

In the name of Allah



Palestine Polytechnic University

Faculty of Engineering and Technology

Department of Civil Engineering and Architecture

Specialization in Civil Engineering Branch Building Engineering

Project Name

Structural design of Palestine TV& media center

Team :

Aziz N. Salaimeh

Sufian Natsheh

Ayman M. Salhab

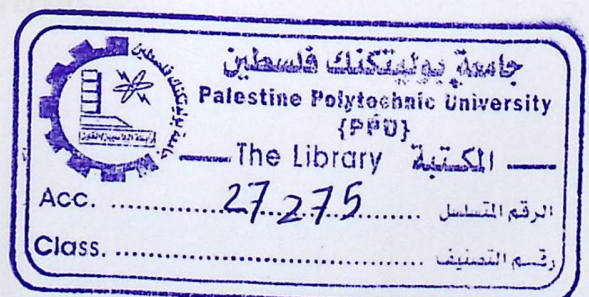
Mohannad Ebiedo

Supervision :

D. Haitham Ayad.

Palestine – Hebron

2013.2014



In the name of Allah

Structural Design and Details of Palestine TV& media center

Team :

Aziz N. Salameh

Sufian Natsheh

Ayman M. Salhab

Mohannad Ebiedo

Supervision :

D. Haitham Ayad.

Palestine – Hebron

2013.2014

Report of graduation project

**Introduce to the Department of Civil Engineering and Architecture in the
College of Engineering and Technology in
Palestine Polytechnic University**

**To meet part of the requirements for obtaining
Bachelor's degree in engineering specialty buildings engineering.**



**Faculty of Engineering and Technology
Department of Civil Engineering and Architecture
Palestine Polytechnic University**

Hebron – Palestine

2013-2014

Certificate of evaluate graduation project

Palestine Polytechnic University

Hebron . Palestine



Structural Design and Details of Palestine TV& media center

Team :

Aziz N. Salaimeh

Sufian Natsheh

Ayman M. Salhab

Mohannad Ebiedo

Based on the Instruction of D.Haytham Ayyad and the approval of all members of the Committee of the discussion, this project was Introduced to the Department of Civil Engineering and Architecture in the College of Engineering and Technology for partial fulfillment of the requirements for the bachelor's degree circle.

supervisor Signature

D. Haitham Ayad

Head of the department Signature

D. Ghassan Dweik

June 2014

Thanks **Dedicate** Appreciation

Thanks to Allah for his guidance and providence!

We, the work team, would like to take the opportunity to write

To those who have always believed in me and given me wings to fly and told me that there are no limits in the sky .

To those who have helped me throughout my learning years without ever grumbling about my curiosity and appetite to knowledge .

To those who have always showered me with unwavering support and care.

To those who know themselves and know what they mean to me without the need of articulation

Those are my family, friends and teachers and for them I dedicate this research, hoping that- by doing so- I am repaying them a little amount of what they owe me.

Thanks and appreciation

Thanks be to Allah for his guidance and providence!

We, the work team, would like to take the opportunity to wholeheartedly thank everyone who supported the project and nurtured it until it has become what it is now.

We especially thank:

- Palestine Polytechnic University, Engineering and Technology College, Civil and Architectural Engineering Department, including all members of the helpful and reverend staff.
- All lecturers in the university. We particularly like to thank Dr. Haytham Ayyad who extended a great effort in drawing the Figure of the project.
- The library of the university including its entire staff for providing us with the needed books and resources.
- Everyone who extended a hand in the project and was of great help

Much appreciated!

Project Summary

Structural Design and Details of Palestine TV& media center

Team :

Aziz N. Salaimeh

Sufian Natsheh

Ayman M. Salhab

Mohannad Ebiedo

Supervision :

D. Haitham Ayyad.

Palestine – Hebron

2013.-2014

The idea of this project is the structural design of Palestinian Radio and Television Building. The project is integrated radio and television building with all needs of the student and researcher, where there is a sufficient number of Decor Workshops, makeup & clothes , Decoration Engineers offices , audience's Studios , Maintenance Workshops , Stores , news Studio , Restaurant , Montage department and all services needed .

The building consists of seven floors with a total area of 29,000 m2 approximately . The design will be based on the requirements of the American Code (ACI -318-08), and the Jordanian Code of loads. And we will use some of software like (Office 2003) and (AutoCAD 2014) to project output, and also we use some of programs for structural design and analysis like, Atir , Safe 12.3, Etaps 9.7.1, Sap 2000, DeconSTDesign, Pca-column.

