

STRUCTURAL STEEL ANALYSIS DESIGN OF COMMERCIAL FACTORIES FOR LOADING AND EARTHQUAKE RESISTANCE

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ABSTRACT

In the university my courses According to the widely used of "Steel buildings " in these days which taught us to analyze and design this flooring system that can achieve the requirements of this structure. Also use several programs like (ETAB, SAP , ROBOT & TEKLA etc), beside that I used different combinations for earthquake resistance. It analysis and design according to the eurocode AISC 14.0 , in addition to that I used the program of Autodesk Robot Structural, AUTO CAD 2013 & Tekla Structure. while I have many components I used that program to help me for designing, by include the loads as input, then I got different diagrams and diameters for each part.

Keyword : Steel connection & Loading

INTRODUCTION

Most design tasks in structural engineering are based both on theoretical models and empirical evidence. The theory is developed into a design procedure, and this procedure is carried out and tested. The results of these tests are then used to formulate new and more accurate analysis procedures. So, to some extent, this is an iterative process.

Centuries of experience with building structures as well as new developments in analytical procedures have led to a greater understanding of how structures work. This is especially true in the area of connections in structures. There is a fairly high degree of uncertainty in the behavior of connections, which makes their analysis and design difficult, and makes experimental verification so important. Many of the elements of the connections have variability in their properties, which, when analyzed together, further increase the uncertainties in the connection. The consequence of this is that careful and accurate design of connections can be crucial to the design of the structure. This thesis attempts to understand and compare various types of connections in structural steel buildings, including both the modeling theory of connections as well as the physical components of connections.

Formulation Of The Problem

1. How to make the building earthquake resistant?
2. How to make the building static and how to make the building economic and construction products relevant for steel structures?
3. Why we will use claddings working in one-way (horizontal) direction and an analyzed?

AIM

1. Agree exactly what a project is meant to do and what it is meant to deliver.

2. Agree the design, timescale, analysis and design and quality of project.
3. Understand and apply methods and approaches adopted by the code regarding the analyzed structure.
4. Comprehend the method of designing steel structures.
5. Establish a link between analysis of structures and the design process.
6. Enhance applying computer package programs such as: ROBOT, AUTOCAD, TEKLA and Microsoft office to determine the calculation process and obtain the required results.
7. Understand the exact and suitable ways to analyze and design.
8. Prepare myself for work environment to be an engineer.

LITERATURE REVIEW

Design of Bolted Connections with Injection Bolts

- **Design rules for static loading**

As of now demonstrated previously, injection bolts might be non-preloaded load move through bearing and shear of the bolt and furthermore preloaded load move through bearing and shear of the bolt and through grinding between the associated plates. The ECCS proposals give design rules for both types of shear connections.

- **Bearing and shear:**

The plan extreme shear load shall not surpass the plan shear opposition of the bolt nor the plan bearing obstruction of the resin.

Serviceability limit state:

$$F_{b.Rd.ser.resin} = \frac{1.0 \cdot K_s \cdot d \cdot t \cdot \beta \cdot f_{b.resin}}{\gamma_{Ms.ser}}$$

Ultimate limit state:

$$F_{b.Rd.ult.resin} = \frac{1.2 K_s \cdot d \cdot t \cdot \beta \cdot f_{b.resin}}{\gamma_{Ms.ult}}$$

Shear resistance of the bolt (Eurocode 3, 1992), shear plane through through the shank of the bolt).

$$F_{v.Rd} = \frac{0.5 f_{Ub} A_s}{\gamma_{Mb}} KN$$

Design Methods for support and Bracing Connections

Accidental activities:

- Drifted snow decided utilizing Annex B of BS EN 1991-1-3 (see Section 3.3.3).
- Opening of a prevailing opening thought to be closed.
Robustness requirements of BS EN 1991-1-7 and its National Annex

Combination

Table 1

Action	Ψ_0	Ψ_1
Imposed loads in buildings, category (see EN 1991-1-1)		
Category A: domestic, residential areas	0.7	0.5
Category B: office areas	0.7	0.5
Category C: congregation areas	0.7	0.7
Category D: shopping areas	0.7	0.7
Category E: storage areas	1.0	0.9
Category H: roofs	0.7	0
Snow loads on buildings (see EN 1991-3)		
- for sites located at altitude H > 1 000m a.s.l.	0.70	0.50
- for sites located at altitude H ≤ 1 000m a.s.l.	0.50	0.20
Wind loads on buildings (see (EN 1991-1-4)	0.5	0.2
Temperature (non-fire) in buildings (see EN 1991 -1 -5)	0.6	0.5

