia.=	16	mm	
L=	605	cm	

<u>Checks :</u>	ĺ		
i- Min. Angle	<u> </u>		
min. a =	52.8	mm	Safe

ii- Buckling		
L <sub>in</sub> =	302.5	cm
L <sub>out</sub> =	453.75	cm
λ <sub>in</sub> =	166.21	Safe
λ <sub>out</sub> =	159.35	Safe

iii- Capacity			_
λ <sub>C</sub> =	1.7878		
F <sub>cr</sub> =	0.4866	t/cm <sup>2</sup>	
P <sub>r</sub> =	5.3794	ton	Safe

<u>iv- Tie Plate</u>			
r <sub>/</sub> =	194.46	cm	Two Tie Plates

#### Element

Staining actions				
Pult =	1	ton		

<b>Sextion Properties</b>				
a =	60	mm		
Area =	13.82	cm²		
r <sub>x</sub> =	1.82	cm		
r <sub>y</sub> =	2.848	cm		
r <sub>v</sub> =	1.17	cm		
r <sub>u</sub> =	2.29	cm		

@ Design Nextical Braking As SHS 2-

Date, f = 12 ton, Lin = 0-5 + 800 = 400 mm.

Lbout = 0.75 + 800 = 600 mm.

Choice: Assume Qc = 1.1 So \ = 102.2 , fer = 1.28 + lcm

from stress, 12 = 0.8 +1.28 + Ag => Ag = 12.11 chaose 420 \* 3.6

From Backling 2 \ = 102.2

 $rac{102.2}{102.2} = \frac{600}{V_{K}}$ ,  $r_{K} = 5.87$  cm.

 $r_{102.2} = \frac{400}{r_g}$ ,  $r_g = 3.91$  cm. Choose 150+5

Finally use 120 + 120 + 5

 $A = 22,9 \text{ cm}^2$ ,  $V_X = 5.74 \text{ cm}$ ,  $V_Y = 3.31 \text{ cm}$ 

cheek Compactness:

 $\frac{h-2(r+t)}{t} = 22 \left\langle \frac{64}{fy} = 41 \right\rangle \text{ coeb & flarge}$ are Non-Compact.

b-2(r+t) = 22 < 41

Check Bulkling - Sout = 600 = 127 (180  $\sqrt{\sin} = \frac{400}{4.69} = 85.3 < 180$ 

Fer = 0.83 t/cm2 - Pr = Ofer Ag = 0.8 + 0.83 + 22.9 = 15.2 > 12 Safe.

 $Last = \frac{12}{0.7 + 4 \times 0.5 \times 0.4 \times 3.6} + 2 \times 0.5 = 69 \text{ mm}$ take A = 100 mm

			Flanges			Web		SEC-B-177-4
Section Dim	ensions	tf=	0.92	cm	tw=	0.59	cm	3EC-B-177-4
		b=	11	cm	hw=	22	cm	Length (cm) 60
							1	
Material:				Geometry:				
Steel Grade	St	t.37		Lb=	0	cm		
				Cb=	1.130			
Straining Actio	ns:			Lbx=	600	cm		SEC-B-177-4
Mux=	4.00	t.m		Lby=	600	cm		
				L=	600	cm		
Qu=	1	t		Lh=	600	cm		
Nu=	0.00	t						
IVU-	0							

				Flexure	
(1) Local Bu	ıckling	ling Compact Flange		Compac	t Web Compact Section
(2) LTB					
Lp=	128.113	cm			
Lr=	477.085	cm		Case A	
Mn=	7.20	t.m	≤	Mp=	7.20002 t.m
Φb*Mn=	6.12	t.m	D/C=	0.654	Safe for flexure about major axis

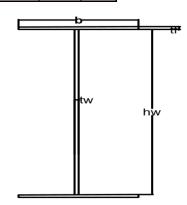
			Axia	l Compression	
λx=	61.295	≤	180		OK
λy= λc=	241.849	≤	180	De	crease λy
λc=	2.6025				
Fcr=	0.2296	t/cm2			
Pn=	7.63	t			
Φc*Pn=	6.10	t	D/C=	0.000	Safe for axial compression

			А	xial Tension	
(1) Stiffnes	condition				
λ=	241.849		≤	300	OK
Lh/60=	10.000		≤	h	ОК
(2) Strengh	nt condition				
Pn=	79.73	t			
Φt*Pn=	67.7688	t	D/C=	0.000	Safe for yielding at tension

	Combine	d (Normal F	orce + Flexure)
M+C	D/C=	0.654	Safe for combined M+C
M+T	D/C=	0.654	Safe for combined M+T

| 15t trial Sx=Mu/(.75\*Fy) = 222.22 | Section Properties | Ix | 3183.10585 | cm4 | Iy | 204.463195 | cm4 | rx = 9.79 | cm | ry = 2.48 | cm | A = 33.22 | cm2 | Aw = 14.07 | cm2 | Sx = 267.04 | cm3 | Sy = 37.18 | cm3 | Zx = 300.00 | cm3 | Zy = 57.57 | cm3 | rt = 2.88 | cm | cm2 | cm2 | cm3 | cm3

Mateial Properties						
Fy= 2.4 t/cm2						
Fu=	3.7	t/cm2				
E=	2100	t/cm2				



			Flanges			Web		HED SOO
Section Din	nensions	tf=	1.9 cm tw= 1.2 cm 22 cm hw= 60 cm  Geometry: Lb= 705 cm Cb= 1.130 Lbx= 282 cm  Cant-M-Beam-2-	HEB 300				
		b=	22	cm	hw=	60	cm	Length (cm) 2
				-				<u></u>
Material:				Geometry:				
Steel Grade	St	:.37		Lb=	705	cm		
				Cb=	1.130			
Straining Acti	ons:			Lbx=	282	cm		Cant-M-Beam-2-1
Mux=	25.20	t.m		Lby=	282	cm		
				L=	282	cm		
Qu=	13	t		Lh=	282	cm		
Nu=	0.00	t					-	
Tu=	0	t						

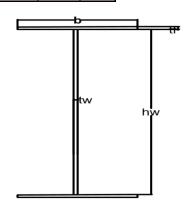
	Flexure								
(1) Local Bu	ıckling	ng Compact		Compac	t Web	Compact Section	Compact Section		
(2) LTB									
Lp=	240.697	cm							
Lr=	828.938	cm		Case B					
Mn=	61.74	t.m	≤	Mp=	87.3158	t.m			
Φb*Mn=	52.48	t.m	D/C=	0.480	Safe	for flexure about major as	kis		

	Axial Compression								
λx=	11.030	≤	180		OK				
λy=	60.501	≤	180		OK				
λy= λc=	0.6510								
Fcr=	2.0094	t/cm2							
Pn=	312.66	t							
Φc*Pn=	250.13	t	D/C=	0.000		Safe for axial compression			

			Д	xial Tension	
(1) Stiffnes	condition				
λ=	60.501		≤	300	OK
Lh/60=	4.700		≤	h	ОК
(2) Strengh	nt condition				
Pn=	373.44	t			
Φt*Pn=	317.424	t	D/C=	0.000	Safe for yielding at tension

	Combined	d (Normal F	orce + Flexure)		Shear Ford	e	
M+C	D/C=	0.480	Safe for combined M+C	Vn=	110.25	t	
M+T	D/C=	0.480	Safe for combined M+T	Φv*Vn=	93.71	t	
				D/C=	0	.139	
				9	Safe for Shear		
			Safe				

Mateial Properties							
Fy= 2.4 t/cm2							
Fu=	3.7	t/cm2					
E=	2100	t/cm2					



		Flanges			Web			SEC-B-132	1
Section Dimensions	tf=	1.6	cm	tw=	1.02	cm		3EC-B-132	2-1
	b=	20	cm	hw=	50	cm		Length (cm)	600
						•			
faterial:			Geometry:						
Steel Grade S	t.37		Lb=	0	cm				
			Cb=	1.130					
training Actions:			Lbx=	600	cm	Ì	Mai	n-B-132-1	
Mux= 18.00	t.m		Lby=	600	cm				
			L=	600	cm				
Qu= 32	t		Lh=	600	cm				
Nu= 0.00	t	•							
Tu= 0	t								

				Flexure			
(1) Local Bu	ickling	Compa	ct Flange	Compac	t Web	Compact Se	ection
(2) LTB							
Lp=	222.646	cm					
Lr=	760.344	cm		Case A			
Mn=	54.48	t.m	≤	Mp=	54.4834	t.m	
Φb*Mn=	46.31	t.m	D/C=	0.389	Safe '	for flexure about m	ajor axis

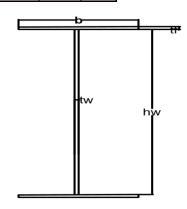
			A			
			AXI	al Compression		
λx=	27.886	≤	180		OK	
λy= λc= Fcr=	139.162	≤	180		OK	
λc=	1.4975					
Fcr=	0.6935	t/cm2				
Pn=	79.75	t				
Фс*Pn=	63.80	t	D/C=	0.000		Safe for axial compression

			Д	xial Tension	
(1) Stiffnes	condition				
λ=	139.162		≤	300	ОК
Lh/60=	10.000		≤	h	OK
(2) Strengh	t condition				
Pn=	276.00	t			
Φt*Pn=	234.6	t	D/C=	0.000	Safe for yielding at tension

	Combine	d (Normal F	orce + Flexure)	
M+C	D/C=	0.389	Safe for combined M+C	Vn=
M+T	D/C=	0.389	Safe for combined M+T	Φv*Vn=
				D/C=

Hint:		
1st trial Sx=I	Mu/(.75*Fy)=	1000.00
Se	ction Propertie	2S
lx	53239.6133	cm4
ly	2137.75503	cm4
rx=	21.52	cm
ry=	4.31	cm
A=	115.00	cm2
Aw=	54.26	cm2
Sx=	2001.49	cm3
Sy=	213.78	cm3
Zx=	2270.14	cm3
Zy=	333.01	cm3
rt=	5.13	cm