Table of Contents

Page #	
i	The title page
ii	project report Page
iii	evaluation certificate graduation project Page
iv	Dedication Page
v	thanks and appreciation Page
vi	Summary page

6	<u>Architectural Description</u>	Chapter 2
7	Introduction	1.2
7	Basic Identification Of Project	2.2
7	The project site	2.3
8	Project Land Location	2.3.1
8	General Climate Of The City	2.3.2
9	Contour Lines Of The Project Land	2.3.3
10	project components description	2.4
10	project plans description	2.4.1
15	project Elevations description	2.4.2
17	Vertical movement description	2.4.3

18	<u>Structural description</u>	Chapter 3
18	Introduction	3.1
19	The purpose of structural design	3.2
20	Theoretical studies of the structural elements of the building	3.3
20	Types of loads	3.4
20	Dead loads	3.4.1
21	Live loads	3.4.2
21	Environmental loads	3.4.3
21	Seismic	3.4.3.1
22	wind	3.4.3.2

22	snow	3.4.3.3
22	Practical tests	3.5
22	Structural elements	3.6
23	Slabs	3.6.1
23	The ribbed slabs	3.6.1.1
24	Solid slabs	3.6.1.2
25	flat slabs	3.6.1.3
25	Beams	3.6.2
26	Columns	3.6.3
26	Shear walls	3.6.4
27	Foundation	3.6.5
28	Stairs	3.6.6
29	Expansion join's	3.6.7

30	Structural Design & Analysis	Chapter 4
31	Introduction	4.1
31	Determination of Slab thickness	4.2
32	Determination the Loads of ribs	4.3
33	Determination of dead load	4.3.1
33	Determination of factored dead & live loads	4.3.2
34	Design of topping	4.4
36	Design of rib 0-17	4.5
38	Design of flexure	4.5.1
38	Design of positive moment of rib 0-17	4.5.1.1
40	Design of negative moment of rib 0-17	4.5.1.2
43	Design of shear of rib 0-17	4.5.2
46	Design of beam 0-55	4.6
47	Determination of Dead load of beam	4.6.1
47	Determination of live load of beam	4.6.1.1
47	Determination of factored dead& live load	4.6.1.2
49	Design of flexure	4.6.2
49	Design of positive moment	4.6.2.1
53	Design of negative moment	4.6.2.2
54	Design of shear	4.6.2.3
60	Cut of Bars Calculations	4.6.2.4

62	Design Of two Way Rib Slab	4.7
62	Determination of Thickness for Two Way Rib Slab	4.7.1
65	Load Calculation	4.7.2
65	Determination of factored dead & live load	4.7.2.1
65	Determination of factored dead & live load	4.7.2.2
66	Design for moment	4.7.3
73	Design for shear	4.7.4
75	Design Flate plate.	4.8
75	Drawing the Plan as DXF	4.8.1
76	Determination of Slab Thickness	4.8.2
77	Determination of the Loads	4.8.3
78	Check the Allowable deflection	4.8.4
79	Design of Moment	4.8.5
80	Design of punching shear	4.8.6
81	Design of Stairs	4.9
81	Determination of Slab Thickness	4.9.1
82	Load Determination	4.9.2
83	Design of flight	4.9.3
84	Design of Shear for flight	4.9.4
85	Design of Bending Moment for Flight	4.9.5
87	Design of landing	4.9.6
88	Design of Shear for Landing	4.9.7
88	Design of bending moment for landing	4.9.8
91	Design of Short Column	4.10
91	Load Calculation	4.10.1
91	Check Slenderness Effect	4.10.2
92	Select the longitudinal bars	4.10.3
92	Design for ties	4.10.4
93	Check for code requirements	4.10.5
94	Design of long column (C6-58)	4.11
94	Check the slenderness effect	4.11.1
94	Calculate e_{min} , M_{min}	4.11.2
95	Determine of Euler buckling load	4.11.3
95	Calculate the moment magnifier factor	4.11.4
96	Select the longitudinal bars	4.11.5
96	Design the stirrups	4.11.6

96	Check for code requirements	4.11.7
97	Design of punching (two way shear) at interior column in flat plate	4.12
97	Load calculations	4.12.1
98	Compute the shear force acting on the critical section	4.12.2
98	Compute ϕV_c	4.12.3
99	Using shear stirrups as shear reinforcement	4.12.4
100	Now check the force stresses at the outer critical section	4.12.5
101	Shear wall design	4.13
102	Check max shear strength permitted	4.13.1
102	Calculate shear strength provided by concrete	4.13.2
103	Determine required horizontal reinforcement	4.13.3
103	Determine required vertical reinforcement	4.13.4
104	Design for flexure (uniformly distributed flexure reinforcement)	4.13.5
106	Design of shear wall using Etabs program	4.13.6
108	Design of Isolated Footing (F4)	4.14
108	Area of Footing	4.14.1
109	Depth of footing	4.14.2
111	Design of flexural reinforcement:	4.14.3
112	Development length of flexural reinforcement	4.14.4
113	Development length of column reinforcement	4.14.5
114	Lap splice of column	4.14.7
115	Design of strip Footing	4. 15
115	Determination of load	4.15.1
116	Check of One Way Shear	4.15.2
116	Design of Bending Moment	4.15.3
117	Development Length of main Reinforcement	4.15.4
119	Design Of Dom	4.16
120	Load calculation	4.16.1
121	Steel members design	4.16.2
122	Design of weld	4.13.4

123	Estimated Cost	Chapter 5
124	Excavation and backfilling work	5.1
124	General notes for excavation	5.1.1
126	The estimated quantities and prices	5.1.2
127	Concrete work	5.2
132	Stone Works	5.3