Serviceability limit state

$$\sum_{j>1} G_{k,j} + Q_{k,1} \sum_{j>1} \Psi_{0,i} Q_{k,i}$$

Table 2. Suggested limits for vertical deflections

Vertical deflection	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
Other beams (except purlins and sheeting rails)	Span/200
Purlins and sheeting rails	To suit the characteristics of the particular cladding

Loading to the Eurocodes:

Table 3. Suggested limits for horizontal deflections:

Horizontal deflection	
Tops of columns in single-story buildings except portal frames.	Height/300.
Columns in gateway frame structures, not supporting crane runways.	To suit the qualities of the specific cladding.
In every story of a structure with more than one story.	Height of that story/300.

Connections

Connections are initially assumed as pins, thereby implying that the centroidal axes of all members intersecting at a node point are coincident. Practical considerations frequently dictate otherwise, and it is quite common for member axes to be eccentric to the assumed node for reasons of fit-up and the physical constraints that are inherent in the truss structure. Such eccentricities induce secondary bending stresses of the node points, which must be accounted for not only by local bending and axial load checks at the ends of all constituent members, but also in connection design. Typical truss joints are illustrated. It is customary to calculate the net bending moment at each node point due to any eccentricities, and proportion this moment to each member connected to the node in relation to member stiffness.

Portal frames

By far the most common structural form for single-storey buildings is the portal Frame. Various configurations of portal frame

Table 4

Section 4.6	Preliminary design of portal frames
Section 4.7	Bracing
Section 4.8	Design of portal frames to BS EN 1993-1-1.

Bracing

Bracing is required to resist lateral loads, principally wind loads, and the destabilising effects of the imperfections defined in Section 5.3 of BS EN 1993-1-1

Bracing at plastic hinges:

$$Q_m = 1.5a_m \frac{N_{f,Ed}}{100}$$

First-order and second-order analysis

For portal frames with shallow roof slopes, provided that the axial compression in the beams or rafters is not significant and $a_1 \leq 3.0$ the 'amplification factor' can be calculated according to: $\left(\frac{1}{1-1/a_{cr}}\right)$

Secondary beams:

Table 5. Beams as secondary beams (IPE/HE sections in S355 steel) of sizes

Cellular beam	Maximum span				
	12	13.5	15	16.5	18
Opening diameter (mm)	300	350	400	450	500
Beam depth (mm)	460	525	570	630	675
Top chord	IPE 360	IPE 400	IPE 400	IPE 400	IPE 450
Bottom chord	HE 260 A	HE 260 A	HE 300 A	HE 360 B	HE 40 M

Table 6. Composite cellular beams as secondary beams (UC sections in S355 steel) of sizes

Cellular beam	Maximum span				
	12	13.5	15	16.5	18
Opening diameter (mm)	300	350	400	450	450
Beam depth (mm)	415	490	540	605	625
Top chord	305 UC 54	356 UC 67	406 UC 67	457 UC 67	457 UC 82
Bottom chord	254 UC 89	305 UC 54	305 UC 137	356 UC 153	356 UC 287

Tension members

tension member (or tie) transmits a direct tensile force

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \quad , \quad N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}}$$

Columns and struts

Members subject to compression are typically referred to as either columns

$$N_{cr} = \frac{Af_y}{N_{cr}} \quad , \quad N_{cr} = \frac{\pi^2 EI}{L_{cr}^2}$$

Column buckling resistance

Is a function of its slenderness, its material strength, cross-sectional shape and method of manufacture. Using BS EN 1993-1-1, column buckling resistance

$$N_{pl,Rd} = \frac{\chi Af_y}{\gamma_{M1}} \quad \text{for class 1, 2 and 3 sections ,}$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad \text{for class 4 sections}$$

buckling reduction factor defined

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1.0$$

$$\Phi = 0.5 \left[1 + a (\lambda - 0.2) + \lambda^2 \right]$$

Table 7. Imperfection factors a

Buckling curve	??	a	b	c	d
Imperfection factor a	0.13	0.21	0.34	0.49	0.78

Beams

Beams are possibly the most fundamental type of member present in a civil engineering structure

