Faculty of Engineering
Structural Engineering Department

Ain Shams University

Steel Design note

Structural Engineering Department-CHEP

Steel Structures Project

Proj N08- Industrial Planet

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Prepared by

ST: Mohamed Hassan Ali Bezawy

Mail:1902242@eng.asu.edu.eg

Senior - Level (2)

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Contents

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

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

$W_{L.L\ Frame}=60-66.67*tan(2.86)=56.67\ kg/m2=0.05667ton/m^2$

Live load on frame (S=6m) = 0.05667 $*$ 6 = 0.34 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 $_{\text{Sec Beams}}$ =30 Kg/m

O.W Walls = 150 Kg/m

Then

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

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

Wind Load

$W_{wind}=Ce*q*k\left(kg/m^2\right)\pm Ci*q*k\rightarrow\left(q=68\ kg/m^2\right)$,

where k (0 $\rightarrow$ 5) = 1.15 , k (5 $\rightarrow$ 15) =1.4

Wind load (+X)

$W_1 = 0.37$ t/m

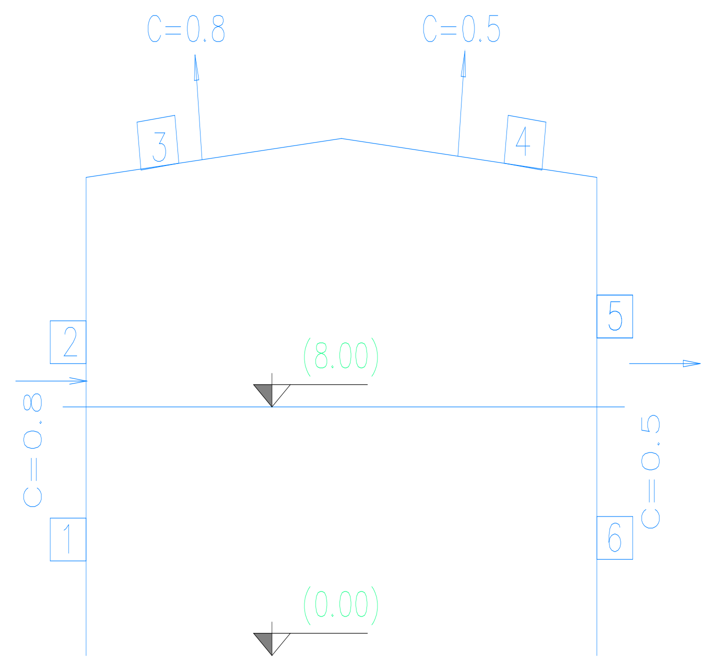
$W_2 = 0.29$ t/m

$W_3 = 0.33$ t/m

$W_4 = 0.204$ t/m

$W_5 = 0.46$ t/m

$W_6 = 0.235$ t/m



Wind load(-X)

$W_1 = 0.38$ t/m

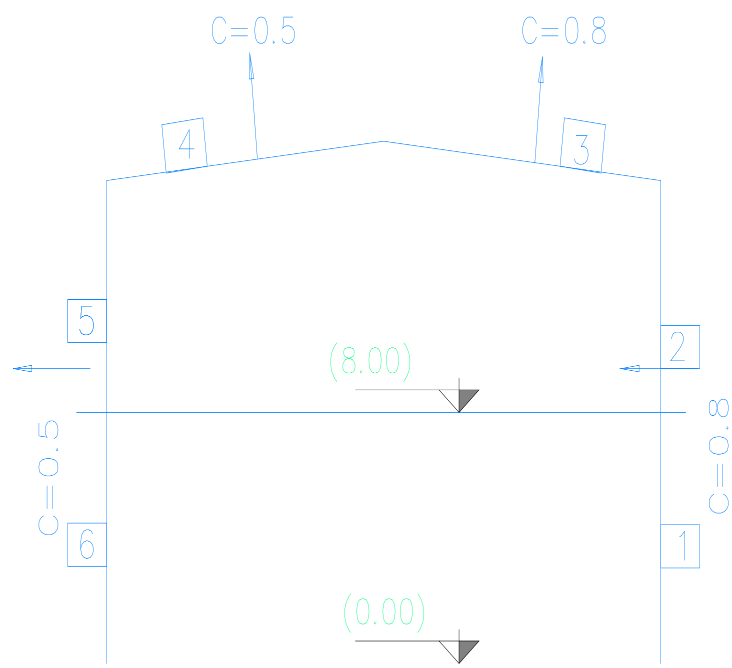
$W_2 = 0.63$ t/m

$w_3 = 0.33$ t/m

$w_4 = 0.204$ t/m

$w_5 = 0.114$ t/m

$w_6 = 0.235$ t/m



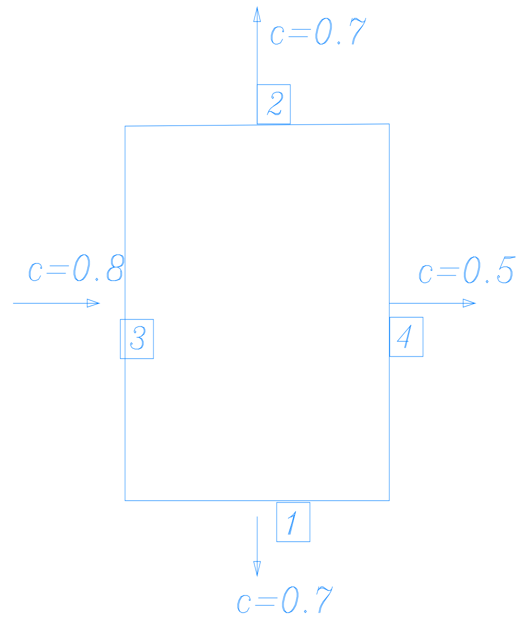
Wind Load [+y]

$W_1 = 0.33$ t/m , $w'_1 = 0.57$ t/m

$W_2 = 0.33$ t/m , $w'_2 = 0.47$ t/m

$W_3 = 0.38$ t/m , $w'_3 = 0.29$ t/m

$W_4 = 0.235$ t/m , $w'_4 = 0.46$ t/m



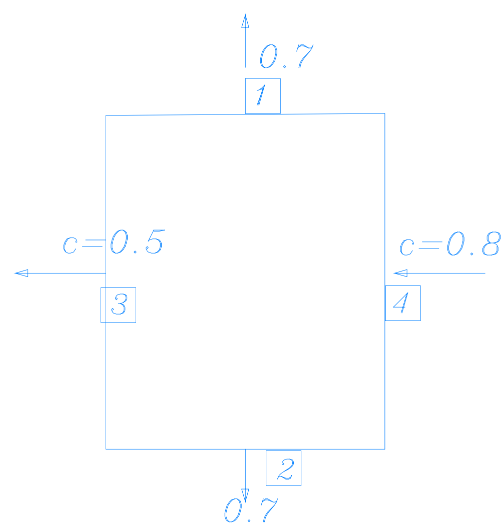
Wind Load [-y]

$W_2 = 0.33$ t/m , $w'_2 = 0.228$ t/m

$W_2 = 0.33$ t/m , $w'_2 = 0.228$ t/m

$W_3 = 0.235$ t/m , $w'_3 = 0.114$ t/m

$W_4 = 0.38$ t/m , $w'_4 = 0.63$ t/m



Max. Straining Action (SAP2000)

Axial Force

Shear Force

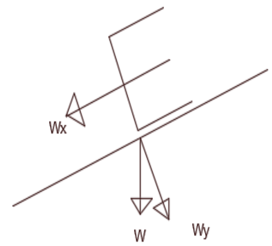
Bending Moment

CHAPTER II

Cold Formed

Purlin

S = 6 m a = 2 m Slope = 1/20 $\alpha = 2.86^{\circ}$



Loads

W_{-c} = 10 Kg\m², O.W = 5 Kg/m²

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

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

$W_{D.L\,y}$ = 30 * sin (2.86) = 1.5 Kg / m`

$W_{L.L}$ = (60 – 66.6 * 1/20) *2 = 113.34 Kg / 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`

Total 1.2 WD. L + 1.6 W L.L

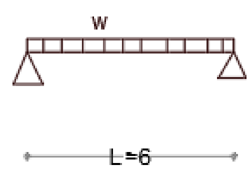
W_x = 29.96 *1.2 + 1.6 * 113.198 = 217.068 Kg\m`/1000 = 0.217 t/m`

W_y = 1.5 *1.2 + 1.6 * 5.65 = 10.84 Kg\m`/1000 = 0.011 t/m`

Straining Action

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

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



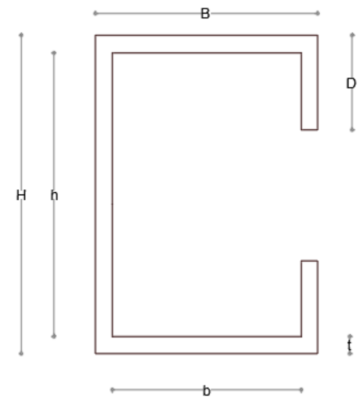
$Q_{-x} = \frac{0.217*6}{2} = 0.651\ ton$

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

Choice of Section

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

Assume $\dashrightarrow$ $S_y = \frac{Sx}{6}$ $\dashrightarrow$ Sx = 6 * Sy



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

Try … Lipped Channel (180 * 80 * 25 * 4 * 8 mm)

(H * B * D * t * r)

Code Limit for Slender Sections

For Web

h= H – 2r – 2 t = 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 mm

b/t = 14 < 40 ……. **OK**

For Lip

d= D – r – t = 25 – 8 – 4 = 20 mm

d/t = 3.25 < 40 ……. **OK**

So … Section Satisfies Code Limits

The Effective Parts of Sections

Flange

Partially Stiff Flange is Compression…. So …ψ = 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} cm^4

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

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

So, take 5.25 – 5 (25 / 56) = 3 ∴Ka = 3

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

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

so, Flange is Fully Effective

Stiff Web

Ψ = -1 is subjected to M, Kα = 23.9, b=h= 156 mm

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

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

So, Web is Fully effective

Lip

Un Stiff Lip

Ψ = 1, Kα = 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

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

$F_{\text{-}b\text{-}c}$ < 0.75 * .85 * F-y = 1.53 t/cm² …. Ok

Check Shear

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

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

$V_{\text{-}all}$ = 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

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.

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

$S_{x-x} = I_{x-x} / Y = 3.7733 * 10^{-5}$ m3. $F_{act} = 0.178 / 3.773 * 10^{-5} * (100)^2 * 1.13 = 0.417 < 0.8 * 2.4$

S0, chequered plate is safe.

Design of Beams: **for B1**

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

$w_{l.L} = 0.4 * 2.5 / 2 = 0.5 \text{ t/m.}$ from sap2000:

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

Check bending:

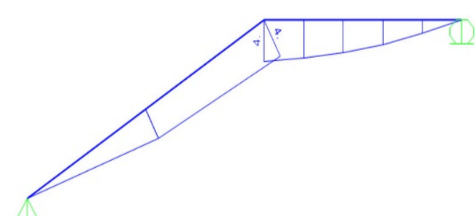
Lb= $\frac{2500}{3} = 833 \text{ mm}$, $L_p = \frac{80 * 2.3}{\sqrt{2.4}} = 96 \ cm$, so Lb <Lp (case A)

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).