Ma	ateial Propertie	es .
Fy=	2.4	t/cm2
Fu=	3.7	t/cm2
E=	2100	t/cm2



			Flanges			Web				Hint:		
Section D	imensions	tf=	0.69	cm	tw=	0.47	cm	HU	NCH-L9		=Mu/(.75*Fy)=	38.89
		b=	7.3	cm	hw=	14	cm	Length	(cm) 400		ection Propertie	
								NO fly bra	icing	lx	651.355497	cm4
laterial:				Geometry:						ly	44.8580818	cm4
teel Grade	St.3	37		Lb=	400	cm		Wir	ndow-Beam-19-1	rx=	6.25	cm
			_	Cb=	1.300					ry=	1.64	cm
raining Act				Lbx=	1201.5	cm			1	A=	16.65	cm2
Mux=	0.70	t.m		Lby=	400	cm				Aw=	7.23	cm2
				L=	400	cm				Sx=	84.70	cm3
Qu=	0.80	t		Lh=	400	cm				Sy=	12.29	cm3
Nu=	0.00	t								Zx=	95.92	cm3
Tu=	0	t								Zy=	19.16	cm3
										rt=	1.91	cm
VI I D	lite	C		Flexure	14/-l-	_	C+i-				lataial Daga	_
) Local Buc	KIING	Compa	ct Flange	Compact	vveb	Co	ompact Section				lateial Propertie	
2) LTB										Fy=	2.4 3.7	t/cm2 t/cm2
) LID										Fu= E=	2100	t/cm2
)=	84.751	cm									2100	t/till2
=	347.894	cm		Case C								
		•									<del>b-</del> -	
	4.25				2.30197							
n=	1.35	t.m	≤	Mp=	2.50197	t.m					- 11	
In= b*Mn=	1.15	t.m t.m	D/C=	0.611			e about major axis					
			D/C=	0.611 Compression		e for flexure	e about major axis					
b*Mn=	1.15	t.m	D/C= Axial	0.611 Compression	Safe	e for flexure	e about major axis	<b>■</b> 				
b*Mn=	1.15	t.m	D/C= Axial	0.611 Compression	Safe Decrease λ	e for flexure	e about major axis	<b>■</b> 			-tv	,
b*Mn=	1.15 192.121 243.724	t.m	D/C= Axial	0.611 Compression	Safe Decrease λ	e for flexure	e about major axis				-tv	/
*Mn= = = = =	1.15 192.121 243.724 2.6227 0.2261 3.77	t.m ≤ ≤	D/C= Axial 180 180	0.611 Compression	Safe Decrease λ ecrease λy	e for flexure					-tv	,
*Mn= = = = = = =	1.15 192.121 243.724 2.6227 0.2261	t.m ≤ ≤ t/cm2	D/C= Axial	0.611 Compression	Safe Decrease λ ecrease λy	e for flexure	e about major axis				-tv	,
*Mn= = = = = = =	1.15 192.121 243.724 2.6227 0.2261 3.77	t.m  ≤ ≤ t/cm2 t	Axial 180 180	0.611  Compression  D  0.000	Safe Decrease λ ecrease λy	e for flexure					-tv	,
b*Mn=  c= /= c= cr= n= c*Pn=	1.15 192.121 243.724 2.6227 0.2261 3.77 3.01	t.m  ≤ ≤ t/cm2 t	Axial 180 180	0.611  Compression  D	Safe Decrease λ ecrease λy	e for flexure					-tv	,
b*Mn=  (= /= := := :r= n= c*Pn=	1.15 192.121 243.724 2.6227 0.2261 3.77 3.01	t.m  ≤ ≤ t/cm2 t	Axial 180 180 D/C=	0.611  Compression  D  0.000  dial Tension	Safe Decrease λ ecrease λy	e for flexure					-tv	,
*Mn= = = = = = = = *Pn= Stiffnes co	1.15 192.121 243.724 2.6227 0.2261 3.77 3.01 ondition 243.724	t.m  ≤ ≤ t/cm2 t	D/C=  Axial 180 180  D/C=  Ax	0.611  Compression  D  0.000  dial Tension  300	Safe Decrease λ ecrease λy	e for flexure					·tv	,
c= c= cr= n= cc*Pn=	1.15 192.121 243.724 2.6227 0.2261 3.77 3.01	t.m  ≤ ≤ t/cm2 t	Axial 180 180 D/C=	0.611  Compression  D  0.000  dial Tension	Safe Decrease λ ecrease λy	e for flexure					rtv	,
t= (= (= (= (= (= (= (= (= (= (= (= (= (=	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667	t.m  ≤ ≤ t/cm2 t	D/C=  Axial 180 180  D/C=  Ax	0.611  Compression  D  0.000  dial Tension  300	Safe Decrease λ ecrease λy	e for flexure					-tv	,
o*Mn= == == == == == == == == == =========	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667	t.m  ≤ ≤ t/cm2 t	D/C=  Axial 180 180  D/C=  Ax	0.611  Compression  D  0.000  dial Tension  300	Safe Decrease λ ecrease λy	e for flexure					-tv	′
*Mn=  = = =	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667 condition	t.m	D/C=  Axial 180 180  D/C=  Ax	0.611  Compression  D  0.000  dial Tension  300	Decrease λ	Safe for axi					·tv	,
b*Mn=  ==================================	1.15 192.121 243.724 2.6227 0.2261 3.77 3.01 243.724 6.667 condition 39.97	t.m	D/C=  Axial 180 180  D/C=  Ax  \$ \$ \$ \$	0.611  Compression  D  0.000  itial Tension  300  h	Decrease λ	Safe for axi	al compression				-tv	,
= = = = = = = = = = = = = = = = = = =	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667  condition 39.97 33.97416  Combined	t.m  \$  \$  t/cm2 t t t (Normal Foil	D/C=  Axial 180 180  D/C=  Ax  \$ \$ \$ \$ \$ \$ \$ \$  Ce + Flexure)	0.611  Compression  D  0.000  itial Tension  300  h	Safe	Safe for axi.  OK OK	al compression  ding at tension  Shear Force				-tv	′
x= y= c= cr= n= bc*Pn= t1) Stiffnes cc= = h/60= 2) Strenght n= n= tt*Pn=	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667 condition 39.97 33.97416  Combined D/C=	t.m	D/C=  Axial 180 180  D/C=  Ax  \$ 5  D/C=  Cce + Flexure) Safe for cc	0.611  Compression  D  0.000  dial Tension  300  h  0.000	Decrease λy	Safe for axi.  OK OK OK Vn=	al compression  ding at tension  Shear Force  10.41 t				-tv	,
x= y= c= cr= n= n= h/60= 2) Strenght n= nt*Pn=	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667  condition 39.97 33.97416  Combined	t.m  \$  \$  t/cm2 t t t (Normal Foil	D/C=  Axial 180 180  D/C=  Ax  \$ 5  D/C=  Cce + Flexure) Safe for cc	0.611  Compression  D  0.000  itial Tension  300  h	Decrease λy	OK OK Safe for yiel  Vn= $\Phi v^*Vn=$	al compression  ding at tension  Shear Force  10.41 t 8.85 t				-tv	,
b*Mn=  = = = = = = = = = = = = = = = = = =	1.15  192.121 243.724 2.6227 0.2261 3.77 3.01  ondition 243.724 6.667 condition 39.97 33.97416  Combined D/C=	t.m	D/C=  Axial 180 180  D/C=  Ax  \$ 5  D/C=  Cce + Flexure) Safe for cc	0.611  Compression  D  0.000  dial Tension  300  h  0.000	Decrease λy	OK OK Safe for yiel  Vn= D/C=	al compression  ding at tension  Shear Force  10.41 t				-tv	,

Safe

# Ain Shams university Faculty of engineering Structural Engineering Department - SCHEP Graduation project Design of columns Refrence : Egyptian code( LRFD )



Frame loaction

M <sub>x 14</sub>	10	t.m
N	66	ton
Q	5	ton

Column ID Main-Col-16-12

h	49	cm
t <sub>w</sub>	1.2	cm
b <sub>F</sub>	30	cm
t <sub>f</sub>	2.3	cm



(i) Web		
d <sub>w</sub>	44.4	cm
t <sub>w</sub>	1.2	cm
λ	dw <del>Ew</del>	37

	(ii) Flange		
	С	15	cm
	t <sub>f</sub>	2.3	cm
table	$\lambda =$	aC tf	4.96
2.12c	λρ		9.88
. 2-29	$\lambda_r$		18.07

2.3	cm
	4.06

α 0.76

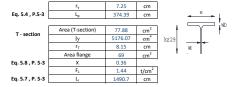
Class.	compact

Class.	compact

#### Section class. compact

#### ( Ch.5 ) ( Compact section )

50.81 143.3



e (b)	Мр	92.22	t.m	
C <sub>b</sub>	1.83		User defi	ne ,5.1 , P.5-1
Mr Mcr	49.4		P.5-1 : P.5-4	
Mn	92.22	t.m	]	
φ φMn	0.85 78.39	t.m	]	Safe
	C <sub>b</sub> Mr Mcr Mn	C <sub>b</sub> 1.83 Mr 49.4 Mcr - Mn 92.22 φ 0.85	C <sub>b</sub> 1.83 Mr 49.4 Mcr Mn 92.22 t.m φ 0.85	C <sub>0</sub> 1.83 User defit Mr 4.9.4 Mcr - P.5-1: P.5-4 Mn 92.22 Lm φ 0.85

#### (9) Check combined M+N Ch.7

Pu Ø Pn	0.2355124
Mu Ø Mn	0.84

Eq.s 7.1a,7.1b Ratio 0.98

Safe

#### (4) section properties

Area	196.8	cm <sup>2</sup>
Ix	84054.38	cm <sup>4</sup>
Iy	10356.39	cm <sup>4</sup>
Sx	3430.79	cm <sup>3</sup>
Sy	690.43	cm <sup>3</sup>

#### (5) Column data

Total length	550	cm
Base type	Hinged	
G 1	10	
Ix (Rafter)	48200	cm <sup>4</sup>
Length of rafter	600	cm
G 2	1.9	
К	1.9	

Table 2.6b , P.2-15 User define User define

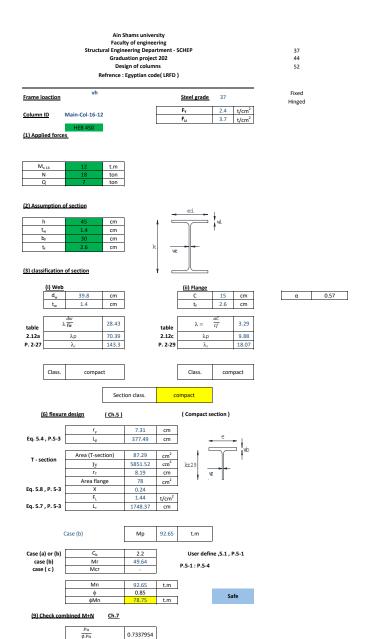
User define , chart (2.7) , P.2-14

	cm	1045	L bx
User define ( from bracing	cm	550	L by = Lb

<u>(5)</u>	flexure design		<u>( N</u>	on-compact	1			
			flange)	270.52	t.m	Eq. 5.16		
			(web)	92.22	t.m	P. 5-5		
		n n	//n	92.22	t.m			
			L <sub>p</sub> '	374.39	cm			
	case ( b )							
	case	( D )						
Case (a)	or (b)		Cb	0.6				
case	(b)		Mr	49.4				
case (	c)	N	Лcr	-				
P. 5-	.5		Иn	51.29	t.m			
		ф		0.85			Sa	fo
		ф	Mn	43.6	t.m		30	ie

(7) Desgin of normal stre	enght .	(Ch.4)	
ĺ	r <sub>x</sub>	20.67	cm
ĺ	r <sub>y</sub>	7.25	cm
	λx	50.56	
	λγ	75.86	
	λmax	75.86	
Eq. 4.4	λc	0.82	
Eq. 4.2 , 4.3	F <sub>cr</sub>	1.78	
Eq. 4.1	Pn	350.3	ton
	ф	0.8	
	φBn	200.24	ton

	φPn	280.24	ton	
(8) Design of shear str	<u>Ch.5</u>			
	h tw	37		
	<sup>112</sup> / <sub>√Fy</sub>	72.3		
	<sup>139</sup> / <sub>√Fy</sub>	89.72		
Eq.s 5.22,5.23,5.24	Vn	76.72	ton	
	ф	0.85		
	φVu	65.21	ton	



Mu Ø Mn

Ratio

Eq.s 7.1a,7.1b

0.23

0.94

Safe

#### (4) section properties

Area	219	cm <sup>2</sup>
Ix	77555.75	cm <sup>4</sup>
Iy	11709.1	cm <sup>4</sup>
Sx	3446.92	cm <sup>3</sup>
Sy	780.61	cm <sup>3</sup>

#### (5) Column data

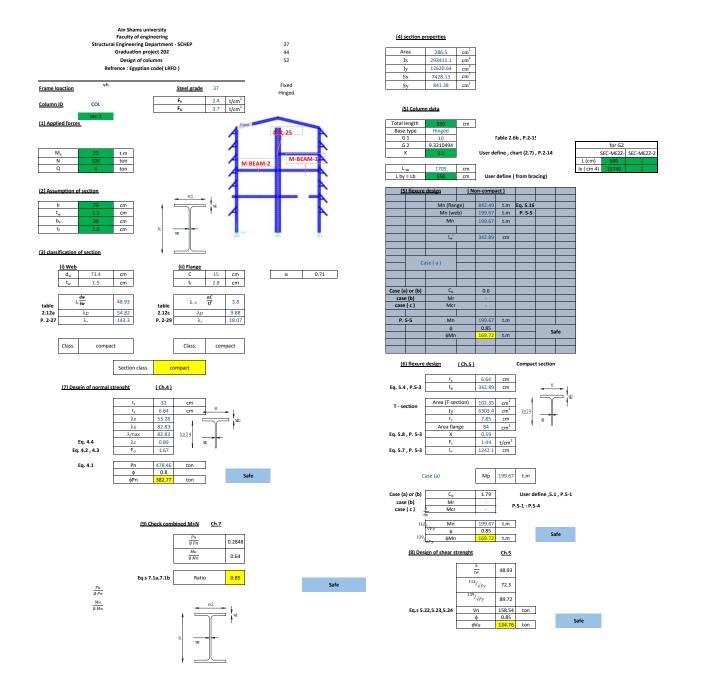
Total length	1600	cm	
Base type	Hinged		•
G 1	10		Table 2.6b , P.2-15
Ix (Rafter)	79890	cm <sup>4</sup>	User define
Length of rafter	2430	cm	User define
G 2	1.47		•
K	3.8		User define , chart (2.7) , P.2-14