Table .8

beam type	span range (m)	notes
Angles	1-6	Used for roof purlins, sheeting rails, etc. where only light loads have to be carried
Cold-formed sections	2-8	Used for roof purlins, sheeting rails, etc. where only light loads have to be carried
Rolled sections: UBs, IPEs, UCs, Hes	1-30	Most frequently used type of section; proportioned to eliminate several possible modes of failure
Open web joists	4-40	Prefabricated using angles or tubes as chords and round bar for the web diagonals, used in place of rolled sections
Cellular beams	6-60	Used for long spans and/or light loads; depth of rolled section increased by 50%; web openings may be used for services etc.
Compound sections	5-30	Used when a single rolled section would not provide sufficient resistance
Plate girders	10-100	Made by welding 3 plates, often automatically, with web depths up to 3-4 m; may need stiffening - see Chapter 18
Trusses	10-100	Fabricated from angles, tubes or, if spanning large distances, rolled sections - see Chapter 20
Box girders	15-200	Fabricated from plate, usually stiffened; used for overhead travelling cranes and bridges due to good torsional and transverse stiffness properties

S355 steel:

All but 1 Universal Beams are Class 1 (1 is Class 2). All but 6 Universal Columns are Class 1 (3 are Class 2,3 are Class 3)

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \quad , \quad M_{c,rd} = \frac{W_{el} f_y}{\gamma_{M0}}$$

$$\varepsilon = \sqrt{235 / f_y}$$

Determination of beam slenderness $\bar{\lambda}_{LT}$

Hence, lateral torsional buckling resistance $M_{b,Rd}$ is given by:

$$M_{b,Rd} = \frac{\chi_{LT} W_y f_y}{\gamma_{M1}}$$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L_{cr}^2} \sqrt{\frac{I_w}{I_z} + \frac{L_{cr}^2 G I_t}{\pi^2 E I_z}}$$

Bolted connections

For many years, riveting was the accepted method used for connecting the members of steel structures

Table. 9

Bolt Diameter	Standard (Dia)	Oversize (Dia)	Short-slot (width X Length)	Long slot (Width X Length)
1 2	9 16	5 8	9 16 ?? 11 16	9 16 ?? 1 4
5 8	11 16	13 16	11 16 ?? 7 8	11 16 ?? 1 9 16
3 4	13 16	15 16	13 16 ?? 1	13 16 ?? 1 7 8
7 8	15 16	1 1 16	15 16 ?? 1 1 8	15 16 ?? 2 3 16
1	1 1 16	1 1 4	1 1 16 ?? 1 5 16	1 1 16 ?? 2 1 2
= 1 1 8	?? + 1 16	+ 5 16	?? + 1 16 ?? ?? + 3 8	?? + 1 16 ?? 2.5 ?? ??

Bearing-Type connections (Bearing strength):

- ✓ If deformations around bolt holes is a design consideration (that is, if we want deformations to be ≤ 0.25 in), that

$$R_n = 1.2 l_c F_u \leq 2.4 dt F_u$$

- ✓ If deformation around bolt holes is not a design consideration (that is, if deformations > 0.25 in are acceptable)

$$R_n = 1.5 l_c F_u \leq 3.0 dt F_u$$

- ✓ For bolts used in connections with long-slotted holes, the slots being perpendicular to the forces.

$$R_u = 1.0 l_c t F_u \leq 2.0 dt F_u$$

RESEARCH METHODOLOGY

Type of use – residential building;

Span – L=6m;

Bay – B1=6m, B2=6m;

1. Building size: 337.9 m²
2. Total plot area = 425.1 m²
3. Percentage of free space = 3 ~ 4%
4. Location in North Cyprus – Turkey – Near East University.
5. Grade of concrete – C30 – S420, S220 STIRRUBS (MPA)
6. Ground Floor height 400 cm.
7. Slab thickness t=20 cm.
8. Type of column = W10x88
9. Type of Beam = W8x58
10. Type of slab beam = W8x40
11. Type of Purlin (1) = MC10x33.6
12. Type of Purlin (2) = L8x6x1
13. Type of Brace = PIPE10STD
14. Live load on slab = 3.0t/m² –
15. Dead load = 0.15t/m²
16. Site located on Seismic Zone 2.