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

Check deflection :

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

MEM. BRACING		Length(m)	Vu(tonf)	Pu(tonf)	M(t.m)	LBx(m)	Lby(m)	Connection type	D	edge dist.	Pitch dist	L-take cm	N1	N2	LOAD CASE	FINAL IPE
نموذج ك.ث (1)	SEC-B-261	6	8.1432		3.16E-15			SHEAR	1.6	30	60	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	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	220
	SEC-B-71		3.6729		8.88E-15			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-179		-3.5E-08		5.94E-12			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	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
	SEC-B-104		6.7082		9.33E-06			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	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	1.6	30	60	120	2	2	1.2D+1.6LL+0.5Lr	220
	SEC-B-140		0.0116		0.02207			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-119		0.1083		0.16376			SHEAR	1.6	30	60	120	2	1	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		3.5386		3.89242			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
	SEC-B-177-4		1.646		4.61663			SHEAR	1.6	30	60	120	2	1	1.2D+1.6LL+0.5Lr	220
نموذج ك.ث (2)	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
	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
نموذج ك.ر (1)	Main-B-23-3				-5.11255			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-38-4				-5.58683			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-18-2				-6.43621			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-85-1		-2.8295	-9.7879	-3.37607			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-77-1		-2.8451	-11.0266	-5.44985			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-24-1			-11.8051	-6.12008			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-33-1		-6.2235	-7.2958	-6.29068			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-78-1		-3.0695	-12.0172	-6.6586			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-11-1		-12.8405		-7.03825			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-4-1		-13.3485		-7.6233			BOLTED	1.6	54	84	57.55	5	5		330
	Main-B-4-1				-12.3159			BOLTED	2.7	54	84	57.55	5	5		330
	Main-B-53-1		-6.6253		-7.87244			BOLTED	2.7	54	84	57.55	5	5		330
	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
نموذج ك.ر (2)	Main-B-3-1		-14.7649		-13.8719			BOLTED/FILLET WELD	2.7	54	84	57.55	5	5		450
	Main-B-18-1		11		13.92064			BOLTED/FILLET WELD	2.7	54	84	57.55	5	5		450
	Main-B-6-1		-10.5385		-15.9015			BOLTED/FILLET WELD	2.7	54	84	57.55	5	5		450
	Main-B-8-1		-17.2963		-18.0098			BOLTED	2.7	54	84	57.55	5	5		450
	Main-B-20-1		-12.4118		-17.8531			BOLTED	2.7	54	84	57.55	5	5		450
	Main-B-25-4				-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				-18.9974			BOLTED	2.7	54	84		5	5		IPE 450
نموذج ك.ر (3)	Main-B-48-1		-12.9388		-21.5345			BOLTED	2.7	54	94	73.45	6	6		IPE 500
	Main-B-12-1		-28.7206		-26.2375			BOLTED	2.7	54		73.45	6	6	FULL PN. GROVE WLD	IPE 500
	Main-B-39-1		-30.097		-34.1155			BOLTED	2.7			73.45	6	6	FULL PN. GROVE WLD	IPE 500
	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
نموذج ع.ر (1)	Main-Col-12		6.45	-2.72	5.79			BOLTED	2.7							IPE 600
	Main-Col-20-8		-10.1411	-2.3367	-4.5388			BOLTED	2.7							IPE 600
	Main-Col-20		7.75	-15.9	11.9			BOLTED	2.7							IPE 600
	Main-Col-18-8		-19.3122	-10.623	-9.51866			BOLTED	2.7							IPE 600
	Main-Col-21		7.12	-14.2	16.34			BOLTED	2.7							IPE 600
	Main-Col-8		7.5	-43	-11.61			BOLTED	2.7							IPE 600
	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
نموذج ع.ر(2)	Main-Col-16		-14.1	-104	23.4			BOLTED	2.7							HEA 800
	Main-Col-25		4.61	-109.21	27.24			BOLTED	2.7							HEA 800
	Main-Col-26		5.8	-116.47	31.71			BOLTED	2.7							HEA 800
	Main-Col-27		12.42	-101.7	13.85			BOLTED	2.7							HEA 800
نموذج ع.ر(3)	Main-Col-23		6.86	-75.9	15.87			BOLTED	2.7							HEA 600
	Main-Col-14		-12.7	-77.7	11.56			BOLTED	2.7							HEA 600
	Main-Col-15		-13.16	-85.24	22.75			BOLTED	2.7							HEA 600
	Main-Col-24		-3.49	-74.79	29.19			BOLTED	2.7							HEA 600
نموذج ع.ر(4)	Main-Col-28		5.84	-65.96	10.75			BOLTED	2.7							HEA 500
	Main-Col-17		-7.45	-56	22.2			BOLTED	2.7							HEA 500
	Main-Col-18		-11.2	-41.3	-11.49			BOLTED	2.7							HEA 500
	Main-Col-19		-7.79	-50.5	32.7			BOLTED	2.7							HEA 500
	Main-Col-67		7.38	-45.9	38.5			BOLTED	2.7							HEA 500
	Main-Col-68		14.23	-56	16.3			BOLTED	2.7							HEA 500
	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
	Main-Col-22		7.4	-50.7	41.68			BOLTED	2.7							HEA 500
	Main-Col-13		3.11	-54	5.017			BOLTED	2.7							HEA 500
نموذج ك.ك (1)	Cant-M-Beam-1-1		-13.1136		-25.1995			BOLTED	2.7	54	94	73.45	6	6		450
	Cant-M-Beam-2-1		-13.1136		-25.1995			BOLTED	2.7	54	94	73.45	6	6		450
	Cant-M-Beam-3-1		-13.2536		-15.7973			BOLTED/FILLET WELD	2.7	54	94	73.45	6	6		450
	Cant-M-Beam-4-1		-14.0387		-27.8046			BOLTED	2.7	54	94	73.45	6	6		450
	Cant-M-Beam-8-1		-5.7865		-8.59008			BOLTED/FILLET WELD	2.7	54	94	73.45	6	6		450
	Window-Beam-19-1		0.78		0.71			BOLTED	1.6							IPE 140

Project	

Element	

Steel Properties			37
F_y=	2.4	t/cm^2	
F_u=	3.7	t/cm^2	

Staining actions		
Pult =	1	ton

Chosen sec	60x6		Double Angle
$t_{G.PL}$ =	1	cm	Bolted
Bolt Dia.=	16	mm	
L =	605	cm	

Sextion Properties		
a =	60	mm
Area =	13.82	cm^2
r_x =	1.82	cm
r_y =	2.848	cm
r_v =	1.17	cm
r_u =	2.29	cm

Checks :

i- Min. Angle			
min. a =	52.8	mm	Safe

ii- Buckling		
L_{in} =	302.5	cm
L_{out} =	453.75	cm
λ_{in} =	166.21	Safe
λ_{out} =	159.35	Safe

iii- Capacity			
λ_C =	1.7878		
F_{cr} =	0.4866	t/cm^2	
P_r =	5.3794	ton	Safe

iv- Tie Plate			
$L^\backslash$ =	194.46	cm	Two Tie Plates

⊛ Design Vertical Bracing As SHS :-

Data , $F = 12$ ton , $L_{in} = 0.5 * 800 = 400$ mm.

$L_{bout} = 0.75 * 800 = 600$ mm.

Choice : Assume $\lambda_c = 1.1$ So $\lambda = 102.2$, $F_{cr} = 1.28\ t/cm^2$

from stress , $12 = 0.8 * 1.28 * A_g \Rightarrow A_g = 12.11$

choose $120 * 3.6$

From Buckling : $\lambda = 102.2$

$\frac{\lambda_{out}}{102.2} = \frac{600}{r_x}$, $r_x = 5.87$ cm.

$\frac{\lambda_{in}}{102.2} = \frac{400}{r_y}$, $r_y = 3.91$ cm.

} Choose $150 * 5$

$\boxed{\text{Finally use } 120 * 120 * 5}$

$A = 22{,}9\ cm^2$, $r_x = 5.74$ cm , $r_y = 3.31$ cm

Check :- Check Compactness :-

$\frac{h - 2(r+t)}{t} = 22 < \frac{64}{\sqrt{F_y}} = 41$ web & Flange are Non-Compact.

$\frac{b - 2(r+t)}{t} = 22 < 41$

Check Buckling :- $\lambda_{out} = \frac{600}{4.69} = 127 < 180$

$\lambda_{in} = \frac{400}{4.69} = 85.3 < 180$

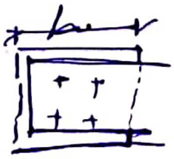
$\lambda_c = 1.67 > 1.1$

$F_{cr} = 0.83\ t/cm^2 \rightarrow P_r = \phi F_{cr} A_g = 0.8 * 0.83 * 22.9$

$= 15.2 > 12$ safe.

$$L_{act} = \frac{12}{0.7 * 4 * 0.5 * 0.4 * 3.6} + 2 * 0.5 = 69 \text{ mm}$$

take for weld $= 100$ mm



DESIGN OF BUILT-UP WELDED (COMPACT/NON-COMPACT) STEEL I SECTIONS

Section Dimensions	Flanges			Web		
	tf=	0.92	cm	tw=	0.59	cm
	b=	11	cm	hw=	22	cm

SEC-B-177-4	
Length (cm)	600

Hint:	
1st 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

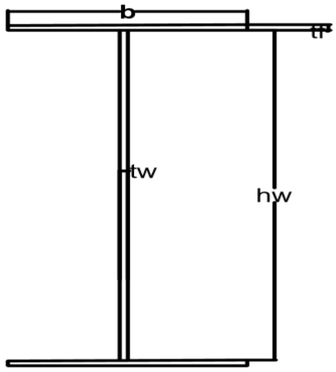
Material:	
Steel Grade	St.37

Geometry:		
Lb=	0	cm
Cb=	1.130	
Lbx=	600	cm
Lby=	600	cm
L=	600	cm
Lh=	600	cm

SEC-B-177-4

Straining Actions:		
Mux=	4.00	t.m
Qu=	1	t
Nu=	0.00	t
Tu=	0	t

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



Flexure							
(1) Local Buckling		Compact Flange		Compact 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		

Axial Compression					
λx=	61.295	≤	180	OK	
λy=	241.849	≤	180	Decrease 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

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

Combined (Normal Force + Flexure)			
M+C	D/C=	0.654	Safe for combined M+C
M+T	D/C=	0.654	Safe for combined M+T

Shear Force		
Vn=	20.25	t
Φv*Vn=	17.22	t
D/C=	0.058	
Safe for Shear		

Safe

DESIGN OF BUILT-UP WELDED (COMPACT/NON-COMPACT) STEEL I SECTIONS

Section Dimensions	Flanges			Web		
	tf=	1.9	cm	tw=	1.2	cm
	b=	22	cm	hw=	60	cm

HEB 500	
Length (cm)	282

Hint:	
1st trial Sx=Mu/(.75*Fy)=	1399.97

Section Properties		
Ix	101705.799	cm4
Iy	3380.50667	cm4
rx=	25.57	cm
ry=	4.66	cm
A=	155.60	cm2
Aw=	76.56	cm2
Sx=	3188.27	cm3
Sy=	307.32	cm3
Zx=	3638.16	cm3
Zy=	481.40	cm3
rt=	5.60	cm