L bx	6080	cm	
L by = Lb	1600	cm	User define ( from bracing)

(5) fle	xure design		L	Non-compact )			
		Mn (	flange)	186.2	t.m	Eq. 5.16	
		Mn	(web)	92.65	t.m	P. 5-5	
		N	∕ln	92.65	t.m		
			L <sub>p</sub> '	377.49	cm		
	case ( b	1					
	case ( b	1					
Case (a) or	(b)		Сь	0.6			
case (b)			Mr	49.64			
case ( c )		N	Лcr	-			
P. 5-5		ı	Иn	32.58	t.m		
			φ	0.85			Safe
		ф	Mn	27.69	t.m		50.0

) Desgin of normal strenght		(Ch.4)	
	r <sub>x</sub>	18.82	cm
	r <sub>y</sub>	7.31	cm
	λx	323.06	
	λγ	218.88	
	λmax	323.06	
Eq. 4.4	λc	3.48	
Eq. 4.2 , 4.3	F <sub>cr</sub>	0.14	
Eq. 4.1	Pn	30.66	ton
Lq. 4.1		0.8	ton
	ф		
	φPn	24.53	ton

(8) Design of shear strenght		<u>Ch.5</u>		
	h tw	28.43		
	<sup>112</sup> / <sub>√Fy</sub>	72.3		
	<sup>139</sup> / <sub>√Fy</sub>	89.72		
q.s 5.22,5.23,5.24	Vn	80.24	ton	ĺ
	ф	0.85		
	φVu	68.2	ton	1



Project	PROJ NO.8

Steel Properties			St.37
Fy	2.4	t/cm2	
Fu	3.6	t/cm2	

Connection Details		
Secondary bear		IPE330
Main beam s	sec.	IPE450
Angle		80x8
tw_sec Beam = 0.75		cm
tw_main Beam =	0.94	cm
t angle=	0.8	cm
t min n1=	0.8	cm
t min n2=	0.75	cm

7
7

<u>Shear</u>			included
qb=	5	t/cm2	
As=	1.568	cm2	
Rvs,b=	4.705	ton	
Rvd,b=	9.41	ton	

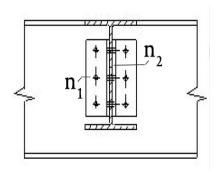
Bearing			
α	1.6		
F <sub>b</sub> =	5.76	t/cm <sup>2</sup>	
Rbr,b n1=	5.161	ton	
Rbr,b n2=	4.838	ton	

Number of Bolts		
R n1_least (t)	4.705	
R n2_double (t)	4.838	
n1	2	
n2	2	

Element	SEC-B-261

Straining Actions		
Q u=	8	ton

<u>Bolts</u>		
d=	1.6	cm
edge dist. =	25	cm
Pitch dist. =	50	cm
Category:	A	1
Grade:	10	.9
F <sub>y</sub> =	9	t/cm2
F <sub>u</sub> =	10	t/cm2



In Case of Category (B):-		
Qs=	#### ton	
Case of loading		
St.	37&44	
Ordinary steel work		
Ps=	####	ton
n1 ser =	####	Bolts
n2 ser =	####	Bolts

Block Shear Rupture (angle)		
A <sub>net shear</sub> =	115.68	cm <sup>2</sup>
A <sub>net tension</sub> =	4.32	cm <sup>2</sup>
0.6F <sub>u</sub> A <sub>n.s</sub> =	249.8688	Ton
F <sub>u</sub> A <sub>n.t</sub> =	15.552	Ton
A <sub>gross shear</sub> =	120	cm <sup>2</sup>
A <sub>gross tension</sub> =	5.76	cm <sup>2</sup>
Rrp =	184.585	Ton

Safe

Project	PROJ NO.8

Steel Properties		St.37	
Fy	2.4	t/cm2	
Fu	3.6	t/cm2	

Connection Details		
Secondary bear	n sec.	<b>IPE220</b>
Main beam s	ec.	<b>IPE450</b>
Angle		80x8
tw_sec Beam =	0.59	cm
tw_main Beam =	0.94	cm
t angle=	0.8	cm
t min n1=	0.8	cm
t min n2=	0.59	cm

J
7

<u>She</u>	<u>ear</u>		included
qb=	5	t/cm2	
As=	1.568	cm2	
Rvs,b=	4.705	ton	
Rvd,b=	9.41	ton	

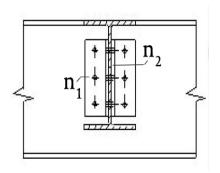
<u>Bearing</u>		
α	1.6	
F <sub>b=</sub>	5.76	t/cm <sup>2</sup>
Rbr,b n1=	5.161	ton
Rbr,b n2=	3.806	ton

Number of Bolts	
R n1_least (t) 4.705	
R n2_double (t)	3.806
n1	2
n2	2

Element	SEC-B-261

Strair	ning Actions	S
Q u=	7	ton

	<u>Bolts</u>	
d=	1.6	cm
edge dist. =	25	cm
Pitch dist. =	50	cm
Category:		
Grade:	10	.9
F <sub>y</sub> =	9	t/cm2
F <sub>u</sub> =	10	t/cm2



In Case of Category (B):-							
Qs=	####	ton					
Case of lo	ading						
St.	378	<b>k</b> 44					
Ordina	ry steel wo	rk					
Ps=	####	ton					
n1 ser =	#### Bolts						
n2 ser =	####	Bolts					

Block Shear Rupture (angle)								
A <sub>net shear</sub> =	115.68	cm <sup>2</sup>						
A <sub>net tension</sub> =	4.32	cm <sup>2</sup>						
0.6F <sub>u</sub> A <sub>n.s</sub> =	249.8688	Ton						
F <sub>u</sub> A <sub>n.t</sub> =	15.552	Ton						
A <sub>gross shear</sub> =	120	cm <sup>2</sup>						
A <sub>gross tension</sub> =	5.76	cm <sup>2</sup>						
Rrp =	184.585	Ton						

= 184.585 Ton Safe

Project				Element	M-B-40	+ M-Col-15	
Steel P	roperties		St.37	Strainin	ng action	ns:-	1
Fy	2.4	t/cm2	51.57	Mu=	41	mt	
Fu	3.6	t/cm2	<u>l</u>	Tu= Qu=	33	t	
Danier D		_	1		Di		
bf=	imensions 40	cm	Í	Column bf=	30	cm	
tf=	2	cm		tf=	3.5	cm	
dw= tw=	58 0.8	cm cm		dw= tw=	1.85	cm	
d=	60	cm			363.55	cm	
Connection	n Geomet	ry:-	1	E	Bolts		)
a=	9.2	cm	1	d=	2.7	cm	
b= tp=	5.4 3	cm		As= Grade	4.4659 10.9	cm Category B	
2W=	18.8	cm		Fy, bolt=	9	t/cm2	
W= B=	9.4	cm cm		Fu, bolt=	10	t/cm2	
Charles	1		-				
Checks Fillet Wled	Between	Beam ar	nd End Plate				
Sf=	10 144	mm cm2	Sw=	76.8	mm cm2		
Aw hz= Aw tot=	220.8	cm2 cm2	Aw VI= Ix=	137565.6	cm2 cm4		
Check Stree	sses F1	0.03303	1 4/2	C-4-		-"	
Point 1 Point 2	f2	0.92392 0.7153	t/cm2 t/cm2	Safe			
	q2	0.42969	t/cm2	C-4-			
	F2	1.03225	t/cm2	Safe			
	r of bolts		Extended v	ith full depth bol	t		
e= p=	5.4 8.1	cm	+	+ ] e		Г	
allowable n/2=	6	bolts	=				+ +
P actual= chosen n=	9.4	cm bolts	+	+			+ + +
			+	+			+ +
			+	+			+)(+)
						L	
			Extended w	ith full depth bolt	ts		Full depth bolts
y1	63.3	cm	у6	14.9	cm		
y2 y3	52.5 43.1	cm cm	y7 y8	0	cm		
у4	33.7	cm	у9	0	cm		
y5 Σy2=	24.3 10569	cm cm2	y10	0	cm		
			a.			ī	
Tension on I First bolt	Bolts	l		Second bo	lt 8.3599	t	ĭ
T1=	12.278	t		Prying Force=	2.1717	t	
Rtb=	25.009	t	Safe	T2+P=	10.532	t	Safe
Shear on b							
Qu/n= Rvb=	2.3571 17.177	t					
			•				
interaction eq=	0.2599	Safe	l				
In Case of C				d=	2.7	cm	
Qs= Ms=	22 27.333	ton		Ordinary Case of load		vork I	
Ts=	0	ton	1	St.		37&44	
Tb=	8.1853	ton	1	Grade Load factor	r=	10.9 1.5	
Qb=	1.5714	ton	]	Ps=	9.25	ton	
interaction eq=	0.453	Safe	1	T=	28.91	ton	
	-		1				
Thickness of er M1=	19.979	cm.t	1				
M2=	25.164	cm.t		•			
tb required=	2.2911	cm	Safe	l			
Check Col			I				
db= Tb=	60 70.69	cm t	ł				
Cb=	70.69	t	1				
Flange Loa	acal Bendi	ing	1	Using Do	oubler P	late	No Need
Rt=	156.19	t	ļ	t=	10	mm	
Tb=	70.69	t	safe	Rt	####	t	
Using			No Need		8000	T	ī
bs= ts=	14 10	cm mm	Local buckling Normal stresses	bs/ts= T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	
			_				
		ng	I	Using Do			No Need
Web Loa				t=	5	mm	
Web Loa Rc= Cb=	146.58 70.69	t	Safe	Rc	####	t	
Rc= Cb=	146.58 70.69	t			####		
Rc=	146.58 70.69	t	Safe No Need Local buckling		####		
Rc= Cb= Using bs= ts=	146.58 70.69 Stiffner 14 10	t t cm mm	No Need Local buckling Normal stresses	Rc bs/ts= T=	####	t	
Rc= Cb= Using bs=	146.58 70.69 Stiffner	t t	No Need Local buckling	Rc bs/ts=	####	t	
Rc= Cb= Using bs= ts=	146.58 70.69 Stiffner 14 10	t t cm mm	No Need Local buckling Normal stresses	Rc bs/ts= T=	####	t	
Rc= Cb= Using bs= ts=	146.58 70.69 Stiffner 14 10 83	t t cm mm	No Need Local buckling Normal stresses	Rc bs/ts= T=	####	t	
Rc=   Cb=   Using   bs=   ts=   ds=   Web 0	146.58 70.69 Stiffner 14 10 83 Crippling	t t	No Need Local buckling Normal stresses	Rc bs/ts= T= Q= Using Do	#### #### ####	t t t	No Need
Rc= Cb= Using bs= ts= ds=	146.58 70.69 Stiffner 14 10 83	t t	No Need Local buckling Normal stresses	Rc bs/ts= T= Q=	#### #### ####	t t	No Need
Rc=   Cb=   Using   bs=   ts=   ds=   Web C   Rn=   Rc=   Cb=	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69	t t t	No Need Local buckling Normal stresses Shear stresses	Bs/ts= T= Q= Using Dc t=	#### #### #### bubler P	t t t	No Need
Rc=   Cb=   Using   bs=   ts=   ds=   Web C   Rn=   Rc=	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69	t t t	No Need Local buckling Normal stresses Shear stresses	Bs/ts= T= Q= Using Dc t=	#### #### #### bubler P	t t t	No Need
Rc=   Cb=   Using   bs=   ts=   ds=     Web C   Rn=   Rc=   Cb=   Using   bs=   ts=   ts=   ts=     Cb=	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69 Stiffner 14 6	t t t t t t cm	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Rc	#### #### bubler P 5 ####	t t t t	No Need
Rc=   Using   Using	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69 Stiffner	t t t cm mm cm t t t t cm	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling	Rc	#### #### bubler P 5 ####	t t t t	No Need
Rc= Cb=  Using bs= ts= ds=  Web C Rn= Rc= Cb= Using bs= ts= ds=	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69 Stiffner 14 6 83	t t cm mm cm t t t t cm	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Rc	#### #### bubler P 5 ####	t t t t	No Need
Rc=   Cb=   Using   bs=   ts=   ds=     Web C   Rn=   Rc=   Cb=   Using   bs=   ts=   ds=     Shear P   Shear P   Shear P   Rc=   Cb=   Shear P   Shear P	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69 Stiffner 14 6 83	t t cm mm cm t t t t cm	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Rc	#### #### bubler P 5 ####	t t t t	No Need
Rc= Cb=  Using bs= ts= ds=  Web C Rn= Rc= Cb=  Using bs= ts= ds=  Using bs= ts= ds=  Shear P Pu 0.4Py=	146.58 70.69 Stiffner 14 10 83 rippling 124.42 87.069 70.69 Stiffner 14 6 83 anel Zone 160 349.01	t t t t t t t t t t t t t t t t t t t	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Rc	#### #### 5 #### #### ####	t t t t t t t t t t t t t t t t t t t	No Need
Rc= Cb=  Using bs= ts= ds=  Web C  Rn= Rc= Cb=  Using bs= ts= ds=  Using hs= Rc= Cb= Re= Rc= Rc= Rc= Rc= Rc= Rc= Rc= Rc= Rc= Rc	146.58 70.69 Stiffner 14 10 83 Crippling 124.42 87.096 70.69 Stiffner 14 6 83 anel Zone 160 349.01 203.8	t t t cm mm cm t t t t t t t t t t t t t	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses	Solution   Solution	#### #### 5 #### #### #### ####	t t t t t t t ordered and t t t t t t t t t t t t t t t t t t t	
Rc= Cb=  Using bs= ts= ds=  Web C Rn= Rc= Cb=  Using bs= ts= ds=  Using bs= ts= ds=  Shear P Pu 0.4Py=	146.58 70.69 Stiffner 14 10 83 rippling 124.42 87.069 70.69 Stiffner 14 6 83 anel Zone 160 349.01	t t t t t t t t t t t t t t t t t t t	No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Rc	#### #### 5 #### #### ####	t t t t t t t t t t t t t t t t t t t	

