Steel Structure Design
Steel Bridge

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Steel Bridge

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ABSTRACT

Steel bridges are an essential component of modern infrastructure, providing a safe and reliable way for people to cross over rivers, highways, and other obstacles. This thesis explores the construction of steel bridges at intersections between two main roads. It will examine the management of such projects from start to finish, discussing the public problems that can arise during construction and how they can be solved. The aim is to provide a safe building to pass vehicles over the intersection, by using Egyptian Code for practice and construction programs and these helped us achieving the required target.

ACKNOWLEDGEMENTS

In the first place, we want to thank Allah, the Most Gracious, the Most Merciful and peace is upon His Prophet. And We owe a lot of thanks to all those who have helped us during this study Furthermore we are appreciating the time and effort they spent providing guidance and advice throughout the year. Great thanks also extend to Prof. Dr. Mohammed El- Ghandour for his help and his efforts with us in this project to product a great project and for his great help, encouragement throughout this work, valuable comments and sound guidance.

-Thank you-

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Chapter 1 (Introduction)

1-1 General

Steel is one of the most advanced building materials available today composition can bring about important cost savings, as well as innovation in building. The advantages of steel as a light in weight and more durable material than concrete results in quicker and easier construction.
Steel can be recycled from job to job, meaning the amount of generated steel waste is small. Steel's sustainability also has a significant positive impact on the environment.
In this project, we will try to reduce the economic cost and try to reach the highest levels of safety and optimal use, also we Aim to Design a stunning bridge to match surrounding landscape.

1-2 Objective and Goals

The basic target of this project is to design road way steel bridge to decrease traffic jam in the intersection of two main roadways. The associated objectives can be itemized as follows:

1- Review the previous projects.

2- Investigate the major factors affecting the design of road way steel bridge.

3- Study the effect of using empty space under the bridge as a commercial partition (cafes, shops...etc).

4- Investigate the cost and time of construction. Also, study the risks that affect every stage in construction process.

1-3 Literature Review

Steel bridges have been widely used in transportation infrastructure due to their high strength-to-weight ratio, durability, and ease of fabrication. Recent studies have focused on improving the design and construction of steel bridges to enhance their performance and reduce their environmental impact.

In summary, recent studies on steel bridges have focused on improving their performance, durability, and sustainability through the use of advanced materials, technologies, and design strategies. These advancements have the potential to improve the safety and longevity of bridges, while reducing their environmental impact and life cycle cost.

Chapter 2 (Architecture Model)

2-1 3D Site

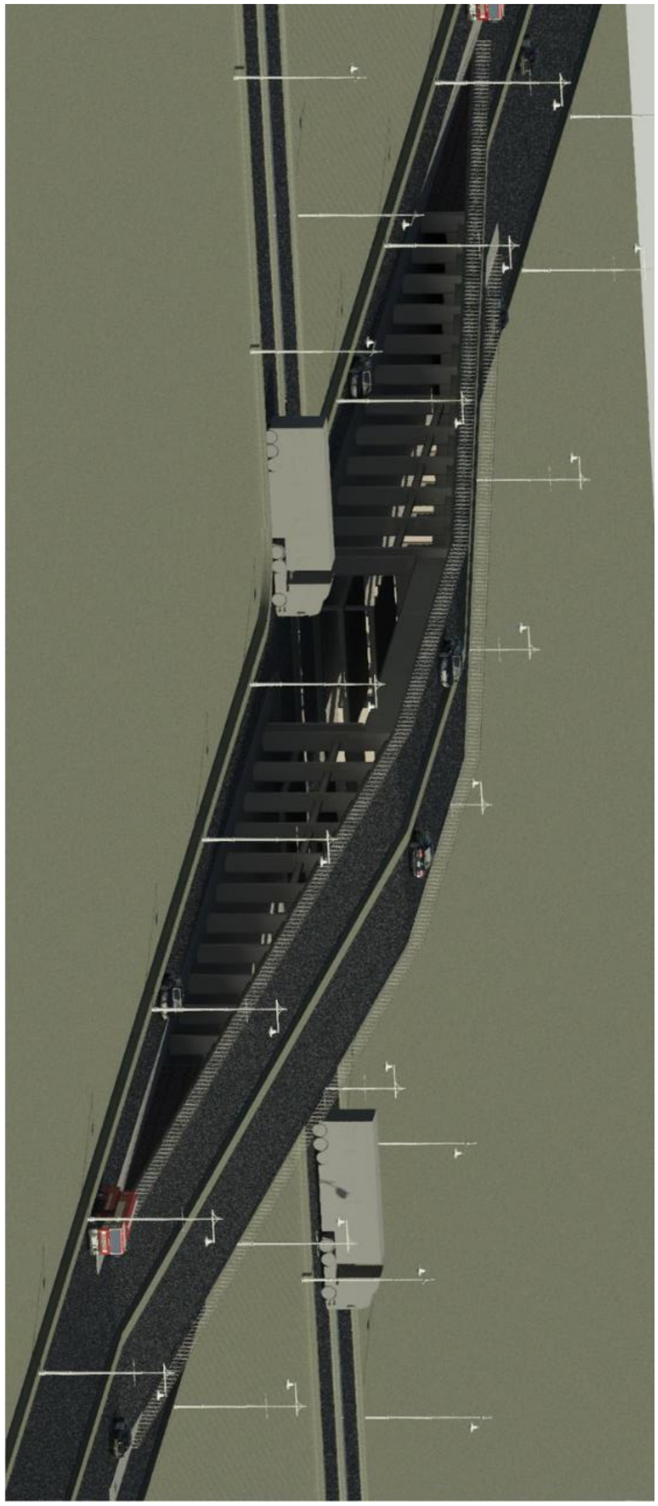


Figure2. 1 3d site

2-2 Site Plan

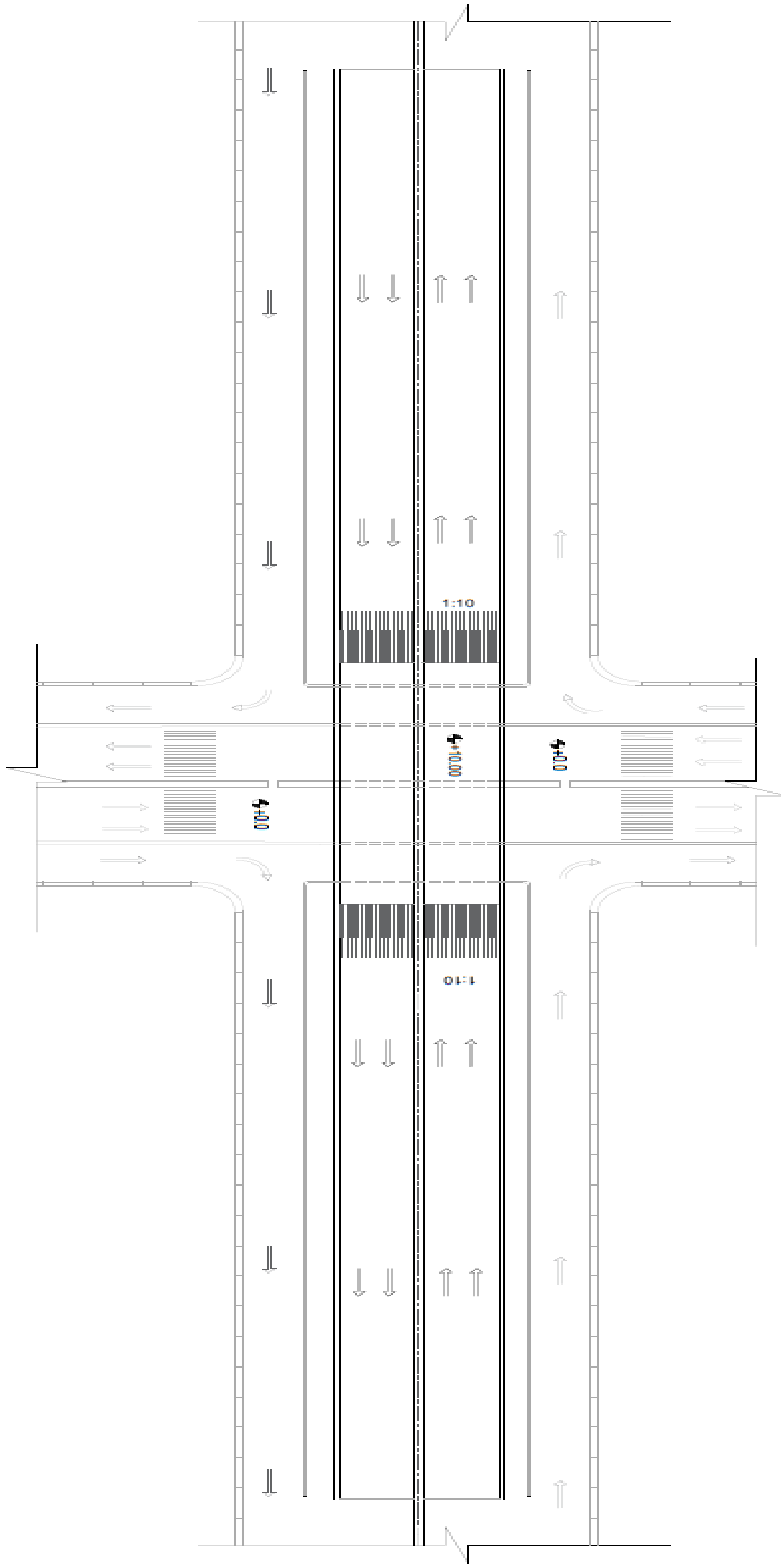


Figure2. 2 Site plan

2.3 First floor Plan

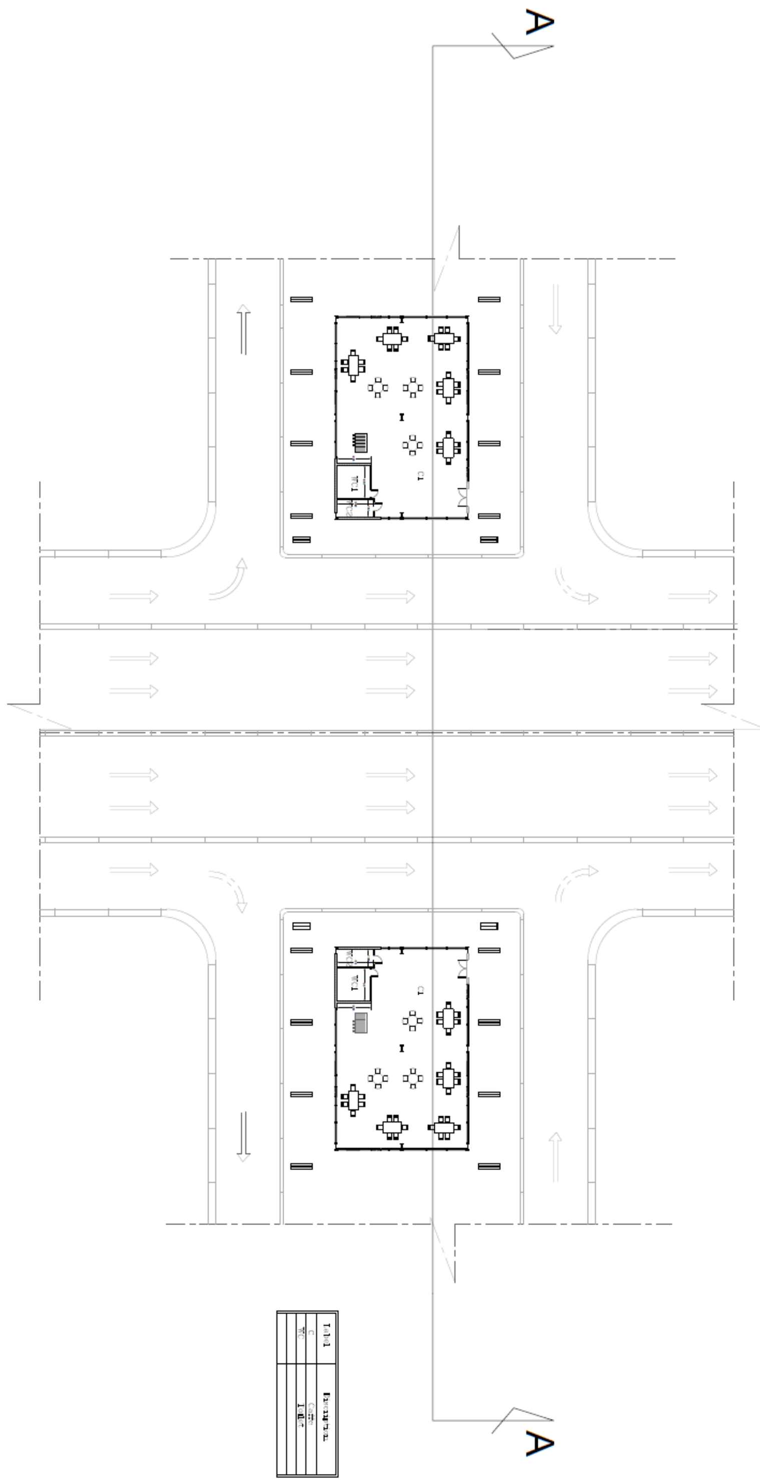


Figure2. 3 First floor Plan

2.4 Side View

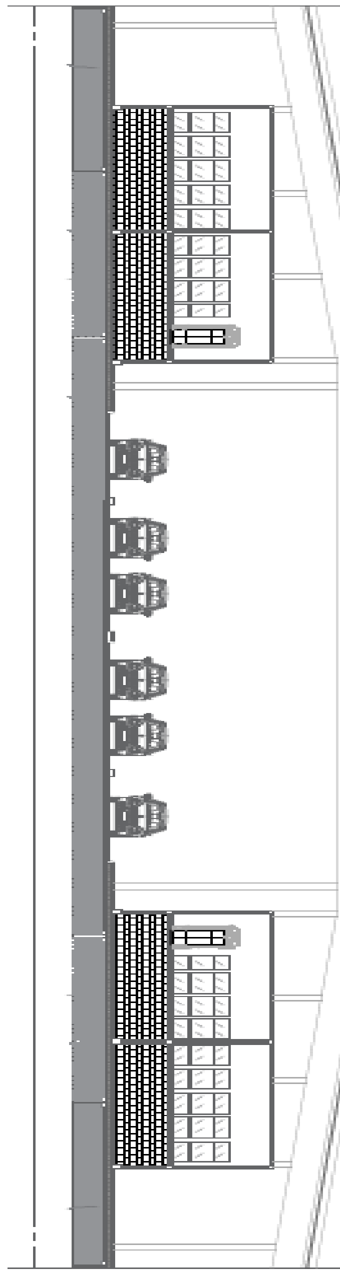


Figure2. 4 Side View

Chapter 3 (Structural Model)

3-1 Structural 3D

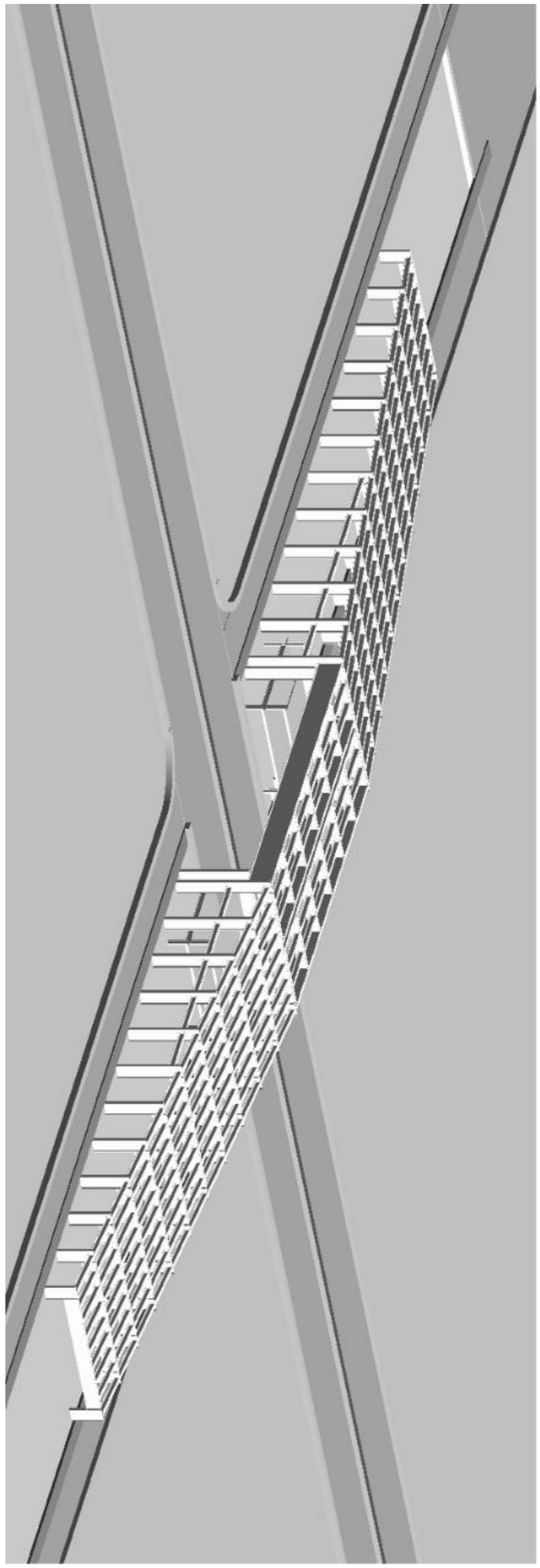


Figure 3. 1 Structural 3D

3-2 Inclined Part

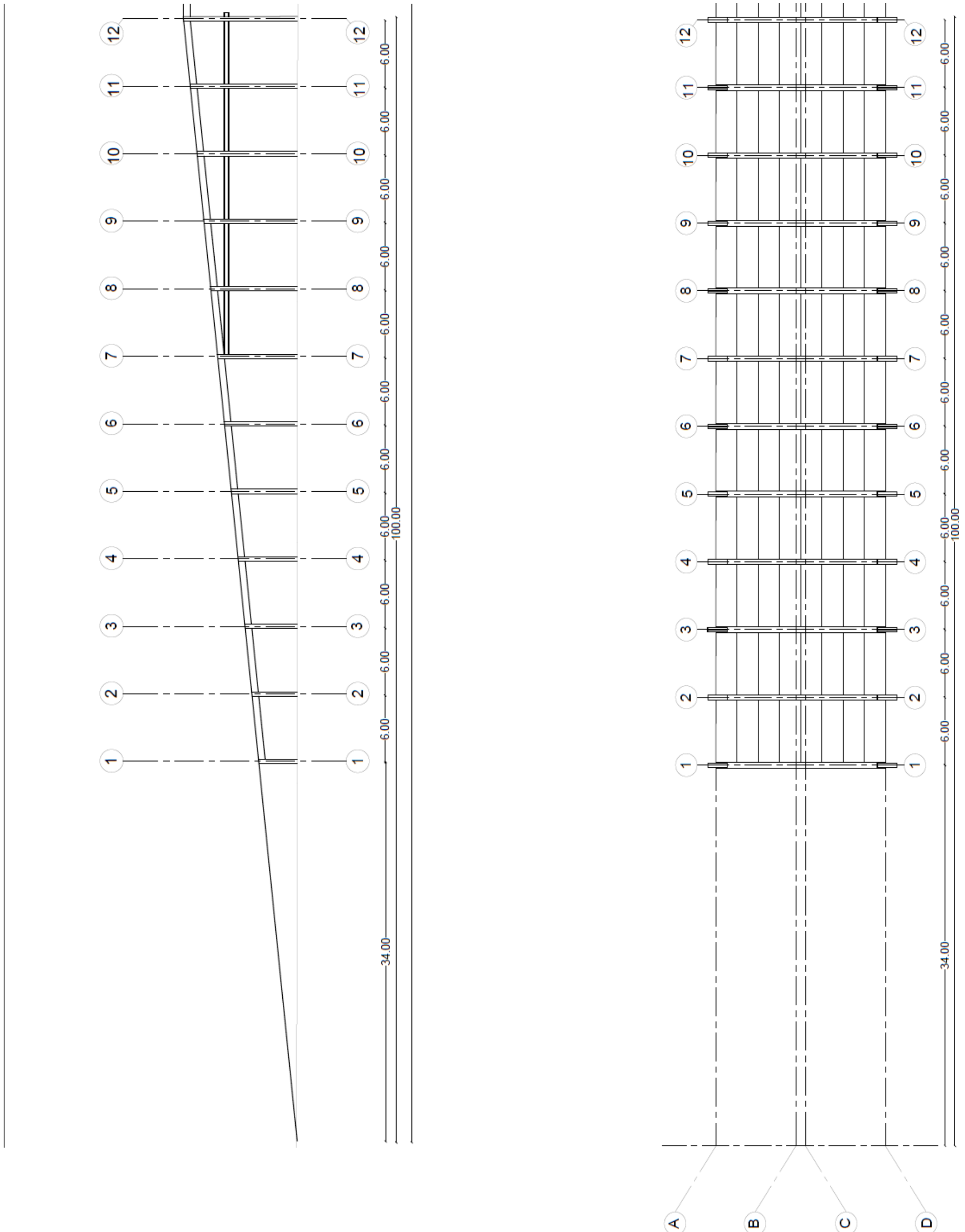


Figure 3. 2 Layout of inclined part plan& side view

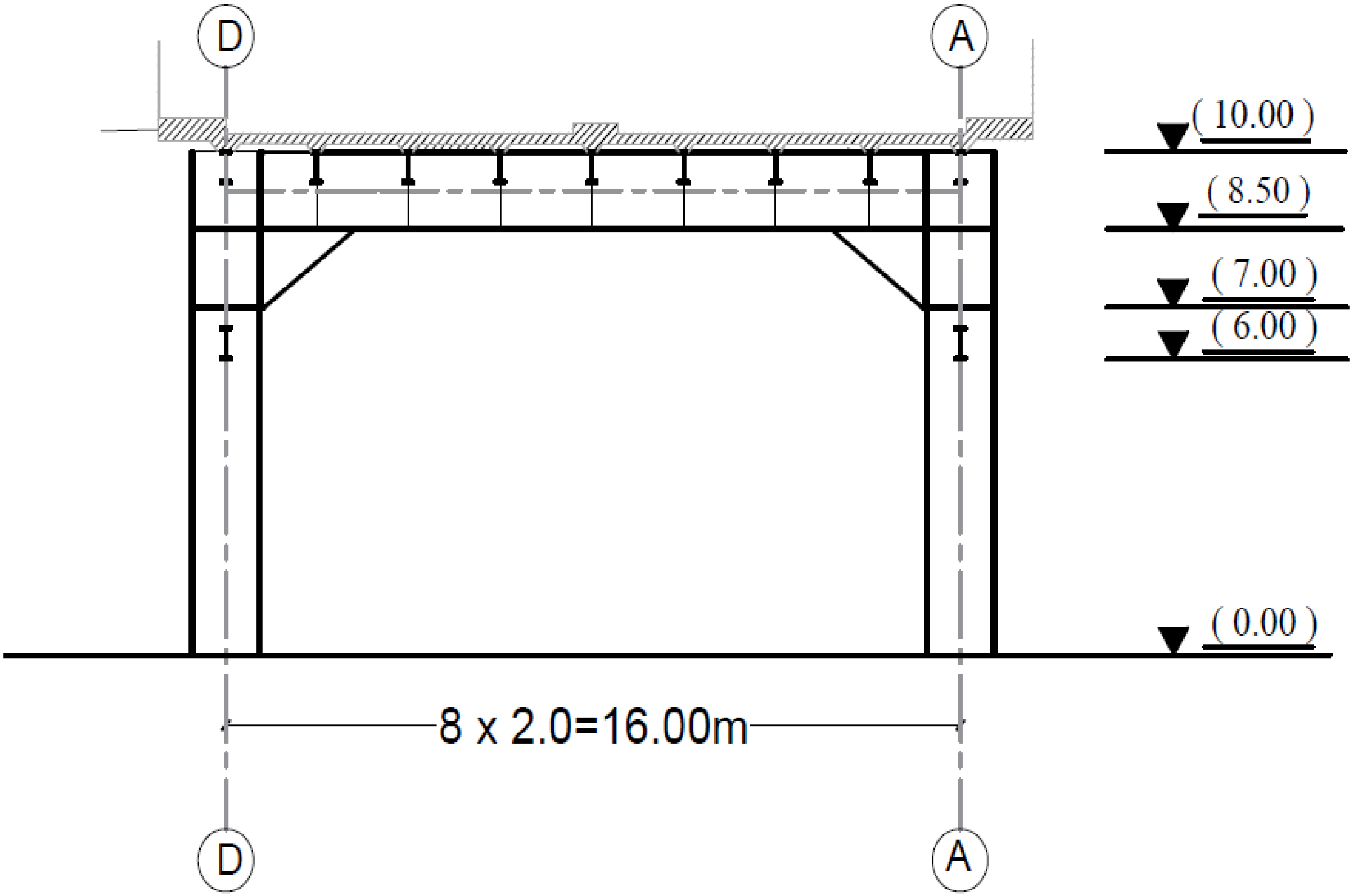


Figure 3. 3 Layout of inclined part elevation

3-3Deck Part (First Trial)