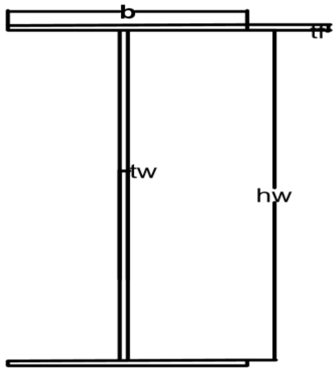
Material:	
Steel Grade	St.37

Geometry:		
Lb=	705	cm
Cb=	1.130	
Lbx=	282	cm
Lby=	282	cm
L=	282	cm
Lh=	282	cm

Cant-M-Beam-2-1

Straining Actions:		
Mux=	25.20	t.m
Qu=	13	t
Nu=	0.00	t
Tu=	0	t

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



Flexure							
(1) Local Buckling		Compact Flange		Compact Web		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 axis		

Axial Compression					
λx=	11.030	≤	180		OK
λy=	60.501	≤	180		OK
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

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

Combined (Normal Force + Flexure)			
M+C	D/C=	0.480	Safe for combined M+C
M+T	D/C=	0.480	Safe for combined M+T

Shear Force		
Vn=	110.25	t
Φv*Vn=	93.71	t
D/C=	0.139	
Safe for Shear		

Safe

DESIGN OF BUILT-UP WELDED (COMPACT/NON-COMPACT) STEEL I SECTIONS

Section Dimensions	Flanges			Web		
	tf=	1.6	cm	tw=	1.02	cm
	b=	20	cm	hw=	50	cm

SEC-B-132-1	
Length (cm)	600

Hint:	
1st trial Sx=Mu/(.75*Fy)=	1000.00

Section Properties		
Ix	53239.6133	cm4
Iy	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

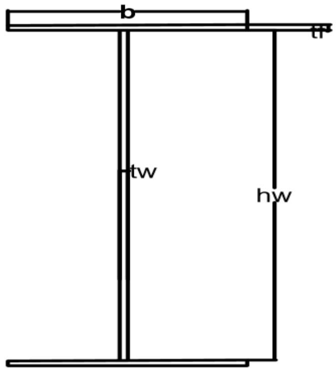
Material:	
Steel Grade	St.37

Geometry:		
Lb=	0	cm
Cb=	1.130	
Lbx=	600	cm
Lby=	600	cm
L=	600	cm
Lh=	600	cm

Main-B-132-1

Straining Actions:		
Mux=	18.00	t.m
Qu=	32	t
Nu=	0.00	t
Tu=	0	t

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



Flexure							
(1) Local Buckling		Compact Flange		Compact Web		Compact Section	
(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 major axis		

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

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

Combined (Normal Force + Flexure)			
M+C	D/C=	0.389	Safe for combined M+C
M+T	D/C=	0.389	Safe for combined M+T

Shear Force		
Vn=	78.14	t
Φv*Vn=	66.42	t
D/C=	0.482	
Safe for Shear		

Safe

DESIGN OF BUILT-UP WELDED (COMPACT/NON-COMPACT) STEEL I SECTIONS

Section Dimensions	Flanges			Web		
	tf=	0.69	cm	tw=	0.47	cm
	b=	7.3	cm	hw=	14	cm

HUNCH-L9	
Length (cm)	400

NO fly bracing

Window-Beam-19-1

Hint:	
1st trial Sx=Mu/(.75*Fy)=	38.89

Section Properties		
Ix	651.355497	cm4
Iy	44.8580818	cm4
rx=	6.25	cm
ry=	1.64	cm
A=	16.65	cm2
Aw=	7.23	cm2
Sx=	84.70	cm3
Sy=	12.29	cm3
Zx=	95.92	cm3
Zy=	19.16	cm3
rt=	1.91	cm

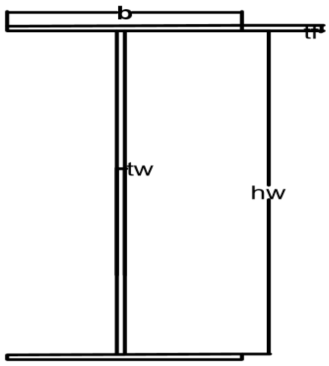
Material:	
Steel Grade	St.37

Straining Actions:		
Mux=	0.70	t.m
Qu=	0.80	t
Nu=	0.00	t
Tu=	0	t

Geometry:		
Lb=	400	cm
Cb=	1.300	
Lbx=	1201.5	cm
Lby=	400	cm
L=	400	cm
Lh=	400	cm

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

Flexure							
(1) Local Buckling		Compact Flange		Compact Web		Compact Section	
(2) LTB							
Lp=	84.751	cm					
Lr=	347.894	cm		Case C			
Mn=	1.35	t.m	≤	Mp=	2.30197	t.m	
Φb*Mn=	1.15	t.m	D/C=	0.611	Safe for flexure about major axis		



Axial Compression					
λx=	192.121	≤	180	Decrease λx	
λy=	243.724	≤	180	Decrease λy	
λc=	2.6227				
Fcr=	0.2261	t/cm2			
Pn=	3.77	t			
Φc*Pn=	3.01	t	D/C=	0.000	Safe for axial compression

Axial Tension					
(1) Stiffnes condition					
λ=	243.724	≤	300	OK	
Lh/60=	6.667	≤	h	OK	
(2) Strenght condition					
Pn=	39.97	t			
Φt*Pn=	33.97416	t	D/C=	0.000	Safe for yielding at tension

Combined (Normal Force + Flexure)			
M+C	D/C=	0.611	Safe for combined M+C
M+T	D/C=	0.611	Safe for combined M+T

Shear Force		
Vn=	10.41	t
Φv*Vn=	8.85	t
D/C=	0.090	
Safe for Shear		

Safe

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

37
44
52
Fixed
Hinged

Frame loaction vh

Steel grade 37

F_y	2.4	t/cm^2
F_u	3.7	t/cm^2

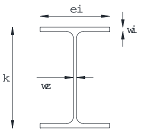
Column ID Main-Col-16-12

(1) Applied forces

$M_{x\,14}$	10	t.m
N	66	ton
Q	5	ton

(2) Assumption of section

h	49	cm
t_w	1.2	cm
b_f	30	cm
t_f	2.3	cm



(3) classification of section

(i) Web

d_w	44.4	cm
t_w	1.2	cm

table 2.12a P. 2-27

$\lambda\ \frac{dw}{tw}$	37
λp	50.81
λ_r	143.3

Class.	compact

(ii) Flange

C	15	cm
t_f	2.3	cm

table 2.12c P. 2-29

$\lambda = \frac{\alpha C}{tf}$	4.96
λp	9.88
λ_r	18.07

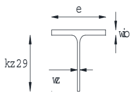
Class.	compact

α	0.76

Section class.	compact

(6) flexure design **(Ch.5)** **(Compact section)**

	r_y	7.25	cm
Eq. 5.4 , P.5-3	L_p	374.39	cm
T - section	Area (T-section)	77.88	cm^2
	Iy	5176.07	cm^4
	r_T	8.15	cm
	Area flange	69	cm^2
Eq. 5.8 , P. 5-3	X	0.36	
	F_L	1.44	t/cm^2
Eq. 5.7 , P. 5-3	L_r	1490.7	cm



Case (b)

Mp	92.22	t.m

Case (a) or (b)	C_b	1.83
case (b)	Mr	49.4
case (c)	Mcr	-

User define ,5.1 , P.5-1
P.5-1 : P.5-4

Mn	92.22	t.m
ϕ	0.85	
ϕMn	78.39	t.m

Safe

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

$\frac{Pu}{\phi\ Pn}$	0.2355124
$\frac{Mu}{\phi\ Mn}$	0.84

Eq.s 7.1a,7.1b	Ratio	0.98

Safe

(4) section properties

Area	196.8	cm^2
Ix	84054.38	cm^4
Iy	10356.39	cm^4
Sx	3430.79	cm^3
Sy	690.43	cm^3

(5) Column data

Total length	550	cm	
Base type	Hinged		
G 1	10		Table 2.6b , P.2-15
Ix (Rafter)	48200	cm^4	User define
Length of rafter	600	cm	User define
G 2	1.9		
K	1.9		User define , chart (2.7) , P.2-14

L_{bx}	1045	cm	
L by = Lb	550	cm	User define (from bracing)

(5) flexure design	**(Non-compact)**		
Mn (flange)	270.52	t.m	Eq. 5.16
Mn (web)	92.22	t.m	P. 5-5
Mn	92.22	t.m	
L_p'	374.39	cm	
case (b)			
Case (a) or (b)	C_b	0.6	
case (b)	Mr	49.4	
case (c)	Mcr	-	
P. 5-5	Mn	51.29	t.m
	ϕ	0.85	
	ϕMn	43.6	t.m

Safe

(7) Desgin of normal strenght **(Ch.4)**

	r_x	20.67	cm
	r_y	7.25	cm
	λx	50.56	
	λy	75.86	
	λmax	75.86	
Eq. 4.4	λc	0.82	
Eq. 4.2 , 4.3	F_{cr}	1.78	
Eq. 4.1	Pn	350.3	ton
	ϕ	0.8	
	ϕPn	280.24	ton

Safe

(8) Design of shear strenght **Ch.5**

	$\frac{h}{tw}$	37	
	$^{112}/\sqrt{Fy}$	72.3	
	$^{139}/\sqrt{Fy}$	89.72	
Eq.s 5.22,5.23,5.24	Vn	76.72	ton
	ϕ	0.85	
	ϕVu	65.21	ton

Safe

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Faculty of engineering
Structural Engineering Department - SCHEP
Graduation project 202
Design of columns
Refrence : Egyptian code(LRFD)

37
44
52

Fixed
Hinged

Frame loaction vh

Steel grade 37

F_Y	2.4	t/cm^2
F_U	3.7	t/cm^2

Column ID Main-Col-16-12

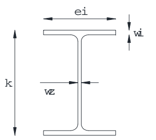
HEB 450

(1) Applied forces

$M_{x\,14}$	12	t.m
N	18	ton
Q	7	ton

(2) Assumption of section

h	45	cm
t_w	1.4	cm
b_F	30	cm
t_f	2.6	cm



(3) classification of section

(i) Web

d_w	39.8	cm
t_w	1.4	cm

table	$\lambda\ \frac{dw}{tw}$	28.43
2.12a	λp	70.39
P. 2-27	λ_r	143.3

Class.	compact

(ii) Flange

C	15	cm
t_f	2.6	cm

table	$\lambda = \frac{\alpha C}{tf}$	3.29
2.12c	λp	9.88
P. 2-29	λ_r	18.07

Class.	compact

α	0.57

Section class.	compact

(4) section properties

Area	219	cm^2
Ix	77555.75	cm^4
Iy	11709.1	cm^4
Sx	3446.92	cm^3
Sy	780.61	cm^3

(5) Column data

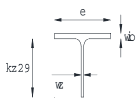
Total length	1600	cm	
Base type	Hinged		
G 1	10		Table 2.6b , P.2-15
Ix (Rafter)	79890	cm^4	User define
Length of rafter	2430	cm	User define
G 2	1.47		
K	3.8		User define , chart (2.7) , P.2-14