134	Appendixes	Chapter 6
135	Appendix A: Architectural Drawings	6.1
136	Appendix B: Structural Drawings	6.2
137	Results	6.3
138	Recommendations	6.4
138	Sources and references	6.5

Index of Tables		
5	Project schedule during the academic year	Table(1.1)
21	Specific density of the materials used	Table (3.1)
21	Live loads	Table (3.2)
22	Loads of snow by sea level	Table(3.3)
33	Dead loads of the one way ribbed slab	Table(4.1)
34	Dead loads of the topping	Table(4.2)
47	Dead load of beam	Table (4.3)
65	Dead loads of two way ribbed slab	Table (4.4)
Index of Figures		
7	general picture of project	Figure (2.1)
8	The location of the land.	Figure (2.2)
9	winds movement	Figure(2.3)
9	The topography of the land	Figure (2.4)
10	Basement floor plan	Figure (2.5)
11	ground floor plan	Figure(2.6)
11	first floor plan	Figure (2.7)
12	second floor plan	Figure(2.8)
12	third floor plan	Figure(2.9)
13	forth floor plan	Figure(2.10)
13	fifth floor plan	Figure(2.11)
14	sixth floor plan	Figure(2.12)

14	seventh floor plan	Figure (2.13)
15	North East Elevation	Figure (2.14)
15	south East Elevation	Figure (2.15)
16	North west Elevation	Figure (2.16)
16	south west Elevation	Figure(2.17)
17	section A.A	Figure(2.18)
17	section B.B	Figure (2.19)
20	dead loads	Figure(3.1)
21	live load	Figure (3.2)
23	real one way rib & sample in the project	Figure (3.3)
24	real two way rib & sample in the project	Figure(3.4)
24	solid slabs	Figure(3.5)
25	flat slabs	Figure(3.6)
25	drop beam	Figure(3.7)
25	hidden beam	Figure(3.8)
26	circular column	Figure (3.9)
26	rectangular column	Figure(3.10)
26	shear wall	Figure(3.11)
27	isolated footing	Figure(3.12)
27	strip footing	Figure (3.13)
28	Mat footing	Figure (3.14)
28	stairs	Figure (3.15)
29	expansion joints	Figure(3.16)
31	Ground Floor Slab	Figure(4.1)
32	Typical Section in Ribed slab	Figure (4.2)
34	Both fixed end static system	Figure(4.3)
36	Rib 0.17 geometry.	Figure(4.4)
36	loading of Rib 0.17	Figure (4.5)
37	Moment Envelop of rib 0.17	Figure(4.6)
37	Shear Envelop of rib 0.17	Figure(4.7)
46	Beam 0.55 Plan	Figure(4.8)
48	Beam (0.55) geometry	Figure(4.9)
48	Beam (0.55) loading	Figure(4.10)
48	Beam (0.55)Moment diagram	Figure(4.11)

49	Beam (0.55) shear & reactions diagram	Figure (4.12)
62	Two way rib slab(0.15).	Figure(4.13)
62	T Section .	Figure(4.14)
66	Moment Slab	Figure(4.15)
75	plan on AutoCAD as DXF	Figure(4.16)
76	plan on SAFE Program	Figure(4.17)
77	dead load	Figure(4.18)
77	live load	Figure(4.19)
78	long term deflection for the flat slab	Figure(4.20)
79	moment in X-direction	Figure(4.21)
79	moment in Y-direction	Figure(4.22)
79	Bottom reinforcement of ϕ 12@20 cm c/c in both direction	Figure(4.23)
80	Top reinforcement of ϕ 14@15cm c/c in both direction	Figure(4.24)
80	punching shear of slab	Figure(4.25)
81	stairs details	Figure(4.26)
83	loads on stairs	Figure(4.27)
83	Moment on stairs	Figure(4.28)
84	Shear on stairs	Figure(4.29)
87	Loads on Landing.	Figure(4.30)
87	Moment Diagram for landing	Figure(4.31)
87	Shear Diagram for landing.	Figure(4.32)
90	Sections Details for stairs.	Figure(4.33)
93	C-1-60 detail	Figure(4.34)
95	The results from PCA_COLUMN program.	Figure(4.35)
96	C6-58 detail	Figure(4.36)
97	The location of column C6-58	Figure(4.37)
99	Detail of stirrups.	Figure(4.38)
100	Ties Details	Figure(4.39)
101	The shear wall.	Figure(4.40)
101	Shear and moment for the shear wall.	Figure(4.41)
105	top view & section for the wall	Figure(4.42)
106	The shear wall (w13).	Figure(4.43)
106	shear force for the shear wall	Figure(4.44)
106	Area of steel required for the shear wall	Figure(4.45)