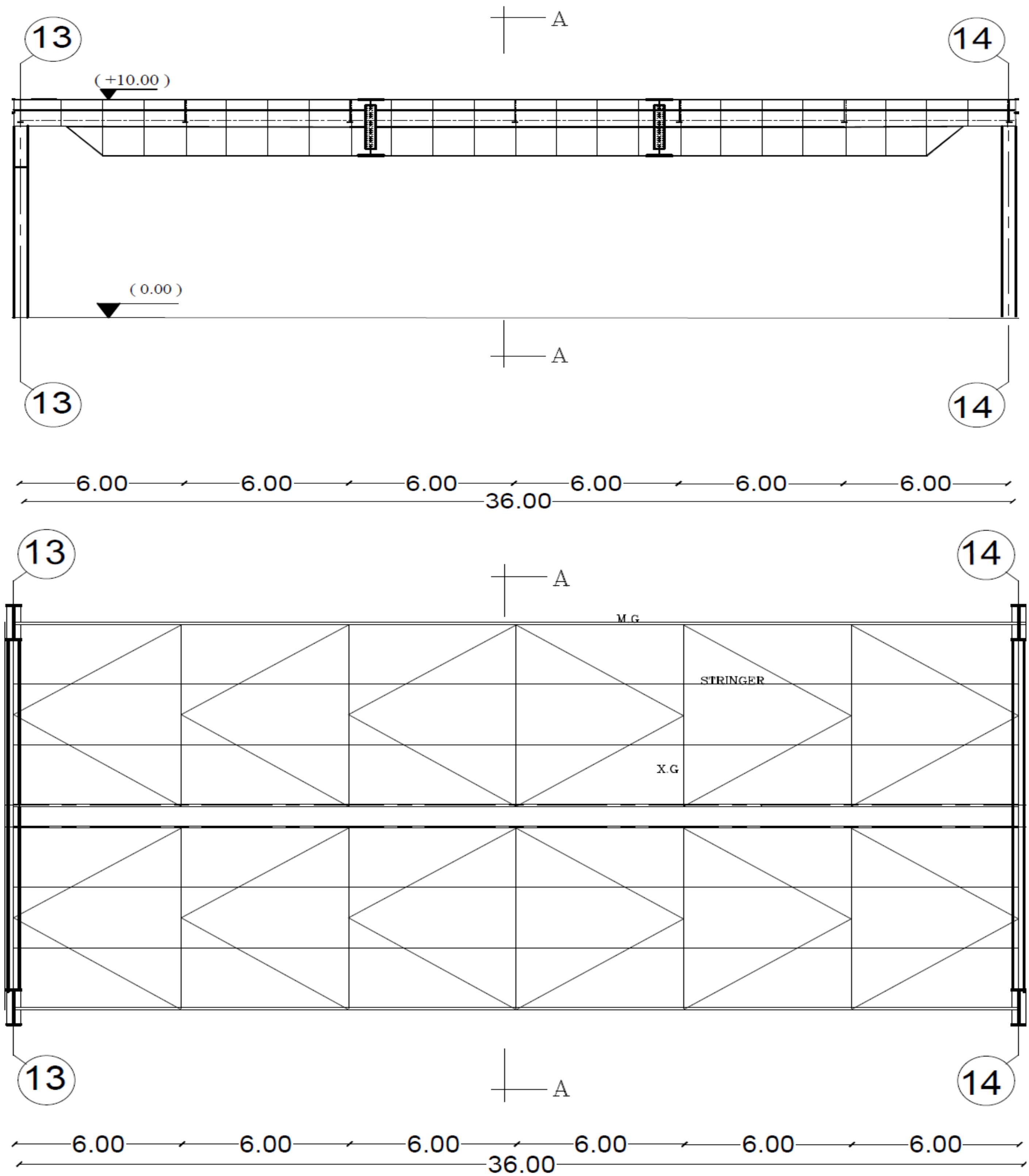


Figure 3. 4 Layout of deck part plan. side view & elevation (first trial)

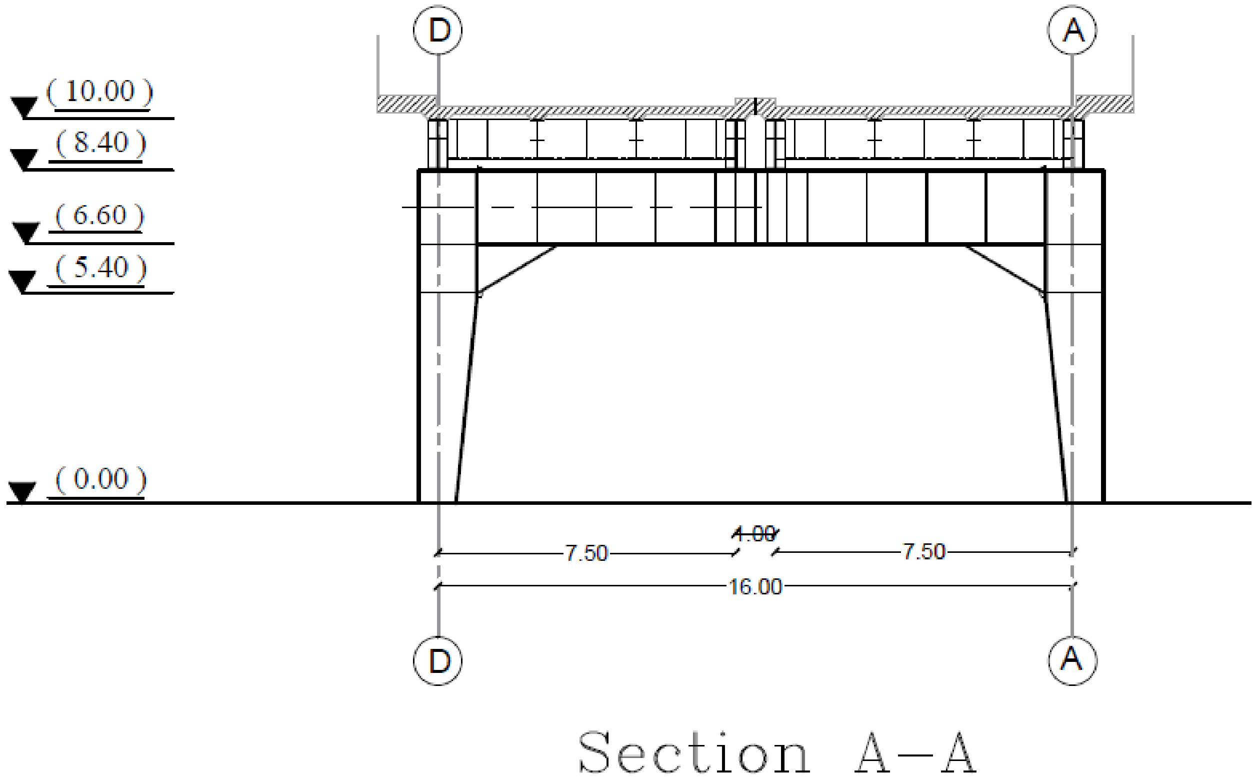


Figure 3. 5 Elevation (first trial)

3-4 Deck Part (Second Trial)

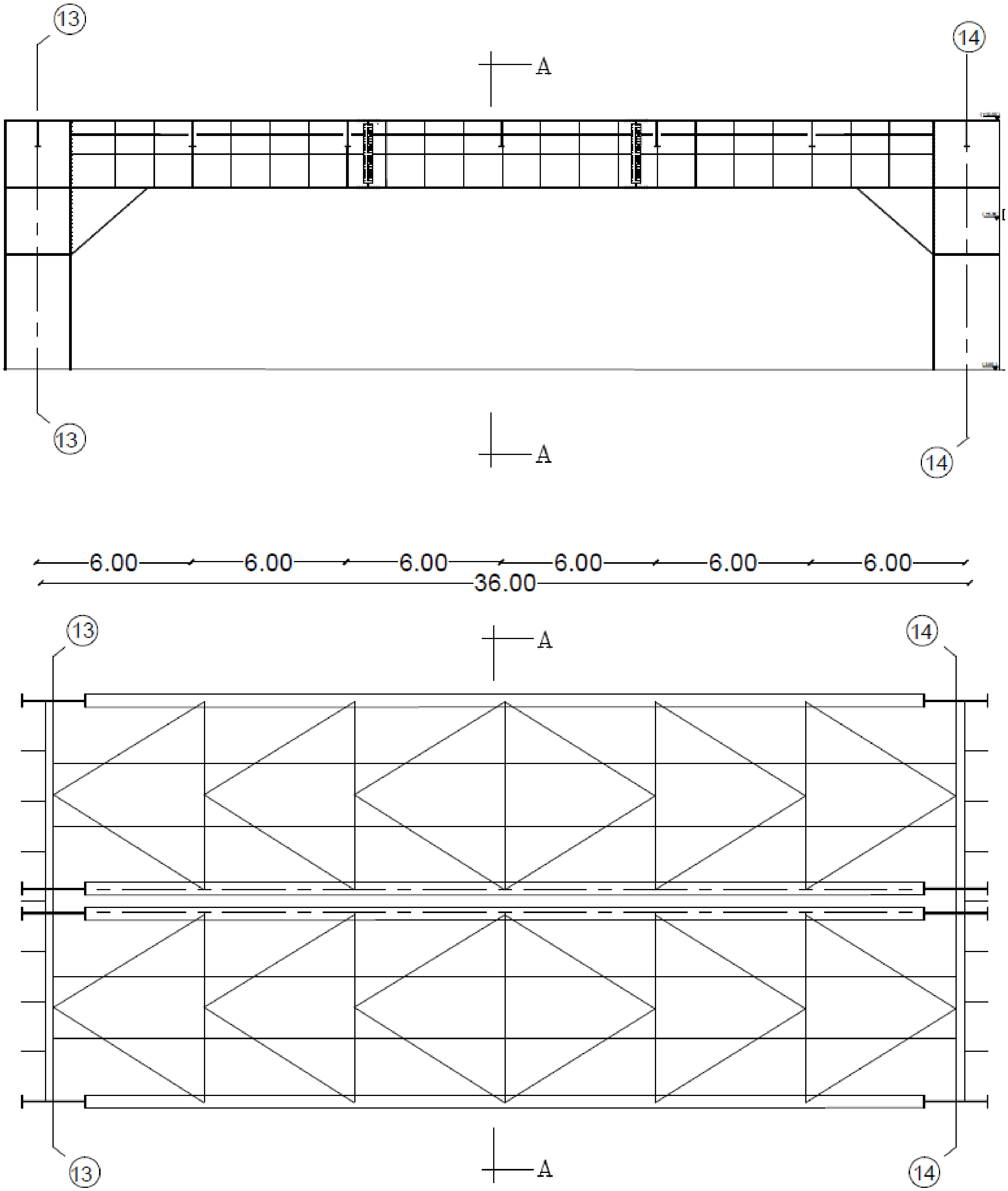


Figure 3. 6 Layout of deck part plan. side view & elevation (second trial)

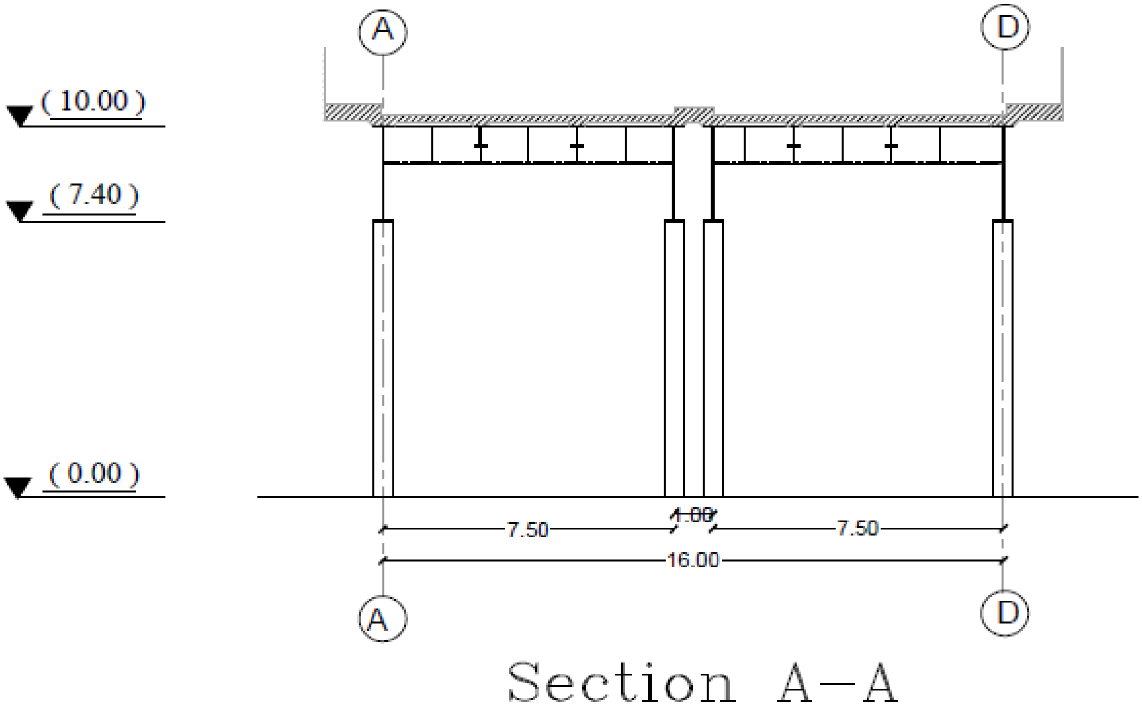


Figure 3. 7 Elevation (second trial)

3-5 Mezzanine Floor

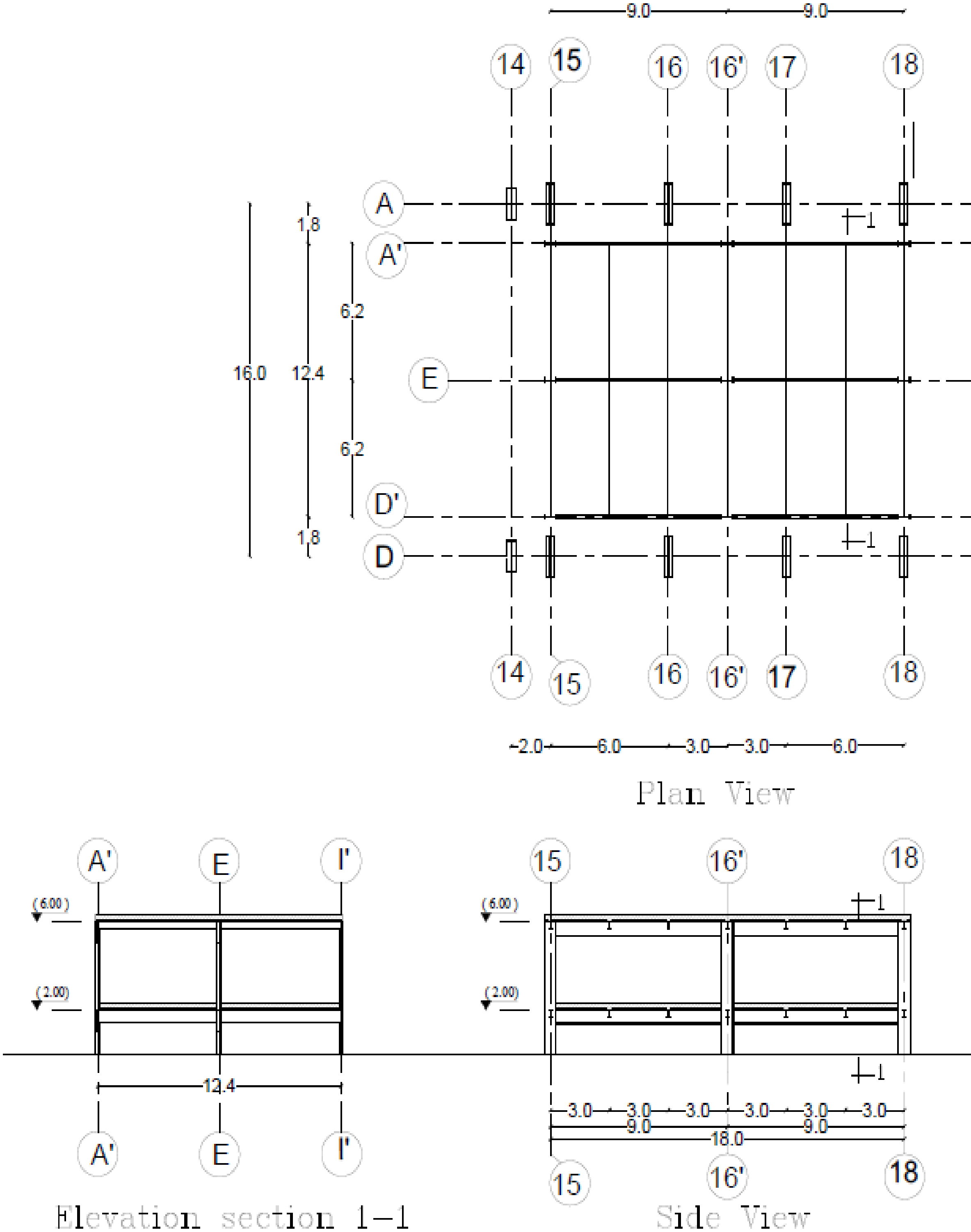


Figure 3. 8 Layout of mezzanine part plan. side view & elevation

Chapter 4 (Design of Inclined Part)