Project				Element	M-B-20	+ M-Col-15	
	roperties 2.4	A/2	St.37	Straini Mu=	ng actio		
Fy Fu	3.6	t/cm2 t/cm2	i	Tu=	0	mt t	
			•	Qu=	17	t	
Beam Di	mension	\$	1	Column	Dimens	ions	İ
bf=	19	cm	1	bf=	30	cm	
tf=	1.46	cm		tf=	3.5	cm	
dw= tw=	43.54 1.4	cm		dw= tw=	83 1.85	cm	
d=	45	cm		Ac	363.55	cm	
			-				· }
Connection a=	4.4	ry:- cm		d=	Bolts 2.7	cm	
b=	5.4	cm	]	As=	4.4659	cm	
tp= 2W=	2	cm		Grade	10.9	Category B	
W=	17.2 8.6	cm	1	Fy, bolt= Fu, bolt=	10	t/cm2 t/cm2	
B=	21	cm	]				
Checks	1						
	Betweer	n Beam ar	nd End Plate				
Sf=	10	mm	Sw=	8	mm		
Aw hz= Aw tot=	68.4 126	cm2 cm2	Aw VI=	57.6 39148.30464	cm2 cm4		
Check Stres		CITIZ	12-	33140.30404	CIII4		
Point 1	F1	1.08051		unsafe			
Point 2	f2 q2	0.82762	t/cm2 t/cm2				
	F2	0.97277		Safe	l		
	r of L		Post 1 1	ittle faith de la ca	de	i	
e=	r of bolts 5.4	cm	extended w	ith full depth bo	nt	l	
p=	8.1	cm	+	+ - e		Γ	
allowable n/2= P actual=	5 8.6	bolts	5				+ +
chosen n=	8.6	bolts	+	+			+   +
			+	+			+   +
			+ +	+			+)(+)
				<u> </u>		L	
y1	49.77	cm	Extended w y6	ith full depth bo	lts cm	l	Full depth bolts
y2 y2	38.97	cm	y7	0	cm		
у3	30.37	cm	y8	0	cm		
y4 y5	21.77 13.17	cm cm	y9 y10	0	cm		
Σγ2=	5565.4	cm2	,				
Tanaian an I	1-14-	1	1	Cd b.	-14		
Tension on E First bolt	soits	l		Second be	4.9112	t	ì
T1=	8.0484	t		Prying Force=	3.2494	t	
Rtb=	25.009	t	Safe	T2+P=	8.1607	t	Safe
Shear on b	olts		_				
Qu/n=	1.4167	t					
Rvb=	17.177	t	ı				
interaction eq=	0.1133	Safe	]				
In Case of C	ataganı l	(p).	1	d=	2.7	cm	i
Qs=	11.333	ton	1		ry steel v		
Ms=	12	ton	]	Case of loa	ding		
Ts=	0	ton	l	St. Grade		37&44 10.9	
Tb=	5.4404	ton	1	Load facto	or=	1.5	
Qb=	0.9444	ton	]	Ps=	9.25	ton	
interaction eq=	0.2903	Safe	1	T=	28.91	ton	
			•				
Thickness of en			1				
M1= M2=	14.298 12.223	cm.t	l				
tb required=	1.8055	cm	Safe				
Chock C-I	umn sat-	-tv					
Check Col db=	umn Sate 45	cm	1				
Tb=	41.341	t	]				
Cb=	41.341	t	j				
Flange Loa			] !	Using D	oubler F	late	No Need
Rt=	156.19	t		t=	10	mm	
Tb=	41.341	t	safe	Rt	####	t	
	Stiffner		No Need				_
bs= ts=	14 10	cm mm	Local buckling Normal stresses	bs/ts= T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	
Web Load	cal Yieldir	ng	] !	Using F	oubler F	late	No Need
Rc=	135.86	t		t=	5	mm	
Cb=	41.341	t	Safe	Rc	####	t	
Using	Stiffner		No Need	Ì			
bs=	14	cm	Local buckling	bs/ts=	####		
ts= ds=	10 83	mm cm	Normal stresses Shear stresses	T= Q=	####	t	
	- 55			Ψ-			
Weh (	rippling		1				
Rn=	123.58	t	]	Using D	oubler F		No Need
Rc= Cb=	86.508 41.341	t	Cafa	t= Rc	5 ####	mm	
		t	Safe	nu	*****	t	
	Stiffner		No Need			•	
bs= ts=	14 6	cm mm	Local buckling Normal stresses	bs/ts= T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	
Shoor D			•				
	anel Zono	•					
Pu=	anel Zone	t					
					oubler F	Plate	No Need

Project				Element	Cant	M-B-2 + M-COL-25	
Steel P	roperties		St.37	St	raining a	ctions:-	1
Fy	2.4	t/cm2		Mu=	27	mt	
Fu	3.6	t/cm2		Tu= Qu=	0 14	t	
Beam Di	mension	s	Ì	Col	lumn Dir	nensions	1
bf=	40	cm		bf=	30	cm	
tf= dw=	2 58	cm		tf= dw=	3.5 83	cm cm	
tw=	0.8 60	cm		tw=	1.85 363.55	cm cm	
				AC			
Connection a=	10.6	ry:-		d=	Bolt 2	cm	
b=	4	cm		As=	2.4504 10.9	cm Catagony B	
tp= 2W=	12.8	cm		Grade Fy, bolt=	9	Category B t/cm2	
W= B=	6.4 30	cm		Fu, bolt=	10	t/cm2	
			ı				
Checks Fillet Wled	Between	n Beam an	d End Plate	1			
Sf= Aw hz=	8 115.2	mm cm2	Sw= Aw VI=	5 48	mm cm2		
Aw tot=	163.2	cm2	lx=	107364.352	cm4		
Check Stres Point 1	ses F1	0.77456	t/cm2	Safe	T		
Point 2	f2	0.60355	t/cm2		ı		
	q2 F2	0.29167	t/cm2 t/cm2	Safe	Ī		
Numbo	r of bolts		Eutondodu	vith full depth bo	-le	ı	
e=	4	cm	Extended	ntii ruii deptii bt	л	1 0 11 11	
p= allowable n/2=	6 9	cm bolts	+	+   6		50	<b>=</b>
P actual=	6.4	cm	+	Lt L		+	+
chosen n=	10	bolts	+ +	+ +		+	+ +
			+	+		+	+
			1				
			Extended w	ith full depth bo	ilts		Full depth bolts
y1	62.6	cm	у6	29	cm		·
y2 y3	54.6 48.2	cm cm	y7 y8	0	cm cm		
y4 y5	41.8 35.4	cm cm	y9 y10	0	cm cm		
Σγ2=	13065	cm2	yıu		CIII		
Tension on E	Bolts	1		Second be	olt	1	
First bolt			Ì	T2=	4.9806	t	
T1= Rtb=	6.4686 13.722	t	Safe	Prying Force= T2+P=	0.7395 5.7201	t	Safe
Shear on b	nits	1					
Qu/n=	1	t					
Rvb=	9.4248	t					
interaction eq=	0.2335	Safe					
In Case of C				d=	2	cm	
Qs= Ms=	9.3333	ton		Case of loa		eel work	
Ts=	0	ton		St. Grade		37&44 10.9	
Tb=	4.3124	ton		Load facto		1.5	
Qb=	0.6667	ton		Ps= T=	4.93 15.43	ton	
interaction eq=	0.4147	Safe			, _5.45		ı
Thickness of en	d plate	<u></u>	_				
M1= M2=	7.8383 12.084						
tb required=	1.9241	cm.t	Safe	]			
Check Col	umn Safe	ety					
db=	60	cm					
Tb= Cb=	46.552 46.552						
Flange Loa	ical Bend	ing	Ì	He	ing Doub	ler Plate	No Need
Rt=	156.19	t		t=	10	mm	
Tb=	46.552	t	safe	Rt	####	t	
Using bs=	Stiffner 14	enc	No Need Local buckling	bs/ts=	####		
ts=	10	cm mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	
	al W		1		lag D	lor Distr	N1. ** *
Web Load	al Yieldir 138.14	ng t	<u></u>	t=	ing Doub	ler Plate mm	No Need
Cb=	46.552	t	Safe	Rc	####	t	
Using			No Need				
bs= ts=	14 10	cm mm	Local buckling Normal stresses	bs/ts= T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	
Web C	rippling		İ				
Rn=	124.42					ler Plate	No Need
Rc= Cb=	87.096 46.552	t	Safe	t= Rc	5 ####	mm t	
				1			
bs=	Stiffner 14	cm	No Need Local buckling	bs/ts=	####		
ts= ds=	6 83	mm cm	Normal stresses Shear stresses	T= Q=	####	t t	
		· · · · ·	5 65565	Ψ-		•	
Shear Pa	anel Zone	9					
	160	t	l				
Pu= 0.4Py=	349.01	t			ing Doub	lor Diato	No Need