107	Maximum story displacement.	Figure(4.46)
108	detail for the footing	Figure(4.47)
109	One way shear.	Figure(4.48)
110	two way shear	Figure(4.49)
111	Max. moment	Figure(4.50)
114	details of the footing	Figure(4.51)
115	location strip footing	Figure(4.52)
118	Strip Footing Detail	Figure(4.53)
119	SAP Model of the Dom	Figure(4.54)
120	Snow load on SAP model	Figure(4.55)
121	Wind load on SAP model	Figure(4.56)
121	Members sections on SAP model	Figure(4.57)
122	Joint between Vertical and horizontal	Figure(4.58)
123	Joint between Vertical and horizontal show the welding	Figure(4.59)
124	Joint between Vertical and top horizontal	Figure(4.60)
125	Joint between Vertical and top horizontal show welding	

List of Abbreviations

- A_c = area of concrete section resisting shear transfer.
- A_s = area of non-prestressed tension reinforcement.
- A_s = area of non-prestressed compression reinforcement.
- A_g = gross area of section.
- A_v = area of shear reinforcement within a distance (S).
- A_t = area of one leg of a closed stirrup resisting tension within a (S).
- b = width of compression face of member.
- b_w = web width, or diameter of circular section.
- C_c = compression resultant of concrete section.
- C_s = compression resultant of compression steel.
- **DL** = dead loads.
- d = distance from extreme compression fiber to centroid of tension reinforcement.
- E_c = modulus of elasticity of concrete.
- f_c = compression strength of concrete .
- F_y = specified yield strength of non-prestressed reinforcement.

- **h** = overall thickness of member.
- **L_n** = length of clear span in long direction of two- way construction, measured face-to-face of supports in slabs without beams and face to face of beam or other supports in other cases.
- **LL** = live loads.
- **L_w** = length of wall.
- **M** = bending moment.
- **M_u** = factored moment at section.
- **M_n** = nominal moment.
- **P_n** = nominal axial load.
- **P_u** = factored axial load
- **S** = Spacing of shear or in direction parallel to longitudinal reinforcement.
- **V_c** = nominal shear strength provided by concrete.
- **V_n** = nominal shear stress.
- **V_s** = nominal shear strength provided by shear reinforcement.
- **V_u** = factored shear force at section.
- **W_c** = weight of concrete. (Kg/m³).
- **W** = width of beam or rib.
- **W_u** = factored load per unit area.
- **Φ** = strength reduction factor.
- **ε_c** = compression strain of concrete = 0.003mm/mm.
- **ε_s** = strain of tension steel.
- **ε_s** = strain of compression steel.
- **ρ** = ratio of steel area .
- **F_{sd,r}** = total additional tension force above the support .
- **V_{ed,0}** = shear force at critical section .
- **N_{ed,0}** = normal tension force at support.
- **a₁** = offset factor.
- **α** = angle of stirrup.

Chapter one

Introduction

1

- 1.1 introduction.
- 1.1 General Identification.
- 1.3 Project Choosing Reasons.
- 1.4 The Project Objectives.
- 1.5 The problem of Project.
- 1.6 The postulate of the project.
- 1.7 The project components.
- 1.8 The timeline table of the project stages

1.1 Introduction

The last century has witnessed a starting of an age of revolution and improvement in all of the life aspects, and with the increasing demand of the people in the cities, it was very essential to cope up with this improvement in all of the fields, in such a way that suits and controls the environment.

All of us knows the importance of media specially TV and Radio stations, in transferring the information in all of the fields: political, financial, advertising, sport, and entertainment, etc..., specially here in Palestine to spread the understanding of the danger of the occupation and to visualize the latest news and activities of the occupation .

So it was necessary to think of a corporation includes many activities supporting the media, so the idea is to establish a media city has been in minds, which includes two buildings TV and Radio building, and Art building.

The city intended to improve the media in Palestine, in a way that serves and enhances the awareness of people making information easy to reach and understand by providing the accessibility for various media activities.

1.2 General Identification

The project is a Radio and Television Building designed in Ramallah city , it provides all requirements needed for suitable workplace Like Decor , Workshops, Makeup & Clothes , Decoration Engineers Offices , Audience's Studios , Maintenance Workshops , Stores , News Studio , Restaurant , Montage Department, Production Montage Department , Financial Department , Central Administration and all services needed .

1.3 Project Choosing Reasons

After more than one month of searching on a good project, we decided to choose the Palestinian Radio and Television Building from a lot of other projects because of the large size of the project and the including of various structural members, the most important thing from this project is to have.

1.4 The Project Objectives

- **Architectural Objectives**

The main architectural aim is to create a design that is unique in views, representative and break the lack of architecture that Palestine suffers. So the city is designed upon the latest architecture views, so we can see the curves and the huge transmission station shaped as the globe.

- **structural Objective**

- 1-Increasing the ability to choose a structural system that suits the objectives of the building.
- 2-To correlate what we have taken In the design courses with the practical thinking.
- 3-To get a new skills and experiences while facing problems and obstacles rising while working in the project, which has not mentioned in the theoretical studying.

1.5 The problem of Project

The problem of the project is to find the most appropriate structural system that satisfies the strength and serviceability requirements, and to design and analyze the structural components that consists the project which is the TV and Media building, so we will analyses an design these components like slabs, beams, columns,...etc, after determining the loads on each of the structural member so we can select the required dimensions and reinforcement, after that all of the design outputs will be presented in the structural drawing that used to transfer the project from being a drawing to the practical field.