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

(5) flexure design			**(Non-compact)**			
		Mn (flange)	186.2	t.m	**Eq. 5.16**	
		Mn (web)	92.65	t.m	**P. 5-5**	
		Mn	92.65	t.m		
		L_p'	377.49	cm		
	case (b)					
Case (a) or (b)		C_b	0.6			
case (b)		Mr	49.64			
case (c)		Mcr	-			
P. 5-5		Mn	32.58	t.m		
		ϕ	0.85			**Safe**
		ϕMn	27.69	t.m		

(6) flexure design **(Ch.5)** **(Compact section)**

	r_y	7.31	cm
Eq. 5.4 , P.5-3	L_p	377.49	cm
T - section	Area (T-section)	87.29	cm^2
	Iy	5851.52	cm^4
	r_T	8.19	cm
	Area flange	78	cm^2
Eq. 5.8 , P. 5-3	X	0.24	
	F_L	1.44	t/cm^2
Eq. 5.7 , P. 5-3	L_r	1748.37	cm



Case (b)

Mp	92.65	t.m

Case (a) or (b)	C_b	2.2	**User define ,5.1 , P.5-1**
case (b)	Mr	49.64	**P.5-1 : P.5-4**
case (c)	Mcr	-	

Mn	92.65	t.m
ϕ	0.85	
ϕMn	78.75	t.m

Safe

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

$\frac{Pu}{\emptyset\ Pn}$	0.7337954
$\frac{Mu}{\emptyset\ Mn}$	0.23

Eq.s 7.1a,7.1b	Ratio	0.94

Safe

(7) Desgin of normal strenght **(Ch.4)**

	r_x	18.82	cm
	r_y	7.31	cm
	λx	323.06	
	λy	218.88	
	λmax	323.06	
Eq. 4.4	λc	3.48	
Eq. 4.2 , 4.3	F_{cr}	0.14	
Eq. 4.1	Pn	30.66	ton
	ϕ	0.8	
	ϕPn	24.53	ton

Safe

(8) Design of shear strenght **Ch.5**

	$\frac{h}{tw}$	28.43	
	$^{112}/_{\sqrt{Fy}}$	72.3	
	$^{139}/_{\sqrt{Fy}}$	89.72	
Eq.s 5.22,5.23,5.24	Vn	80.24	ton
	ϕ	0.85	
	ϕVu	68.2	ton

Safe

Ain Shams university
Faculty of engineering
Structural Engineering Department - SCHEP
Graduation project 202
Design of columns
Refrence : Egyptian code(LRFD)

37
44
52

Frame loaction vh

Steel grade 37

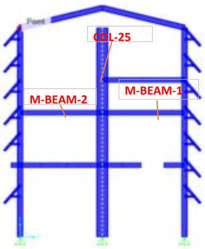
F_Y	2.4	t/cm^2
F_U	3.7	t/cm^2

Fixed
Hinged

Column ID COL

sec 3

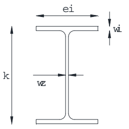
(1) Applied forces



M_x	23	t.m
N	109	ton
Q	4	ton

(2) Assumption of section

h	79	cm
t_w	1.5	cm
b_F	30	cm
t_f	2.8	cm



(3) classification of section

(i) Web

d_w	73.4	cm
t_w	1.5	cm

table 2.12a P. 2-27

$\lambda \frac{dw}{tw}$	48.93
λp	54.82
λ_r	143.3

Class.	compact

(ii) Flange

C	15	cm
t_f	2.8	cm

α	0.71

table 2.12c P. 2-29

$\lambda = \frac{\alpha C}{tf}$	3.8
λp	9.88
λ_r	18.07

Class.	compact

Section class.	compact

(4) section properties

Area	286.5	cm^2
Ix	293411.1	cm^4
Iy	12620.64	cm^4
Sx	7428.13	cm^3
Sy	841.38	cm^3

(5) Column data

Total length	550	cm
Base type	Hinged	
G 1	10	
G 2	9.3210494	
K	3.1	

Table 2.6b , P.2-15

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

for G2		
	SEC-MEZZ-1	SEC-MEZZ-2
L (cm)	600	1
Ix (cm 4)	33740	1

L_{bx}	1705	cm
L by = Lb	550	cm

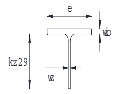
User define (from bracing)

(5) flexure design		**(Non-compact)**			
	Mn (flange)	842.49	t.m	**Eq. 5.16**	
	Mn (web)	199.67	t.m	**P. 5-5**	
	Mn	199.67	t.m		
	L_p'	342.89	cm		
Case (a)					
Case (a) or (b)	C_b	0.6			
case (b)	Mr	-			
case (c)	Mcr	-			
P. 5-5	Mn	199.67	t.m		
	ϕ	0.85			**Safe**
	ϕMn	169.72	t.m		

(6) flexure design **(Ch.5)** **Compact section**

Eq. 5.4 , P.5-3

r_y	6.64	cm
L_p	342.89	cm



T - section

Area (T-section)	102.35	cm^2
Iy	6303.4	cm^4
r_T	7.85	cm
Area flange	84	cm^2
X	0.59	
F_L	1.44	t/cm^2
L_r	1242.1	cm

Eq. 5.8 , P. 5-3

Eq. 5.7 , P. 5-3

Case (a)

Mp	199.67	t.m

Case (a) or (b)
case (b)
case (c)

C_b	1.79
Mr	-
Mcr	-

User define ,5.1 , P.5-1

P.5-1 : P.5-4

$\frac{h}{tw}$

$112/\sqrt{Fy}$

$139/\sqrt{Fy}$

Mn	199.67	t.m
ϕ	0.85	
ϕMn	169.72	t.m

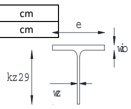
Safe

(7) Desgin of normal strenght **(Ch.4)**

r_x	32	cm
r_y	6.64	cm
λx	53.28	
λy	82.83	
λmax	82.83	
λc	0.89	
F_{cr}	1.67	

Eq. 4.4

Eq. 4.2 , 4.3



Eq. 4.1

Pn	478.46	ton
ϕ	0.8	
ϕPn	382.77	ton

Safe

(8) Design of shear strenght **Ch.5**

$\frac{h}{tw}$	48.93
$112/\sqrt{Fy}$	72.3
$139/\sqrt{Fy}$	89.72

Eq.s 5.22,5.23,5.24

Vn	158.54	ton
ϕ	0.85	
ϕVu	134.76	ton

Safe

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

$\frac{Pu}{\emptyset Pn}$	0.2848
$\frac{Mu}{\emptyset Mn}$	0.64

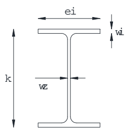
Eq.s 7.1a,7.1b

Ratio	0.85

Safe

$\frac{Pu}{\emptyset Pn}$

$\frac{Mu}{\emptyset Mn}$



Project	PROJ N0.8

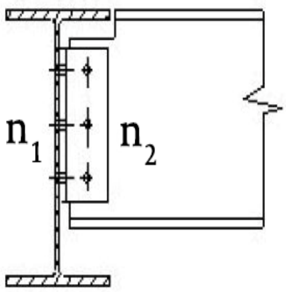
Element	SEC-B-261

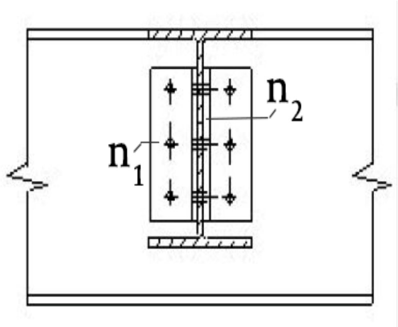
Steel Properties			St.37
Fy	2.4	t/cm2	
Fu	3.6	t/cm2	

Straining Actions		
Q u=	8	ton

Connection Details		
Secondary beam sec.	IPE330	
Main beam 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

Bolts		
d=	1.6	cm
edge dist. =	25	cm
Pitch dist. =	50	cm
Category :	A	
Grade :	10.9	
F_y =	9	t/cm2
F_u =	10	t/cm2





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

Bearing		
α	1.6	
F_{b} =	5.76	t/cm^2
Rbr,b n1=	5.161	ton
Rbr,b n2=	4.838	ton

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

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

Block Shear Rupture (angle)			
$A_{net\ shear}$ =	115.68	cm^2	
$A_{net\ tension}$ =	4.32	cm^2	
$0.6F_u\ A_{n.s}$ =	249.8688	Ton	
$F_u\ A_{n.t}$ =	15.552	Ton	
$A_{gross\ shear}$ =	120	cm^2	
$A_{gross\ tension}$ =	5.76	cm^2	
Rrp =	184.585	Ton	Safe

Project	PROJ N0.8

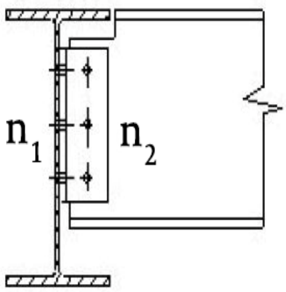
Element	SEC-B-261

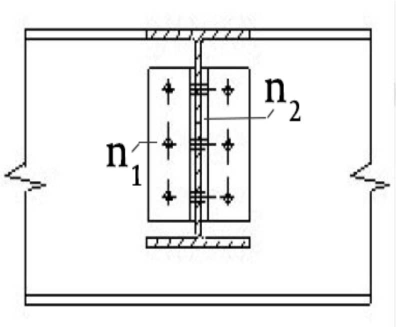
Steel Properties		St.37
Fy	2.4	t/cm2
Fu	3.6	t/cm2

Straining Actions		
Q u=	7	ton

Connection Details		
Secondary beam sec.		IPE220
Main beam sec.		IPE450
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

Bolts		
d=	1.6	cm
edge dist. =	25	cm
Pitch dist. =	50	cm
Category :	C	
Grade :	10.9	
F_y =	9	t/cm2
F_u =	10	t/cm2





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

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

Bearing		
α	1.6	
$F_{b\,=}$	5.76	t/cm^2
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

Block Shear Rupture (angle)			
$A_{net\ shear}$ =	115.68	cm^2	
$A_{net\ tension}$ =	4.32	cm^2	
$0.6F_u\ A_{n.s}$ =	249.8688	Ton	
$F_u\ A_{n.t}$ =	15.552	Ton	
$A_{gross\ shear}$ =	120	cm^2	
$A_{gross\ tension}$ =	5.76	cm^2	
Rrp =	184.585	Ton	Safe

Project		Element	M-B-40 + M-Col-15

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

Straining actions:-		
Mu=	41	mt
Tu=	0	t
Qu=	33	t

Beam Dimensions		
bf=	40	cm
tf=	2	cm
dw=	58	cm
tw=	0.8	cm
d=	60	cm

Column Dimensions		
bf=	30	cm
tf=	3.5	cm
dw=	83	cm
tw=	1.85	cm
Ac	363.55	cm

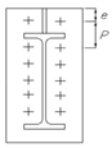
Connection Geometry:-		
a=	9.2	cm
b=	5.4	cm
tp=	3	cm
2W=	18.8	cm
W=	9.4	cm
B=	30	cm