Scheme of the building is given on the following visualization:

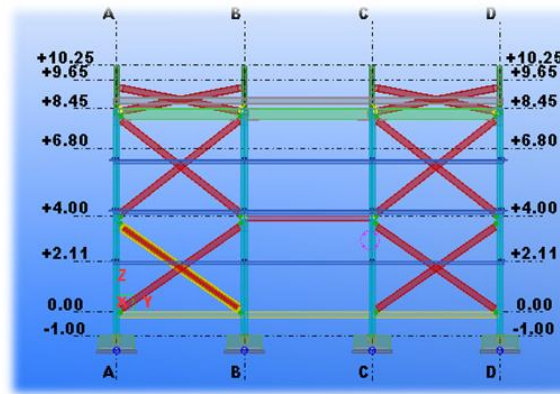


Fig. 1

Structural analysis

The structural model for the analysis was created in software ROBOT. Following input data is used for model consideration:

- Beams in plane xz are rigidly connected to the steel columns.
- The beams in plane yz are hinged at both ends. Releases for hinged connections are indicated in following directions: Ry, Rz.
- Elements defining bracing system are also hinged at both ends.
- Supports are pinned. Fixed directions of pinned support: Ux, Uy, Uz, Rz.11
- Bracings in axis A1-A2, D1-D2, A3-A4, D3-D4 are represented by one bar per frame assuming that it will work in tension and compression.
- The concrete slab has a strong influence on the global stiffness of the structure. In ROBOT 3D model concrete slab was modeled by a horizontal bracing system, connected to main columns. Connection of these bracings are hinged.
- To identify the type of analysis which should be performed (1st or 2nd order) we calculate a_{cr} for ultimate limit state combinations.

In all combinations $a_{cr} > 10$. Therefore, according to EN 1993-1-1 1st order elastic analysis should be performed.

MATERIAL:

ASTM A36 STEEL PLATE

CHARACTERISTICS

ASTM A36 plate is a low carbon steel that exhibits good strength coupled with formability. It is easy to machine and fabricate and can be securely welded.

A36 STEEL PLATE SIZES

3/16" – 28"

TYPICAL CHEMICAL PROPERTIES

Table 10

Carbon, Max %	0.026
Manganese, Max %	0
Phosphorus, Max %	0.04
Sulphur, Max %	0.05
Silicon, Max %	0.4
Copper, Max %	0.20

ASTM A36 BEAM

ASTM A36 beam is an intermediate tensile strength carbon steel that is easy to form, machine and weld

✓ TYPICAL MECHANICAL PROPERTIES

Table 11

Tensile Strength, ksi	58-80
Yield Point, ksi	36

ASTM A36 CHANNEL

ASTM A36 is a hot-rolled, low-alloy carbon steel that exhibits good tensile strength. Its properties include ductility, ease of machinability and welding.

SIZES

Depths: 0.5" thru 18"

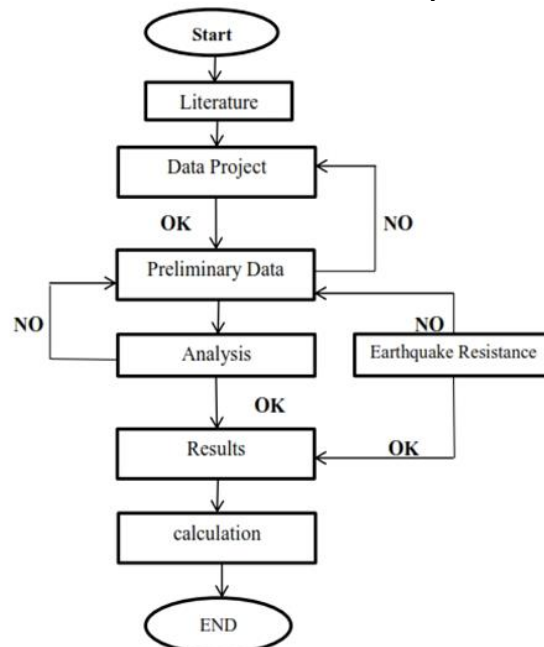
Flange Widths: 0.125" thru 5"

TYPICAL MECHANICAL PROPERTIES

Table 12.