4.1 Stringer

Loading

Dead load

OW = 0.15 t/m^2

W $_{flooring}$ = 2.5*0.21+2.2*0.08= 0.7 t/m^2

W_{DL} = (0.15+0.7) *2= 1.55 t/m

M_{DL} = (1.55*6^2)/8 = 6.975 t.m

Q_{DL} = (1.55*6)/2 = 4.65 t

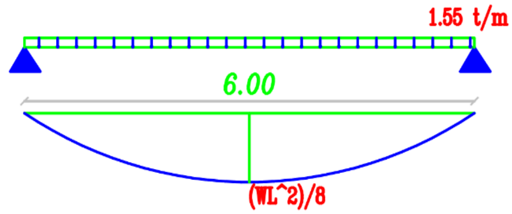


Figure4. 1 Max moment of dead load

Live load

R_{LL} = 15*1+10*0.5 = 20 t

A_{900} = 0.5*4*1 - (0.5*0.75*1.5) = 1.44 m^2

A_{250} = 0.5*0.75*1.5 = 0.56 m^2

W_{LL}= (0.9*1.44) +(0.25*0.56) = 1.43 t/m

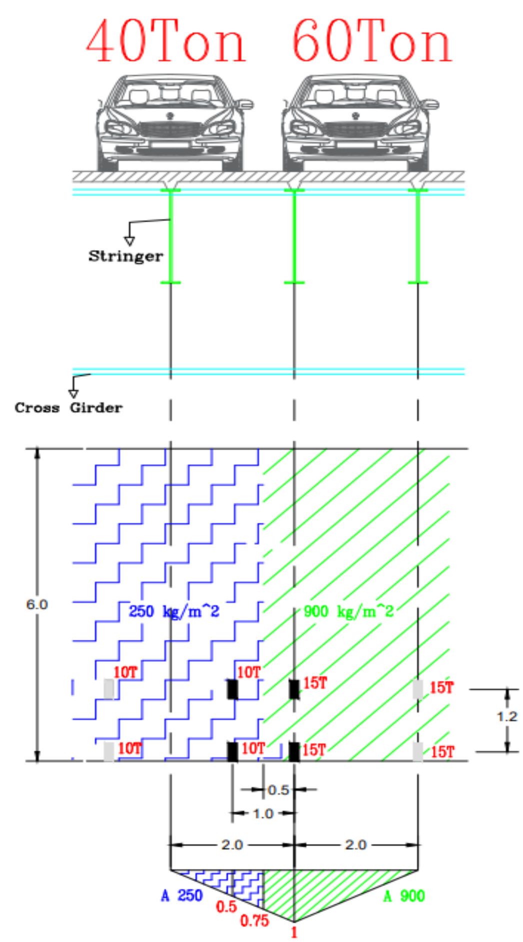


figure4. 2 Live load

Moment Live load

$M_{LL} = 20*(1.485+0.945) + 1.43*(0.5*1.485*6\ ) = 55$ t.m

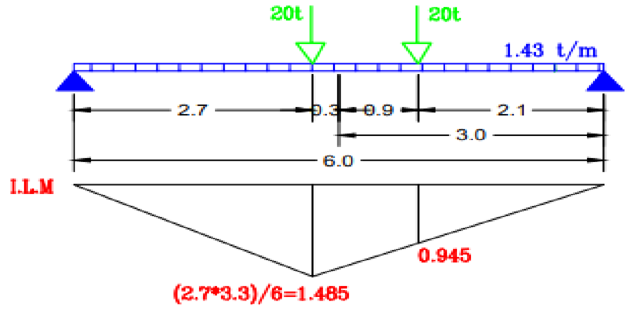


figure4. 3 Max moment of live load

Shear live load

$Q_{LL} = 20*(1+0.8) + 1.43*(0.5*1*6) = 40.5$ t

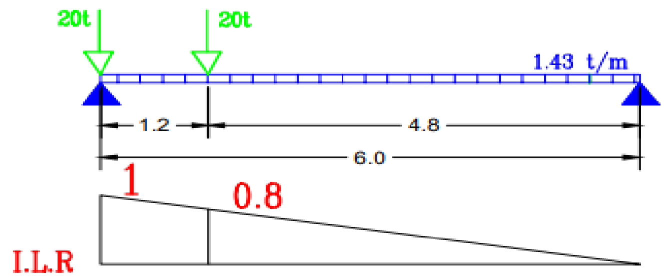


figure4. 4 Max shear of live load

Design Values

$M_{LL} = 55*0.9 = 49.5$t.m

$M_{min} = 7*0.9 = 6.3$t.m

$M_{max} = 49.5+6.3 = 55.8$t.m

$M_{max\ fatigue} = 6.3+0.6*49.5 = 36$t.m

$Q_{max} = 40.5+4.65 = 45.15$ t

[1]

Checks

SEC. BEAM ID :- (SB-1)

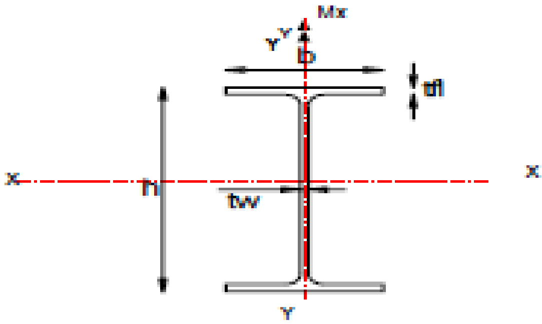
Steel grade	St.52			
F_y	-	3.60	t/cm^2	
F_u	-	5.20	t/cm^2	

1)- APPLIED FORCES :-

				Case Combination
M+ive	=	55.80	mt	a
Q	=	45.15	t	

2)- CHOISE OF SECTION :-

The section is Rolled section		IPE 600	
b_{FL}	=	220.0	mm
t_{FL}	=	19.0	mm
h	=	600.0	mm
t_{WEB}	=	12.0	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	6.00	m
Lu of compressin flange	=	0.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\bar{Y}$	=	30.00	cm
A	=	156.00	cm^2
I_x	=	92080.00	cm^4
I_y	=	3390.00	cm^4
S_x	=	3070.00	cm^3
r_x	=	24.30	cm
r_y	=	4.66	cm

CHECK COMPACTNESS :-

d_w/t_w	=	42.833	Web is	Compact
C/t_f	=	4.211	Flange is	Compact
The sec is	Compact			

CHECK NORMAL STRESSES :-

Cb	=	1.13					
Lu_{max}	=	$(20.b_f) / (\sqrt{fy})$ =		2.319	m	2.319	m
		$(1380.A_f) / (fy.d) \times C_b$ =		3.02	m		
		Lateral torsional buckling of comp.flange		There is no LTB			
Fltb	=	2.304	t/cm^2				
F_{bcx}	=	2.304	t/cm^2				
f_{bcx}	=	1.82	t/cm^2	<	2.304	SAFE	

CHECK SHEAR STRESSES :-

q_w	=	0.73	t/cm^2	<	1.260	SAFE

CHECK DEFLICTION DUO TO LIVE LOAD :-

W_{LL}	=	11.00	t/m			
Beam type	=	continues				
δ_{LL}	=	0.960	cm	<	1.00	SAFE

CHECK FATIGUE	Mll	=	49.50	mt		
Mfatigue	=	29.70	mt			
Ffatigue	=	0.97		<	1.680	SAFE

[1]

4.2Frame

Loading

Dead load

$OW = 0.3$ t/m^2

$W_{\text{flooring}} = 2.5*0.21+2.2*0.08= 0.7$ t/m^2

$W_{DL} = (0.3+0.7) *6= 4.5$ t/m

Live load

$R_{LL\,(15)} = 15*(1+0.8) = 20$ t
$R_{LL\,(10)} = 10*(1+0.8) = 18$ t
$R_{LL\,(5)} = 5*(1+0.8) = 9$ t

$W_{LL\,(900)} = 0.9*0.5*12*1 = 5.4$ t/m
$W_{LL\,(250)} = 0.25*0.5*12*1 = 1.5$ t/m

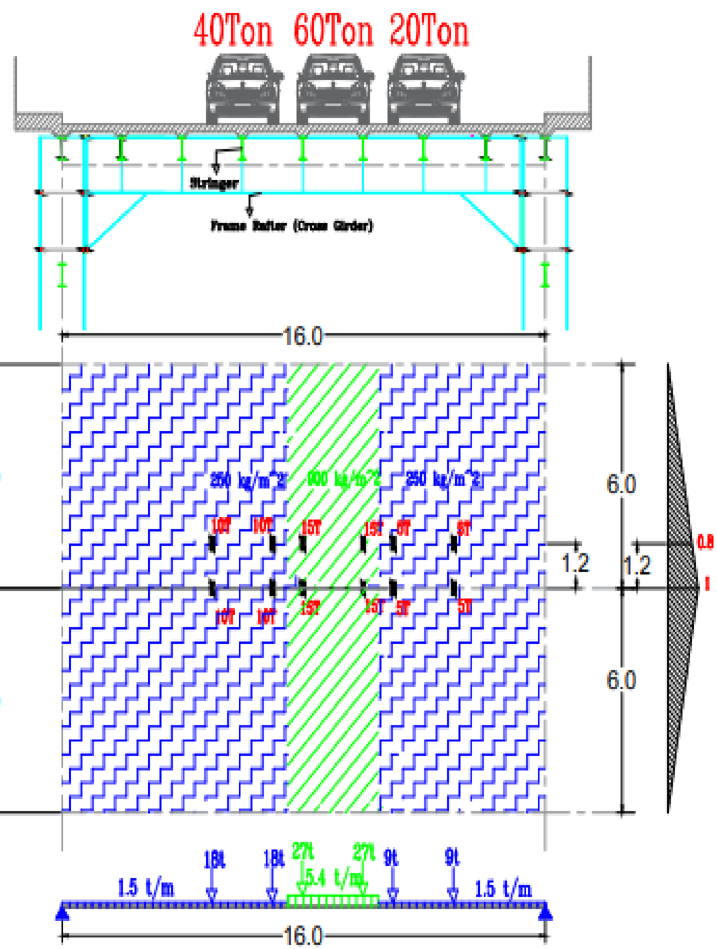


figure4. 5 Live load distribution [1]

[1]

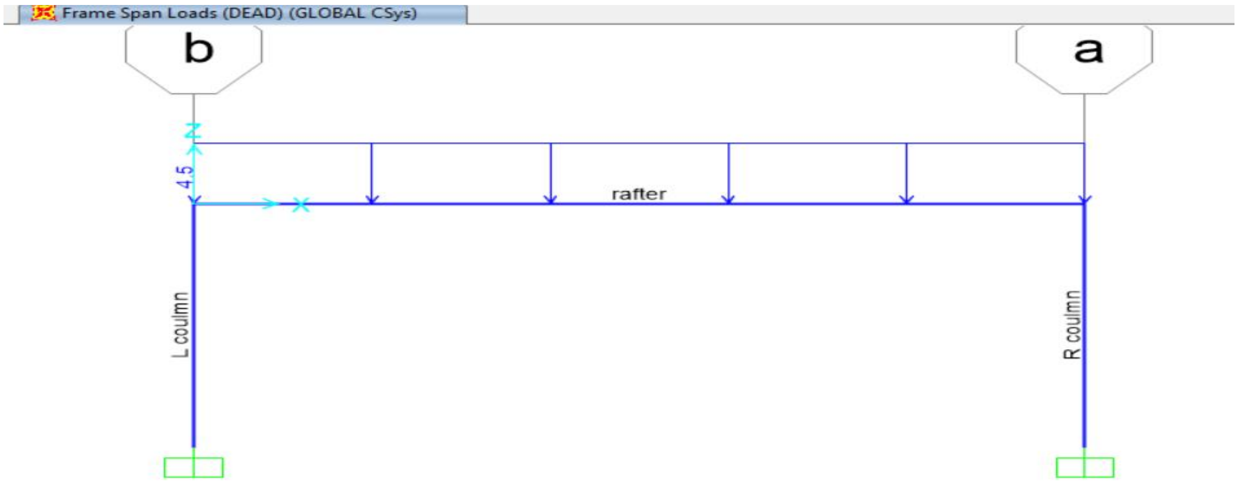


figure4. 6 Dead load

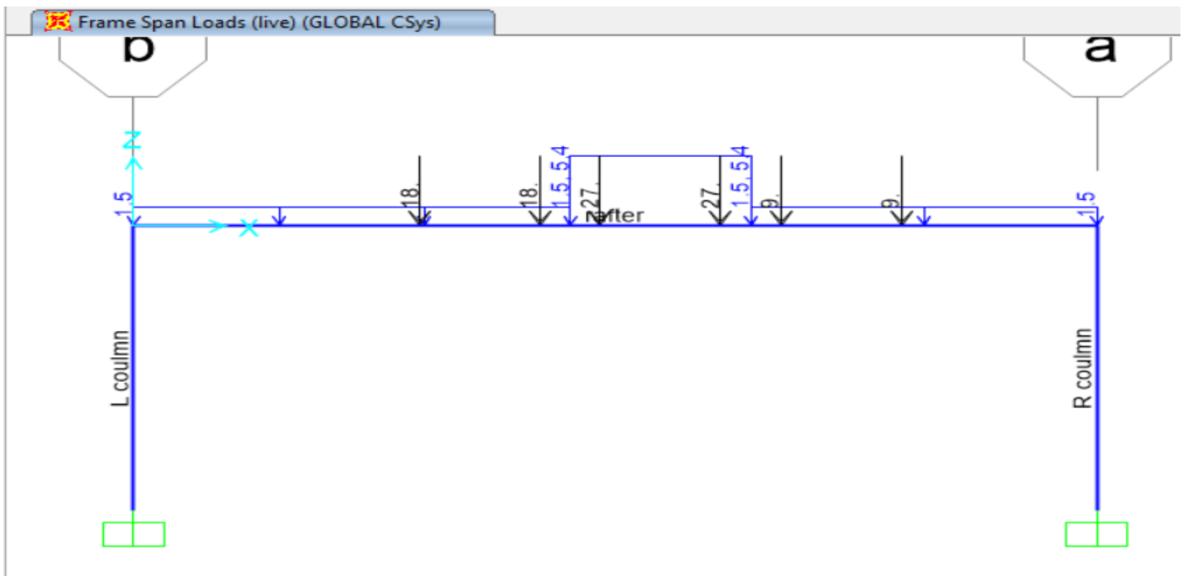


figure4. 7 Live loading of max moment case

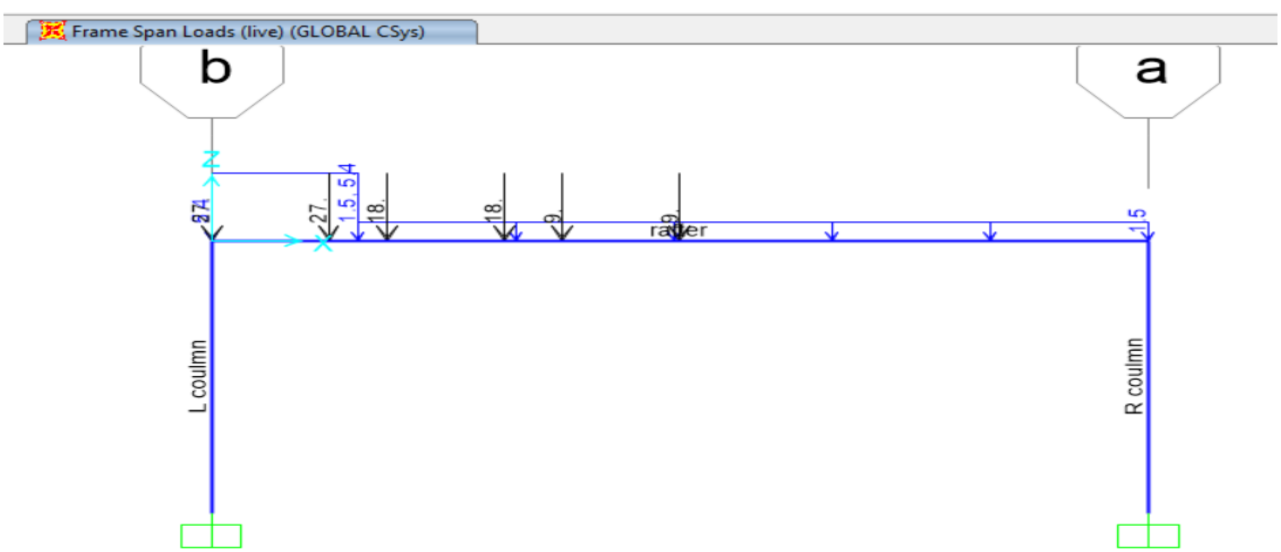


figure4. 8 Live loading of max shear case

SAP Analysis

1- At H frame = 10 m

MAX moment at H=10m					
Frame	**Station**	**OutputCase**	**N**	**Q**	**M**
Text	m	Text	Tonf	Tonf	Tonf-m
column L	10	DL+LL	-105.4476	39.7817	264.2801
column L	10	DL+LL	-105.4476	39.7817	-133.537
coulmn R	0	DL+LL	-110.2524	-39.7817	-266.943
coulmn R	10	DL+LL	-110.2524	-39.7817	130.8739
rafter	0	DL+LL	-39.7817	-105.4476	-264.28
rafter	7.75	DL+LL	-39.7817	-20.9976	300.264
rafter	16	DL+LL	-39.7817	110.2524	-266.943

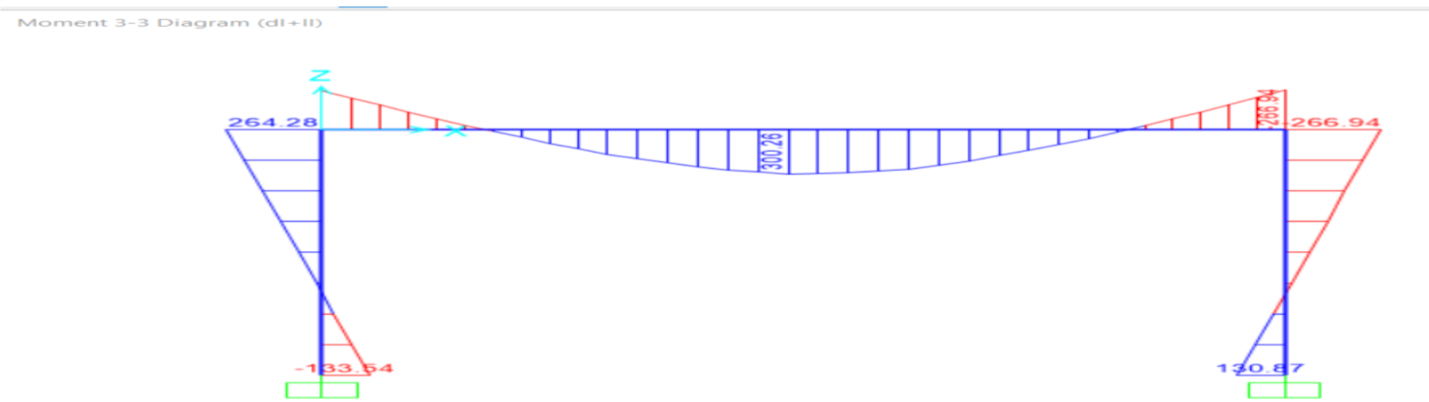
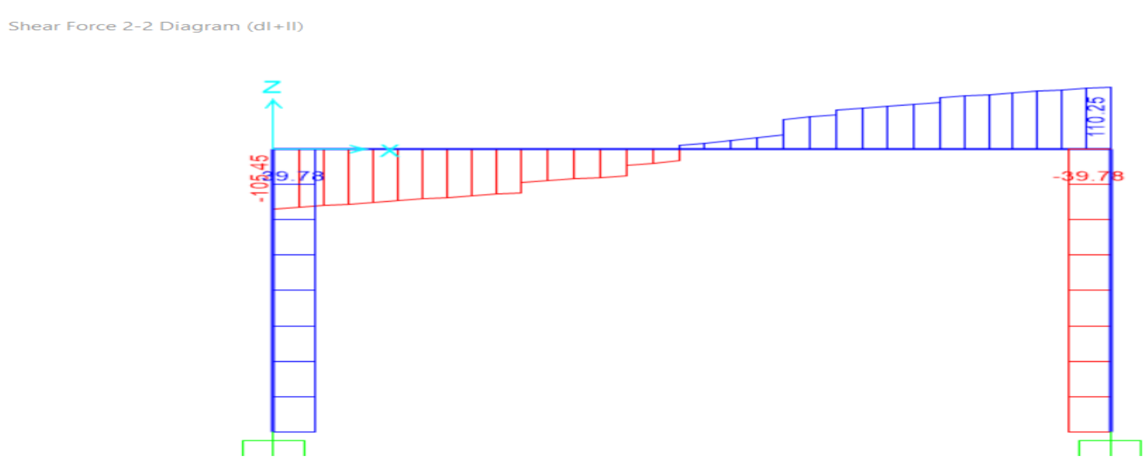


figure4. 9 Max moment on frame from loading of max moment



Ffigure4. 10 Max shear on frame from loading of max moment

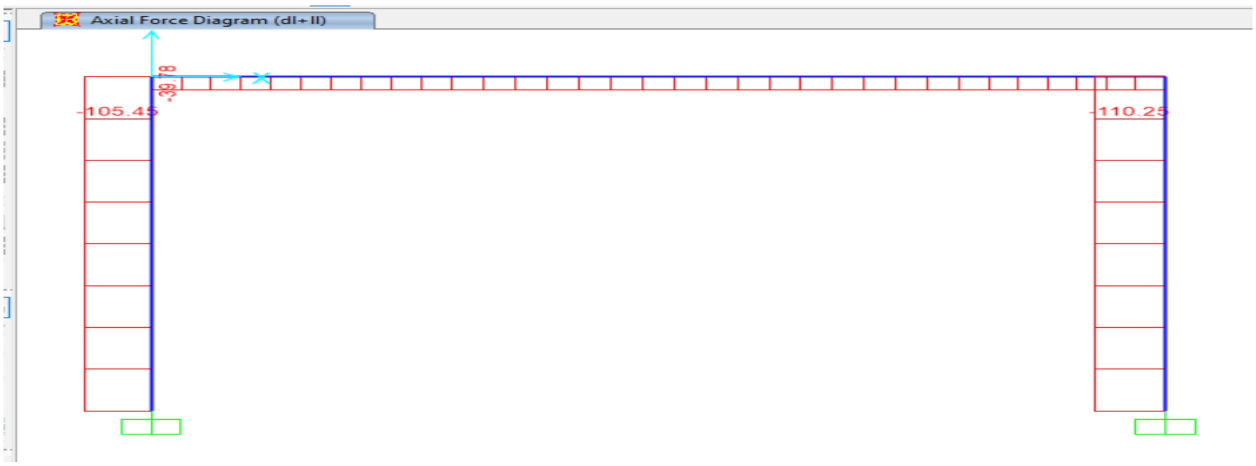


figure4. 11 Normal force on frame from loading of max moment

MAX Shear at H=10m					
Frame	**Station**	**OutputCase**	**N**	**Q**	**M**
Text	m	Text	Tonf	Tonf	Tonf-m
column L	0	DL+LL	-146.0915	27.9088	197.16574
column L	10	DL+LL	-146.0915	27.9088	-81.92243
coulmn R	0	DL+LL	-67.6585	-27.9088	-175.51346
coulmn R	10	DL+LL	-67.6585	-27.9088	103.57471
rafter	0	DL+LL	-27.9088	-119.0915	-197.16574
rafter	7.75	DL+LL	-27.9088	7.6585	178.33338
rafter	16	DL+LL	-27.9088	67.6585	-175.51346