							•
Project				Element	Cant-	M-B-2 + M-COL-25	
Steel P	roperties		St.37	St	raining a	ctions:-	
Fy Fu	2.4 3.6	t/cm2 t/cm2		Mu= Tu=	27 0	mt t	
	5.0	ty cinz	ı	Qu=	14	t	
Beam D	mension	s	1	Co	lumn Din	nensions	
bf=	40	cm		bf=	30	cm	
tf= dw=	58	cm		tf= dw=	3.5 83	cm cm	
tw=	0.8	cm		tw=	1.85	cm	
d=	60	cm	l	Ac	363.55	cm	ļ
Connection				d=	Bolt 2		
a= b=	10.6 4	cm		As=	2.4504	cm cm	
tp= 2W=	2 12.8	cm		Grade Fy, bolt=	10.9 9	Category B t/cm2	
W=	6.4	cm		Fu, bolt=	10	t/cm2	
B=	30	cm					
Checks	D-4	. D	nd End Plate	Ī			
Sf=	8	mm	Sw=	5	mm	1	
Aw hz= Aw tot=	115.2 163.2	cm2 cm2	Aw VI= Ix=	48 107364.352	cm2 cm4		
Check Stres				107304.332	CIII4	l	
Point 1 Point 2	F1 f2	0.77456	t/cm2 t/cm2	Safe	l		
FOIII 2	q2	0.29167	t/cm2				
	F2	0.78707	t/cm2	Safe	l		
	r of bolts		Extended w	rith full depth bo	olt		
e= p=	4 6	cm	+	+ 20			
allowable n/2=	9	bolts	=			1	₹
P actual= chosen n=	6.4 10	cm bolts	+	+ -		+ +	+
			+ +	+ +		+	+
			+	+			⇒
	62.5			ith full depth bo		i	Full depth bolts
y1 y2	62.6 54.6	cm cm	y6 y7	29 0	cm cm		
у3	48.2	cm	у8	0	cm		
y4 y5	41.8 35.4	cm	y9 y10	0	cm		
Σγ2=	13065	cm2					
Tension on I	Bolts			Second b			i
First bolt T1=	6.4686	t	1	T2= Prying Force=	4.9806 0.7395	t t	
Rtb=	13.722	t	Safe	T2+P=	5.7201	t	Safe
Shear on b	olts	1					
Qu/n=	1	t					
Rvb=	9.4248		<u> </u>				
interaction eq=	0.2335	Safe					
In Case of C				d=	2	cm	
Qs= Ms=	9.3333	ton		Case of loa	dinary st	eel work	
Ts=	0	ton		St. Grade		37&44	
Tb=	4.3124	ton		Load facto		10.9	
Qb=	0.6667	ton		Ps=	4.93 15.43	ton	
interaction eq=	0.4147	Safe	]	-	15.45	ton	l
Thickness of er	d plate	1					
M1=	7.8383						
M2= tb required=	12.084		Safe	l			
			 1	ı			
Check Col	umn Safe 60	cm					
Tb=	46.552	t	]				
Cb=	46.552	t	]				
Flange Loa					ing Doub		No Need
Rt= Tb=	156.19 46.552	t	safe	t= Rt	10	mm t	
Using	Stiffner		No Need	<u></u>			. <u></u> -
bs=	14	cm	Local buckling	bs/ts=	####		
ts= ds=	10 83	mm cm	Normal stresses Shear stresses	T= Q=	####	t	
				•			
Web Loa	al Yieldir	ng	1	Us	ing Doub	ler Plate	No Need
Rc=	138.14	t	cat-	t=	5	mm	
Cb=	46.552	t	Safe	Rc	*****	t	
Using bs=	Stiffner 14	cm	No Need Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	
Web (			]				
	rippling		1		ing Doub		No Need
Rn=	124.42	t				mm	
		t t	Safe	t= Rc	####	t	
Rn= Rc= Cb=	124.42 87.096 46.552	t				t	
Rn= Rc= Cb= Using bs=	124.42 87.096 46.552 Stiffner	t t	No Need Local buckling	Rc bs/ts=	####		
Rn= Rc= Cb= Using bs= ts=	124.42 87.096 46.552 Stiffner 14 6	t t	No Need Local buckling Normal stresses	Rc bs/ts= T=	####	t	
Rn= Rc= Cb= Using bs=	124.42 87.096 46.552 Stiffner	t t	No Need Local buckling	Rc bs/ts=	####	t	
Rn= Rc= Cb= Using bs= ts=	124.42 87.096 46.552 Stiffner 14 6 83	t t	No Need Local buckling Normal stresses	Rc bs/ts= T=	####	t	
Rn= Rc= Cb= Using bs= ts= ds=	124.42 87.096 46.552 Stiffner 14 6 83	t t	No Need Local buckling Normal stresses	Rc bs/ts= T= Q=	####	t t	No Need

Project				Element	M-B-(3	7)/(25) + C3	
	roperties		St.37	Strain	ing actio		
Fy Fu	2.4 3.6	t/cm2 t/cm2		Mu= Tu=	15 0	mt t	
				Qu=	12	t	
Beam Di	imension	\$	I	Column	n Dimens	ions	1
bf=	19	cm		bf=	30	cm	
tf=	1.46	cm		tf=	2.5 54	cm	
dw= tw=	42.08 0.94	cm		dw= tw=	1.3	cm	
d=	43.54	cm		Ac	220.2	cm	
Connection	n Geomet	rv:-	I		Bolts		1
a=	4.7	cm		d=	2.7	cm	
b=	5.4 2	cm		As= Grade	4.4659 10.9	cm Category B	
tp= 2W=	16.4	cm		Fy, bolt=	9	t/cm2	
W=	8.2	cm		Fu, bolt=	10	t/cm2	
B=	21	cm	l				
Checks	1			ı			
Fillet Wled	Betweer 10	n Beam ar	d End Plate Sw=	8	mm		
Aw hz=	68.4	cm2	Aw VI=	55.7312	cm2		
Aw tot=	124.13	cm2	lx=	36411.00881	cm4		
Check Stres	sses F1	0.93804	t/cm2	Safe	1		
Point 2	f2	0.71748	t/cm2				
	q2	0.21532	t/cm2	Cofo	1		
	F2	v.avab1	t/cm2	Safe		_	
	r of bolts		Extended v	ith full depth b	olt		
e= p=	5.4 8.1	cm cm	+	+ ] e		-	
allowable n/2=	5	bolts		<u>⊢</u>     ρ			+ +
P actual= chosen n=	8.2	cm bolts	+ +	+   -'			+ +
cnosen n=		DOITS	+	+			+ + +
			+	+			+ +
			1 ±	<u>_</u>			
	40.24			ith full depth bo		i	Full depth bolts
y1 y2	48.24 37.44	cm cm	y6 y7	0	cm		
у3	29.24	cm	y8	0	cm		
y4 y5	21.04 12.84	cm cm	y9 y10	0	cm		
Σy2=	5191.4	cm2	y 10		CIII		
		1	=			i	
Tension on E First bolt	soits	l		Second b	4.2243	t	Ì
T1=	6.9693	t		Prying Force=	2.5961	t	
Rtb=	25.009	t	Safe	T2+P=	6.8204	t	Safe
Shear on b	olts		_				
Qu/n=	1	t					
Rvb=	17.177	t	l				
interaction eq=	0.081	Safe					
In Case of C	`ategory i	'R)	ı	d=	2.7	cm	ì
Qs=	8	ton		Ordina	ry steel v		
Ms= Ts=	10 0	ton		Case of loa St.	ding	1 37&44	
IS=	U	ton		Grade		10.9	
Tb=	4.6462	ton		Load facto		1.5	
Qb=	0.6667	ton		Ps= T=	9.25 28.91	ton	
interaction eq=	0.2328	Safe					
Thickness of en	nd plate	1					
M1=	12.202	cm.t					
M2=	10.61	cm.t		ī			
tb required=	1.7081	cm	Safe				
Check Col							
db= Tb=	43.54 35.646	cm t					
Cb=	35.646	t					
Ele	acal Pr '	ing	I	Heles *	Doubler F	late	No Need
Flange Loa Rt=	79.688		<u></u>	t=	10	mm	No Need
Tb=	35.646	t	safe	Rt	####	t	
Using	Stiffner		No Need				
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts= ds=	10 54	mm cm	Normal stresses Shear stresses	T= Q=	####	t	
us-	J**	un	Jilear stresses	ν-	*****		
v. · ·	ent with the		1		Janet -	llate	No No.
Web Load	72.499			Using E	Ooubler F	late mm	No Need
Cb=	35.646		Safe	Rc	####	t	
Heine	Stiffner		No Need		_	_	_
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	54	cm	Shear stresses	Q=	####	t	
Mat 6	`rinnlina						
Rn=	rippling 62.077	t		Using E	Ooubler F	late	No Need
Rc=	43.454	t		t=	5	mm	
Cb=	35.646	t	Safe	Rc	####	t	
Using	Stiffner		No Need	<u></u>			
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts= ds=	6 54	mm cm	Normal stresses Shear stresses	T= Q=	####	t	
	34	ÇI		~		•	
	2 ng! 7		i				
Pu=	anel Zone	t					
0.4Py=	211.39 93.881	t			Ooubler F	late mm	No Need
Rv=		t		t=			

Project				Element	MB14-N	MC22-AXIS 2	
Stor	el Properties		St.37	S t	raining ac	tions:-	
Fy	2.4	t/cm2	31.37	Mu=	22	mt	
Fu	3.6	t/cm2		Tu=	0	t	
•				Qu=	10	t	
			_		•		
Bear	n Dimensions	5		Co	lumn Dime	ensions	
bf=	19	cm		bf=	22	cm	
tf=	1.46	cm		tf=	1.6	cm	
dw=	43.54	cm		dw=	56.8	cm	
tw=	1.4	cm		tw=	1.2	cm	
d=	45	cm	l	Ac	138.56	cm	
Connoc	tion Geomet	n	T		Bolts	-	Ì
	6.8		ł	d=		cm	
a= b=	4.7	cm		As=	2.7 4.46593	cm	
tp=	3	cm		Grade	10.9	Category B	
2W=	21	cm		Fy, bolt=	9	t/cm2	
W=	10.5	cm		Fu, bolt=	10	t/cm2	
n=	12	bolts	1	,			<u> </u>
			_		e S	· e	
For	ces on bolts		I		- +	+   a	
Tb=	50.52825	t			P = =	= 28+€	
Cb=	50.52825	t	ļ				
	7				<u>+</u>	<del>+</del>	
Checks Filler M	Ilad Baturaan	. Doom on	d Fuel Diate	1	6,	,	
S f=	/led Between	mm	S w=	8	mm	1	
	inge & col Fla		S W=	1.477434	t/cm2	unsafe	İ
	veb & col Fla		Fw=	0.179433	t/cm2	Safe	
		-6-		0.275.00	·, ·	94.0	
Bolts			_				
	sion on Bolts		Į	Shea	r&Tension	on bolts	
Text,b=	12.632062	t		Qu/n=	0.83333		
Prying force=		t		Rvb=	17.1767	t	
Text,b + P=	16.436506	t		1			
Rtb=	25.009214	t	Safe	I			
interact	ion eq	0.43429	Safe	ı			
In Case of Ca	tegory (B):-	Ī					
Qs=	6.6666667	ton	Ī	d=	2.7	cm	
Ms=	14.666667	ton	]		dinary stee		
Ts=	0	ton		Case of	loading		
Tb=	32.592593	ton	Į	S		37&44	
		7			ade	10.9	
Shear&Tensi		0.2222		Ps=	9.25	ton	
interact	ion eq	0.34191	Safe	T=	28.91	ton	
Thickness of	end plate	Ī					
M1=	25.870213	cm.t	Ī				
M2=	33.500481	cm.t	1				
tb required=		cm	Safe	]			
				_			
Check	Column Safe	ty					
1. Flanc	e Loacal Bend	ding	ī	116	ing Double	r Plate	
Rt=	32.64	t t		t=	5	mm	
Tb=	50.52825	t	unsafe	Rt	56.2275	t	Ok
		-				-	
Us	ing Stiffner		<u></u>	L			
bs=	10.4	cm	Local buckling	bs/ts=	10.4		Safe
ts=	10	mm	Normal stresses	T=	42.432	t	Safe
ds=	56.8	cm	Shear stresses	Q=	139.046	t	Safe
	less-lift L		T		ina D - 1 :	u Diet-	<b>a</b>
	Loacal Yield		ł		ing Double		No Need
Rc=	58.71456	t	Soto.	t=	5	mm t	
Cb=	50.52825	t	Safe	Rc	####	t	
Us	ing Stiffner		No Need	1			
bs=	10.4	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	56.8	cm	Shear stresses	Q=	####	t	
	lah Culus II		T				
	eb Crippling		ł	11-	ing Davik!	r Diato	
Rn=	44.882031	t	1		ing Double		
Rc= Cb=	31.417421 50.52825	t	unsafe	t= Rc	5 54.6206	mm t	Ok
	30.32023		unsaie	· · · ·	34.0200		OK.
Us	ing Stiffner			<u> </u>			
bs=	10.4	cm	Local buckling	bs/ts=	17.3333		unsafe
ts=	6	mm	Normal stresses	T=	25.4592	t	Safe
ds=	56.8	cm	Shear stresses	Q=	83.4278	t	Safe
A. Sha	ear Panel Zon	e	T				
Pu=	20	t	t				
0.4Py=	133.0176	t	1	[ le	ing Double	er Plate	No Need
	88.128	t	1	t=	5	mm	
Rv=	00.140						