Yield Point ksi, min	36 min
Tensile Strength ksi	58-80
% Elong. 2" Bend	23

Table 13. Flowchart Of Project



DRAWING BY TEKLA

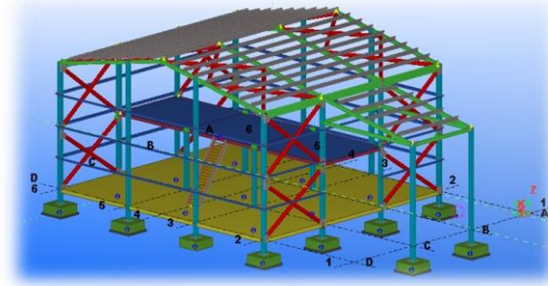


Fig 2

RESULTS AND DISCUSSION

Structural analysis

- Beams in plane xz are rigidly connected to the steel columns.
- The beams in plane yz are hinged at both ends. Releases for hinged connections are indicated in following directions: Ry, Rz.

Table .14

		Limit values
beam-to-column	W8x58 - W10x88	0.26
beam-to-beam	W8x58 -	0.98
beam-column (web) connection	W8x58 - W10x88	0.54
Fixed Column base design	W10x88	0.87
Bracing	PIPE10STD	
Horizontal deflection (Δ)	2	2.4
Horizontal deflection first floor (δ_1)	1.3	1.33
Bolt bearing on the beam Direction x		2.50
		0.69
Bolt bearing on the beam Direction z		2.50
		0.81

Bracings in axis A1-A2, D1-D2, A3-A4, D3-D4 are represented by one bar per frame assuming that it will work in tension and compression

Table . 15

	a_{cr} (mode 1)
Combination 1	10.95
Combination 2	10.95
Combination 3	11.46
Combination 4	11.46
Combination 5	11.66
Combination 6	11.66

- Design of fixed beam-to-column connection (Frame knee connections)

Weakest component:

BOLT RUPTURE

Remarks

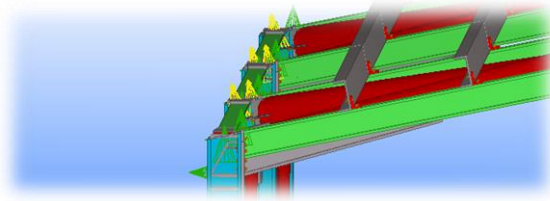


Fig3

The thickness of bracket web is less than the thickness of beam web
 $8 \text{ [mm]} < 13 \text{ [mm]}$

The thickness of bracket flange is less than the thickness of beam flange
 $12 \text{ [mm]} < 21 \text{ [mm]}$

Table . 16

Beam resistances	0.01
Column resistances	0.14
Connection resistance for compression	0.07
Connection resistance for bending	0.26
Connection resistance for shear	0.02
Weld resistance	24.30 24.20 12.15

Table . 17 SUMMARY TABLE OF FORCES

Nr	h_j	$F_{Tj,Rd}$	$F_{Ttc,Rd}$	$F_{Twc,Rd}$	$F_{Tsp,Rd}$	$F_{Twb,Rd}$	$F_{T,Rd}$	$B_{p,Rd}$
1	312	117.99	117.99	380.85	117.99	347.67	117.99	574.39
2	242	91.50	117.99	380.85	117.99	347.67	117.99	574.39
3	142	53.67	117.99	380.85	117.99	347.67	117.99	574.39
4	92	34.75	117.99	380.85	117.99	347.67	117.99	574.39
5	42	15.83	117.99	380.85	117.99	347.67	117.99	574.39

- Design of fixed beam-to-beam connection

Weakest component

FRONT PLATE – TENSION

Remarks

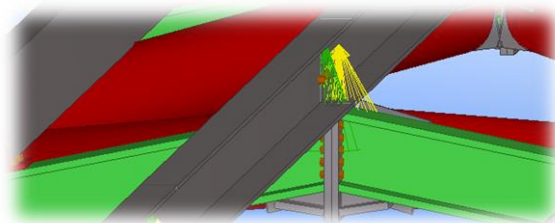


Fig 4

Distance of bolts from an edge is too small

$$20 \text{ [mm]} < 20 \text{ [mm]}$$

Bolts vertical spacing is too small.