1.6 The postulate of the project

Our study will aimed to prepare the required structural drawings of the various structural members existing in the building in such a way that takes the architectural design as the main outlet.

1.7 Chapters of Project :

Chapter one:

General Introduction of the project.

Chapter Two:

The architectural description of the project.

Chapter Three:

The structural studying of the project including: the structural members, the loads, and the function description.

Chapter Four:

The structural analysis and the design of some structural members like: beams, columns, and slabs.

Chapter Five:

Estimated cost.

Chapter Six :

Results and recommendations.

1.8 The timeline table of the project stages:

The table No.(1-1) shows the timeline table of the project stages upon the proposed steps within the first semester.

Table(1-1). The timeline table of the project stages

Suggested time	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	
Project Selection	█	█	█	█	█	█	█																										
Site Study							█	█	█	█																							
Collect information about the project									█	█	█																						
Architectural Study of the building										█	█	█	█																				
Structural Study of the building											█	█	█	█																			
Preparation of Graduation Project Introduction															█																		
Final presentation of introduction to project Graduation																█																	
Structural Analysis																	█	█	█	█													
Structural Design																					█	█	█	█	█	█	█	█	█	█	█	█	█
Preparation of construction drawings of the project																						█	█	█	█	█	█	█	█	█	█	█	█
Writing project																																	
Standby time																																	
Representation of the project																																	

2.3.1 Project Land Location,
 2.3.2 General Climate Of The City,
 2.3.3 Contour Lines Of The Project Land
 2.4 project components description.
 2.4.1 project plans description.
 2.4.2 project Elevations description.
 2.4.3 Vertical movement description.

Chapter Two

Architectural Description

2

2.1 Introduction.

2.2 Basic Identification.

2.3 Project Site.

2.3.1 Project Land Location,

2.3.2 General Climate Of The City .

2.3.3 Contour Lines Of The Project Land .

2.4 project components description.

2.4.1 project plans description.

2.4.2 project Elevations description.

2.4.3 Vertical movement description.

2.1 Introduction

The soul of architecture is to design a structure that will be suited for humans to live in, work in, play in, etc. It is also to give comfort to its users, to make them feel comfortable, make them feel uplifted, make them feel that someone cared about their well-being enough to design something that they would enjoy. A good Architect does more than just design buildings, he or she understands how people's surroundings make them feel, and creates an environment that will meet their needs and desires.

Doing architecture requires strong technical knowledge in the fields of engineering, logistics, geometry, building techniques, functional design and ergonomics. It also requires a certain sensibility to arts and aesthetics. Finally, it also requires a preoccupation for human questions and society's problems. Architecture is a very broad and humanistic field that is at the same time technical, artistic and social.

2.2 Basic Identification Of Project

From the importance of both traditional and new media to the flow of information in the fields of national, social and economics, we work towards creating a unified media entity able to accommodate the requirements of the future in which it seems that the gap between the possible and the impossible will almost fade.