							-
Project				Element	P	XIS 3	
Ste	el Properties		St.37	St	raining act	tions:-	
Fy	2.4	t/cm2	51.57	Mu=	24	mt	
Fu	3.6	t/cm2		Tu=	0	t	
			_	Qu=	14	t	
Deer	m Dimensions		7		luman Dima		ì
bf=	19	cm	1	bf=	lumn Dime	cm	
tf=	1.46	cm		tf=	2.3	cm	
dw=	43.54	cm	1	dw=	55.4	cm	
tw=	1.4	cm		tw=	1.3	cm	
d=	45	cm	l	Ac	210.02	cm	
Connec	tion Geomet	rv:-	ī		Bolts		
a=	6.8	cm	İ	d=	2.7	cm	
b=	4.7	cm		As=	4.46593	cm	
tp=	2.7	cm		Grade	10.9	Category B	
2W=	21	cm		Fy, bolt=	9	t/cm2	
W= n=	10.5	cm bolts		Fu, bolt=	10	t/cm2	
		DOILS	ı		- 21		
Foi	rces on bolts		I			+ 13.4	
Tb=	55.121727	t			7-	28+6	
Cb=	55.121727	t	Į		+		
Checks	7				냭	₹	
	Vled Betweer	Beam an	d End Plate	<u> </u>		_	
S f=	10	mm	S w=	8	mm		Ī
	ange & col Fla		Fw=	1.611746	t/cm2	unsafe	
Beam v	web & col Fla	nge	Fw=	0.251206	t/cm2	Safe	
Bolts	7						
	sion on Bolts		Ī	Shea	r&Tension	on bolts	
Text,b=	13.780432	t	Ī	Qu/n=	1.16667	t	
Prying force=		t	l	Rvb=	17.1767	t	
Text,b + P= Rtb=	18.10385 25.009214	t	Safe	1			
interact		0.52863	Safe	1			
	•		•				
In Case of Ca			•				ì
Qs=	9.3333333	ton		d=	2.7	cm	
Ms= Ts=	16 0	ton			dinary stee loading	I WORK	
Tb=	35.55556	ton			t.	37&44	
		,	=		ade	10.9	
Shear&Tensi		0.20455	C-4-	Ps=	9.25	ton	
interact	lion eq	0.39155	Safe	T=	28.91	ton	
Thickness o	f end plate		_				
M1=	29.399242	cm.t					
M2=	35.368787	cm.t		1			
tb required=	2.5699836	cm	Safe	J			
Check	Column Safe	ty	Ī				
			-				
	e Loacal Ben				ing Double		No Need
Rt= Tb=	67.4475 55.121727	t	Safe	t= Rt	5 ####	mm t	
10-	33.121727		Jaie	IX.	******		
Us	sing Stiffner		No Need	L			
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts=	10 55.4	mm	Normal stresses	T=	####	t	
ds=	33.4	cm	Shear stresses	Q=	*****	t	
			_				
	Loacal Yield		Ī		ing Double		No Need
Rc=	73.68504	t	6.5	t=	5	mm	
Cb=	55.121727	t	Safe	Rc	####	t	
Us	sing Stiffner		No Need	1			
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	55.4	cm	Shear stresses	Q=	####	t	
3- V	Veb Crippling		Ī				
Rn=	59.726637	t	J	Us	ing Double	r Plate	
Rc=	41.808646	t	<u> </u>	t=	5	mm	
Cb=	55.121727	t	unsafe	Rc	69.4073	t	Ok
Us	sing Stiffner			1			
bs=	14.2	cm	Local buckling	bs/ts=	23.6667		unsafe
ts=	6	mm	Normal stresses	T=	34.7616	t	Safe
ds=	55.4	cm	Shear stresses	Q=	81.3715	t	Safe
A. Sh	ear Panel Zon	ie	ī				
Pu=	20	t	İ				
0.4Py=	201.6192	t	1	Us	ing Double	r Plate	No Need
Rv=	95.472	t	<b></b>	t=	5	mm	
Cb or Tb=	53.333333	t	Safe	Rv	####	t	

Project				Element	MB38-I	MC24-AXIS4	
C+-	al Duamantias		C+ 27	1 6		tions.	1
Fy	el Properties 2.4	t/cm2	St.37	Mu=	raining ac	mt	
Fu	3.6	t/cm2		Tu=	0	t	
i u	3.0	t/till2	1	Qu=	21	t	
				- 44			
Bea	m Dimension	S	Ī	Co	lumn Dime	ensions	
bf=	19	cm	Ĭ	bf=	30	cm	
tf=	1.46	cm		tf=	2.3	cm	
dw=	43.54	cm		dw=	55.4	cm	
tw=	1.4	cm		tw=	1.3	cm	
d=	45	cm	l	Ac	210.02	cm	
Conno	ction Geomet	n.	7	r	Bolts	1	I
a=	6.8	cm	ł	d=	2.7	cm	
b=	4.7	cm		As=	4.46593	cm	
tp=	2.7	cm		Grade	10.9	Category B	
2W=	21	cm		Fy, bolt=	9	t/cm2	
W=	10.5	cm		Fu, bolt=	10	t/cm2	
n=	12	bolts	1			2W	
			-			e S	_e_
	rces on bolts					+	+   a
Tb=	59.715204	t					=  _2b+t <sub>0</sub>
Cb=	59.715204	t	Į.				
Chacks	7					_	+
Checks Fillet V	l Vled Betweer	Ream and	d End Plate	1		اجا	ᆗ
S f=	10	mm	S w=	8	mm	$b_b$	-
	ange & col Fla		Fw=	1.746059		unsafe	I
	web & col Fla		Fw=	0.376809		Safe	
						-	•
Bolts			_				
	sion on Bolts		I		r&Tensior		
Text,b=	14.928801	t		Qu/n=	1.75	t	
	4.6837028	t		Rvb=	17.1767	t	
Text,b + P=	19.612504	t		1			
Rtb=	25.009214	0.62537	Safe Safe				
interac	tion eq	0.02337	Jale	1			
In Case of Ca	ategory (B):-	Ī					
Qs=	14	ton	Ī	d=	2.7	cm	
Ms=	17.333333	ton		Or	dinary stee	el work	
Ts=	0	ton			loading	_	
Tb=	38.518519	ton	ļ	S		37&44	
Ch 0 T	b . la .	r			ade	10.9	
Shear&Tens interac		0.45922	Safe	Ps= T=	9.25 28.91	ton ton	
interac	tion eq	0.43322	Jale	1-	20.31	ton	
Thickness o	f end plate	Ī					
M1=	31.849179	cm.t					
M2=	38.316186	cm.t		_			
tb required=	2.6749237	cm	Safe				
Ch I	. C-l C-f-		7				
Cneck	Column Safe	ty	1				
1- Flans	ge Loacal Ben	ding	7	Us	ing Double	er Plate	No Need
Rt=	67.4475	t	1	t=	5	mm	
Tb=	59.715204	t	Safe	Rt	####	t	
							-
	sing Stiffner		No Need			,	
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	55.4	cm	Shear stresses	Q=	####	t	<u> </u>
2- Wel	b Loacal Yield	ing	Ī	Us	ing Double	er Plate	No Need
Rc=	73.68504	t	<u></u>	t=	5	mm	
Cb=	59.715204	t	Safe	Rc	####	t	
							<u> </u>
	sing Stiffner		No Need			1	
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts= ds=	10 55.4	mm	Normal stresses	T=	####	t	
us-	33.4	cm	Shear stresses	Q=	*****	·	
3- V	Web Crippling		Ī				
Rn=	59.726637	t	J	Us	ing Double	er Plate	
Rc=	41.808646	t		t=	5	mm	
Cb=	59.715204	t	unsafe	Rc	69.4073	t	Ok
				1			
	sing Stiffner		Lead built	h-/4	22 000-		
bs=	14.2	cm	Local buckling	bs/ts=	23.6667		unsafe
ts= ds=	55.4	mm cm	Normal stresses Shear stresses	T= Q=	34.7616 81.3715	t	Safe Safe
us-	33.4	CIII	oncar scresses	, <b>ų</b> -	01.5/13		Jaie
4- Sh	ear Panel Zon	ie	Ī				
Pu=	20	t	J				
0.4Py=	201.6192	t	]	Us	ing Double	er Plate	No Need
Rv=	95.472	t		t=	5	mm	