Bolts		
d=	2.7	cm
As=	4.4659	cm
Grade	10.9	Category B
Fy, bolt=	9	t/cm2
Fu, bolt=	10	t/cm2

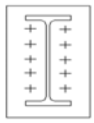
Checks

Fillet Wled Between Beam and End Plate					
Sf=	10	mm	Sw=	8	mm
Aw hz=	144	cm2	Aw Vl=	76.8	cm2
Aw tot=	220.8	cm2	Ix=	137565.6	cm4
Check Stresses					
Point 1	F1	0.92392	t/cm2	Safe	
Point 2	f2	0.7153	t/cm2		
	q2	0.42969	t/cm2		
	F2	1.03225	t/cm2	Safe	

Number of bolts			Extended with full depth bolt
e=	5.4	cm	
p=	8.1	cm	
allowable n/2=	6	bolts	
P actual=	9.4	cm	
chosen n=	10	bolts	



Extended with full depth bolts



Full depth bolts

y1	63.3	cm	y6	14.9	cm
y2	52.5	cm	y7	0	cm
y3	43.1	cm	y8	0	cm
y4	33.7	cm	y9	0	cm
y5	24.3	cm	y10	0	cm
Σy2=	10569	cm2			

Tension on Bolts			
First bolt			
T1=	12.278	t	
Rtb=	25.009	t	Safe

Second bolt			
T2=	8.3599	t	
Prying Force=	2.1717	t	
T2+P=	10.532	t	Safe

Shear on bolts		
Qu/n=	2.3571	t
Rvb=	17.177	t

interaction eq=	0.2599	Safe

In Case of Category (B):-		
Qs=	22	ton
Ms=	27.333	ton
Ts=	0	ton
Tb=	8.1853	ton
Qb=	1.5714	ton

interaction eq=	0.453	Safe

d=	2.7	cm
Ordinary steel work		
Case of loading		I
St.		37&44
Grade		10.9
Load factor=		1.5
Ps=	9.25	ton
T=	28.91	ton

Thickness of end plate			
M1=	19.979	cm.t	
M2=	25.164	cm.t	
tb required=	2.2911	cm	Safe

Check Column Safety		
db=	60	cm
Tb=	70.69	t
Cb=	70.69	t

Flange Loacal Bending				Using Doubler Plate			No Need
Rt=	156.19	t		t=	10	mm	
Tb=	70.69	t	safe	Rt	####	t	

Using Stiffner			No Need			
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	146.58	t		t=	5	mm	
Cb=	70.69	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Crippling				Using Doubler Plate			No Need
Rn=	124.42	t					
Rc=	87.096	t		t=	5	mm	
Cb=	70.69	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	6	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Shear Panel Zone				Using Doubler Plate			No Need
Pu=	160	t					
0.4Py=	349.01	t					
Rv=	203.8	t		t=	5	mm	
Cb or Tb=	68.333	t	Safe	Rv	####	t	

Project		Element	M-B-20 + M-Col-15

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

Straining actions:-		
Mu=	18	mt
Tu=	0	t
Qu=	17	t

Beam Dimensions		
bf=	19	cm
tf=	1.46	cm
dw=	43.54	cm
tw=	1.4	cm
d=	45	cm

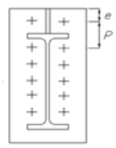
Column Dimensions		
bf=	30	cm
tf=	3.5	cm
dw=	83	cm
tw=	1.85	cm
Ac	363.55	cm

Connection Geometry:-		
a=	4.4	cm
b=	5.4	cm
tp=	2	cm
2W=	17.2	cm
W=	8.6	cm
B=	21	cm

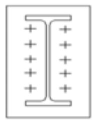
Bolts		
d=	2.7	cm
As=	4.4659	cm
Grade	10.9	Category B
Fy, bolt=	9	t/cm2
Fu, bolt=	10	t/cm2

Checks					
Fillet Wled Between Beam and End Plate					
Sf=	10	mm	Sw=	8	mm
Aw hz=	68.4	cm2	Aw Vl=	57.6	cm2
Aw tot=	126	cm2	Ix=	39148.30464	cm4
Check Stresses					
Point 1	F1	1.08051	t/cm2	unsafe	
Point 2	f2	0.82762	t/cm2		
	q2	0.29514	t/cm2		
	F2	0.97277	t/cm2	Safe	

Number of bolts			Extended with full depth bolt
e=	5.4	cm	
p=	8.1	cm	
allowable n/2=	5	bolts	
P actual=	8.6	cm	
chosen n=	8	bolts	



Extended with full depth bolts



Full depth bolts

y1	49.77	cm	y6	0	cm
y2	38.97	cm	y7	0	cm
y3	30.37	cm	y8	0	cm
y4	21.77	cm	y9	0	cm
y5	13.17	cm	y10	0	cm
Σy2=	5565.4	cm2			

Tension on Bolts			
First bolt			
T1=	8.0484	t	
Rtb=	25.009	t	Safe

Second bolt			
T2=	4.9112	t	
Prying Force=	3.2494	t	
T2+P=	8.1607	t	Safe

Shear on bolts		
Qu/n=	1.4167	t
Rvb=	17.177	t

interaction eq=	0.1133	Safe

In Case of Category (B):-		
Qs=	11.333	ton
Ms=	12	ton
Ts=	0	ton
Tb=	5.4404	ton
Qb=	0.9444	ton

d=	2.7	cm
Ordinary steel work		
Case of loading		I
St.		37&44
Grade		10.9
Load factor=		1.5
Ps=	9.25	ton
T=	28.91	ton

interaction eq=	0.2903	Safe

Thickness of end plate			
M1=	14.298	cm.t	
M2=	12.223	cm.t	
tb required=	1.8055	cm	Safe

Check Column Safety		
db=	45	cm
Tb=	41.341	t
Cb=	41.341	t

Flange Loacal Bending			
Rt=	156.19	t	
Tb=	41.341	t	safe

Using Doubler Plate			No Need
t=	10	mm	
Rt	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Loacal Yielding			
Rc=	135.86	t	
Cb=	41.341	t	Safe

Using Doubler Plate			No Need
t=	5	mm	
Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Crippling			
Rn=	123.58	t	
Rc=	86.508	t	
Cb=	41.341	t	Safe

Using Doubler Plate			No Need
t=	5	mm	
Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	6	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Shear Panel Zone			
Pu=	160	t	
0.4Py=	349.01	t	
Rv=	203.8	t	
Cb or Tb=	40	t	Safe

Using Doubler Plate			No Need
t=	5	mm	
Rv	####	t	

Project		Element	Cant-M-B-2 + M-COL-25

Steel Properties		St.37	Straining actions:-		
Fy	2.4	t/cm2	Mu=	27	mt
Fu	3.6	t/cm2	Tu=	0	t
			Qu=	14	t

Beam Dimensions			Column Dimensions		
bf=	40	cm	bf=	30	cm
tf=	2	cm	tf=	3.5	cm
dw=	58	cm	dw=	83	cm
tw=	0.8	cm	tw=	1.85	cm
d=	60	cm	Ac	363.55	cm

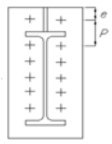
Connection Geometry:-			Bolts		
a=	10.6	cm	d=	2	cm
b=	4	cm	As=	2.4504	cm
tp=	2	cm	Grade	10.9	Category B
2W=	12.8	cm	Fy, bolt=	9	t/cm2
W=	6.4	cm	Fu, bolt=	10	t/cm2
B=	30	cm			

Checks

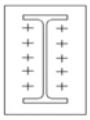
Fillet Wled Between Beam and End Plate					
Sf=	8	mm	Sw=	5	mm
Aw hz=	115.2	cm2	Aw Vl=	48	cm2
Aw tot=	163.2	cm2	Ix=	107364.352	cm4

Check Stresses				
Point 1	F1	0.77456	t/cm2	Safe
Point 2	f2	0.60355	t/cm2	
	q2	0.29167	t/cm2	
	F2	0.78707	t/cm2	Safe

Number of bolts			Extended with full depth bolt
e=	4	cm	
p=	6	cm	
allowable n/2=	9	bolts	
P actual=	6.4	cm	
chosen n=	10	bolts	



Extended with full depth bolts



Full depth bolts

y1	62.6	cm	y6	29	cm
y2	54.6	cm	y7	0	cm
y3	48.2	cm	y8	0	cm
y4	41.8	cm	y9	0	cm
y5	35.4	cm	y10	0	cm
Σy2=	13065	cm2			

Tension on Bolts				Second bolt			
First bolt				T2=	4.9806	t	
T1=	6.4686	t		Prying Force=	0.7395	t	
Rtb=	13.722	t	Safe	T2+P=	5.7201	t	Safe

Shear on bolts		
Qu/n=	1	t
Rvb=	9.4248	t
interaction eq=	0.2335	Safe

In Case of Category (B):-			d=	2	cm
Qs=	9.3333	ton	Ordinary steel work		
Ms=	18	ton	Case of loading	I	
Ts=	0	ton	St.	37&44	
			Grade	10.9	
Tb=	4.3124	ton	Load factor=	1.5	
Qb=	0.6667	ton	Ps=	4.93	ton
			T=	15.43	ton
interaction eq=	0.4147	Safe			

Thickness of end plate			
M1=	7.8383	cm.t	
M2=	12.084	cm.t	
tb required=	1.9241	cm	Safe

Check Column Safety		
db=	60	cm
Tb=	46.552	t
Cb=	46.552	t

Flange Loacal Bending				Using Doubler Plate			No Need
Rt=	156.19	t		t=	10	mm	
Tb=	46.552	t	safe	Rt	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	138.14	t		t=	5	mm	
Cb=	46.552	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Cripping							
Rn=	124.42	t		Using Doubler Plate			No Need
Rc=	87.096	t		t=	5	mm	
Cb=	46.552	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	6	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Shear Panel Zone							
Pu=	160	t					
0.4Py=	349.01	t		Using Doubler Plate			No Need
Rv=	203.8	t		t=	5	mm	
Cb or Tb=	45	t	Safe	Rv	####	t	

Project		Element	Cant-M-B-2 + M-COL-25

Steel Properties		St.37	Straining actions:-		
Fy	2.4	t/cm2	Mu=	27	mt
Fu	3.6	t/cm2	Tu=	0	t
			Qu=	14	t

Beam Dimensions			Column Dimensions		
bf=	40	cm	bf=	30	cm
tf=	2	cm	tf=	3.5	cm
dw=	58	cm	dw=	83	cm
tw=	0.8	cm	tw=	1.85	cm
d=	60	cm	Ac	363.55	cm

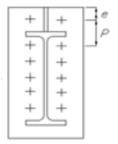
Connection Geometry:-			Bolts		
a=	10.6	cm	d=	2	cm
b=	4	cm	As=	2.4504	cm
tp=	2	cm	Grade	10.9	Category B
2W=	12.8	cm	Fy, bolt=	9	t/cm2
W=	6.4	cm	Fu, bolt=	10	t/cm2
B=	30	cm			