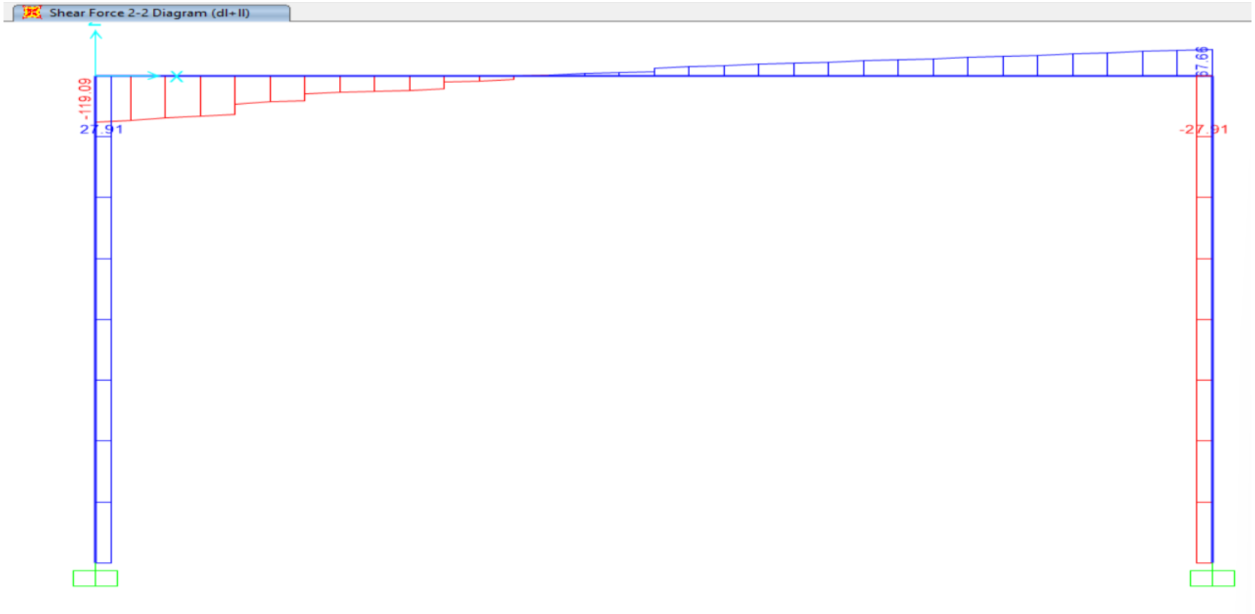


figure4. 12 Max shear on frame from loading of max sheer

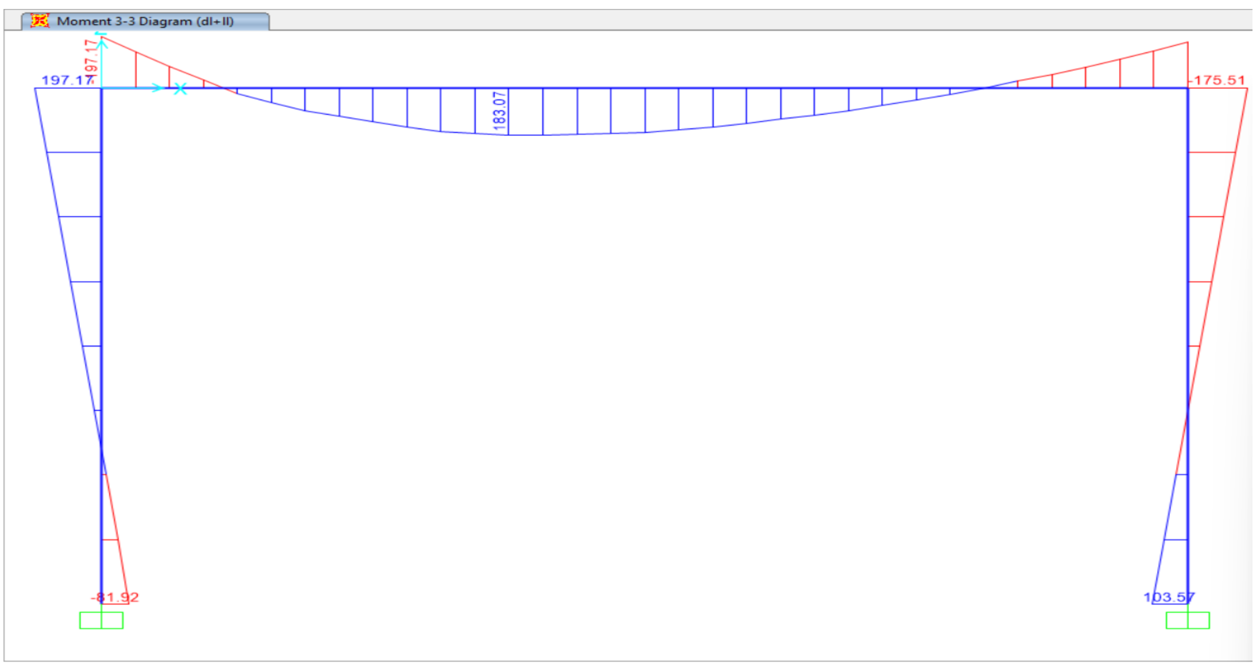


figure4. 13 Max moment on frame from loading of max shear

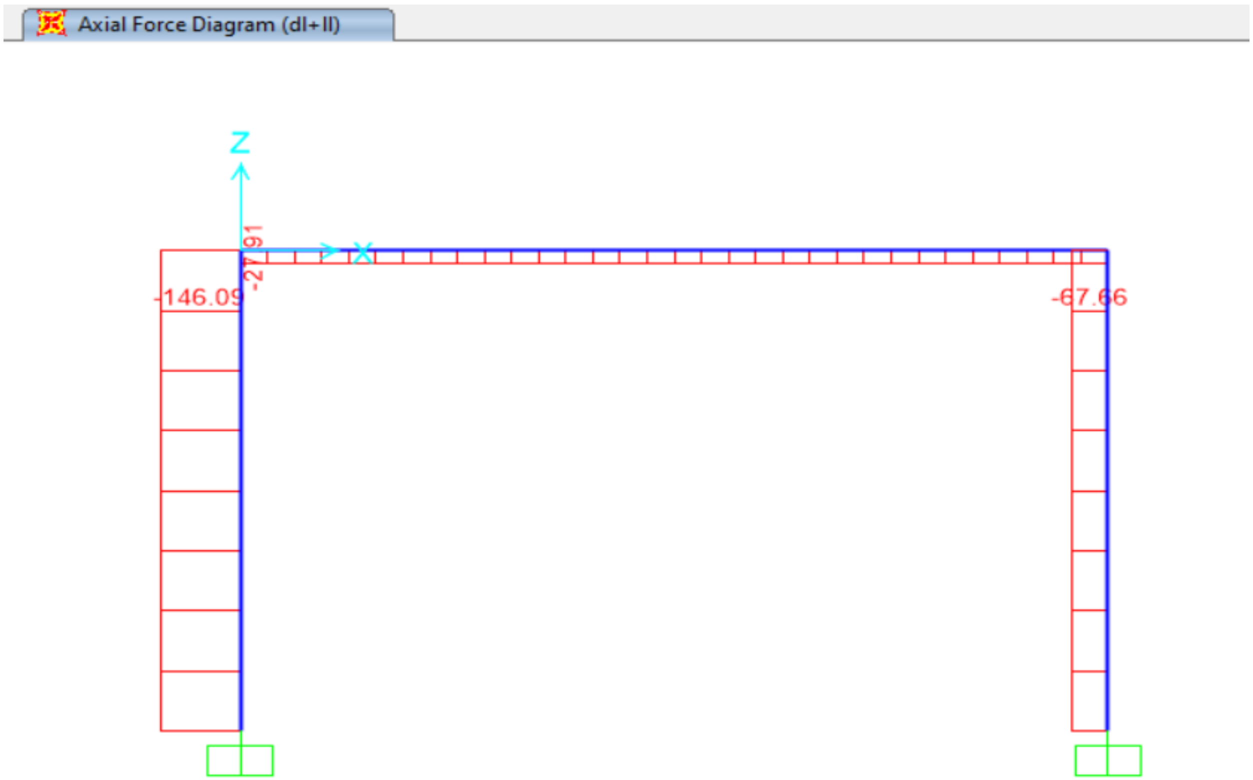


figure4. 14 Normal force on frame from loading of max shear

Checks

Design column H frame = 10m

COLUMN ID :- (SC-1)

DESIGN CASE :- (D.L+L.L) a

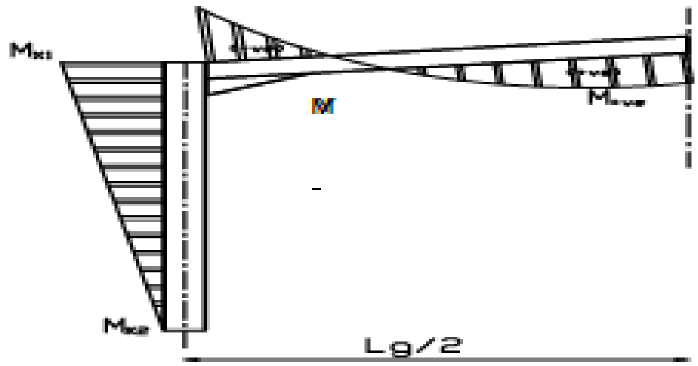
Steel grade St.52

F_y	=	3.60	t/cm^2
F_u	=	5.20	t/cm^2

1)- APPLIED FORCES :-

Mx_1	=	267.00	mt
Mx_2	=	-133.00	mt
M_y	=	0.00	mt
N	=	146.00	t

if Mx2 at the other side put -ve sign



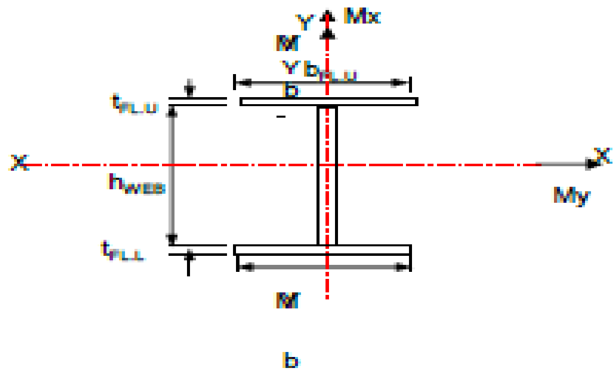
2)-DIM. OF SECTION :-

The section is Bulit up section

$b_{FL.U}$	=	400.00	mm
$t_{FL.U}$	=	24.00	mm
h_{WEB}	=	1500.00	mm
t_{WEB}	=	22.00	mm
$b_{FL.L}$	=	400.00	mm
$t_{FL.L}$	=	24.00	mm

3)-COLUMN DATA :-

Total length of column	=	10.00	m
Lu act.of comp. Flange	=	4.00	m
Length subject to buckling in plan	=	6.00	m
Length subject to buckling out plan	=	6.00	m
Length of girder	=	16.00	m
Ix (rafter)	=	1209990.00	cm^4
Base type	=	Fixed	



4)- PROPERTIES OF SECTION :-

A	=	522.00	cm^2
$\overline{Y}$	=	77.40	cm
$\overline{X}$	=	20.00	cm
I_x	=	1733678.64	cm^4
I_y	=	25733.10	cm^4
S_x	=	22398.95	cm^3
S_y	=	1286.66	cm^3
r_x	=	57.63	cm
r_y	=	7.02	cm

5)- CHECK COMPACTNESS :-

d_w/t_w	=	68.182	Web is	Non compact
C/t_f	=	8.333	Flange is	Non compact

The sec is Non compact

6)- CHECK STRESSES :-

f_{bcx}	=	1.19	t/cm^2
f_{bcy}	=	0.00	t/cm^2
F_{bcy}	=	2.59	t/cm^2
f_{ca}	=	0.28	t/cm^2
α	=	-0.50	

C_b	=	1.30				
Lu_{max}	=	$(20.b_f) / (\sqrt{fy})$ =	4.216	m	3.193	m
		$(1380.A_f) / (fy.d) \times C_b$ =	3.19	m		
	(Lateral torsional buckling of comp.flange)		There is L.T.B			
Fltb	=	2.088	t/cm^2			
F_{bcx}	=	2.088	t/cm^2			
G_A	=	1	(according to base type)			
G_B	=	2.29				
K	=	1.50	E.C.O.P (P.61)			
L_{bin}	=	7.20	m			
L_{bout}	=	6.00	m			
λ_{in}	=	12.49				
λ_{out}	=	85.46				
			λ_{max} =	85.46	< 180	SAFE
F_c	=	1.11	t/cm^2			
f_{ca} / F_c	=	0.25				
C_{mx}	=	0.85				
A_1	=	1.00				
C_{my}	=	1.00				
A_2	=	1.37				

<u>APPLYING THE INTERACTION EQUATION :</u>

$(f_{ca} / F_c) + (f_{bcx} / F_{bcx}) A_1 +(f_{bcy} / F_{bcy}) A_2$ = 0.822 < 1.00 SAFE

[1]

Design rafter

MAIN. BEAM ID :- (MB-1)

Steel grade	St. 52		
F_y	=	3.60	t/cm^2
F_u	=	5.20	t/cm^2

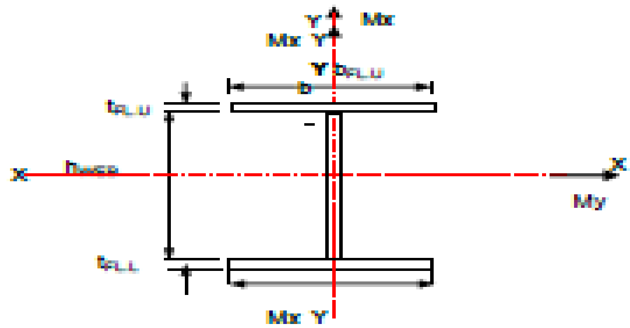
1)- APPLIED FORCES :- **Case Combination**

M+ive	=	300.00	mt	a
Q	=	119.00	t	

2)- CHOISE OF SECTION :-

The section is Built up section

$b_{FL.U}$	=	380	mm
$t_{FL.U}$	=	18	mm
h_{web}	=	1500	mm
t_{web}	=	15	mm
$b_{FL.L}$	=	380	mm
$t_{FL.L}$	=	18	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	16.00	m
Lu of compressin flange	=	2.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\overline{Y}$	=	76.80	cm
A	=	361.80	cm^2
I_x	=	1209990.74	cm^4
S_x	=	15755.09	cm^3

CHECK COMPACTNESS :-

d_w/t_w	=	100.00	Web is	Non compact
C/t_f	=	10.14	Flange is	Non compact
The sec is	Non compact			

CHECK NORMAL STRESSES :-

Cb	=	1.13				
Lu_{max}	=	$(20.b_f) / (\sqrt{fy})$ =	4.006	m	1.975	m
		$(1380.A_f) / (fy.d) \times C_b$ =	1.98	m		

There is Lateral Torsional Buckling in comp.flange

Fltb	=	2.088	t/cm^2			
F_{bcx}	=	2.088	t/cm^2			
f_{bcx}	=	1.90	t/cm^2	<	2.088	SAFE

CHECK SHEAR STRESSES :-

q_w	=	0.53	t/cm^2	<	1.260	SAFE

CHECK DEFLICTION DUO TO LIVE LOAD :-

$W_{L.L}$	=	7.15	t/m			
Beam type	=	continues				
$\delta_{L.L}$	=	2.401	cm	<	2.67	SAFE

CHECK FATIGUE	Mll	=	229.00	mt		
Mfatigue	=	137.40	mt			
Ffatigue	=	0.87		<	1.680	SAFE

Chapter 5 (Design of Deck Part (First Trial))

5.1Main girder

Loading

Dead load

$W_{flooring} = 0.7$ t/m^2

$W_{sin} = 150+4(33)+0.03(33)^2 = 314.88$ kg/m^2

$W_{dl} = 0.7*3.75+0.314*3.75 = 39$ t/m^2

$M_{dl} = 3.9*33^2/8-2*(3.9*1.5^2/8) = 526.5$ t.m

$Q_{dl} = 64.4$ t.m

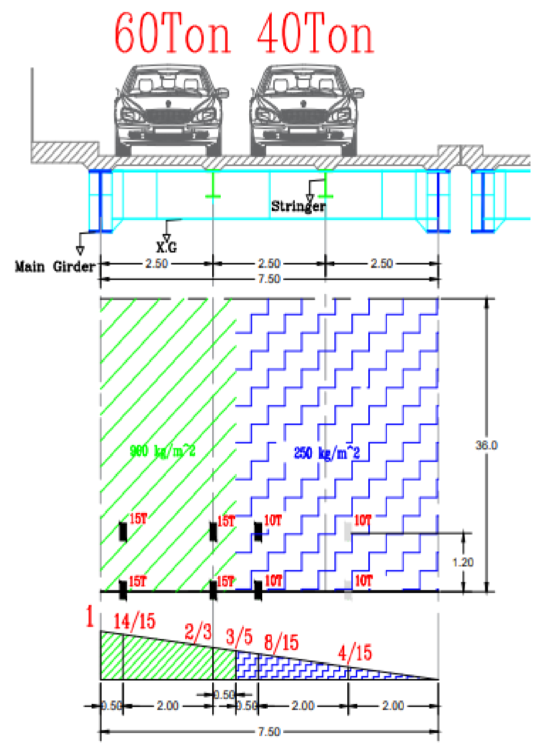


Figure5. 1 Live load of main girder

Live load

$R_{LL} = 15*(0.667+0.933)+10*(0.533+0.2667) = 32$ t

$A_{900} = 0.5*1*7.5-1.35 = 2.4$ m^2

$A_{250} = 0.2*0.6*4.5 = 1.35$ m^2

$W_{LL} = (0.9*2.4) +(0.25*1.35) = 2.5$ t/m

Moment Live load

$M_{LL} = 32*(8.24+7.65) + 2.5*(0.5*33*8.24) = 848.38$ t.m

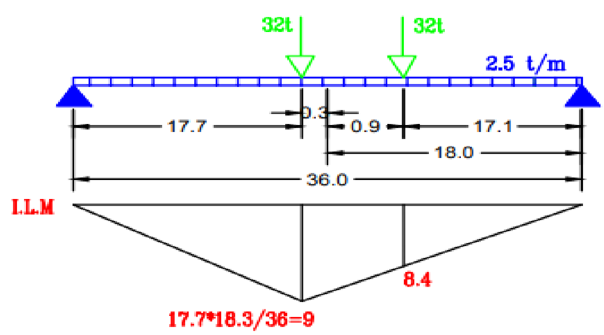


Figure5. 2 Max moment live load of main girder

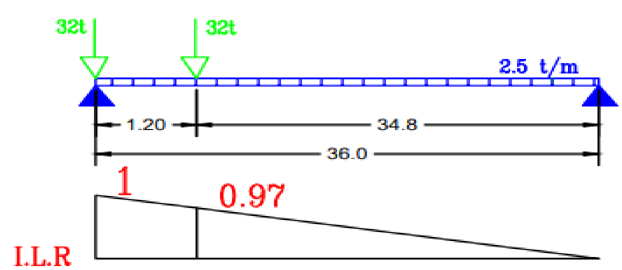
Shear live load

$Q_{R\,LL} = 32*(1+0.96)+2.5*0.5*1*33+0.5*0.045*1.5*25 = 104$ t

Design Values

$M_{max} = 526.5+848.38 = 1374.6$ t.m

$Q_{max} = 104.4+64.4 = 168.4$ t



[1]

Figure5. 3 Max shear live load of main girder

Checks

Steel grade	St.52		
F_y	=	3.60	t/cm^2
F_u	=	5.20	t/cm^2

1)- APPLIED FORCES :-

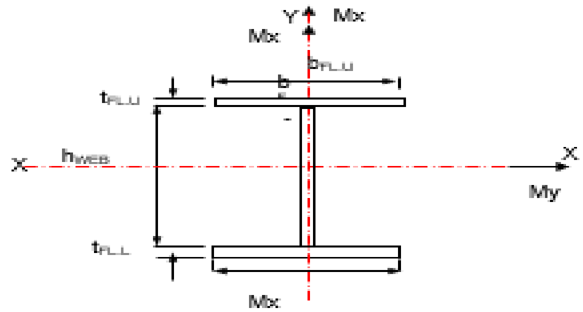
Case Combination

M+ive	=	1374.00	mt	a
Q	=	168.00	t	

2)- CHOISE OF SECTION :-

The section is Built up section

b_{FLU}	=	500	mm
t_{FLU}	=	30	mm
h_{web}	=	2500	mm
t_{web}	=	30	mm
b_{FLL}	=	500	mm
t_{FLL}	=	30	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	36.00	m
Lu of compressin flange	=	6.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\overline{Y}$	=	128.00	cm
A	=	1050.00	cm^2
I_x	=	8707150.00	cm^4
S_x	=	68024.61	cm^3

CHECK COMPACTNESS :-

d_w/t_w	=	83.33	Web is	Non compact
C/t_f	=	7.83	Flange is	Compact

The sec is Non compact

CHECK NORMAL STRESSES :-

Cb = 1.30

Lu_{max} = $(20.b_f)/(\sqrt{fy})$ = 5.270 m ; $(1380.A_f)/(fy.d) \times C_b$ = 2.99 m → 2.990 m

There is Lateral Torsional Buckling in comp.flange

Fltb	=	2.088	t/cm^2			
F_{bcx}	=	2.088	t/cm^2			
f_{bcx}	=	2.02	t/cm^2	<	2.088	SAFE

CHECK SHEAR STRESSES :-

q_w	=	0.22	t/cm^2	<	1.260	SAFE

Fatigue

$F_{sr\ act}$ = (848.38*0.6*100)/68024 =0.74 t/cm^2 < $F_{sr\ all}$ =1.68 t/cm^2 safe

Deflection

Δ = (5/384) *[(6.23*10^-2*3600^4)/2100*8707150] = 5.2 < 3600/600 =6 cm safe

Design of End Bearing Stiffener

1) Length:

$$b_{st} \geq \frac{250}{30} + 5 = 13.3cm$$

$$b_{st} \leq \frac{50-2.5}{2} = 23.75cm$$

2) Design as compression section:

F= Q_{design}=168.4t , L_b=0.8*250=200cm

Area= $12tw^2+bst*t_{st}*2$

Stress:- A= $\frac{Q}{Fall}$ $12(2.5)^2+23.75*t_{st}*2= \frac{168.4}{0.58*2.8}$ $\longrightarrow$ Take tst=1.8cm