fig(2-1) –general picture of project

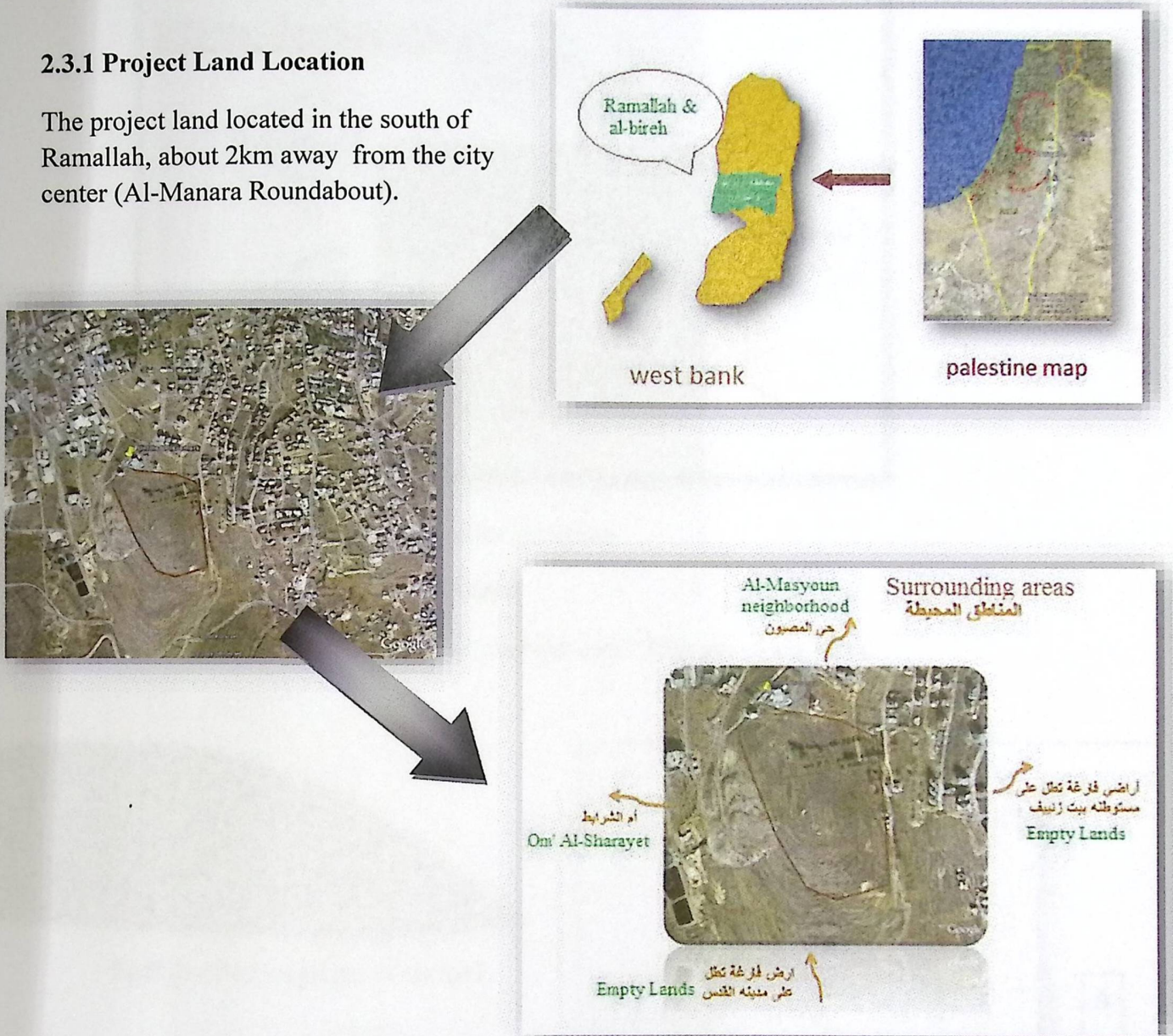
For that the idea is to establish an integrate “Media City” that include a coalition of journalists, governmental agencies, publishing houses and all other organizations involved in the production of radio, television, film, news and advertising .

2.3 Project Site

Its recommend that you pursue land that has already been approved by the local authorities as an "approved building lot". That means all surveys; soil testing, wetlands conservation, and site engineering work have been completed and approved. While raw land costs less, you will have to spend money to complete the required tests, surveys and engineering work before you can get the land approved for building.

2.3.1 Project Land Location

The project land located in the south of Ramallah, about 2km away from the city center (Al-Manara Roundabout).



fig(2-2) –The location of the land.

2.3.2 General Climate of The City

This area generally enjoys a Mediterranean Climate of a dry summer and mild, rainy winter with occasional snowfall. The recorded average of Ramallah's rainfall is about 694 mm (27 in) and (minimum rainfall is 307 mm (12 in) and maximum rainfall is 1,591 mm (63 in))

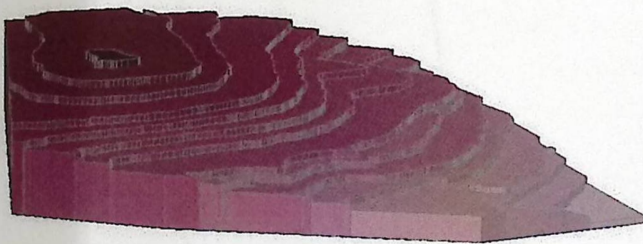
while the western and south western winds dominate, the northern winds are light and the eastern winds still blow on occasion.



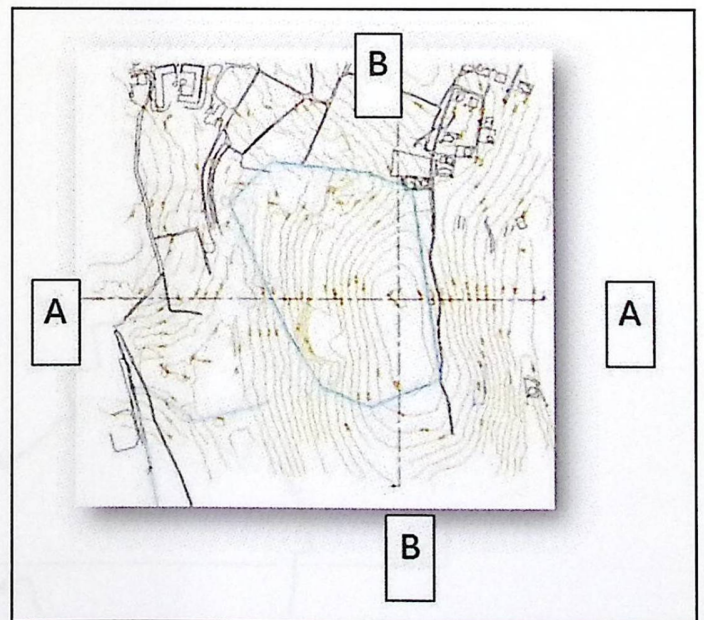
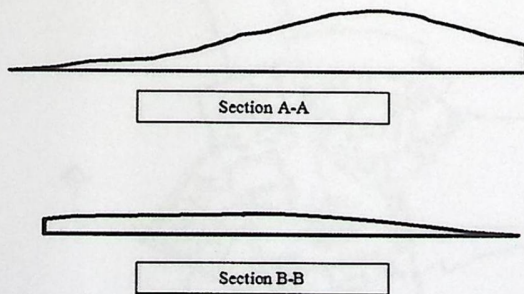
fig(2-3) –winds movement