				Element	MB22-N	AC29-AXIS 7	
-	al Duc · ·		0.05				Ì
	el Properties		St.37		raining ac		1
Fy	2.4	t/cm2		Mu=	14	mt	1
Fu	3.6	t/cm2	l	Tu=	6	t	Ī
				Qu=	6	t	
Rear	n Dimensions	<u> </u>	<b>i</b> 1	Co	lumn Dime	ensions	
bf=	19	cm	1	bf=	22	cm	1
tf=	1.46	cm		tf=	1.6	cm	Ī
dw=	42.08	cm		dw=	56.8	cm	Ī
tw=	0.94	cm		tw=	1.2	cm	Ī
d=	43.54	cm		Ac	138.56	cm	Ī
-						-	
Connec	tion Geomet	ry:-			<u>Bolts</u>		Ī
a=	5	cm		d=	2	cm	Ī
b=	3.3	cm		As=	2.45044	cm	Ī
tp=	2	cm		Grade	10.9	Category B	Ī
2W=	21	cm		Fy, bolt=	9	t/cm2	1
W=	10.5	cm		Fu, bolt=	10	t/cm2	Ī
n=	12	bolts			2W	_	
			•		e_S	e	
	ces on bolts			,	, =	26+t	
Tb=	36.269962				41-		
Cb=	30.269962	t	ļ				
_	-				ئىل ئ		
Checks	4-45 :		drade' :	ı	66	,	
	Vled Between					Ì	
S f=	10	mm	S w=	5	mm		1
	ange & col Fla		Fw=	1.060525	t/cm2	unsafe	
Beam v	veb & col Flai	ige	Fw=	0.178232	t/cm2	Safe	
Bolts	7						
	sion on Bolts		<b>j</b>	Chen	r&Tension	on holts	1
Text,b=	9.0674905		}	Qu/n=	0.5	on poits	1
Prying force=				Rvb=	9.42478	t	Ī
Text,b + P=	11.649068	t		KVD=	9.42478	ι	
Rtb=	13.722477	t	Safe	Ī			
interact		0.72345	Safe				
interact	lion eq	0.72343	Jaic				
In Case of Ca	tegory (B):-	ſ					
Qs=	4	ton		d=	2	cm	Ī
Ms=	9.3333333	ton			dinary stee		Ī
Ts=	4	ton			loading	1	Ī
Tb=	23.436227	ton		S	t.	37&44	1
	•		•	Gra	ade	10.9	1
Shear&Tensi	on on bolts	ĺ		Ps=	4.93	ton	Ī
interact	ion eq	0.44733	Safe	T=	15.43	ton	Ī
		-					
Thickness of	f end plate		•				
M1=	12.907885	cm.t					
M2=	17.014833	cm.t					
tb required=	1.7825188	cm	Safe				
	01 01						
Check	Column Safe	ty					
4 51	- II B		1		in a Daniela	- DI-+-	
	e Loacal Bend	aing			ing Double		
D4							
Rt=	32.64	t	uncafo	t=	5	mm	Ok
Rt= Tb=	32.64 36.269962	t	unsafe	t= Rt	56.2275	t	Ok
Tb=	36.269962		unsafe				Ok
Tb=	36.269962 sing Stiffner	t		Rt	56.2275		
Tb= Us bs=	36.269962 sing Stiffner 10.4	t cm	Local buckling	Rt bs/ts=	10.4	t	Safe
Tb= Us bs= ts=	36.269962 sing Stiffner 10.4 10	cm mm	Local buckling Normal stresses	Rt bs/ts= T=	10.4 42.432	t	Safe Safe
Tb= Us bs=	36.269962 sing Stiffner 10.4	t cm	Local buckling	Rt bs/ts=	10.4	t	Safe
Tb= Us bs= ts=	36.269962 sing Stiffner 10.4 10	cm mm	Local buckling Normal stresses	Rt bs/ts= T=	10.4 42.432	t	Safe Safe
Tb= Us bs= ts= ds=	36.269962 sing Stiffner 10.4 10	cm mm cm	Local buckling Normal stresses	Rt bs/ts= T= Q=	10.4 42.432	t t	Safe Safe
Tb= Us bs= ts= ds=	36.269962 sing Stiffner 10.4 10 56.8	cm mm cm	Local buckling Normal stresses	Rt bs/ts= T= Q=	10.4 42.432 139.046	t t t	Safe Safe Safe
Tb=  Us  bs=  ts=  ds=	36.269962 sing Stiffner 10.4 10 56.8	cm mm cm	Local buckling Normal stresses	Bs/ts= T= Q= Us	10.4 42.432 139.046	t t	Safe Safe Safe
Tb= Us bs= ts= ds=  2- Web Rc=	36.269962 sing Stiffner 10.4 10 56.8 b Loacal Yield 53.24256	cm mm cm	Local buckling Normal stresses Shear stresses	Bs/ts= T= Q= Us t=	10.4 42.432 139.046 ing Double	t t t t r Plate mm	Safe Safe Safe
Us bs= ts= ds=  2- Web Rc= Cb=	36.269962 sing Stiffner 10.4 10 56.8 b Loacal Yield 53.24256	cm mm cm	Local buckling Normal stresses Shear stresses	Bs/ts= T= Q= Us t=	10.4 42.432 139.046 ing Double	t t t t r Plate mm	Safe Safe Safe
Us bs= ts= ds=  2- Web Rc= Cb=	36.269962 sing Stiffner 10.4 10 56.8 D Loacal Yield 53.24256 30.269962	cm mm cm	Local buckling Normal stresses Shear stresses	Bs/ts= T= Q= Us t=	10.4 42.432 139.046 ing Double	t t t t r Plate mm	Safe Safe Safe
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=	36.269962 sing Stiffner 10.4 10 56.8 b Loacal Yield 53.24256 30.269962 sing Stiffner	cm mm cm	Local buckling Normal stresses Shear stresses Safe No Need	Bs/ts= T= Q= Us t= Rc	10.4 42.432 139.046 ing Double 5 ####	t t t t r Plate mm	Safe Safe Safe
Tb=  Us bs= ts= ds=  2-Web Rc= Cb=  Us bs=	36.269962 sing Stiffner 10.4 10 56.8 Loacal Yield 53.24256 30.269962 sing Stiffner 10.4	cm mm cm	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling	Ds/ts=   T=   Q=   Us   t=   Rc   Ds/ts=	10.4 42.432 139.046 ing Double 5 ####	t t t t	Safe Safe Safe
Tb=  Us bs= ts= ds=  2- Wet Rc= Cb=  Us bs= ts=	36.269962 sing Stiffner 10.4 10 56.8 Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10	cm mm cm	Local buckling Normal stresses Shear stresses Safe No Need Local buckling Normal stresses	Bs/ts=   T=   Us   t=   Rc   Bs/ts=   T=   T=   T=   T=   T=   T=   T=	56.2275  10.4  42.432 139.046  ing Double  5 ####  ####	t t t t t r Plate mm t	Safe Safe Safe
Tb=  Us bs= ts= ds=  2-Web Rc= Cb=  Us bs= ts= ds=	36.269962 sing Stiffner 10.4 10 56.8 D Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8	cm mm cm t	Local buckling Normal stresses Shear stresses Safe No Need Local buckling Normal stresses	Bs/ts=   T=   Us   t=   Rc   Bs/ts=   T=   T=   T=   T=   T=   T=   T=	56.2275  10.4  42.432 139.046  ing Double  5 ####  ####	t t t t t r Plate mm t	Safe Safe Safe
Tb=  Us bs= ts= ds=  2-Web Rc= Cb=  Us bs= ts= ds=  3-V	36.269962 sing Stiffner 10.4 10 56.8 c Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8	cm mm cm t	Local buckling Normal stresses Shear stresses Safe No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Rc   Bs/ts=   T=   Q=   Q=   T=   Q=   T=   Q=   T=   T	10.4 42.432 139.046 ing Double 5 #### #### ####	t t t t tr Plate mm t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Wet Rc= Cb=  Us bs= ts= ds=  4	36.269962 sing Stiffner 10.4 10 56.8 Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031	cm mm cm t	Local buckling Normal stresses Shear stresses Safe No Need Local buckling Normal stresses	Rt	56.2275  10.4  42.432 139.046  5 ####  #### #### ####	t t t t t t t t t t t t t t t t t t t	Safe Safe Safe
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  3- V Rn= Rc=	36.269962 sing Stiffner 10.4 10 56.8 b Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421	cm mm cm t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses	Bs/ts= T= Q=	10.4 42.432 139.046 5 #### #### #### #### ####	t t t tr Plate mm t t t t trer Plate	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Wet Rc= Cb=  Us bs= ts= ds=  4	36.269962 sing Stiffner 10.4 10 56.8 Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031	cm mm cm t	Local buckling Normal stresses Shear stresses Safe No Need Local buckling Normal stresses	Rt	56.2275  10.4  42.432 139.046  5 ####  #### #### ####	t t t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2-Web Rc= Cb=  Us bs= ts= ds=  3-W Rn= Rc= Cb=	36.269962 sing Stiffner 10.4 10 56.8 c Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421 30.269962	cm mm cm t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses	Bs/ts= T= Q=	10.4 42.432 139.046 5 #### #### #### #### ####	t t t tr Plate mm t t t t trer Plate	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Wet Rc= Cb=  Us bs= ts= ds=  3- V Rn= Rc= Cb=  Us	36.269962  sing Stiffner  10.4  10  56.8  Loacal Yield  53.24256  30.269962  sing Stiffner  10.4  10  56.8  Veb Crippling  44.882031  31.417421  30.269962  sing Stiffner	cm mm cm t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need	St	10.4 42.432 139.046 5 #### #### #### #### #### ####	t t t tr Plate mm t t t t trer Plate	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  3- W Rn= Rc= Cb=  Us Some	36.269962 sing Stiffner 10.4 10 56.8 D Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4	cm mm cm t t t t t t cm	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling	Bs/ts=   T=   Q=   Us   t=   Rc   Us   t=   Q=   Us   t=   Rc   Us   t=   Rc   E   Rc   E   E   C   E   E   E   E   E   E   E	10.4 42.432 139.046 5 #### #### #### #### ####	t t t tr Plate mm t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Wet Rc= Cb=  Us bs= ts= ds=  Us bs= ts= ds=  Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us	36.269962 sing Stiffner 10.4 10 56.8 c Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6	cm mm cm t t t t t t t t t t t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   T=   Rc   Us   T=   Rc   E=   Rc   E=   T=   T=   T=   T=   T=   T=   T=	10.4 42.432 139.046 5 #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  3- W Rn= Rc= Cb=  Us Some	36.269962 sing Stiffner 10.4 10 56.8 D Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4	cm mm cm t t t t t t cm	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling	Bs/ts=   T=   Q=   Us   t=   Rc   Us   t=   Q=   Us   t=   Rc   Us   t=   Rc   E   Rc   E   E   C   E   E   E   E   E   E   E	10.4 42.432 139.046 5 #### #### #### #### ####	t t t tr Plate mm t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Wet Rc= Cb=  Us bs= ts= ds=  Us bs= ts= ds=  Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us Solution Us	36.269962 sing Stiffner 10.4 10 56.8 c Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6	cm mm cm t t t t t t t t t t t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   T=   Rc   Us   T=   Rc   E=   Rc   E=   T=   T=   T=   T=   T=   T=   T=	10.4 42.432 139.046 5 #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  3- V Rn= Rc= Cb=  Us bs= ts= ds=	36.269962 sing Stiffner 10.4 10 56.8 b Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6 56.8	cm mm cm cm t t t t t t t t t t t cm mm cm	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   T=   Rc   Us   T=   Rc   E=   Rc   E=   T=   T=   T=   T=   T=   T=   T=	10.4 42.432 139.046 5 #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  Us bs= ts= ds=  Us A-She	36.269962 sing Stiffner 10.4 10 56.8 c Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6 56.8	cm mm cm t t t t t t t t t t t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   T=   Rc   Us   T=   Rc   E=   Rc   E=   T=   T=   T=   T=   T=   T=   T=	10.4 42.432 139.046 5 #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  3- V Rn= Rc= Cb=  Us bs= ts= ds=  4- Sht	36.269962 sing Stiffner 10.4 10 56.8 c Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6 56.8	cm mm cm cm cm cm cm cm cm cm cm cm cm c	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   T=   Rc   E   Rc   E   C   T=   Q=   C   C   C   C   C   C   C   C   C	10.4 42.432 139.046 5 #### #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2-Wet Rc= Cb=  Us bs= ts= ds=  3-V Rn= Rc= Cb=  Us bs= ts= ds=  4-She Pu= 0.4Py=	36.269962 sing Stiffner 10.4 10 56.8  Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8  Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6 56.8	cm mm cm cm t t t t t t t t t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Rc   Us   T=   Rc   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   Us   T=   Q=   Us   Us   Us   Us   Us   Us   Us   U	10.4 42.432 139.046  ing Double 5 #### #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2- Web Rc= Cb=  Us bs= ts= ds=  3- V Rn= Rc= Cb=  Us bs= ts= ds=  4- Sht Pu= 0.4Py= Rv=	36.269962 sing Stiffner 10.4 10 56.8 b Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8 Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6 56.8	cm mm cm cm t t t t t t t t t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses	Bs/ts= T= Q=   Us t= Rc	10.4 42.432 139.046 5 #### #### #### #### #### #### ####	t t t t r Plate mm t t t t t t r Plate mm t t r Plate mm t	Safe Safe Safe No Need
Tb=  Us bs= ts= ds=  2-Wet Rc= Cb=  Us bs= ts= ds=  3-V Rn= Rc= Cb=  Us bs= ts= ds=  4-She Pu= 0.4Py=	36.269962 sing Stiffner 10.4 10 56.8  Loacal Yield 53.24256 30.269962 sing Stiffner 10.4 10 56.8  Veb Crippling 44.882031 31.417421 30.269962 sing Stiffner 10.4 6 56.8	cm mm cm cm t t t t t t t t t t t t t t	Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses Shear stresses  Safe  No Need Local buckling Normal stresses	Bs/ts=   T=   Q=   Us   T=   Q=   Us   T=   Rc   Us   T=   Rc   T=   Q=   Us   T=   Q=   Us   T=   Q=   Us   Us   T=   Q=   Us   Us   Us   Us   Us   Us   Us   U	10.4 42.432 139.046  ing Double 5 #### #### #### #### #### #### ####	t t t t tr Plate mm t t t t t t t t t t t t t t t t t	Safe Safe Safe No Need