$$35 \text{ [mm]} < 37 \text{ [mm]}$$

Table . 18

Beam resistances	0.00
Connection resistance for compression	0.03
Connection resistance for bending	0.98
Connection resistance for shear	0.00
Weld resistance	97.03
	97.3
	48.52

SUMMARY TABLE OF FORCES

The remaining bolts are inactive (they do not carry loads) because resistance of one of the connection components has been used up or these bolts are positioned below the Table . 15

Table 19 center of rotation

Nr	h _j	F _{y,Rd}	F _{Ltc,Rd}	F _{Lwc,Rd}	F _{Lsp,Rd}	F _{Lwb,Rd}	F _{LtRd}	B _{p,Rd}
1	151	117.99	-	-	117.99	549.70	117.99	574.39
2	116	90.67	-	-	117.99	549.70	117.99	574.39
3	81	63.35	-	-	117.99	549.70	117.99	574.39
4	46	36.03	-	-	117.99	549.70	117.99	574.39
5	-34	-	-	-	117.99	-	117.99	574.39
6	-84	-	-	-	117.99	-	117.99	574.39

- Calculation of the beam-column (web) connection

Remarks

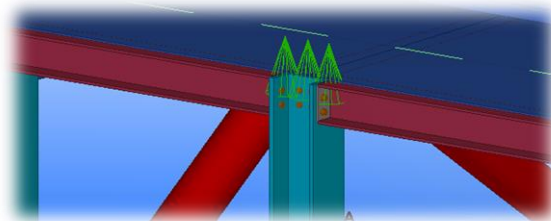


Fig 5

Distance between plate horizontal edge and beam upper flange is too small

$$21 \text{ [mm]} < 30 \text{ [mm]}$$

Length of plate connecting the beam is too large

$$180 \text{ [mm]} > 161 \text{ [mm]}$$

Distance between bolt and edge of beam flange is too small

$$21 \text{ [mm]} < 26 \text{ [mm]}$$

Distance between bolt and plate horizontal edge is too small

$$20 \text{ [mm]} < 20 \text{ [mm]}$$

Table . 20

Bolts connecting beam with plate Direction x	k_{1x} 2.50 σ_{bx} 0.69
Direction z	k_{1z} 2.50 σ_{bz} 0.81
Bolt bearing on the plate Direction x	k_{1x} 1.62 σ_{bx} 0.69
Direction z	k_{1z} 2.50 σ_{bz} 0.40
Verification of the section due to block tearing (Plate)	-1.16
Verification of the section due to block tearing (beam)	-2.31
Weld resistance	-60.22 120.45

- Fixed column base design

Weakest component:

BASE PLATE – BENDING

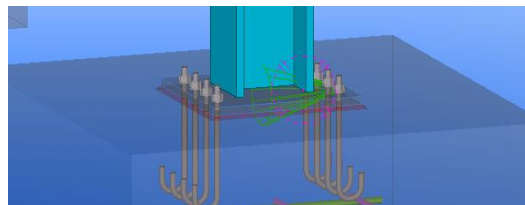


Fig 6
Table . 21

Connection capacity	0.87
SHEAR	0.16
Welds between the column and the base plate	0.09

CONCLUSION

Finally, I am going to summary what we have done in this project during the semester briefly:

1. Each student had his own dimensions of the construction then we learnt how to use ETAB program to design the buildings but I used ROBOT program instead of ETAB because I am familiar with ROBOT in addition both programs have the same functions after that we moved to another step which is putting the loads in the program (all the loads), after loading process we had learnt how to design the connection by using ROBOT program and we did so for our building actually it was sensitive step because the safety of construction depends on connection safety mainly
2. Then we used TEKLA program to draw the project after that we converted our drawings from TEKLA program to AUTOCAD program.
3. At the end, we had used four programs we got too much benefit from these programs and by finishing this project now we are qualified to work and design with a real projects but we still need more experience.

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