Checks

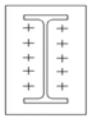
Fillet Wled Between Beam and End Plate					
Sf=	8	mm	Sw=	5	mm
Aw hz=	115.2	cm2	Aw Vl=	48	cm2
Aw tot=	163.2	cm2	Ix=	107364.352	cm4

Check Stresses				
Point 1	F1	0.77456	t/cm2	Safe
Point 2	f2	0.60355	t/cm2	
	q2	0.29167	t/cm2	
	F2	0.78707	t/cm2	Safe

Number of bolts			Extended with full depth bolt
e=	4	cm	
p=	6	cm	
allowable n/2=	9	bolts	
P actual=	6.4	cm	
chosen n=	10	bolts	



Extended with full depth bolts



Full depth bolts

y1	62.6	cm	y6	29	cm
y2	54.6	cm	y7	0	cm
y3	48.2	cm	y8	0	cm
y4	41.8	cm	y9	0	cm
y5	35.4	cm	y10	0	cm
Σy2=	13065	cm2			

Tension on Bolts				Second bolt			
First bolt				T2=	4.9806	t	
T1=	6.4686	t		Prying Force=	0.7395	t	
Rtb=	13.722	t	Safe	T2+P=	5.7201	t	Safe

Shear on bolts		
Qu/n=	1	t
Rvb=	9.4248	t

interaction eq=	0.2335	Safe

In Case of Category (B):-			d=	2	cm
Qs=	9.3333	ton	Ordinary steel work		
Ms=	18	ton	Case of loading		I
Ts=	0	ton	St.		37&44
			Grade		10.9
Tb=	4.3124	ton	Load factor=		1.5
Qb=	0.6667	ton	Ps=	4.93	ton
			T=	15.43	ton

interaction eq=	0.4147	Safe

Thickness of end plate			
M1=	7.8383	cm.t	
M2=	12.084	cm.t	
tb required=	1.9241	cm	Safe

Check Column Safety		
db=	60	cm
Tb=	46.552	t
Cb=	46.552	t

Flange Loacal Bending				Using Doubler Plate			No Need
Rt=	156.19	t		t=	10	mm	
Tb=	46.552	t	safe	Rt	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	138.14	t		t=	5	mm	
Cb=	46.552	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Web Cripplling							
Rn=	124.42	t		Using Doubler Plate			No Need
Rc=	87.096	t		t=	5	mm	
Cb=	46.552	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14	cm	Local buckling	bs/ts=	####		
ts=	6	mm	Normal stresses	T=	####	t	
ds=	83	cm	Shear stresses	Q=	####	t	

Shear Panel Zone							
Pu=	160	t					
0.4Py=	349.01	t		Using Doubler Plate			No Need
Rv=	203.8	t		t=	5	mm	
Cb or Tb=	45	t	Safe	Rv	####	t	

Project		Element	M-B-(37)/(25) + C3

Steel Properties			St.37	Straining actions:-		
Fy	2.4	t/cm2		Mu=	15	mt
Fu	3.6	t/cm2		Tu=	0	t
				Qu=	12	t

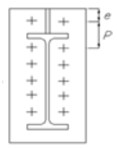
Beam Dimensions				Column Dimensions		
bf=	19	cm		bf=	30	cm
tf=	1.46	cm		tf=	2.5	cm
dw=	42.08	cm		dw=	54	cm
tw=	0.94	cm		tw=	1.3	cm
d=	43.54	cm		Ac	220.2	cm

Connection Geometry:-				Bolts		
a=	4.7	cm		d=	2.7	cm
b=	5.4	cm		As=	4.4659	cm
tp=	2	cm		Grade	10.9	Category B
2W=	16.4	cm		Fy, bolt=	9	t/cm2
W=	8.2	cm		Fu, bolt=	10	t/cm2
B=	21	cm				

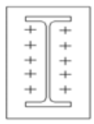
Checks

Fillet Wled Between Beam and End Plate					
Sf=	10	mm	Sw=	8	mm
Aw hz=	68.4	cm2	Aw Vl=	55.7312	cm2
Aw tot=	124.13	cm2	Ix=	36411.00881	cm4
Check Stresses					
Point 1	F1	0.93804	t/cm2	Safe	
Point 2	f2	0.71748	t/cm2		
	q2	0.21532	t/cm2		
	F2	0.80861	t/cm2	Safe	

Number of bolts			Extended with full depth bolt
e=	5.4	cm	
p=	8.1	cm	
allowable n/2=	5	bolts	
P actual=	8.2	cm	
chosen n=	8	bolts	



Extended with full depth bolts


Full depth bolts

y1	48.24	cm	y6	0	cm
y2	37.44	cm	y7	0	cm
y3	29.24	cm	y8	0	cm
y4	21.04	cm	y9	0	cm
y5	12.84	cm	y10	0	cm
Σy2=	5191.4	cm2			

Tension on Bolts				Second bolt			
First bolt				T2=	4.2243	t	
T1=	6.9693	t		Prying Force=	2.5961	t	
Rtb=	25.009	t	Safe	T2+P=	6.8204	t	Safe

Shear on bolts		
Qu/n=	1	t
Rvb=	17.177	t

interaction eq=	0.081	Safe

In Case of Category (B):-				d=	2.7	cm
Qs=	8	ton		Ordinary steel work		
Ms=	10	ton		Case of loading		I
Ts=	0	ton		St.		37&44
				Grade		10.9
Tb=	4.6462	ton		Load factor=		1.5
Qb=	0.6667	ton		Ps=	9.25	ton
				T=	28.91	ton

interaction eq=	0.2328	Safe

Thickness of end plate			
M1=	12.202	cm.t	
M2=	10.61	cm.t	
tb required=	1.7081	cm	Safe

Check Column Safety		
db=	43.54	cm
Tb=	35.646	t
Cb=	35.646	t

Flange Loacal Bending				Using Doubler Plate			No Need
Rt=	79.688	t		t=	10	mm	
Tb=	35.646	t	safe	Rt	####	t	

Using 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	

Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	72.499	t		t=	5	mm	
Cb=	35.646	t	Safe	Rc	####	t	

Using 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	

Web Crippling				Using Doubler Plate			No Need
Rn=	62.077	t					
Rc=	43.454	t		t=	5	mm	
Cb=	35.646	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	14.2	cm	Local buckling	bs/ts=	####		
ts=	6	mm	Normal stresses	T=	####	t	
ds=	54	cm	Shear stresses	Q=	####	t	

Shear Panel Zone				Using Doubler Plate			No Need
Pu=	160	t					
0.4Py=	211.39	t					
Rv=	93.881	t		t=	5	mm	
Cb or Tb=	34.451	t	Safe	Rv	####	t	

Project		Element	MB14-MC22-AXIS 2

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

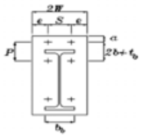
Straining actions:-		
Mu=	22	mt
Tu=	0	t
Qu=	10	t

Beam Dimensions		
bf=	19	cm
tf=	1.46	cm
dw=	43.54	cm
tw=	1.4	cm
d=	45	cm

Column Dimensions		
bf=	22	cm
tf=	1.6	cm
dw=	56.8	cm
tw=	1.2	cm
Ac	138.56	cm

Connection Geometry:-		
a=	6.8	cm
b=	4.7	cm
tp=	3	cm
2W=	21	cm
W=	10.5	cm
n=	12	bolts

Bolts		
d=	2.7	cm
As=	4.46593	cm
Grade	10.9	Category B
Fy, bolt=	9	t/cm2
Fu, bolt=	10	t/cm2



Forces on bolts		
Tb=	50.52825	t
Cb=	50.52825	t

Checks

Fillet Wled Between Beam and End Plate					
S f=	10	mm	S w=	8	mm	
Beam flange & col Flange		Fw=	1.477434	t/cm2	unsafe
Beam web & col Flange		Fw=	0.179433	t/cm2	Safe

Bolts

Tension on Bolts			
Text,b=	12.632062	t	
Prying force=	3.8044431	t	
Text,b + P=	16.436506	t	
Rtb=	25.009214	t	Safe
interaction eq		0.43429	Safe

Shear&Tension on bolts		
Qu/n=	0.83333	t
Rvb=	17.1767	t

In Case of Category (B):-		
Qs=	6.6666667	ton
Ms=	14.666667	ton
Ts=	0	ton
Tb=	32.592593	ton

d=	2.7	cm
Ordinary steel work		
Case of loading		I
St.		37&44
Grade		10.9
Ps=	9.25	ton
T=	28.91	ton

Shear&Tension on bolts		
interaction eq	0.34191	Safe

Thickness of end plate			
M1=	25.870213	cm.t	
M2=	33.500481	cm.t	
tb required=	2.5011848	cm	Safe

Check Column Safety

1- Flange Loacal Bending				Using Doubler Plate			
Rt=	32.64	t		t=	5	mm	
Tb=	50.52825	t	unsafe	Rt	56.2275	t	Ok

Using Stiffner							
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

2- Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	58.71456	t		t=	5	mm	
Cb=	50.52825	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	10.4	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	56.8	cm	Shear stresses	Q=	####	t	

3- Web Crippling				Using Doubler Plate			
Rn=	44.882031	t					
Rc=	31.417421	t		t=	5	mm	
Cb=	50.52825	t	unsafe	Rc	54.6206	t	Ok

Using Stiffner							
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

4- Shear Panel Zone							
Pu=	20	t					
0.4Py=	133.0176	t		Using Doubler Plate			No Need
Rv=	88.128	t		t=	5	mm	
Cb or Tb=	48.888889	t	Safe	Rv	####	t	

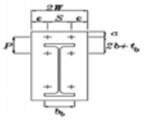
Project		Element	AXIS 3

Steel Properties			St.37	Straining actions:-		
Fy	2.4	t/cm2		Mu=	24	mt
Fu	3.6	t/cm2		Tu=	0	t
				Qu=	14	t

Beam Dimensions				Column Dimensions		
bf=	19	cm		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		Ac	210.02	cm

Connection Geometry:-				Bolts		
a=	6.8	cm		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				

Forces on bolts		
Tb=	55.121727	t
Cb=	55.121727	t



Checks

Fillet Wled Between Beam and End Plate						
S f=	10	mm	S w=	8	mm	
Beam flange & col Flange			Fw=	1.611746	t/cm2	unsafe
Beam web & col Flange			Fw=	0.251206	t/cm2	Safe

Bolts

Tension on Bolts				Shear&Tension on bolts		
Text,b=	13.780432	t		Qu/n=	1.16667	t
Prying force=	4.3234179	t		Rvb=	17.1767	t
Text,b + P=	18.10385	t				
Rtb=	25.009214	t	Safe			
interaction eq		0.52863	Safe			

In Case of Category (B):-						
Qs=	9.3333333	ton		d=	2.7	cm
Ms=	16	ton		Ordinary steel work		
Ts=	0	ton		Case of loading		I
Tb=	35.555556	ton		St.		37&44
				Grade		10.9
Shear&Tension on bolts				Ps=	9.25	ton
interaction eq		0.39155	Safe	T=	28.91	ton