2.3.3 Contour Lines of The Project Land

The project land on the top of a hill flat summit with 780m above sea level.



fig(2-4) –The topography of the land



2.4 project components description

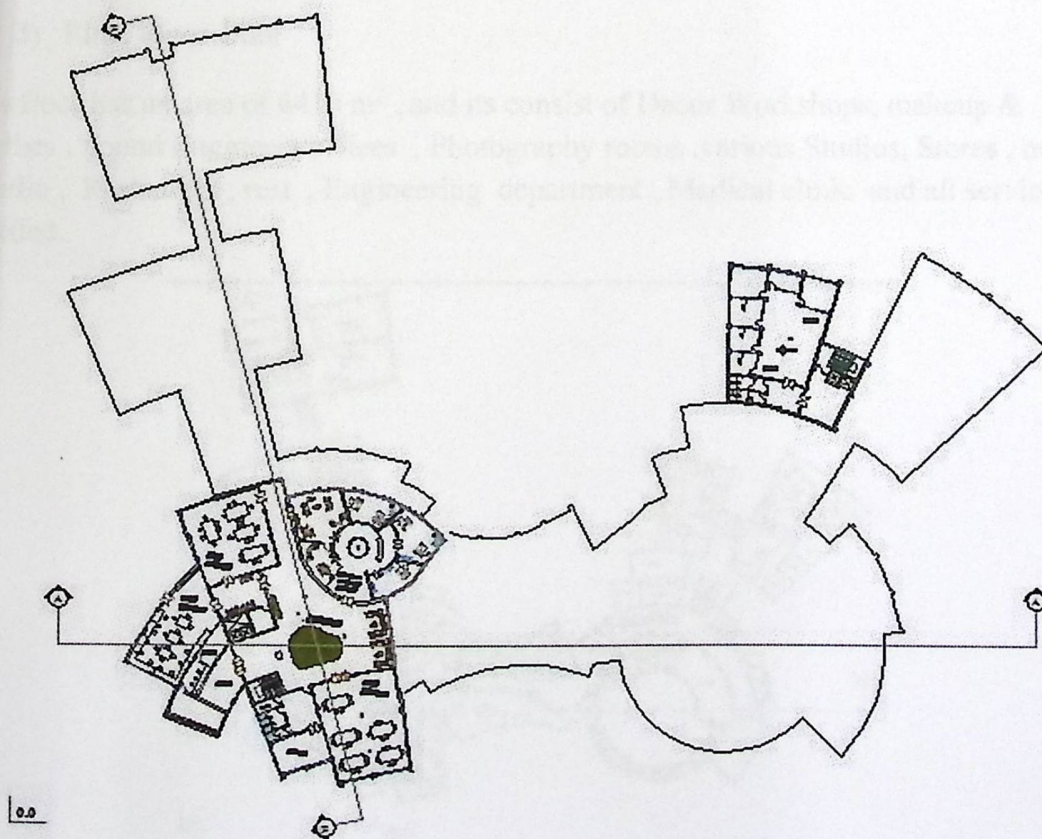
The designer used circular segments with different centers to attract attention, and made various conglomerates on a shape of layers which show the copy and paste action. Finally he used a steel sphere like the earth to show that media make's the world small .

2.4.1 Project plans description

The total area of the building is 29305m².

1) basement Floor Plan

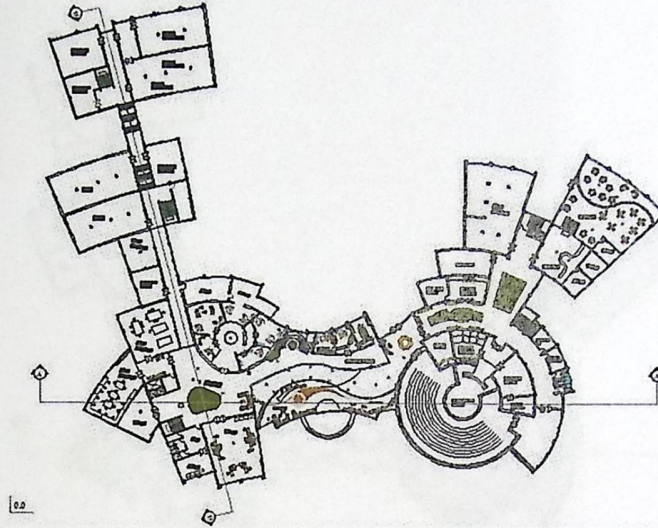
The area of this floor is 1915 m², and its consist of Security room , Decor Workshops , Decoration Engineers offices , audience's Studio and all services needed.