3) Check of local Buckling:

$$\frac{23.75}{1.8} = 13.19cm < \frac{25}{\sqrt{2.8}} = 14.9\ cm$$ Safe

4) Stress: $\lambda x=\frac{Lb}{ix}$, $ix = \sqrt{\frac{Ix}{A}}$

$A=12(2.5)^2+23.75*1.8*2=160.5cm^2$

$$Ix= \frac{(2*23.75+2.5)^3}{12} = 10416.67cm^4$$

$$ix=\sqrt{\frac{10416.67}{160.5}} = 8.1cm\ , \quad \lambda x=\frac{200}{8.1} = 24.8cm$$

$Fall=1.6-0.000085(24.8)^2 =1.547\ t/cm^2$

$$Fact=\frac{168.4}{160.5} = 1.05\frac{t}{cm^2} < 1.547\frac{t}{cm^2}$$ $\longrightarrow$ Safe

5) Design Weld: $\frac{Qdesign}{4S*hw} < 0.2Fu$

$\frac{168.4}{4S*250} = 0.2(4.4)$ $\longrightarrow$ Sw=0.2cm $\longrightarrow$ Take minimum =0.6cm

Design of Horizontal Stiffener

1) d/5: Iy > 4hw*tw^3

$$Iy=\frac{50^3*1.8}{12}=18750cm^4 > 4(250)*2.5^3=15625\ cm^4\ Ok\ SAFE$$

2) d/2: Iy > hw*tw^3

$$Iy=\frac{(16*2+2.5)^3*1.2}{12}=4106.36\ cm^4 > 3906.25\ cm^4\ OK\ SAFE$$

[1]

5-2 Design of Stringer

Loading

Dead load

OW = 0.15 t/m^2

$W_{flooring}$ = 2.5*0.21+2.2*0.08= 0.7 t/m^2

W_{DL} = (0.15+0.7) *2.5= 2.125 t/m

M_{DL} = (2.125*6^2)/8 = 9.56 t.m

Q_{DL} = (2.125*6)/2 = 6.375 t

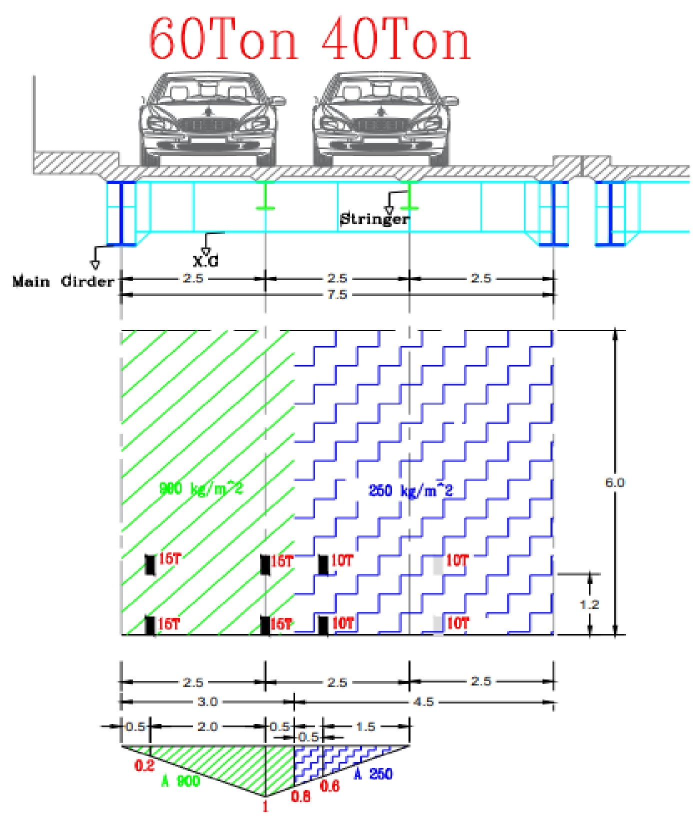


Figure5. 4 live load of stringer

Live load

R_{LL} = 15*(1++0.2)+10*0.6 = 24 t

A_{250} = 0.5*2*0.8 = 0.8 m^2

A_{900} = (0.5*5*1)-0.8 = 1.7 m^2

W_{LL}= (0.9*1.7) +(0.25*0.8) = 1.73 t/m

Moment Live load

M_{LL} = 24*(1.485+0.945) + 1.73*(0.5*1.485*6) = 66 t.

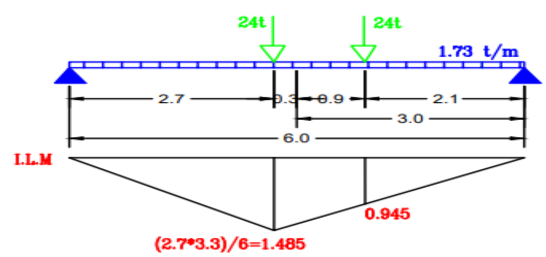


Figure5. 5 Max moment live load of stringer

Shear live load

Q_{LL} = 24*(1+0.8) + 1.37*(0.5*1*6) = 48.39 t

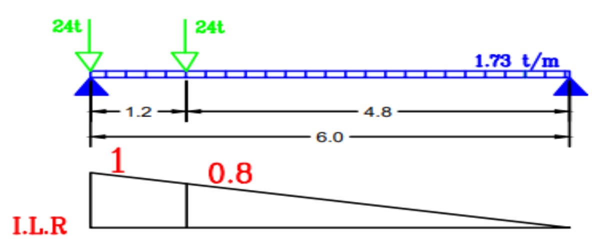


Figure5. 6 Max shear live load of stringer

Design Values

M_{LL} = 66+0.9 = 59.4 t.m
M_{min} = 9.56*0.9 = 8.6 t.m
M_{max} = 59.4+8.6 = 68 t.m
$M_{max\ fatigue}$ = 8.6 +0.6*59.4 = 44.24 t.m
Q_{max} = 48.39+6.375 = 54.76 t

Choose section

Use HEA 550

Properties

Zx =4150 cm^3

Ix=111900 Cm^4

Section compact   Fbc= 0.64Fy

Check

Stress

F_{act} = (68*100)/4150 =1.63 t/cm^2 < $F_{b\ all}$ = 0.64*3.6=2.3 t/cm^2   safe

Shear

qact = 54.7/43.8*1.25 = 0.99 t/cm^2 < q_{all} = 0.35*3.6 =1.26   safe

Fatigue

$F_{sr\ act}$ = (95.4*0.6*100)/4150 =0.86 t/cm^2   < $F_{sr\ all}$ =1.68 t/cm^2   safe

Deflection

Δ = (5/384) *[(13.2*10^-2*600^4)/2100*111900] =0.94 < 600/6=1 safe

[1]

5.3 Design of cross girder

Loading

Dead load

OW = 0.3 t/m^2

W flooring = 2.5*0.21+2.2*0.08= 0.7 t/m^2

W_{DL} = (0.3+0.7) *6= 4.5 t/m

M_{DL} = (4.5*7.5^2) /8= 31.6 t.m

Q_{DL} = (4.5*7.5) /2= 16.8 t

Live load

$R_{LL\,(15)}$ = 15*(1+0.8) = 27 t

$R_{LL\,(10)}$ = 10*(1+0.8) = 18 t

$W_{LL\,(900)}$ = 0.9*0.5*12*1 = 5.4 t/m

$W_{LL\,(250)}$ = 0.25*0.5*12*1 = 1.5 t/m

Moment Live load

$A_{1.5}$ = 0.5*1.62*3.65+0.5*0.74+0.85=3.271m^2

$A_{5.4}$= 0.5*1.85*7.5-3.271=3.665M^2

M LL = 27*1.85+27*0.74+18*1.4+18*0.51 +5.4*3.665+3.271*1.5= 129 t.m

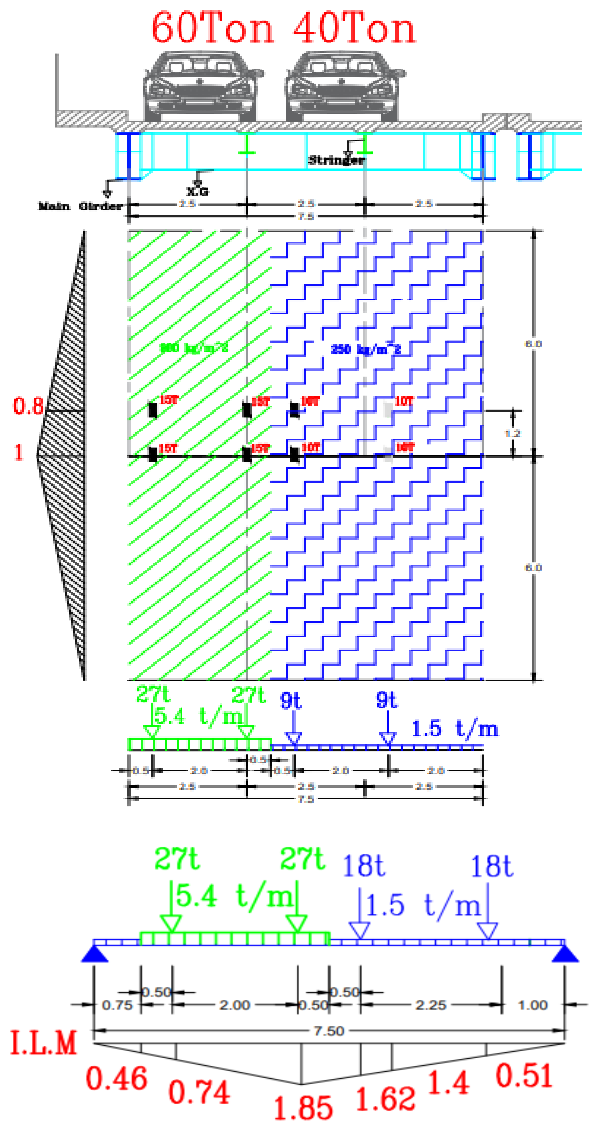


Figure5. 8 Live load of cross girder

Sheer Live load

$A_{1.5}$= 0.5*0.6*4.5=1.35M^2

$A_{5.4}$= 0.5*1*7.5-1.35=2.4m^2

Q_{LL} = 27*(1+0.733) +18(0.53+0.4) +5.4*2.4+1.35*1.5
= 78.5 t.m

Design value

M_{max} =129+31.6=160.6t.m

Q_{max} =16.8+78.5=95.3t.m

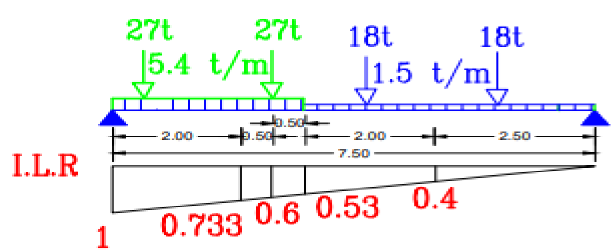


Figure5. 9 Max shear live load of cross girder

Checks

MAIN. BEAM ID : (MB-1)

Steel grade	St.52		
F_y	=	3.60	t/cm²
F_u	=	5.20	t/cm²

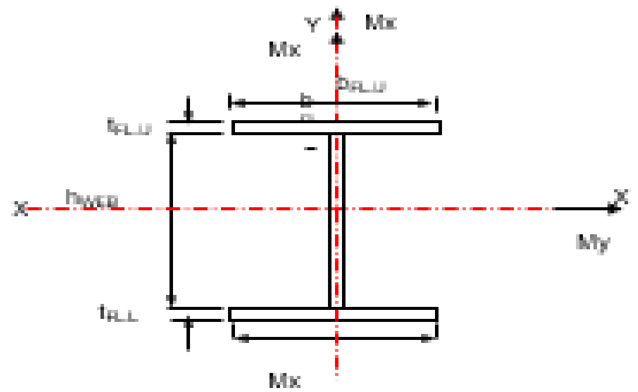
1)- APPLIED FORCES :- **Case Combination**

M+ive	=	160.60	mt	a
Q	=	95.30	t	

2)- CHOISE OF SECTION :-

The section is Built up section

b_{FLU}	=	200	mm
t_{FLU}	=	30	mm
h_{web}	=	1000	mm
t_{web}	=	20	mm
b_{FLL}	=	200	mm
t_{FLL}	=	30	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	7.50	m
Lu of compressin flange	=	2.50	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\overline{Y}$	=	53.00	cm
A	=	320.00	cm²
I_x	=	485026.67	cm⁴
S_x	=	9151.45	cm³

CHECK COMPACTNESS :-

d_w/t_w	=	50.00	Web is	Compact
C/t_f	=	3.00	Flange is	Compact
The sec is	Compact			

CHECK NORMAL STRESSES :-

Cb	=	1.30				
Lu_{max}	=	$(20.b_f)/(\sqrt{fy})$ =	2.108	m	2.108	m
		$(1380.A_f)/(fy.d) \times C_b$ =	2.99	m		

There is Lateral Torsional Buckling in comp.flange

Fltb	=	2.088	t/cm²			
F_{bcx}	=	2.088	t/cm²			
f_{bcx}	=	1.75	t/cm²	<	2.088	SAFE

CHECK SHEAR STRESSES :-

q_w	=	0.48	t/cm²	<	1.260	SAFE

Check Fatigue = (0.6*129)/9151.45 = 0.84 < 1.64 saf

[1]

5-4 Design of upper bracing systems

1) Case of unloaded bridge (without the live part): (q = 200 kg/m2)

$W_1 = 0.2 * [2.5+0.2+0.25] = 0.59$ t/m

$R_I = 0.59 * 36/2 = 10.62$ t

$F_I = R_I / \pm 2$ sine $\qquad F_I = 10.62 / \pm 2 \cos(32) = 5.94$ t

2) Case of loaded bridge (with live part): (q = 100 kg/m2)

$W_2 = 0.1 * [2.5+0.2+0.25+3] = 0.59$ t/m = WI = 0.59 t/m

$R_2 = 0.59 * 36/2 = 10.62$ t

F2 = RI/ ±2 sine $\qquad$ F2 = 10.62/ ±2 cos (32) = 5.94 t

3) Case of unloaded bridge during construction: (q = 200 kg/m2)

W3 = 0.2*0.7*2*[2.5] = 0.7 t/m

R3 = 00.7 * 36/2 = 12.6 t

F3 = RI/ ±2 sine $\qquad$ F3 = 12.6/ ±2 cos (32) = 7.42 t

Note: In case of Deck Bridge, the case of during construction (case 3) usually gives the critical force in bracing members. But, as mentioned before, the allowable stress in this case will be increased by 25 %. So, the designed force can be obtained as follows:

F3 = 7.42/ 1.25 = 5.9 t

Design force F *des.* = ± 5.9 t (Case 1)

Design as compression members:

$L = 7\ M$ Design force $F_{des.} = \pm 5.9\ t$

$L_{bin} = L = 7\ M$ $L_{OUT} = 1.2*7 = 8.4\ M$

Choose 2 angles 180 x 180 x 16

1) Check as compression member:

$\lambda x = L_{bin} / r_x$ $\lambda x = 700 / 5.51 = 127.27 > 140$ ok

$\lambda y = L_{out} / r_u$ $\lambda x = (1.2 * 700) / 0.45*18 = 103 > 140$ ok

$F_c = (7500/\lambda^2)*0.85 = 0.39$ t/cm2

$F_{act} = 75.93/2*55.40 = 0.053$ t/cm2 < $F_c = 0.39$ t/cm2 safe

2) Check as tension member

- Check depth:

  $L/D = 700/18 = 38 < 40$ ok

- Check stresses: (Use M20 bearing type)

  $A_{net} = 2*[55.4-2*2.2*1.8] = 94.9$ t/cm2

  $F_{act} = 5.93/94.9 = 0.062$ t/cm2 < $0.85*1.6 = 1.36$ t/cm2 ok

[1]

Chapter 6 Design of mezzanine

6-1 Design of Secondary beam

1. Loading (first floor)

$T_{RC} = 12$ CM

$W_{rc} = 0.12*2500 = 300$ kg/m^2

$W_t = (w_{rc} + cover) * a + ow + LL*a + O.W$

$W_t = (300+150)*3 + 50 + 400*3 = 2600$ kg/m

2. design Values

$M_X = 2.6*6.2^2 /8 = 12.493$ t.m

$Q_y = 2.6*6.2 /2 = 8.06$ t

3. Design

SEC. BEAM ID :- (SB-1)

Steel grade		St.44	
F_y	=	2.80	t/cm^2
F_u	=	4.40	t/cm^2

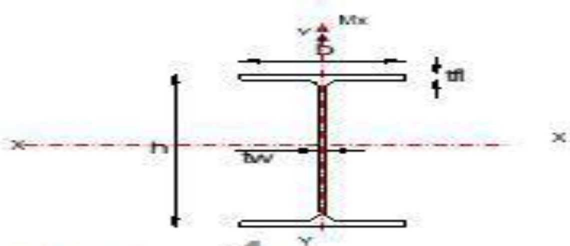
1)- APPLIED FORCES :- **Case Combination** a

M+ive	=	12.49	mt
Q	=	8.06	t

2)- CHOISE OF SECTION :-

The section is Rolled section **IPE 360**

b_{FL}	=	170.0	mm
t_{FL}	=	12.7	mm
h	=	360.0	mm
t_{WEB}	=	8.0	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	6.20	m
Lu of compressin flange	=	0.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

Y	=	18.00	cm
A	=	72.70	cm^2
I_x	=	16270.00	cm^4
I_y	=	1040.00	cm^4
S_x	=	904.00	cm^3
r_x	=	15.00	cm
r_y	=	3.79	cm

CHECK COMPACTNESS :-

d_w/t_w	=	37.325	Web is	**Compact**
C/t_f	=	4.961	Flange is	**Compact**
The sec is	**Compact**			

CHECK NORMAL STRESSES :-

Cb	=	1.30				
Lu_{act}	=	$(20.b_f)/(\sqrt{fy})$ =	2.032	m	2.032	m
		$(1380.A_f)/(fy.d) \times C_b$ =	3.84	m		
Lateral torsional buckling of comp.flange			**There is no LTB**			
Fltb	=	1.792	t/cm^2			
F_{bcx}	=	1.792	t/cm^2			
f_{bcx}	=	1.38	t/cm^2	<	1.792	**SAFE**

CHECK SHEAR STRESSES :-

q_{act}	=	0.34	t/cm^2	<	0.980	**SAFE**

CHECK DEFLICTION DUO TO LIVE LOAD :-

W_{LL}	=	1.20	t/m			
Beam type	=	simple				
δ_{LL}	=	0.676	cm	<	3.10	**SAFE**

1. Loading (second floor)

$W_t = (300+150)*3 + 50+150*3 = 1850$ kg/m

2. Design Values

$M_X = 1.85*6.2^2 /8 = 8.89$ t.m

$Q_y = 1.85*6.2 /2 = 5.7$ t

3. Design

SEC. BEAM ID :- (SB-1)

Steel grade	St.44		
F_y	=	2.80	t/cm²
F_u	=	4.40	t/cm²

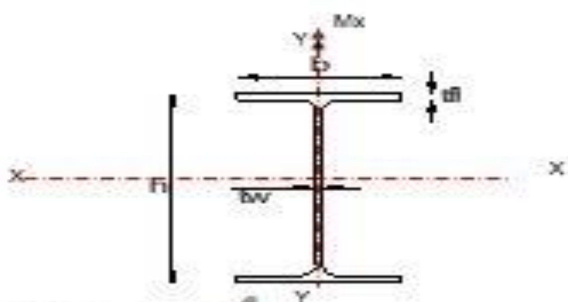
1)- APPLIED FORCES :- Case Combination

M+ive	=	8.89	mt	a
Q	=	5.70	t	

2)- CHOISE OF SECTION :-

The section is Rolled section IPE 300

b_{FL}	=	150.0	mm
t_{FL}	=	10.7	mm
h	=	300.0	mm
t_{WEB}	=	7.1	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	6.20	m
Lu of compression flange	=	0.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\bar{Y}$	=	15.00	cm
A	=	53.80	cm²
I_x	=	8360.00	cm⁴
I_y	=	604.00	cm⁴
S_x	=	557.00	cm³
r_x	=	12.50	cm
r_y	=	3.35	cm

CHECK COMPACTNESS :-

d_w/t_w	=	35.014	Web is	Compact
C/t_f	=	5.276	Flange is	Compact
The sec is	Compact			

CHECK NORMAL STRESSES :-

Cb = 1.30

Lu_{max} = (20.b_f) / ($\sqrt{F_y}$) = 1.793 m; (1380.A_f) / (F_y.d) x C_b = 3.43 m → 1.793 m

Lateral torsional buckling of comp.flange — There is no LTB

Fltb	=	1.792	t/cm²			
F_{bcx}	=	1.792	t/cm²			
f_{cx}	=	1.60	t/cm²	<	1.792	SAFE

CHECK SHEAR STRESSES :-

q_{xx}	=	0.32	t/cm²	<	0.980	SAFE

CHECK DEFLICTION DUO TO LIVE LOAD :-

W_{LL}	=	0.45	t/m			
Beam type	=	simple				
δ_{LL}	=	0.493	cm	<	3.10	SAFE

[1]