Thickness of end plate			
M1=	29.399242	cm.t	
M2=	35.368787	cm.t	
tb required=	2.5699836	cm	Safe

Check Column Safety

1- Flange Loacal Bending				Using Doubler Plate			No Need
Rt=	67.4475	t		t=	5	mm	
Tb=	55.121727	t	Safe	Rt	####	t	

Using 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	

2- Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	73.68504	t		t=	5	mm	
Cb=	55.121727	t	Safe	Rc	####	t	

Using 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	

3- Web Crippling							
Rn=	59.726637	t		Using Doubler Plate			
Rc=	41.808646	t		t=	5	mm	
Cb=	55.121727	t	unsafe	Rc	69.4073	t	Ok

Using Stiffner							
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

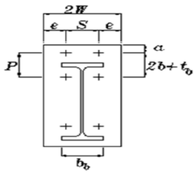
4- Shear Panel Zone							
Pu=	20	t					
0.4Py=	201.6192	t		Using Doubler Plate			No Need
Rv=	95.472	t		t=	5	mm	
Cb or Tb=	53.333333	t	Safe	Rv	####	t	

Project		Element	MB38-MC24-AXIS4

Steel Properties			St.37	Straining actions:-		
Fy	2.4	t/cm2		Mu=	26	mt
Fu	3.6	t/cm2		Tu=	0	t
				Qu=	21	t

Beam Dimensions				Column Dimensions		
bf=	19	cm		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		Ac	210.02	cm

Connection Geometry:-				Bolts		
a=	6.8	cm		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				



Forces on bolts		
Tb=	59.715204	t
Cb=	59.715204	t

Checks

Fillet Wled Between Beam and End Plate						
S f=	10	mm	S w=	8	mm	
Beam flange & col Flange			Fw=	1.746059	t/cm2	unsafe
Beam web & col Flange			Fw=	0.376809	t/cm2	Safe

Bolts

Tension on Bolts					Shear&Tension on bolts		
Text,b=	14.928801	t			Qu/n=	1.75	t
Prying force=	4.6837028	t			Rvb=	17.1767	t
Text,b + P=	19.612504	t					
Rtb=	25.009214	t	Safe				
interaction eq		0.62537	Safe				

In Case of Category (B):-				d=	2.7	cm
Qs=	14	ton		Ordinary steel work		
Ms=	17.333333	ton		Case of loading		I
Ts=	0	ton		St.		37&44
Tb=	38.518519	ton		Grade		10.9
Shear&Tension on bolts				Ps=	9.25	ton
interaction eq		0.45922	Safe	T=	28.91	ton

Thickness of end plate			
M1=	31.849179	cm.t	
M2=	38.316186	cm.t	
tb required=	2.6749237	cm	Safe

Check Column Safety

1- Flange Loacal Bending				Using Doubler Plate			No Need
Rt=	67.4475	t		t=	5	mm	
Tb=	59.715204	t	Safe	Rt	####	t	

Using 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	

2- Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	73.68504	t		t=	5	mm	
Cb=	59.715204	t	Safe	Rc	####	t	

Using 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	

3- Web Crippling							
Rn=	59.726637	t		Using Doubler Plate			
Rc=	41.808646	t		t=	5	mm	
Cb=	59.715204	t	unsafe	Rc	69.4073	t	Ok

Using Stiffner							
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

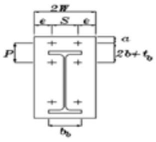
4- Shear Panel Zone							
Pu=	20	t					
0.4Py=	201.6192	t		Using Doubler Plate			No Need
Rv=	95.472	t		t=	5	mm	
Cb or Tb=	57.777778	t	Safe	Rv	####	t	

Project		Element	MB22-MC29-AXIS 7

Steel Properties			St.37	Straining actions:-		
Fy	2.4	t/cm2		Mu=	14	mt
Fu	3.6	t/cm2		Tu=	6	t
				Qu=	6	t

Beam Dimensions				Column Dimensions		
bf=	19	cm		bf=	22	cm
tf=	1.46	cm		tf=	1.6	cm
dw=	42.08	cm		dw=	56.8	cm
tw=	0.94	cm		tw=	1.2	cm
d=	43.54	cm		Ac	138.56	cm

Connection Geometry:-				Bolts		
a=	5	cm		d=	2	cm
b=	3.3	cm		As=	2.45044	cm
tp=	2	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				



Forces on bolts		
Tb=	36.269962	t
Cb=	30.269962	t

Checks

Fillet Wled Between Beam and End Plate						
S f=	10	mm	S w=	5	mm	
Beam flange & col Flange			Fw=	1.060525	t/cm2	unsafe
Beam web & col Flange			Fw=	0.178232	t/cm2	Safe

Bolts

Tension on Bolts				Shear&Tension on bolts		
Text,b=	9.0674905	t		Qu/n=	0.5	t
Prying force=	2.5815771	t		Rvb=	9.42478	t
Text,b + P=	11.649068	t				
Rtb=	13.722477	t	Safe			
interaction eq		0.72345	Safe			

In Case of Category (B):-						
Qs=	4	ton		d=	2	cm
Ms=	9.3333333	ton		Ordinary steel work		
Ts=	4	ton		Case of loading		I
Tb=	23.436227	ton		St.		37&44
				Grade		10.9
Shear&Tension on bolts				Ps=	4.93	ton
interaction eq		0.44733	Safe	T=	15.43	ton

Thickness of end plate			
M1=	12.907885	cm.t	
M2=	17.014833	cm.t	
tb required=	1.7825188	cm	Safe

Check Column Safety

1- Flange Loacal Bending				Using Doubler Plate			
Rt=	32.64	t		t=	5	mm	
Tb=	36.269962	t	unsafe	Rt	56.2275	t	Ok

Using Stiffner							
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

2- Web Loacal Yielding				Using Doubler Plate			No Need
Rc=	53.24256	t		t=	5	mm	
Cb=	30.269962	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	10.4	cm	Local buckling	bs/ts=	####		
ts=	10	mm	Normal stresses	T=	####	t	
ds=	56.8	cm	Shear stresses	Q=	####	t	

3- Web Crippling							
Rn=	44.882031	t		Using Doubler Plate			No Need
Rc=	31.417421	t		t=	5	mm	
Cb=	30.269962	t	Safe	Rc	####	t	

Using Stiffner			No Need				
bs=	10.4	cm	Local buckling	bs/ts=	####		
ts=	6	mm	Normal stresses	T=	####	t	
ds=	56.8	cm	Shear stresses	Q=	####	t	

4- Shear Panel Zone							
Pu=	20	t					
0.4Py=	133.0176	t		Using Doubler Plate			No Need
Rv=	88.128	t		t=	5	mm	
Cb or Tb=	32.154341	t	Safe	Rv	####	t	

C_1 - splice $Q = 18$ ton , IPE 600

$b_f = 220$ mm , $t_f = 19$ mm, $t_w = 12$ m , $h-2c = 514$ mm

$h = 600$ mm.

For S_1 : $b_1 = 22$ cm , $t_1 = 0.5 * 1.9 = 0.95$ cm $= \boxed{1 \text{ cm}}$

For S_2 : $b_2 = 0.5[22 - 1.2 - 2*0.8 - 1 \text{cm}] = 9.1$ cm $= \boxed{9.5 \text{ cm}}$

$22 * 1.9 = 22*1 + 2*9.5*t_2$

$t_2 = 1.05$ cm $\simeq \boxed{1.1 \text{ cm}}$

For S_3 : $60 * 1.2 = 2 * b_3 \cdot t_3$

$\therefore b_3 = 60 - 2*1.9 - 1 \text{cm} = \boxed{55.2 \text{ cm}}$

$t_3 = 0.65 \simeq \boxed{0.7 \text{ cm}}$

Get required No. of Bolts :- [for flange] :-

$C = 22 * 1.9 * 0.8 * 3.6 = 120.40$ ton.

$R_{s,Rd} = 0.9 * 1 * 2 * 0.4 * 0.7 * 10 * \frac{\pi}{4}(2.2)^2 = 19.16$ ton

$R_{s,bd} = 1.34 * 5.2 * 2.2 * 1.9 = 29.12$ ton.

$n_1 = \frac{120.4}{19.16} = 6.28 \simeq 4$ rows $*$ 2 Bolts per row.

Check net section fracture (tens. fl) :-

$f_t = \frac{120.4}{22 * 1.9} = 2.88$ t/cm² $< 0.84 * \frac{[22*1.9 - 3(2.4)*1.9]}{22*1.9}$

$= 2.938$ t/cm²

O.K.

For WEB: $n_2 = \frac{55.2}{4*2.2} = 6.27 \rightarrow P_{act} = \frac{55.2}{8}$

Then $P_{act} = 4*2.2 = 8.8$ cm ; $e = 4.4$ cm.

$Y = (60 - 55.2) + 4.4 + \frac{8.8}{2} = 13.6$ cm.

$f_1 = \frac{0.8*3.6}{[\frac{60}{2}+1.9]} * \frac{60}{2} = 2.71$ t/cm²

$$f_2 = \frac{0.8 * 3.6}{\left[\frac{60}{2} + 1.9\right]} * \left[\frac{60}{2} - 13.6\right] = 1.48 \ t/cm^2.$$

$$f_{H2} = \frac{2.71 + 1.48}{2} * 13.6 * 1.2 = 34.2 \ ton.$$

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

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

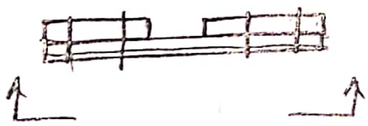
$$R = 17.14 \ ton$$

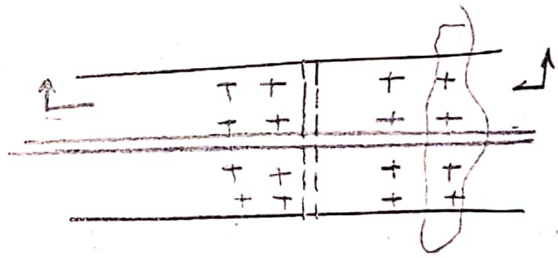
* Bearing Resistance :- $t_w = t_{min} = 1.2 \ cm.$

$$F_{b,Rd} = 0.8 * 2.5 * 0.67 * 2.4 * 1.2 = 20.1 \ ton.$$

$$R = 17.14 < R_{least} = 20.11 \quad o.k.$$

$$G_1 \left[\begin{array}{l} n_1 = 4 \text{ rows} * 2 \text{ Bolts} \\ n_2 = 2 \text{ rows} * 8 \text{ Bolts} \end{array} \right.$$





C_2 - splice, $Q = 13$ ton, HEA 600.

$b_f = 30\,cm$, $t_f = 25\,mm = 2.5\,cm$, $t_w = 13\,mm$, $h - 2t_f = 590\,mm$

For S_1: $b_1 = \boxed{30\,cm}$, $t_1 = \boxed{1.3\,cm}$

For S_2: $b_2 = \boxed{13.1\,cm}$, $t_2 = \boxed{1.4\,cm}$

For S_3: $59 * 1.3 = 2\, b_3 . t_3$

$\boxed{b_3 = 53\,cm} \rightarrow \boxed{t_3 = 0.7\,cm}$

for tens. fL : $T = 30 * 2.5 * 0.8 * 3.6 = 216$ ton

$R_{s,Rd} = 15.83$ ton, M 20 G 10.9 Cat B.