C1 - Splice & = 18 ton, IPE 600 bf = 220 mm, tf = 19 mm, tw=12m, h-2c= 514m h = 600mm. For S1: by = 22cm, t1 = 0.5 \* 1.9 = 0.95.cm  $for S_2$ :  $b_2 = 0.5 [22 - 1.2 - 2 + 0.8 - 1 cm] = 9.1 cm$  $22 * 1.9 = 22 * 1 + 2 * 9.5 * t_2$ t2 = 1.05 cm ~ 1.1 cm  $for S_3$ ,  $60 + 1.2 = 2 + b_3 \cdot t_3$ : b3 = 60-2+1.9-1cm = [55.2 cm] t3 = 0.65 = 0.7cm Cret required No. of Bolds 2- [for flange] 2-C = 22 + 1.9 + 0.8 + 3.6 = 120.40 ton. Rs, Rd = 0.9 x 1 + 2 \* 0.4 + 0.7 \* 10 \* \$\frac{17}{4}(2.2)^2 = 19.16 ton Rs, bd = 1.34 \* 5-2 \* 2.2 + 1.9 = 29.12 ton. n, = 120-4 = 6-28 = 4 rows + 2 Bolts per Check pet section fracture (tens. fl) 2 $f_t = \frac{120.4}{22+1.9} = 2.88 t (cm^2 < 0.84 * [22*1.9-3(2.4)*1.9]$ = 2.938 t/cm2 for WEB:  $n_2 = \frac{66.2}{11+2.2} = 6.27 \rightarrow Pret = \frac{65.2}{8}$ Then Put = 4 x 2.2 = 8.8 cm; C= 4.4 cm.  $\gamma = (60 - 55.2) + 4.4 + \frac{8.8}{2} = 13.6 cm.$  $f_1 = \frac{0.8 + 3.6}{\sqrt{60} + 1.97} + \frac{60}{2} = 2.71$  t lem<sup>2</sup>

$$f_2 = \frac{0.8 \pm 3.6}{\left[\frac{60}{2} + 1.9\right]} * \left[\frac{60}{2} - 13.6\right] = 1.48 \text{ Hcm}^2.$$

$$f_{H2} = 2.71 + 1.48 + 13.6 (1.2) = 34.2 \text{ ton}.$$

$$fH2 = \frac{2.71 + 1.48}{2} + 13.6 + 1.2 = 34.2 ton.$$

$$H = \frac{34.2}{2 \text{ pars}} = .17.1 \text{ ton}.7$$

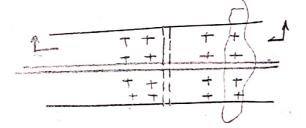
$$H = \frac{34.2}{2 \text{ pws}} = .17.1 \text{ ton-}$$

$$V = \frac{18}{2 + 8} = 1.125$$

$$R = 17.14 \text{ ton-}$$

\* Bearing Pesistance: tw = tmin = 1.2 cm.





 $C_2 - splice$ , Q = 13 ton , HEA 600.  $b_f = 80 cm , t_f = 25 mm , tou = 13 mm , h-26 = 890 mm$ 

For 
$$S_8$$
:  $59 * 1.3 = 2 b_3 . t_3$ 

$$D_3 = 53 cm \longrightarrow [t_3 = 0.7 cm]$$

for tens. fl: T = 30 x 2.5 x 0.8 x 3.6 = 216 ton

Chelk Net fracture (tens. fL): row.

$$ft = \frac{216}{30 \, \# 2.5} = 2.8 \quad \text{(0.8)}$$

For cveB:- 
$$h_2 = \frac{53}{4 + 2} = 6-6$$

take 8

Paet = 8 cm, eact = 4 cm.

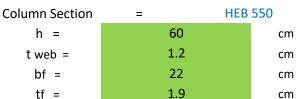
(2 rows \* 8 Bolts).

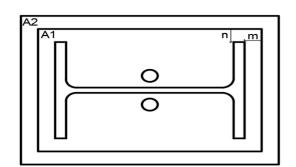
C1 Steel Grade St.37

#### **Description:**

#### 1)- INPUT DATA :-

Nu =	43	t
Qu =	7.5	t
Fcu =	250	Kg/cm <sup>2</sup>
S.weld =	1	cm





Fy

Fu

t/cm<sup>2</sup>

t/cm<sup>2</sup>

2.4

3.7

#### 2)- End Plate dimensions:

m =	2.5	cm	n =	2.5	cm			
Lp =	65	cm	bp =	27	cm			
Ap (A1) =	1755	cm <sup>2</sup>	Ac (A2 ) =	2775	cm <sup>2</sup>	$\sqrt{A2/A1}$	≤ 2	ok
tp =	2	cm	tp,Choo	sen =	2	cm		

#### 3)- Check Concrete Capacity:

Nu	≤	0.6 x	0.67 x Fcu x A	\1 x √ ( A2	/A1)
43	ton	≤	221.79	ton	Safe

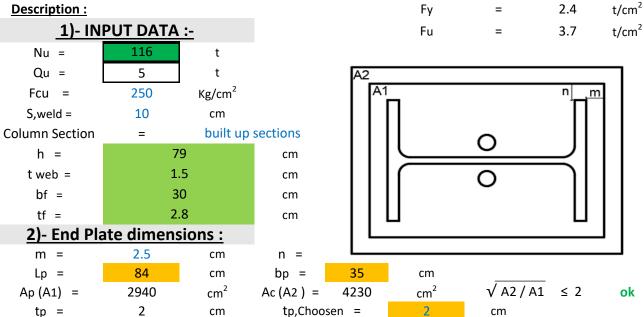
#### 4)- Check welding Safety:

ф.Rnw =	1.1*0.7 x 0.4 x Fu =	1.140	t/cm <sup>2</sup>
φ.Rnw,eff =	1.1*0.77 x 0.4 x Fu =	1.254	t/cm <sup>2</sup>

A w,flange =	78	cm <sup>2</sup>	Aw,total =	182.8	cm <sup>2</sup>
A w,web =	104.8	cm <sup>2</sup>			
Ruw,N =	0.24	t/cm2	Safe		
Q <sub>D</sub> =	0.32	t/cm <sup>2</sup>	Safe		
Ruw,eff =	0.97	t/cm2	Safe		

n.of anchors =	4		M	20	Grade	10.9	
$\alpha = (0.8 \text{ x e})/d$	2.4		As =	2.45	cm <sup>2</sup>		
Length=40*3=120	cm		Fub =	10	t/cm <sup>2</sup>		
Ru,t =	10.75	t					
Ru,v =	1.88	t					
φt.Rnt =	0.8 x 0	).66 x Fub x As =	12.94	t	Safe		
φbr.Rnbr =	0.7 x d	$x tmin x \alpha x Fu =$	24.86	t	7.5	+	Safe
φv.Rv =	.85*0.6 x	0.6 x Fub x As x n =	7.50	t	7.5	ι	Jaie

Steel Grade Fy



#### 3)- Check Concrete Capacity:

Nu	≤	$0.6 \times 0.67 \times \text{Fcu} \times \text{A1} \times \sqrt{\text{(A2/A1)}}$				
116	ton	≤	354.41	ton	Safe	

#### 4)- Check welding Safety:

ф.Rnw =	1.1*0.7 x 0.4 x Fu =	1.140	t/cm <sup>2</sup>
φ.Rnw,eff =	1.1*0.77 x 0.4 x Fu =	1.254	t/cm <sup>2</sup>

A w,flange =	1058	cm <sup>2</sup>	Aw,total =	2414	cm <sup>2</sup>
A w,web =	1356	cm <sup>2</sup>			
Ruw,N =	0.05	t/cm2	Safe		
Q <sub>D</sub> =	0.06	t/cm <sup>2</sup>	Safe		
Ruw,eff =	0.19	t/cm2	Safe		

n.of anchors =	4		M	30	Grade	10.9	
$\alpha = (0.8 \times e)/d$	2.4		As =	5.61	cm <sup>2</sup>		
Length=40*3=120	cm		Fub =	10	t/cm <sup>2</sup>		
Ru,t =	29	t					
Ru,v =	1.25	t					
φt.Rnt =	0.8 x	0.66 x Fub x As =	29.62	t	Safe		
φbr.Rnbr =	0.7 x d	d x tmin x α x Fu =	37.3	t	17 17		Cofo
фv.Rv =	.85*0.6 x	: 0.6 x Fub x As x n =	17.17	t	17.17	ι	Safe

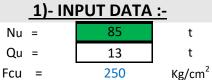
cm

Steel Grade

cm

cm

#### **Description:**

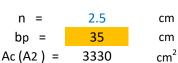


Column Section	= built up s	sections
h =	59	cm
t web =	1.3	cm

10

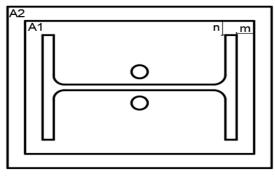
## 2)- End Plate dimensions:

$$m = 2.5$$
 cm  
 $Lp = 64$  cm  
 $Ap (A1) = 2240$  cm<sup>2</sup>



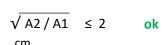
2054

cm<sup>2</sup>



Fy

Fu



2.4

3.7

t/cm<sup>2</sup>

t/cm<sup>2</sup>

#### 3)- Check Concrete Capacity:

Nu	≤	$0.6 \times 0.67 \times \text{Fcu} \times \text{A1} \times \sqrt{\text{(A2/A1)}}$					
85	ton	≤	274.48	ton	Safe		

#### 4)- Check welding Safety:

φ.Rnw =	1.1*0.7 x 0.4 x Fu =	1.140	t/cm <sup>2</sup>
φ.Rnw,eff =	1.1*0.77 x 0.4 x Fu =	1.254	t/cm <sup>2</sup>

A w,flange =	1074	cm <sup>2</sup>	Aw,total =
A w,web =	980	cm <sup>2</sup>	
Ruw,N =	0.04	t/cm2	Safe
Q <sub>D</sub> =	0.07	t/cm <sup>2</sup>	Safe
Ruw,eff =	0.19	t/cm2	Safe

n.of anchors =	4		M	27	Grade	10.9	
$\alpha = (0.8 \times e)/d$	2.4		As =	4.59	cm <sup>2</sup>		
Length=40*3=120	cm		Fub =	10	t/cm <sup>2</sup>		
Ru,t =	21.25	t					
Ru,v =	3.25	t					
φt.Rnt =	0.8 x 0	0.66 x Fub x As =	24.24	t	Safe		
φbr.Rnbr =	$0.7 \times d$	$I \times tmin \times \alpha \times Fu =$	33.57	t	14.05	+	Safe
φv.Rv =	.85*0.6 x	0.6 x Fub x As x n =	14.05	t	14.05	ι	Sale

Steel Grade **Description:** Fy 2.4 t/cm<sup>2</sup> 1)- INPUT DATA :t/cm<sup>2</sup> Fu 3.7 Nu = 65 Qu = 5 t Kg/cm<sup>2</sup> Fcu = 250 S,weld = 10 cm built up sections Column Section h = 49 cm t web = 1.2 cm bf = 30 cm tf = cm 2)- End Plate dimensions: m = 2.5 n = 2.5 cm Lp = 54 35 cm bp = cm  $\sqrt{A2/A1} \leq 2$  $cm^2$  $cm^2$ Ap(A1) =1890 Ac(A2) =2880 ok 2 tp,Choosen = tp = 3)- Check Concrete Capacity:  $0.6 \times 0.67 \times \text{Fcu} \times \text{A1} \times \sqrt{(\text{A2}/\text{A1})}$ 

234.47

ton

Safe

## 4)- Check welding Safety:

ton

65

φ.Rnw =	1.1*0.7 x 0.4 x Fu =	1.140	t/cm <sup>2</sup>
ф.Rnw,eff =	1.1*0.77 x 0.4 x Fu =	1.254	t/cm <sup>2</sup>

A w,flange =	1084	cm <sup>2</sup>	Aw,total =	1880	cm <sup>2</sup>
A w,web =	796	cm <sup>2</sup>			
Ruw,N =	0.03	t/cm2	Safe		
Q <sub>D</sub> =	0.06	t/cm <sup>2</sup>	Safe		
Ruw.eff =	0.16	t/cm2	Safe		

≤

n.of anchors =	4		M	27	Grade	10.9	
$\alpha = (0.8 \text{ x e})/d$	2.4		As =	4.59	cm <sup>2</sup>		
Length=40*3=120	cm		Fub =	10	t/cm <sup>2</sup>		
Ru,t =	16.25	t					
Ru,v =	1.25	t					
φt.Rnt =	0.8 x 0	0.66 x Fub x As =	24.24	t	Safe		
φbr.Rnbr =	0.7 x c	l x tmin x α x Fu =	33.57	t	14.05		Cofo
φν.Rv =	.85*0.6 x	0.6 x Fub x As x n =	14.05	t	14.05	ι	Safe