6-2 Design of Main beam

Load

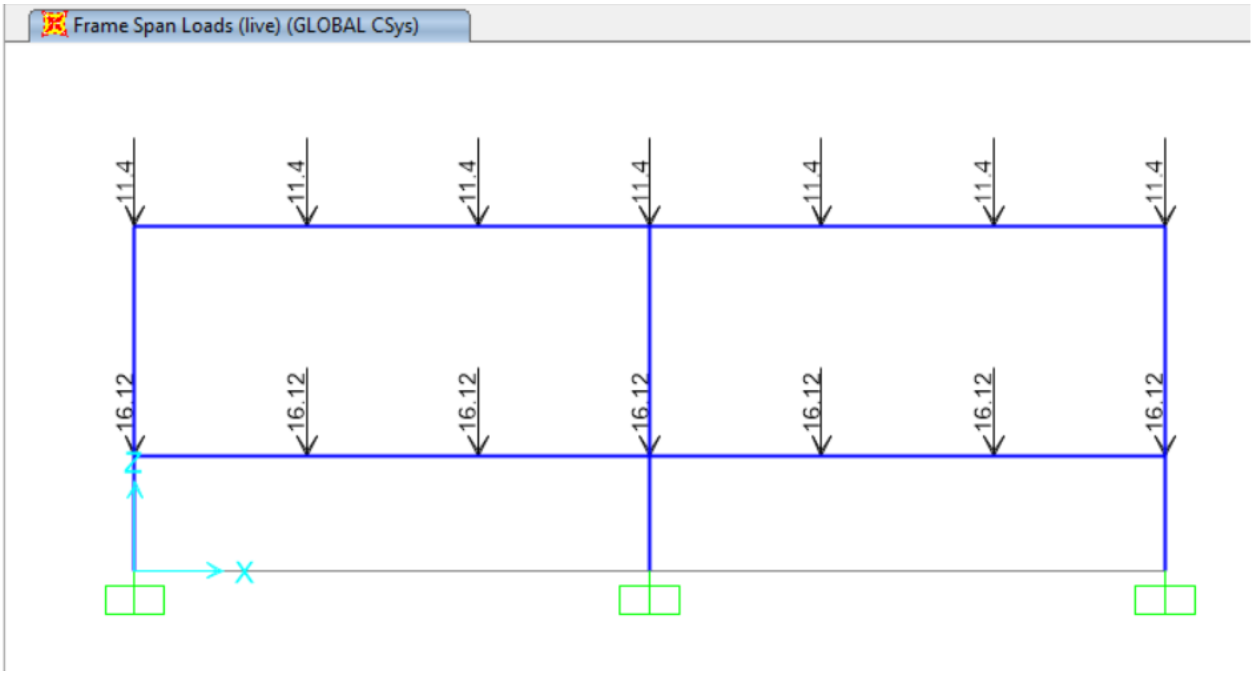


figure 6. 1 Load of main beam

SAP

Axial force

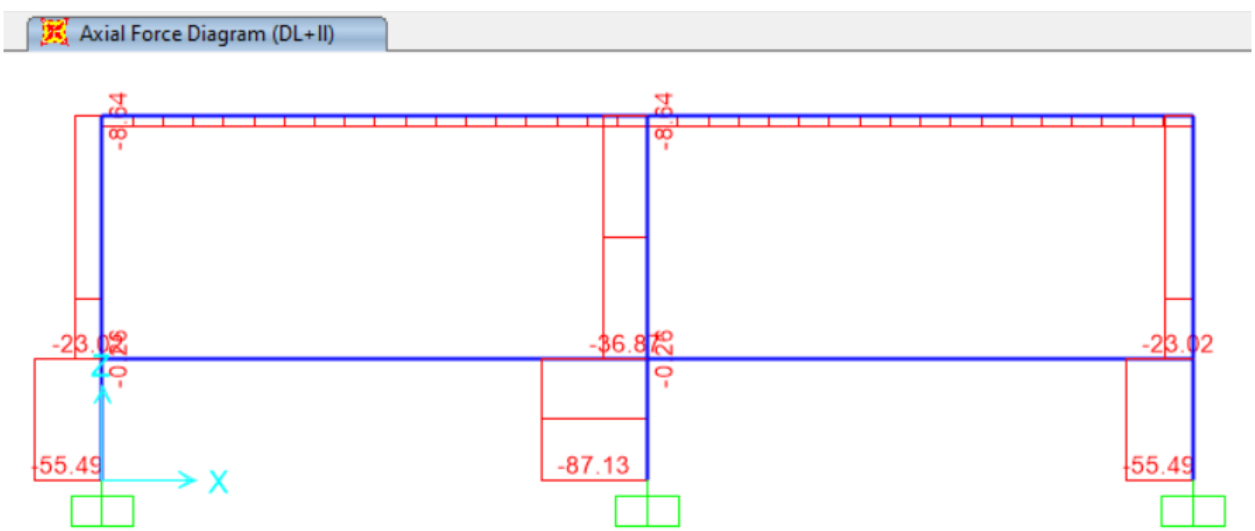


figure 6. 2 Axial Force

Shear force

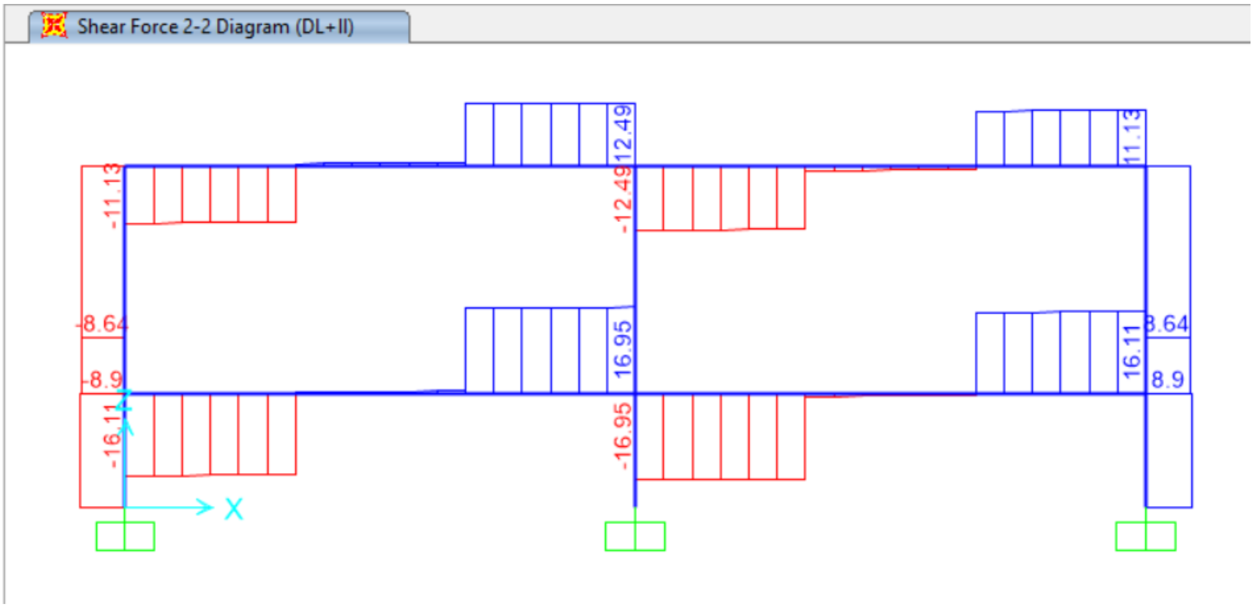


figure 6. 3 Shear force

Moment

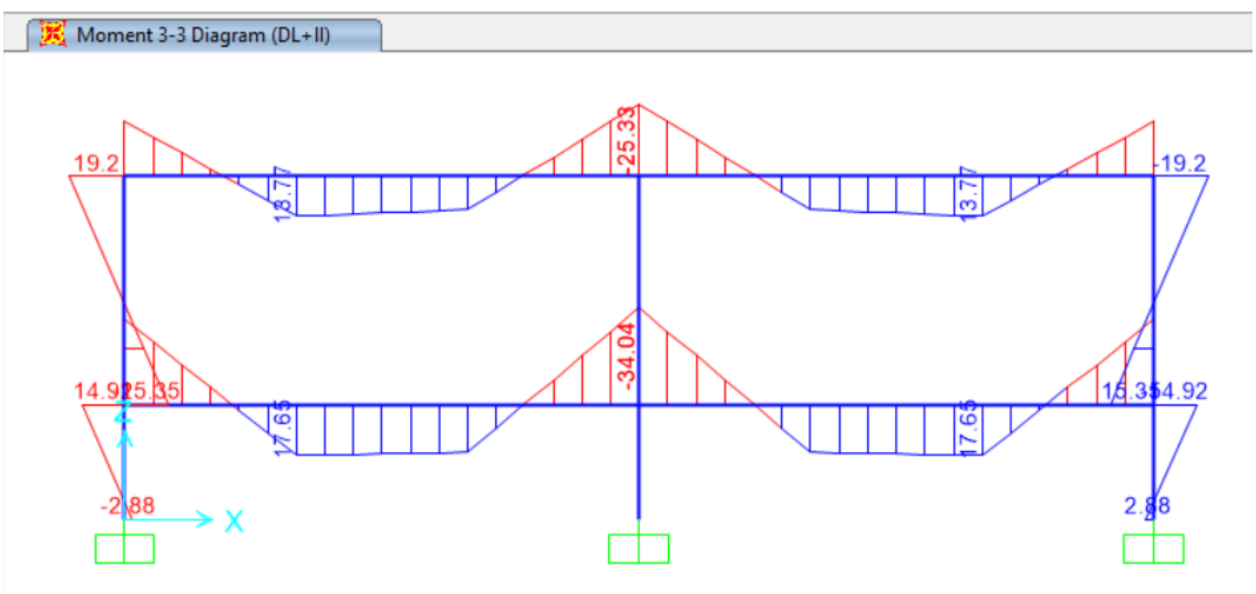


figure 6. 4 Moment

Design main beam

First floor

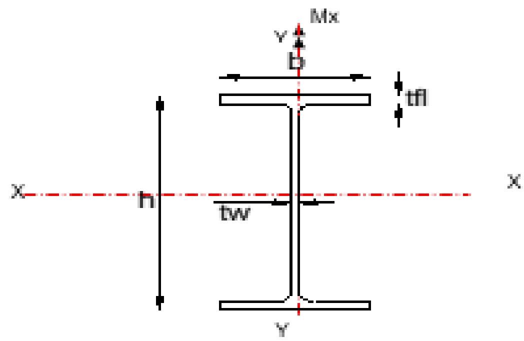
SEC. BEAM ID :-	(SB-1)					Refrences
		Steel grade	St.52			ECP
		F_y	=	3.60	t/cm^2	
		F_u	=	5.20	t/cm^2	

1)- APPLIED FORCES :- — **Case Combination**

M+ive	=	34.00	mt	a
Q	=	16.00	t	

2)- CHOISE OF SECTION :-

The section is Rolled section		IPE 500	
b_{FL}	=	200.0	mm
t_{FL}	=	16.0	mm
h	=	500.0	mm
t_{WEB}	=	10.2	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	9.00	m
Lu of compressin flange	=	3.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\bar{Y}$	=	25.00	cm
A	=	116.00	cm^2
I_x	=	48200.00	cm^4
I_y	=	2140.00	cm^4
S_x	=	1930.00	cm^3
r_x	=	20.40	cm
r_y	=	4.31	cm

CHECK COMPACTNESS :-

d_w/t_w	=	41.765	Web is	Compact	table(2.1a)
C/t_f	=	4.619	Flange is	Compact	table(2.1c)
The sec is	Compact				

CHECK NORMAL STRESSES :-

Cb	=	1.30						table(2.2)
Lu_{max}	=	$(20.b_f)/(\sqrt{fy})$ =		2.108	m	2.108	m	ECP 16
		$(1380.A_f)/(fy.d) \times C_b$ =		3.19	m			eqn. 2.18
Lateral torsional buckling of comp.flange				There is LTB				
Fltb	=	2.088	t/cm^2					
F_{bcx}	=	2.088	t/cm^2					
f_{bcx}	=	1.76	t/cm^2	<	2.088	SAFE		

CHECK SHEAR STRESSES :-

q_w	=	0.37	t/cm^2	<	1.260	SAFE	eqn. 2.2

CHECK DEFLICTION DUO TO LIVE LOAD :-

$W_{L.L}$	=	1.50	t/m				
Beam type	=	simple					
$\delta_{L.L}$	=	1.266	cm	<	4.50	SAFE	ECP 132 table(3.1)

[1]

second floor

F_y	=	3.60	t/cm²	
F_u	=	5.20	t/cm²	

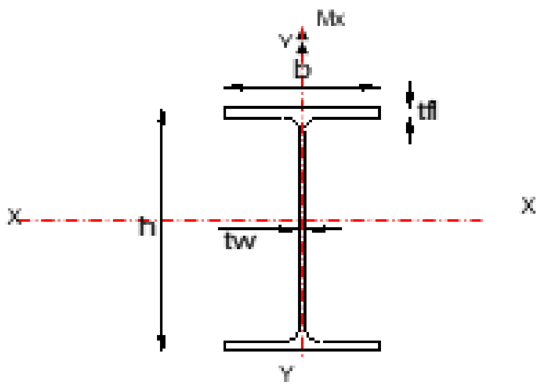
1)- APPLIED FORCES :- **Case Combination**

M+ive	=	25.30	mt	a
Q	=	12.40	t	

2)- CHOISE OF SECTION :-

The section is Rolled section **IPE 450**

b_{FL}	=	190.0	mm
t_{FL}	=	14.6	mm
h	=	450.0	mm
t_{WEB}	=	9.4	mm



3)- BEAM DATA :-

Total length of Beam (L)	=	9.00	m
Lu of compressin flange	=	3.00	m

4)- CHECK SECTION :-

PROPERTIES OF SECTION :-

$\overline{Y}$	=	22.50	cm
A	=	98.80	cm²
I_x	=	33740.00	cm⁴
I_y	=	1680.00	cm⁴
S_x	=	1500.00	cm³
r_x	=	18.50	cm
r_y	=	4.12	cm

CHECK COMPACTNESS :-

d_w/t_w	=	40.298	Web is	**Compact**	table(2.1a)
C/t_f	=	4.747	Flange is	**Compact**	table(2.1c)
The sec is		**Compact**			

CHECK NORMAL STRESSES :-

Cb	=	1.30					table(2.2)
Lu_{max}	=	$(20.b_f)/(\sqrt{fy})$ =	2.003	m	2.003	m	ECP 16
		$(1380.A_f)/(fy.d) \times C_b$ =	3.07	m			eqn. 2.18
Lateral torsional buckling of comp.flange			**There is LTB**				
Fltb	=	2.088	t/cm²				
F_{bcx}	=	2.088	t/cm²				
f_{bcx}	=	1.69	t/cm²	<	2.088	**SAFE**	

CHECK SHEAR STRESSES :-

q_{act}	=	0.35	t/cm²	<	1.260	**SAFE**	eqn. 2.2

CHECK DEFLICTION DUO TO LIVE LOAD :-

W_{LL}	=	1.50	t/m				
Beam type	=	simple					
δ_{act}	=	1.809	cm	<	4.50	**SAFE**	ECP 132

[1]

Design column

First floor

COLUMN ID :- (SC-2)

DESIGN CASE :- (D.L+L.L)

Steel grade St.44

F_y = 2.80 t/cm^2

F_u = 4.40 t/cm^2

1)- APPLIED FORCES :-

Mx	=	19.20	mt
N	=	87.00	t

2)-DIM. OF SECTION :-

The section is IPE 600

M

b

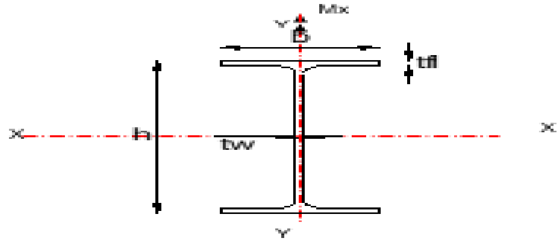
$b_{fl.u}$	=	220.0	mm
$t_{fl.u}$	=	19.0	mm
h_{web}	=	600.0	mm
t_{web}	=	12.0	mm
$b_{fl.L}$	=	220.0	mm
$t_{fl.L}$	=	19.0	mm

3)-COLUMN DATA :-

Total length of column	=	2.00	m
Lu act.of comp. Flange	=	1.70	m
Length subject to buckling in plan	=	1.80	m
Length subject to buckling out plan	=	2.00	m
Girts spacing	=	2.00	m

4)- PROPERTIES OF SECTION :-

$\overline{Y}$	=	300.00	cm
A	=	156.00	cm^2
Ix	=	92080.00	cm^4
Iy	=	3390.00	cm^4
S_x	=	3070.00	cm^3
r_x	=	24.30	cm
r_y	=	4.66	cm



5)- CHECK COMPACTNESS :-

d_w/t_w	=	42.833	Compact
c/t_f	=	5.789	Compact

The sec is **Compact**

6)- CHECK STRESSES :-

f_{bcx}	=	0.63	t/cm^2
f_{ca}	=	0.56	t/cm^2

C_b = 1.30

Lu_{max} = (20.b_f) / (√ fy) = 2.840 m

(1380.A_f) / (fy.d) x C_b = 5.21 m → 2.840 m

(Lateral torsional buckling of comp.flange **No L.T.B**

Fltb	=	1.792	t/cm^2
F_{bcx}	=	1.792	t/cm^2

Use Knee Bracing no

K_{in}	=	2.00	E.C.O.P (P.53)
K_{out}	=	1.00	
L_{bin}	=	3.60	m
L_{bout}	=	2.00	m
λ_{in}	=	14.81	
λ_{out}	=	42.92	

λ_{max} = 42.92 < 180 **SAFE**

f_c	=	1.44	t/cm^2
f_{ca}/f_c	=	0.39	
A_1	=	1.02	

APPLYING THE INTERACTION EQUATION :

$(f_{ca}/f_c) + (f_{bcx}/F_{bcx})\,A_1$ = 0.741 < 1.00 **SAFE** second floor

COLUMN ID :- (SC-2)

DESIGN CASE :- (D.L+L.L) a

Steel grade St.44

F_y	=	2.80	t/cm^2
F_u	=	4.40	t/cm^2

1)- APPLIED FORCES :-

Mx	=	19.20	mt
N	=	36.00	t

2)-DIM. OF SECTION :-

The section is IPE 500

b_{fl_U}	=	200.0	mm
t_{fl_U}	=	16.0	mm
h_{web}	=	500.0	mm
t_{web}	=	10.2	mm
b_{fl_L}	=	200.0	mm
t_{fl_L}	=	16.0	mm

M

b

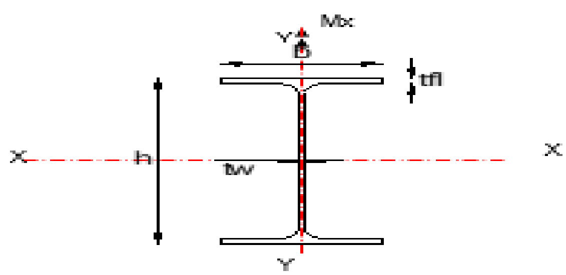
-

3)-COLUMN DATA :-

Total length of column	=	4.00	m
Lu act.of comp. Flange	=	2.30	m
Length subject to buckling in plan	=	2.07	m
Length subject to buckling out plan	=	4.00	m
Girts spacing	=	2.00	m

4)- PROPERTIES OF SECTION :-

$\overline{Y}$	=	250.00	cm
A	=	116.00	cm^2
Ix	=	48200.00	cm^4
Iy	=	2140.00	cm^4
S_x	=	1930.00	cm^3
r_x	=	20.40	cm
r_y	=	4.31	cm



5)- CHECK COMPACTNESS :-

d_w/t_w	=	41.765	Compact
c/t_f	=	6.250	Compact

The sec is Compact

6)- CHECK STRESSES :-

f_{bcx}	=	0.99	t/cm^2
f_{ca}	=	0.31	t/cm^2

C_b	=	1.30			
Lu_{max}	=	$(20.b_f) / (\sqrt{fy})$ =	2.582 m	2.582	m
		$(1380.A_f) / (fy.d) \times C_b$ =	4.78 m		

(Lateral torsional buckling of comp.flange

No L.T.B

Use Knee Bracing no

Fltb	=	1.792	t/cm^2
F_{bcx}	=	1.792	t/cm^2

K_{in}	=	2.00	E.C.O.P (P.53)
K_{out}	=	1.00	
L_{bin}	=	4.14	m
L_{bout}	=	4.00	m
λ_{in}	=	20.29	
λ_{out}	=	92.81	

λ_{max} = 92.81 < 180 SAFE

f_c	=	0.87	t/cm^2
f_{ca} / f_c	=	0.36	
A_1	=	1.02	

APPLYING THE INTERACTION EQUATION :

$(f_{ca} / f_c) + (f_{bcx} / F_{bcx}) A_1$ = 0.922 < 1.00 SAFE

Chapter 7 (Design of Connections)

6-1 Design of Connection between Stringer and Cross Girder

Continuous connection:-

M_{-ve} = 0.75*75.56 =56.67 t.m Q max = 54.76 t

Use High Strength Bolts M22 grade (10.9)

1) Bolts connecting the web of stringer to the framing angles:

N1 = Q max / (2*Ps) = 54.76/2*4.77 = 5.7 use 6 bolt

2) Bolts connecting the framing angles to the web of cross girder:

N2 = Q max / (Ps) = 54.76 /4.77 = 11.46 use 12 bolt

3) Bolts in upper and lower tie plates (to transmit negative bending moment):

T=C= M_{-ve} /H stringer=56.67*100/54.6=104.9 t.