$R_{b,Rd} = 18.11$ ton

$n_1 = \frac{216}{15.83} = 13.6$
$= 16$

$= 4$ rows $*$ 4 Bolts per row.

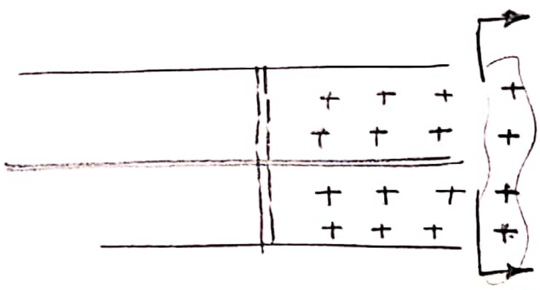
⊕ Check net fracture (tens. fL):

$f_t = \frac{216}{30 * 2.5} = 2.8 < 0.8 \square = 3.7$

o.k.

For web:- $n_2 = \frac{53}{4 * 2} = 6.6$

take 8



$P_{act} = 8\,cm$, $e_{act} = 4\,cm$.

(2 rows $*$ 8 Bolts).

Design of Hinged Base

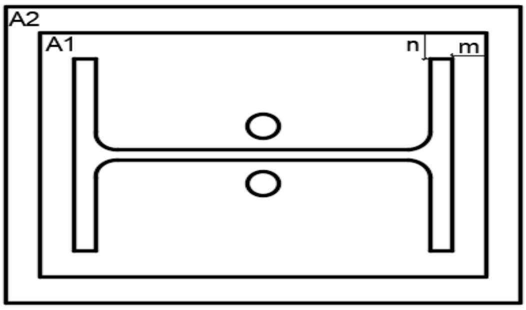
C1

Steel Grade	St.37		
Fy	=	2.4	t/cm^2
Fu	=	3.7	t/cm^2

Description :

1)- INPUT DATA :-

Nu =	43	t
Qu =	7.5	t
Fcu =	250	Kg/cm^2
S,weld =	1	cm
Column Section =	HEB 550	
h =	60	cm
t web =	1.2	cm
bf =	22	cm
tf =	1.9	cm



2)- End Plate dimensions :

m =	2.5	cm	n =	2.5	cm		
Lp =	65	cm	bp =	27	cm		
Ap (A1) =	1755	cm^2	Ac (A2) =	2775	cm^2	$\sqrt{A2 / A1} \leq 2$	ok
tp =	2	cm	tp,Choosen =	2	cm		

3)- Check Concrete Capacity :

$$Nu \leq 0.6 \times 0.67 \times Fcu \times A1 \times \sqrt{(A2 / A1)}$$

43 ton ≤ 221.79 ton **Safe**

4)- Check welding Safety :

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

A w,flange =	78	cm^2	Aw,total =	182.8	cm^2
A w,web =	104.8	cm^2			
Ruw,N =	0.24	t/cm2	Safe		
QD =	0.32	t/cm^2	Safe		
Ruw,eff =	0.97	t/cm2	Safe		

5)- Anchor Bolts :

n.of anchors =	4	M	20	Grade	10.9
α = (0.8 x e)/d	2.4	As =	2.45	cm^2	
Length=40*3=120 cm		Fub =	10	t/cm^2	
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 α x Fu =	24.86	t	7.5	t	Safe	
ϕv.Rv =	.85*0.6 x 0.6 x Fub x As x n =	7.50	t				

Design of Hinged Base

C2

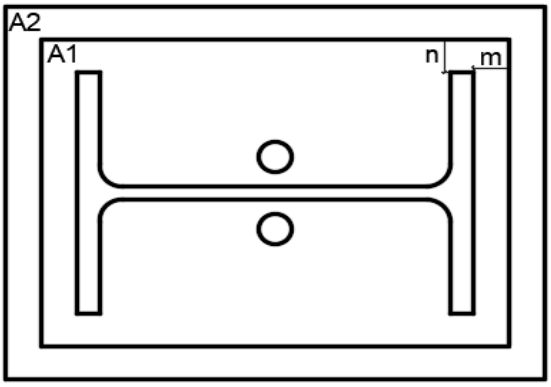
Steel Grade: St.37

Fy	=	2.4	t/cm^2
Fu	=	3.7	t/cm^2

Description :

1)- INPUT DATA :-

Nu =	116	t
Qu =	5	t
Fcu =	250	Kg/cm^2
S,weld =	10	cm
Column Section =	built up sections	
h =	79	cm
t web =	1.5	cm
bf =	30	cm
tf =	2.8	cm



2)- End Plate dimensions :

m =	2.5	cm	n =		
Lp =	84	cm	bp =	35	cm
Ap (A1) =	2940	cm^2	Ac (A2) =	4230	cm^2
tp =	2	cm	tp,Choosen =	2	cm

$\sqrt{A2 / A1} \le 2$ **ok**

3)- Check Concrete Capacity :

$Nu \le 0.6 \times 0.67 \times Fcu \times A1 \times \sqrt{(A2 / A1)}$

116 ton ≤ 354.41 ton **Safe**

4)- Check welding Safety :

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

A w,flange =	1058	cm^2	Aw,total =	2414	cm^2
A w,web =	1356	cm^2			
Ruw,N =	0.05	t/cm2	**Safe**		
QD =	0.06	t/cm^2	**Safe**		
Ruw,eff =	0.19	t/cm2	**Safe**		

5)- Anchor Bolts :

n.of anchors =	4	M	30	Grade	10.9
α = (0.8 x e)/d	2.4	As =	5.61	cm^2	
Length=40*3=120 cm		Fub =	10	t/cm^2	
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 x tmin x α x Fu =	37.3	t	17.17	t	**Safe**
ϕv.Rv =	.85*0.6 x 0.6 x Fub x As x n =	17.17	t			

Design of Hinged Base

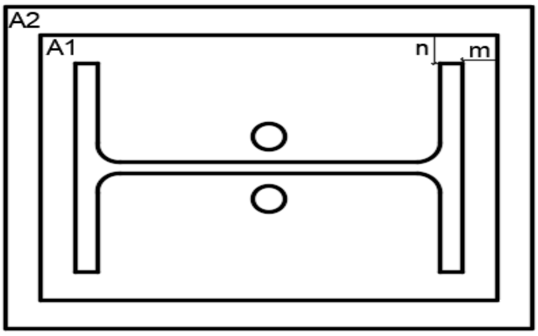
C3

Steel Grade: St.37 | Fy = 2.4 t/cm^2 | Fu = 3.7 t/cm^2

Description :

1)- INPUT DATA :-

Nu =	85	t
Qu =	13	t
Fcu =	250	Kg/cm^2
S,weld =	10	cm
Column Section =	built up sections	
h =	59	cm
t web =	1.3	cm
bf =	30	cm
tf =	2.5	cm



2)- End Plate dimensions :

m =	2.5	cm	n =	2.5	cm
Lp =	64	cm	bp =	35	cm
Ap (A1) =	2240	cm^2	Ac (A2) =	3330	cm^2
tp =	2	cm	tp,Choosen =	2	cm

$\sqrt{A2 / A1} \le 2$ ok

3)- Check Concrete Capacity :

Nu ≤ $0.6 \times 0.67 \times Fcu \times A1 \times \sqrt{(A2 / A1)}$

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^2
ϕ.Rnw,eff =	1.1*0.77 x 0.4 x Fu =	**1.254**	t/cm^2

A w,flange =	1074	cm^2	Aw,total =	2054	cm^2
A w,web =	980	cm^2			
Ruw,N =	0.04	t/cm2	**Safe**		
QD =	0.07	t/cm^2	**Safe**		
Ruw,eff =	0.19	t/cm2	**Safe**		

5)- Anchor Bolts :

n.of anchors = 4 | M 27 | Grade 10.9

α = (0.8 x e)/d 2.4 | As = 4.59 cm^2

Length=40*3=120 cm | Fub = 10 t/cm^2

Ru,t =		21.25	t	
Ru,v =		3.25	t	
ϕt.Rnt =	0.8 x 0.66 x Fub x As =	24.24	t	**Safe**
ϕbr.Rnbr =	0.7 x d x tmin x α x Fu =	33.57	t	14.05 t **Safe**
ϕv.Rv =	.85*0.6 x 0.6 x Fub x As x n =	14.05	t	

Design of Hinged Base

C4

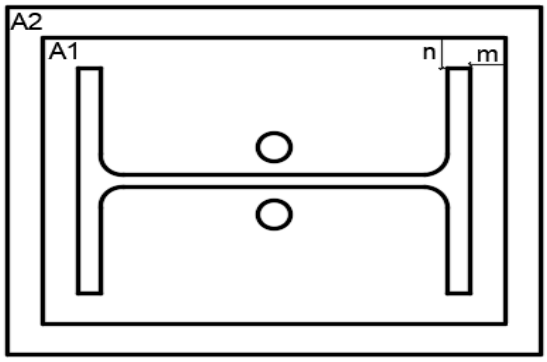
Steel Grade = St.37

Fy	=	2.4	t/cm^2
Fu	=	3.7	t/cm^2

Description :

1)- INPUT DATA :-

Nu =	65	t
Qu =	5	t
Fcu =	250	Kg/cm^2
S,weld =	10	cm
Column Section =	built up sections	
h =	49	cm
t web =	1.2	cm
bf =	30	cm
tf =	2.3	cm



2)- End Plate dimensions :

m =	2.5	cm	n =	2.5	cm		
Lp =	54	cm	bp =	35	cm		
Ap (A1) =	1890	cm^2	Ac (A2) =	2880	cm^2	$\sqrt{A2 / A1} \leq 2$	ok
tp =	2	cm	tp,Choosen =	2	cm		

3)- Check Concrete Capacity :

$Nu \leq 0.6 \times 0.67 \times Fcu \times A1 \times \sqrt{(A2 / A1)}$

65 ton ≤ 234.47 ton **Safe**

4)- Check welding Safety :

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

A w,flange =	1084	cm^2	Aw,total =	1880	cm^2
A w,web =	796	cm^2			
Ruw,N =	0.03	t/cm2	**Safe**		
QD =	0.06	t/cm^2	**Safe**		
Ruw,eff =	0.16	t/cm2	**Safe**		

5)- Anchor Bolts :

n.of anchors =	4	M	27	Grade	10.9
α = (0.8 x e)/d	2.4	As =	4.59	cm^2	
Length=40*3=120 cm		Fub =	10	t/cm^2	

Ru,t =	16.25	t
Ru,v =	1.25	t

ϕt.Rnt =	0.8 x 0.66 x Fub x As =	24.24	t	**Safe**			
ϕbr.Rnbr =	0.7 x d x tmin x α x Fu =	33.57	t	14.05	t	**Safe**	
ϕv.Rv =	.85*0.6 x 0.6 x Fub x As x n =	14.05	t				