N3 = N4= (C OR T) / (Ps) = 104.9/4.77 = 22 use 22 bolt

4) check plate :-

Use T_{plate} = 16mm

$F=T/ A_{net} < 0.58\ fy$

A_{net} = (62-2.4*2)*1.6 =91.52 cm^2

F= 104.9 / 91.52 = 1.146 < 0.58 * 3.6 = 2.088 safe

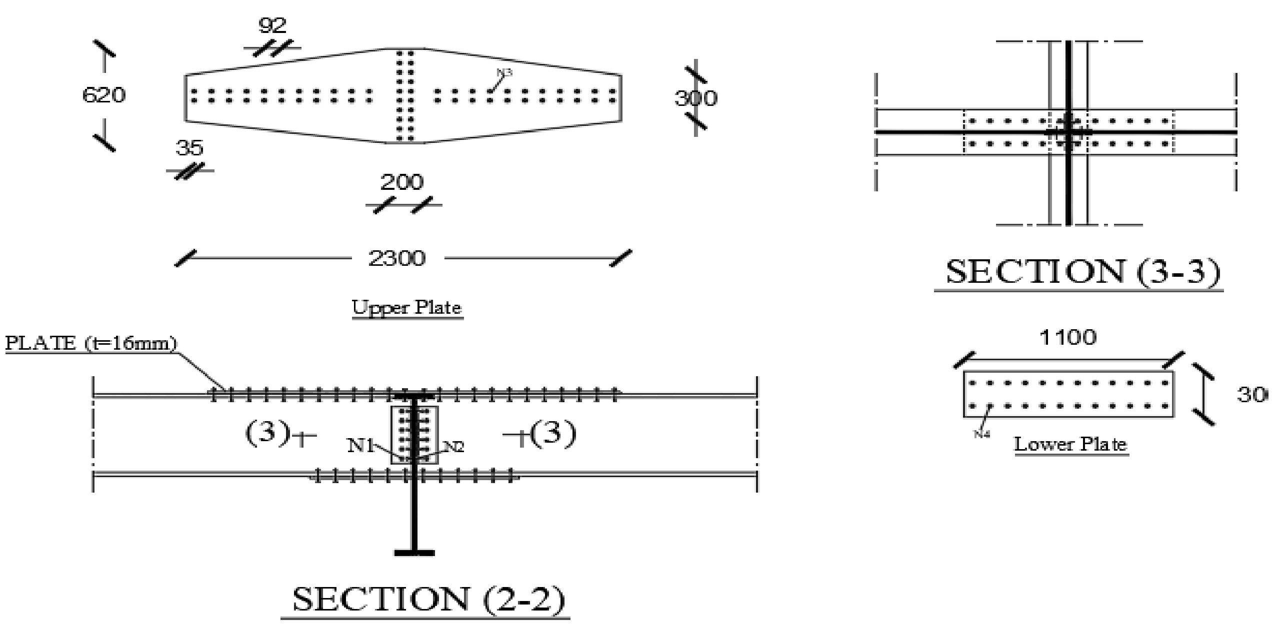


Figure7. 1 Connection between Stringer and Cross Girder

6-2 Design of Connection Between Cross Girder and Main girder

simple connection

Q max =95.3 t Use High Strength Bolts M22 grade (10.9)

Use angle 100*100*10

1) Bolts connecting the web of Cross Girder to the framing angles:

N1= Q max/ 2Ps = 95.3/(2*4.77) = 9.9 bolt use 10 bolt

2) Bolts connecting the framing angles to the web of Main girder:

N2= Q max/ Ps = 95.3/ (4.77) = 19.9 bolt use 20 bolt

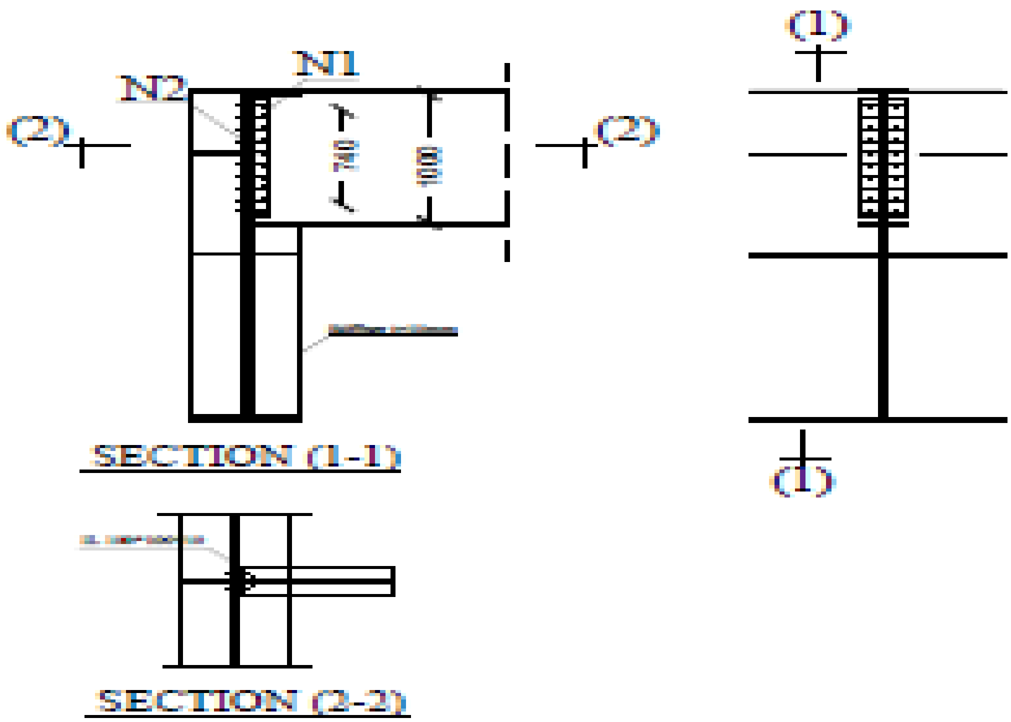


Figure7. 2 Connection Between Cross Girder and Main girder

6-3 Design of bolted field splices

Splice plate S1.	bl = bf = 50 cm	$T1= t_f /2 = 3/2= 1.5$ cm
Splice plate S2:	take b2 = 22 cm	$T2= t_f /2 = 3/2= 1.5$ cm
Splice plate S3:	b3 = 2.38cm	T3=2cm

Splice of flange plate:

Force in flange plate:

C=T= 1356*100/250 =542.4 ton

N_1 = 542.4/2*4.77=56.8 bolt

Use 60 bolt ϕ22 * 6 rows

Splice of web plate:

Assume pitch P =7 cm

No. of bolts per one row = b3/Pitch = 238/7 = 34 bolts

Take N = 34 M22per row

Get actual pitch: Pact = b3/N chosen = 238/26 = 9.15 cm

Take P =7cm and edge e= 3.3cm

y = 1.5+0.5+3.3+7/2 = 8.8cm

f1= ((0.58*3.6)/(250/2 + 3))* 250/2 =2 t/cm^2

f2= ((0.58*3.6)/ (250/2 + 3))* (250/2-8.8) =1.89t/cm^2

F_h = (2+1.85/2) *8.8*3 =50.82ton

The force on the first top bolt (critical) is therefore checked as follows:

H=50.82/ No. of vertical rows 6= 8.4ton V=85/6*34= 0.41ton

R=8.41 ton ≤ 2Ps =2*4.77=9.54 safe

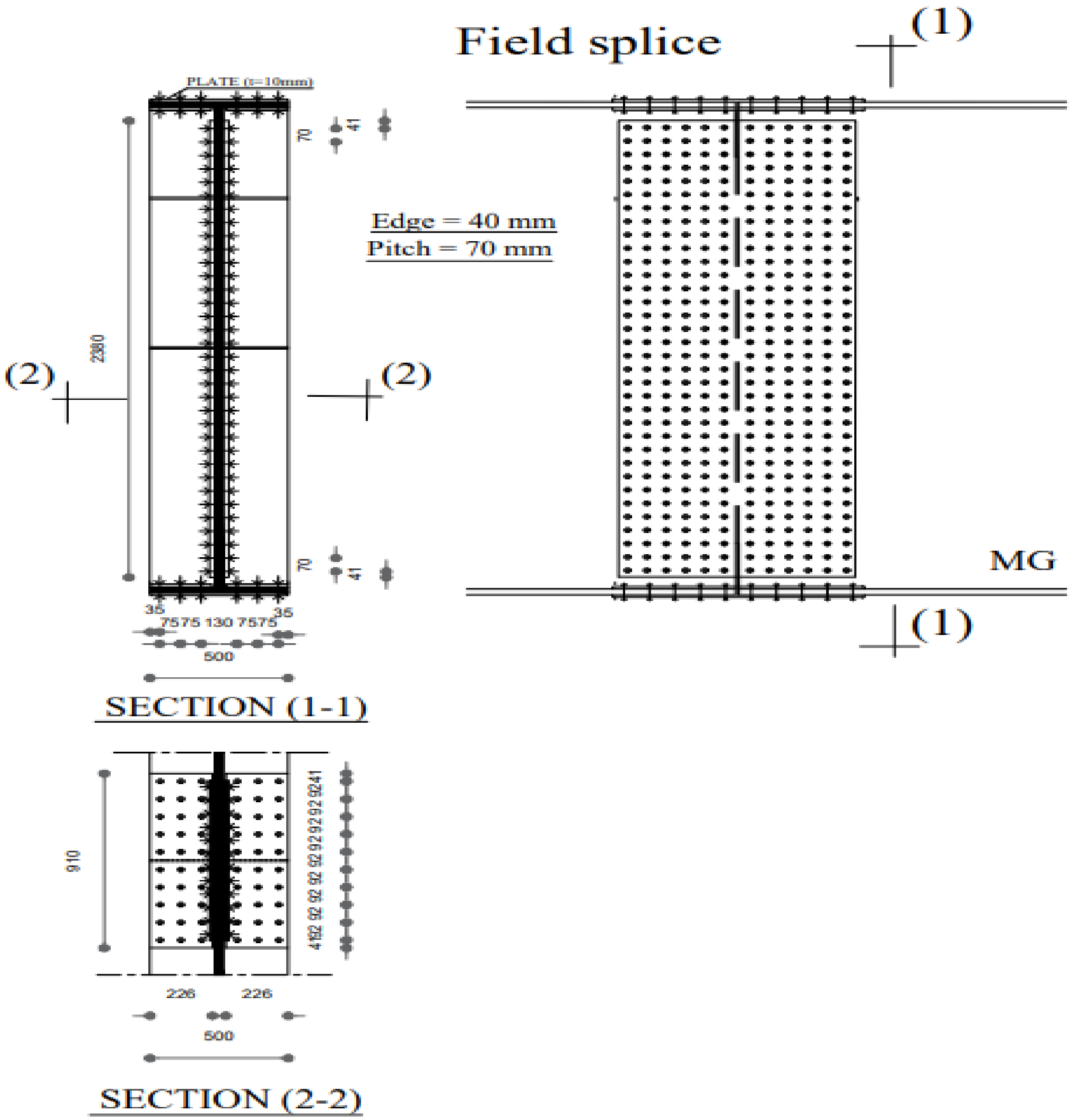


Figure7. 3 Field splices

6-4 Corner connection for frame @10 m

1) Connection Dim:- (M20)(10.9)

   B= 40cm H=310cm

2) Get ``X`` Distance:-

   $\frac{B*X^2}{2}$=2As((y1-x)+(y2-x)+……)

$$\frac{40*X^2}{2} = 2(2.45)((297.6 - x) + (281.6 - x) + (265.6 - x) + (249.6 - x) + (233.6 - x) + (217.6 - x) + (201.6 - x) + (185.6 - x) + (169.6 - x) + (145.8 - x) + (129.8 - x) + (113.8 - x) + (97.8 - x) + (81.8 - x) + (65.8 - x) + (49.8 - x) + (33.8 - x) + (17.8 - x))$$

   So, X = 24.25cm

3) Get IV:-

   $\frac{B*X^3}{3}$+2As((y1-x)²+(y2-x)²+……)

$$\frac{40*24.25^3}{3} + 2(2.45)((297.6 - 24.25)^2 + (281.6 - 24.25)^2 + (265.6 - 24.25)^2 + (249.6 - 24.25)^2 + (233.6 - 24.25)^2 + (217.6 - 24.25)^2 + (201.6 - 24.25)^2 + (185.6 - 24.25)^2 + (169.6 - 24.25)^2 + (145.8 - 24.25)^2 + (129.8 - 24.25)^2 + (113.8 - 24.25)^2 + (97.8 - 24.25)^2 + (81.8 - 24.25)^2 + (65.8 - 24.25)^2 + (49.8 - 24.25)^2 + (33.8 - 24.25)^2 + (17.8 - 24.25)^2)$$

So, IV= 2419516.85 cm^4

4) Check of bolts :-

   Text b1= $\frac{267*100}{2419516.85}(297.6 - 24.25)(2.45) = 7.4t < 0.8(15.43) = 12.3t$

   Text b2= $\frac{267*100}{2419516.85}(281.6 - 24.25)(2.45) = 6.95t < 0.8(15.43) = 12.3t$

   Text b3= $\frac{267*100}{2419516.85}(265.6 - 24.25)(2.45) = 6.5t < 0.8(15.43) = 12.3t$

   Text b4= $\frac{267*100}{2419516.85}(249.6 - 24.25)(2.45) = 6.1t < 0.8(15.43) = 12.3t$

   Text b5= $\frac{267*100}{2419516.85}(233.6 - 24.25)(2.45) = 5.66t < 0.8(15.43) = 12.3t$

   Text b6= $\frac{267*100}{2419516.85}(217.6 - 24.25)(2.45) = 5.22t < 0.8(15.43) = 12.3t$

Text b7= $\frac{267*100}{2419516.85}(201.6-24.25)(2.45) = 4.8t < 0.8(15.43) = 12.3t$

Text b8= $\frac{267*100}{2419516.85}(185.6-24.25)(2.45) = 4.36t < 0.8(15.43) = 12.3t$

Text b9= $\frac{267*100}{2419516.85}(169.6-24.25)(2.45) = 3.93t < 0.8(15.43) = 12.3t$

Text b10= $\frac{267*100}{2419516.85}(145.8-24.25)(2.45) = 3.3t < 0.8(15.43) = 12.3t$

Text b11= $\frac{267*100}{2419516.85}(129.8-24.25)(2.45) = 2.85t < 0.8(15.43) = 12.3t$

Text b12= $\frac{267*100}{2419516.85}(113.8-24.25)(2.45) = 2.42t < 0.8(15.43) = 12.3t$

Text b13= $\frac{267*100}{2419516.85}(97.8-24.25)(2.45) = 2t < 0.8(15.43) = 12.3t$

Text b14= $\frac{267*100}{2419516.85}(81.8-24.25)(2.45) = 1.55t < 0.8(15.43) = 12.3t$

Text b15= $\frac{267*100}{2419516.85}(65.8-24.25)(2.45) = 1.12t < 0.8(15.43) = 12.3t$

Text b16= $\frac{267*100}{2419516.85}(49.8-24.25)(2.45) = 0.69t < 0.8(15.43) = 12.3t$

Text b17= $\frac{267*100}{2419516.85}(33.8-24.25)(2.45) = 0.26t < 0.8(15.43) = 12.3t$

Text b18= $\frac{267*100}{2419516.85}(17.8-24.25)(2.45) = -0.17t < 0.8(15.43) = 12.3t$

Text avg= $\frac{7.4+6.95+6.5+6.1+5.66+5.22+4.8+4.36+3.93+3.3+2.85+2.42+2+1.55+1.12+0.69+0.26}{17}$=
3.83t < 0.6(15.43)=9.26t

5) Check of Shear :-

$\frac{119}{36} = 3.3t <$ 4.82t (Mps)

6) Check Fc :-

$\frac{267*100}{2419516.85}(24.25) = 0.27\frac{t}{cm^2} < 0.58(3.6) = 2.088\frac{t}{cm^2}$

7) Check of weld :-

sw=0.8cm

Av= 4(120*0.8) = 384 cm^2

Ah=2(40*0.8) + 8(16*0.8) = 166.4 cm^2

At=384+166.4=550.4 cm²

$$Ix=4(\frac{0.8*120^3}{12}+(0.8*120)\left(\frac{150.9}{2}\right)^2)+8(0.4(40)*0.8*(\frac{150}{2}-1.8-\frac{0.8}{2})^2)+2(40*0.8*(150+\frac{0.8}{2})^2)=4637191.62cm^4$$

8) Checks :-

$$\frac{267*100}{4637191.62}(150+0.8)=0.87\frac{t}{cm^2}<0.2(5.2)=1.04\frac{t}{cm^2}$$

Q= $\frac{119}{384}=0.31\frac{t}{cm^2}<1.04\frac{t}{cm^2}$

Feq= = $\sqrt{(0.87)^2+(0.31)^2}=0.92\frac{t}{cm^2}$ < 1.04 t/cm²

CONNECTION IS SAFE TO USE

[1]

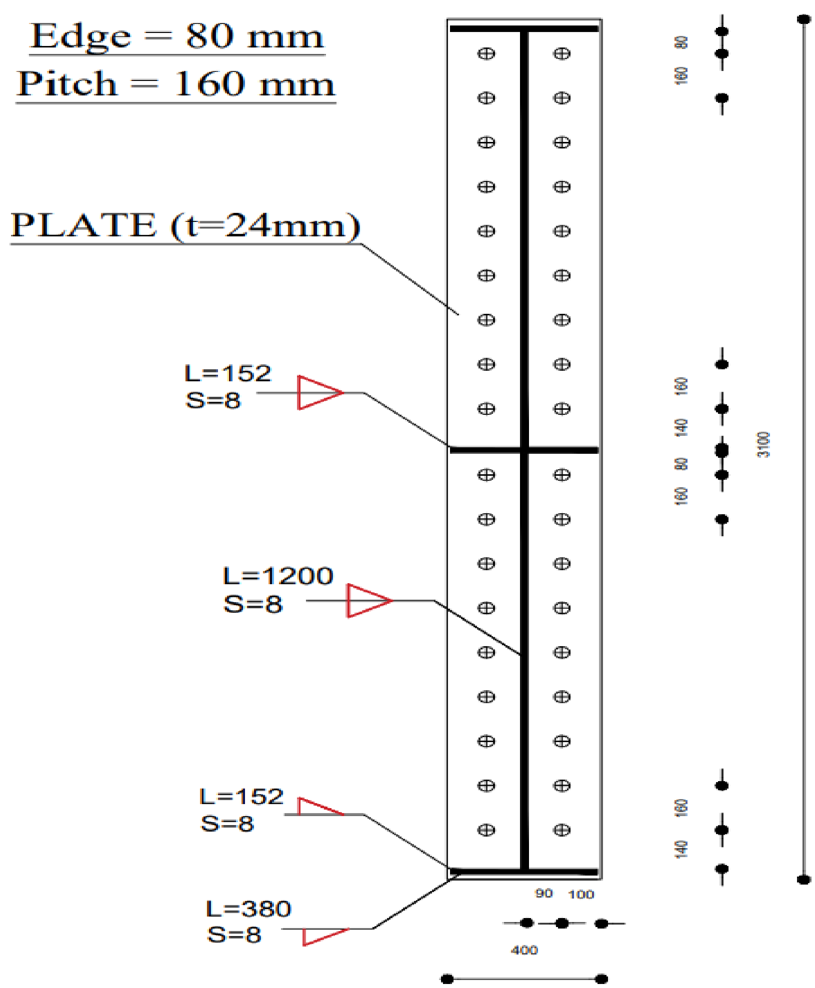


Figure7. 4 Corner connection

6-5 Fixed Base of frame at 10m

1) Assume Dimensions:-

   L= 154.8*1.5= 235cm , B= 93cm

2) Check of horizontal weld:- Assume sw= 0.8cm

   $A_t = 2(235*0.8) + 4(75*0.8) = 616 cm^2$

   $I_y = 2(\frac{0.8*235^3}{12}) + 4\left(\frac{0.8*75^3}{12} + (0.8*75)\left(\frac{235}{2} - \frac{75}{2}\right)^2\right) = 3378883\ cm\hat{}4$

   $F_1 = \frac{-146}{616} - \frac{134*100}{3378883}(\frac{235}{2}) = -0.7\frac{t}{cm^2} < 0.2(5.2) = 1.04\ t/cm^2$

   $F_2 = \frac{-146}{616} + \frac{134*100}{3378883}(\frac{235}{2}) = 0.23\frac{t}{cm^2} < 0.2(5.2) = 1.04\ t/cm^2$

   Shear = $\frac{40}{616} = 0.065\ t/cm^2 < 1.04\ t/cm^2$

   $F_{eq} = \sqrt{0.7^2 + (3*(0.065)^2)} = 0.71\frac{t}{cm^2} < 1.04\frac{t}{cm^2}$

3) Check of vertical weld:-

   $C = \frac{-146}{2} - \frac{134*100}{154.8} = -159.6t * 0.6 = 95.76\ t$

   $T = \frac{-146}{2} + \frac{134*100}{154.8} = 13.6\ t$

   $Q_{act} = \frac{\text{T or } 0.6\text{C}}{\text{Lv}*\text{sw}*\text{n}} \leq 0.2(Fu)$ $L_v = \frac{95.76}{2*0.8*0.2*5.2} + 2(0.8) = 60cm$

4) Check of Bearing Plate :- f1,2 $= \frac{-N}{B*L} \pm \frac{6M}{B*L^2} \leq fconc.(50\sim 70\ kg/cm^2)$

   $F_1 = \frac{-146}{93*235} - \frac{6*134*100}{93*235^2} = -0.022\frac{t}{cm^2}$ < fconc.

   $F_2 = \frac{-146}{93*235} + \frac{6*134*100}{93*235^2} = 0.009\frac{t}{cm^2}$ < fconc.

5) Check of Anchor Bolt:-

C=0.5*3a*Fc*B , T =??

ΣM@T= 0.0

$0.5*3a*0.07*93*(235-a-10) = 146(\frac{235}{2} - 10) + 134*100 \longrightarrow a = 14.13cm$

C= 0.5*3(14.13) *0.07*93 = 138 t

$\Sigma_y = 0$

$146 + T = 138 \longrightarrow T= 8\ t$

$Q_{act}= \frac{T}{\frac{\pi}{4}*Ø^2*0.7*n} \le 0.33fub \longrightarrow \frac{8}{\frac{\pi}{4}*Ø^2*0.7*4} = 0.33*0.85*5.2 \longrightarrow Ø = 2cm$

[1]

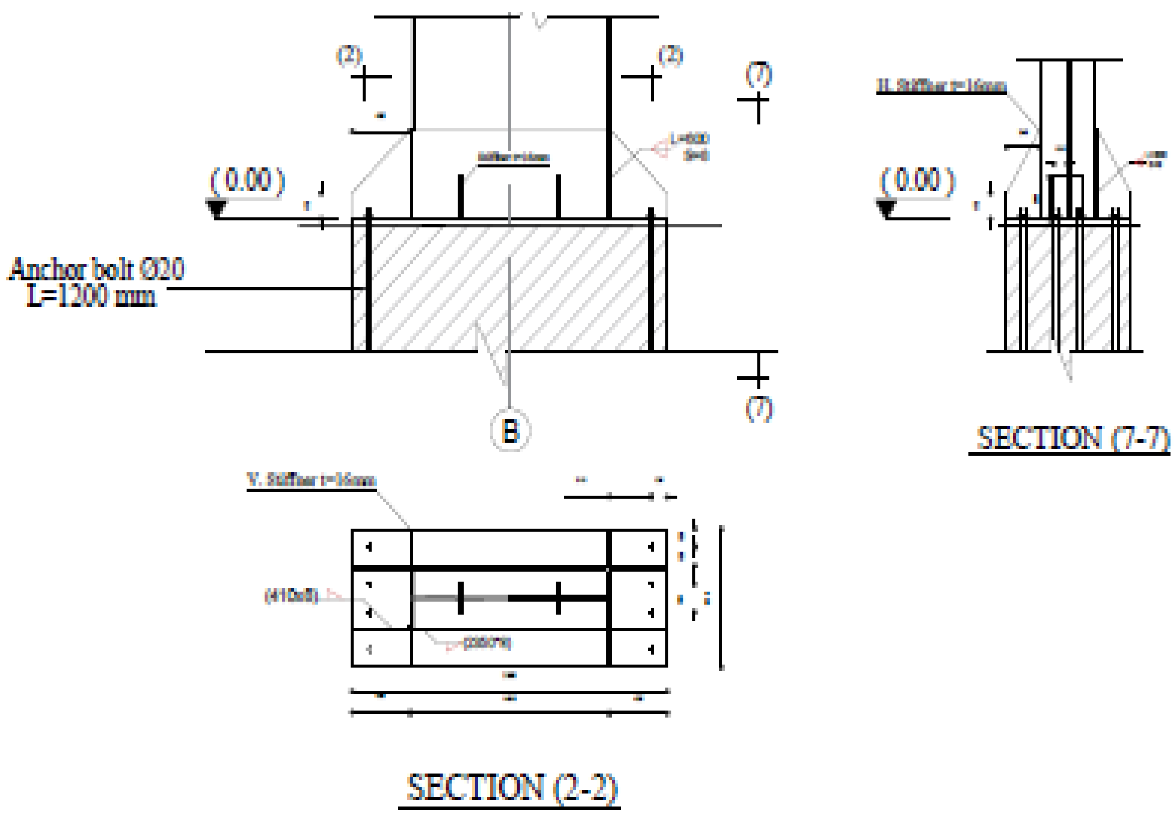
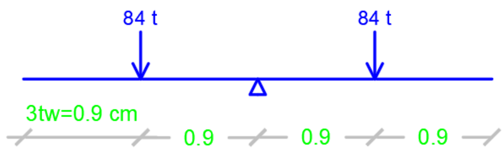


Figure7. 5 Corner connection

6-6 Connection between Main Girder & Portal Frame

Qy from MG = 168 ton

1) Sole plate:

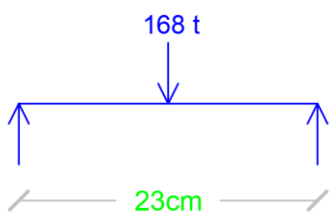


M = 84*0.9 = 75.6 t.cm

$$t = \sqrt{\frac{6\text{M}}{\text{b} * \text{fb}}} = \sqrt{\frac{6 * 75.6}{60 * 1.8}} = 2.05cm$$

Take: t=2.2cm, b=60cm

2) Upper bearing Plate:

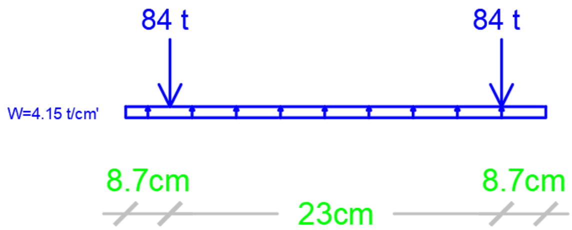


M = 168*23/4 = 966 t.cm

$$t = \sqrt{\frac{6 * 966}{70 * 1.8}} = 6.78cm$$

Take: t=7cm, b=70cm

3) Lower Plate:



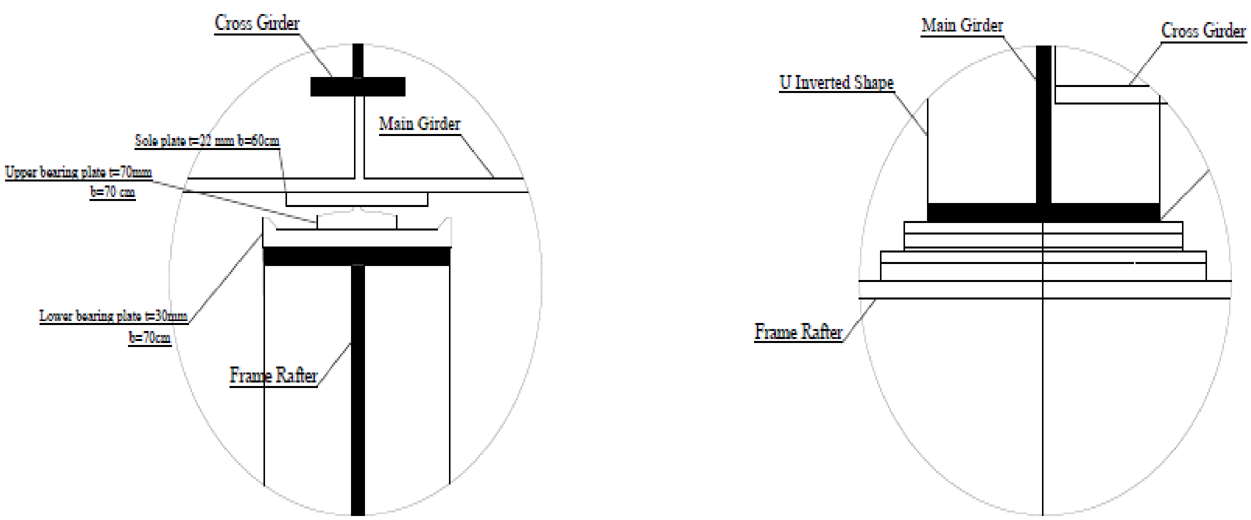
M1 = 4.15*8.7^2/2 = 157.9 t.cm

M2 = 4.15*23^2/2 = 117.3 t.cm

$$t = \sqrt{\frac{6 * 157.9}{70 * 1.8}} = 2.74cm$$

Take: t=3cm, b=70cm

[4]



Detailed Support A
Elevation View

Detailed Support A
Side View

Figure7. 6 Connection between Main Girder & Portal Frame

6-7 Shear Flow Calculation (Inclined part: Frame @10m)

$$T = \frac{\mathrm{Qy}}{\mathrm{Ix}} * \frac{\mathrm{Sx}}{\mathrm{b}} = Sw * Lw * 0.2fu$$

$$Sx = \mathrm{Af} * (0.5 * \mathrm{hw} + 0.5 * tf)$$

1) Rafter

Qy = 119 ton ⟶ (From Max Shear case)

Ix = 1209990 cm4

Sx = 38*1.8*(0.5*150+0.5*1.8) = 5191.56 cm3

$\frac{119}{1209990} * \frac{5191.56}{1} = Sw * 1 * 0.2 * 5.2$ ⟶ Sw= 0.5 cm

Therefore: Use Minimum Sw = 6mm

2) Column

Q_y = 39.8 ton ⟶ (From Max Moment Case)

I_x = 1733678 cm4

S_x = 40*2.4*(0.5*150+0.5*2.4) = 7315.2 cm3

$\frac{39.8}{1733678} * \frac{7315.2}{1} = Sw * 1 * 0.2 * 5.2$ ⟶ Sw= 0.16 cm

Therefore: Use Minimum Sw = 6mm

[1]

Chapter 8 (Management)

7-1 Comparison between 4 MG System Resting on portal frame (System 1) & 4 portal frames each acting as MG (System 2)

Steel unit Weight = 7.85 t/m3   Volume = Area*Length   Weight = Volume * Unit Weight

4 MG System resting on portal frame.

MG:

1. before Curtailment

A = 0.105 m2 , V = 0.105*15.2 = 1.6 m3 , Weight = 12.5 ton

2. after Curtailment

A = 0.0875 m2 , V = 0.0875*7.4*2 = 1.295 m3 , Weight = 10.17 ton

3. Inclined Part

A = 0.07125 m2 , V = 0.07125*1.5*2 = 0.21645 m3 , Weight = 1.7 ton

4. Depth decrease

A = 0.055 m2 , V = 0.055*1.73*2 = 0.19 m3 , Weight = 1.5 ton

Total weight of 1 MG= 12.5+10.17+1.7+1.5 = 25.87 ton

Frame:

1. Rafter:

A = 0.0744 m2 , V = 0.0744*14 = 1.0416 m3 , Weight = 8.18 ton

2. Hunch:

A = 0.0744 m2 , V = 0.0744*2 = 0.1488 m3 , Weight = 1.17 ton

3. Column:

Straight part: A = 0.072 m2 , V = 0.072*3.28 = 0.236 m3 , Weight = 1.85 ton

Inclined part: A = 0.0636 m2 , V = 0.0636*5.3 = 0.337 m3 , Weight = 2.65 ton

Total weight of column = (1.85+2.65)*2 = 9*2 = 18 ton

Total weight of 1 frame = 8.18+1.17+18 = 27.35 ton

Total weight of System = 2*27.35 + 4* 25.87 = 158.2 ton

4 portal frames each acting as MG

Rafter:

A = 0.113 m2 , V = 0.113*33.4 = 3.7742 m3 , Weight = 29.63 ton

Hunch:

A = 0.113 m2 , V = 0.113*3.2 = 0.3616 m3 , Weight = 2.84 ton

Column:

A = 0.121 m2 , V = 0.121*10 = 1.21 m3 , Weight = 9.5 * 2 = 19 ton

Total weight of 1 frame = 29.63+2.84+19 = 51.47

Total weight of System = 41.97 *4 = 205.9 ton

Therefore, we get that It's better to Use 1st Option

(4 MG System resting on portal frame)

7-2 Quantity survey of material:

	Type of member	No.of Member	B flange (mm)	T flange(mm)	h web(mm)	T web(mm)	Area (mm^2)	Length(m)	Volume (m^3)	Weight (ton)
Deck Part	Cross girder of deck part	14	200	30	1000	20	32000	6	2.688	21.1008
	Main girder of deck part b4 Car	4	500	30	2500	30	105000	15.2	6.384	50.1144
	Main girder of deck part car part	4	500	25	2500	25	87500	14.8	5.18	40.663
	Main girder after depth decrease	4	500	25	1200	25	55000	3.46	0.7612	5.97542
	Main girder Inclined Part	4					71250	3	0.855	6.71175
	Rafter of frame of deck part	2	400	30	1800	28	74400	14	2.0832	16.35312
	Column Straight part	4	500	30	1500	28	72000	3.28	0.94464	7.415424
	Column cart part (deck part)	4					63600	5.3	1.34832	10.584312
	Hunch	2	400	30	1800	28	74400	2	0.2976	2.33616
Inclined part	(Rafter) frame of Inclined Part	24	380	18	1500	15	36180	14.5	12.59064	98.836524
	(column) frame of Inclined Part	4	400	24	1500	22	52200	10	2.088	16.3908
	(column) frame of Inclined Part	4	400	24	1500	22	52200	9.4	1.96272	15.407352
	(column) frame of Inclined Part	4	400	24	1500	22	52200	8.8	1.83744	14.423904
	(column) frame of Inclined Part	4	400	24	1500	22	52200	8.2	1.71216	13.440456
	(column) frame of Inclined Part	4	400	24	1500	22	52200	7.6	1.58688	12.457008
	(column) frame of Inclined Part	4	400	24	1500	22	52200	7	1.4616	11.47356
	(column) frame of Inclined Part	4	400	24	1500	22	52200	6.4	1.33632	10.490112
	(column) frame of Inclined Part	4	400	24	1500	22	52200	5.8	1.21104	9.506664
	(column) frame of Inclined Part	4	400	24	1500	22	52200	5.2	1.08576	8.523216
	(column) frame of Inclined Part	4	400	24	1500	22	52200	4.6	0.96048	7.539768
	(column) frame of Inclined Part	4	400	24	1500	22	52200	4	0.8352	6.55632
	(column) frame of Inclined Part	4	400	24	1500	22	52200	3.4	0.70992	5.572872
	Hunch	24	380	18	1500	15	36180	2	1.73664	13.632624
								Total weight		391.872942

Type of plate	No.of plate	T (mm)	B (mm)	Area(mm^2)	H (m)	Volume (m^3)	Weight (ton)
Fixed pase of Inclined Part	48	50	930	46500	2.35	5.2452	41.17482
corner plate inclined part	48	24	400	9600	3.1	1.42848	11.213568
Fixed pase of deck Part	4	30	900	27000	1.8	0.1944	1.52604
corner plate dec part	4	30	500	15000	3.37	0.2022	1.58727
splice plate	16	20	860	17200	2.38	0.654976	5.1415616
					Total weight		55.501698

quantity of stringer					
Type of member	Type Section	No.of Member	Weight (kg/m)	Length (m)	Total weight (ton)
Stringr of inclined part	IPE600	198	122	6	144.936
Stringr of deck part	HEA550	24	168	6	24.192
			Total weight		169.128

Mezzanine Floor					
Type of member	Type Section	No.of Members	weight (kg/m)	Length (m)	Total weight (ton)
Column first floor	IPE600	9	122	2	2.196
Column socend floor	IPE500	9	90.7	4	3.2652
Main Beam First floor	IPE500	6	90.7	9	4.8978
Main Beam Socend floor	IPE450	6	77.6	9	4.1904
Secondary beam First floor	IPE360	14	57.1	6.2	4.95628
Secondary beam socend floor	IPE300	14	42.2	6.2	3.66296
			Total weight		23.16864

Upper Bracing System	2L 100*100*10	24	30.2	6.32	4.580736
Site gerts	IPE600	20	122	6	14.64
			Total weight		19.220736

Total weight of project	658.892016

7-3 Time Estimation:

Steel Bridge

Activity ID	Activity Name	Original Duration	Start	Finish
Steel Bridge		158	01-Aug-23	07-Mar-24
Preliminary Work		75	01-Aug-2	13-Nov-23
B.100	Surveying Works	7	01-Aug-2	09-Aug-23
B.110	Site Offices	3	10-Aug-2	14-Aug-23
B.630	Production of Members	75	01-Aug-2	13-Nov-23
Inclined Part (1)		100	01-Aug-2	18-Dec-23
Earth Works		32	01-Aug-2	13-Sep-23
B.120	Excavation	7	15-Aug-2	23-Aug-23
B.130	Replacement	3	01-Aug-2	03-Aug-23
B.140	Insulation	5	25-Aug-2	31-Aug-23
B.150	Backfilling	3	01-Sep-2	05-Sep-23
B.160	Backfilling For Ramp	7	05-Sep-2	13-Sep-23
Concrete Works		97	04-Aug-2	18-Dec-23
B.170	P.C Footings	5	04-Aug-2	10-Aug-23
B.180	R.C Footings	10	11-Aug-2	24-Aug-23
B.190	Retaining Wall	7	25-Aug-2	04-Sep-23
B.200	Inclined Slap	7	08-Dec-2	18-Dec-23
Steel Installing Works		18	14-Nov-2	07-Dec-23
B.210	Installing Columns	5	14-Nov-2	20-Nov-23
B.220	Installing Rafter	5	21-Nov-2	27-Nov-23
B.230	Installing stringer	8	28-Nov-2	07-Dec-23
Deck Part		141	24-Aug-2	07-Mar-24
Earth Works		109	24-Aug-2	23-Jan-24
B.240	Excavation	7	24-Aug-2	01-Sep-23
B.250	Replacement	3	19-Dec-2	21-Dec-23
B.260	Insulation	5	12-Jan-2	18-Jan-24
B.270	Backfilling	3	19-Jan-2	23-Jan-24
Concrete Works		55	22-Dec-2	07-Mar-24
B.280	P.C Footings	5	22-Dec-2	28-Dec-23
B.290	R.C Footings	10	29-Dec-2	11-Jan-24
B.300	Deck Slap	7	28-Feb-2	07-Mar-24
Steel Installing Works		25	24-Jan-2	27-Feb-24
B.310	Installing Columns	2	24-Jan-2	25-Jan-24
B.320	Installing Rafter	2	26-Jan-2	29-Jan-24
B.330	Installing Main girder	6	30-Jan-2	06-Feb-24
B.340	Installing cross girder	6	07-Feb-2	14-Feb-24
B.350	Bracing	3	15-Feb-2	19-Feb-24
B.360	Installing stringer	6	20-Feb-2	27-Feb-24
Inclined Part (2)		76	04-Sep-2	18-Dec-23
Earth Works		36	04-Sep-2	23-Oct-23
B.370	Excavation	7	04-Sep-2	12-Sep-23
B.380	Replacement	3	13-Sep-2	15-Sep-23
B.390	Insulation	5	09-Oct-23	13-Oct-23
B.400	Backfilling	3	16-Oct-23	18-Oct-23
B.610	Backfilling for ramp	5	17-Oct-23	23-Oct-23
Concrete Works		66	18-Sep-2	18-Dec-23
B.410	P.C Footings	5	18-Sep-2	22-Sep-23
B.420	R.C Footings	10	25-Sep-2	06-Oct-23
B.430	Retaining Wall	6	09-Oct-23	16-Oct-23
B.440	Inclined Slap	7	08-Dec-2	18-Dec-23
Steel Installing Works		18	14-Nov-2	07-Dec-23
B.450	Installing Columns	5	14-Nov-2	20-Nov-23
B.460	Installing Rafter	5	21-Nov-2	27-Nov-23
B.470	Installing stringer	8	28-Nov-2	07-Dec-23
Mezzanine Floor		110	13-Sep-2	13-Feb-24
Earth Works		92	13-Sep-2	18-Jan-24
B.480	Excavation	3	13-Sep-2	15-Sep-23
B.490	Replacement	2	19-Dec-2	20-Dec-23
B.500	Insulation	3	11-Jan-2	15-Jan-24
B.510	Backfilling	3	16-Jan-2	18-Jan-24
Concrete Works		39	21-Dec-2	13-Feb-24
B.520	P.C Footings	5	21-Dec-2	27-Dec-23
B.530	R.C Footings	10	28-Dec-2	10-Jan-24
B.540	Slap	5	07-Feb-2	13-Feb-24
Steel Installing Works		13	19-Jan-2	06-Feb-24
B.550	Installing Columns	4	19-Jan-2	24-Jan-24
B.560	Installing Main Beam	3	25-Jan-2	29-Jan-24
B.570	Installing Secondry Beam	6	30-Jan-2	06-Feb-24
Finishing Works		46	19-Dec-2	20-Feb-24
B.580	Asphalt Works	20	28-Dec-2	24-Jan-24
B.590	Installing Rail	7	19-Dec-2	27-Dec-23
B.600	Installing Lampposts	9	25-Jan-2	06-Feb-24
B.620	Glass and Painting	5	14-Feb-2	20-Feb-24

Figure8. 1 Duration of project

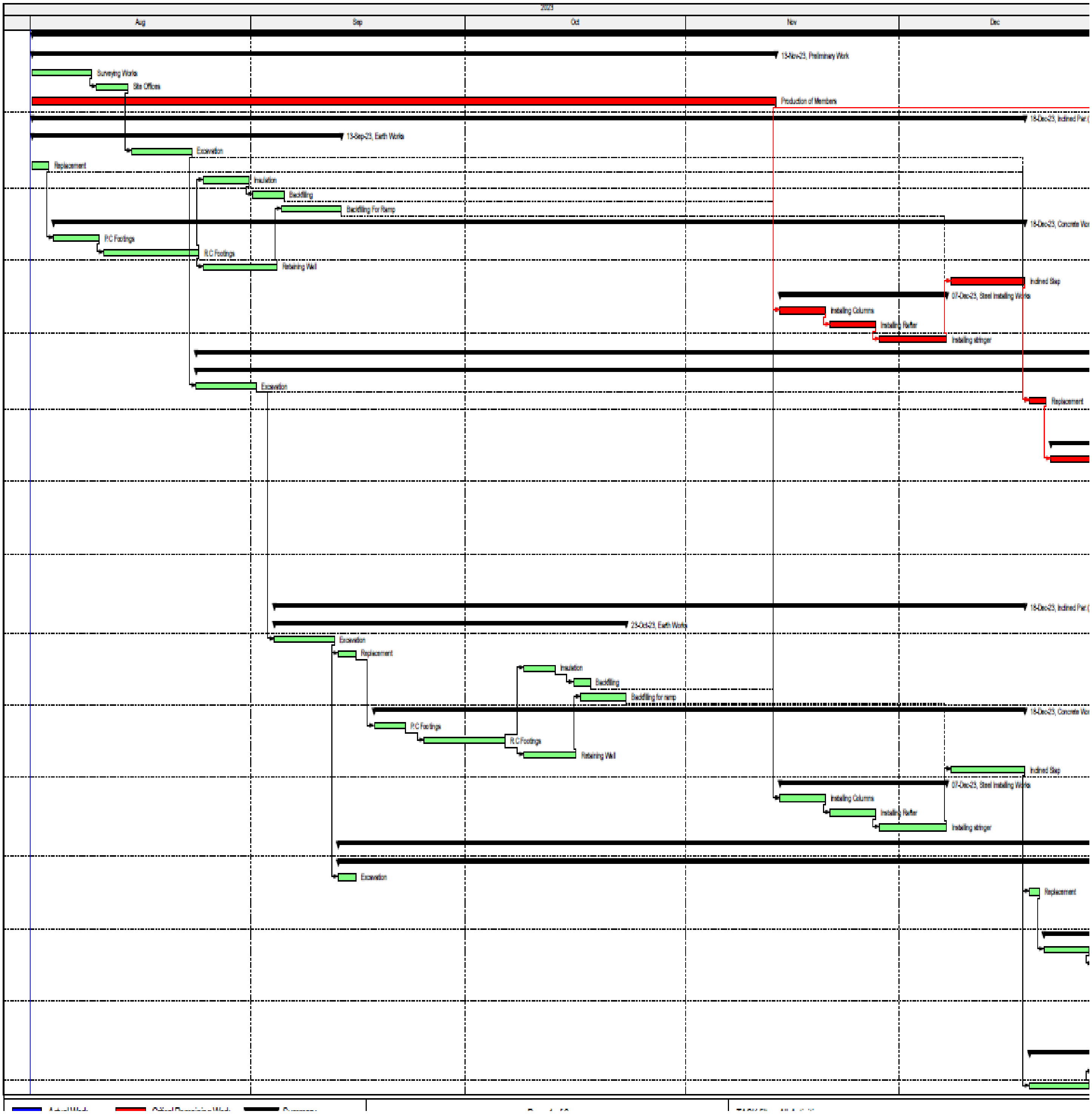


Figure8. 2 Flow chart

Chapter 9 (Conclusion)

This thesis explores the construction of a steel bridge at the intersection of two main roads, with a focus on the management and design aspects. The research has shown that this type of construction can solve public problems and offer significant benefits to those living and working in the area. It is clear that careful planning, design, and management are essential for any successful roadway steel bridge project. Through this thesis, we have gained an understanding of how these factors interact to create a successful outcome. As such, we can confidently conclude that roadway steel bridges can be beneficial for those living in their vicinity.

(Reference)

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