fig(2-5) –Basement floor

2) Ground Floor Plan

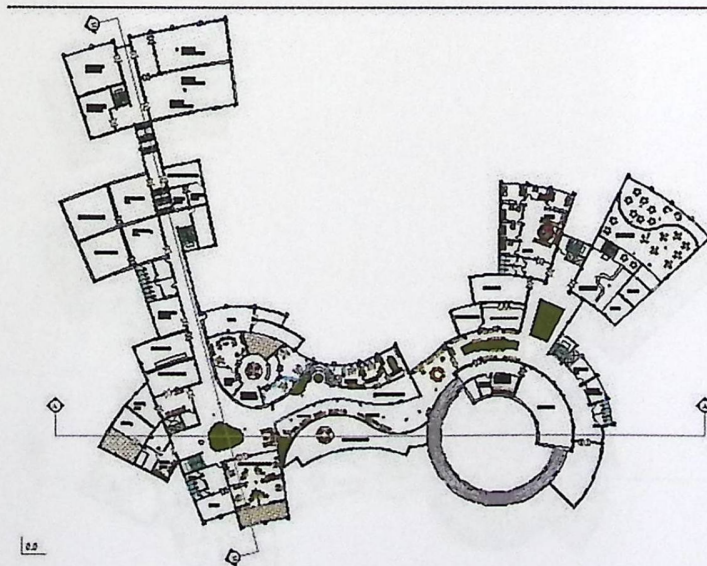
this floor has an area of 7065 m² , the main floor which consist of Decor Workshops, makeup & clothes , Decoration Engineers offices , audience's Studios , Maintenance Workshops , Stores , news Studio , Restaurant , Montage department and all services needed .



fig(2-6) –ground floor plan

3) First Floor Plan

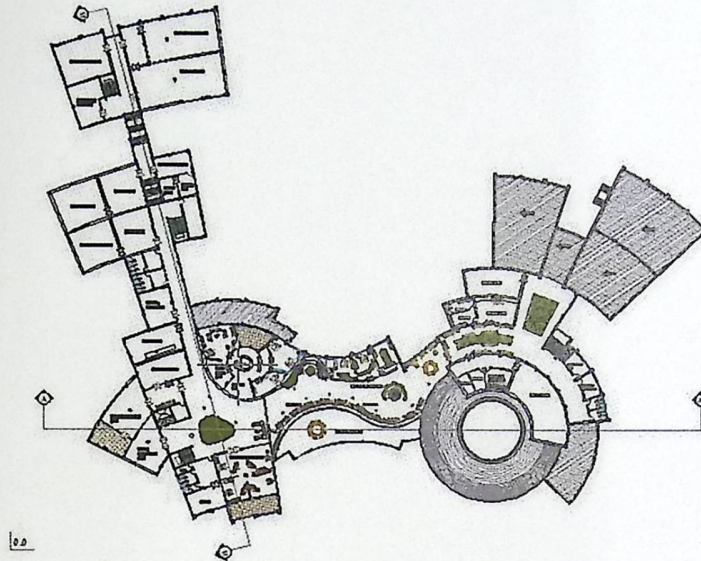
this floor has an area of 6410 m² , and its consist of Decor Workshops, makeup & clothes , Sound Engineers offices , Photography rooms ,various Studios, Stores , news Studio , Restaurant , rest , Engineering department , Medical clinic and all services needed .



fig(2-7) –first floor plan

4) Second Floor Plan

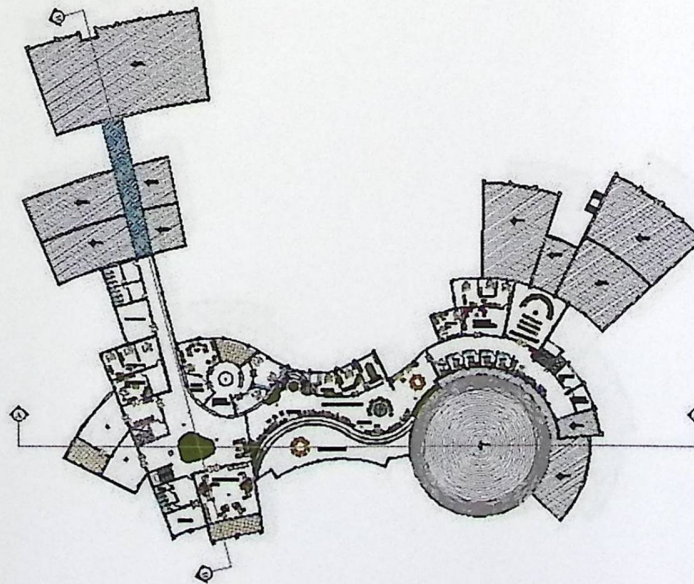
The area of this floor is 5815 m² , and its consist of Decor Workshops , makeup & clothes ,Sound Engineers offices , Photography rooms ,various Studios , Stores , news Studio , Montage department and all services needed .



fig(2-8) –second floor plan

5) Third Floor Plan

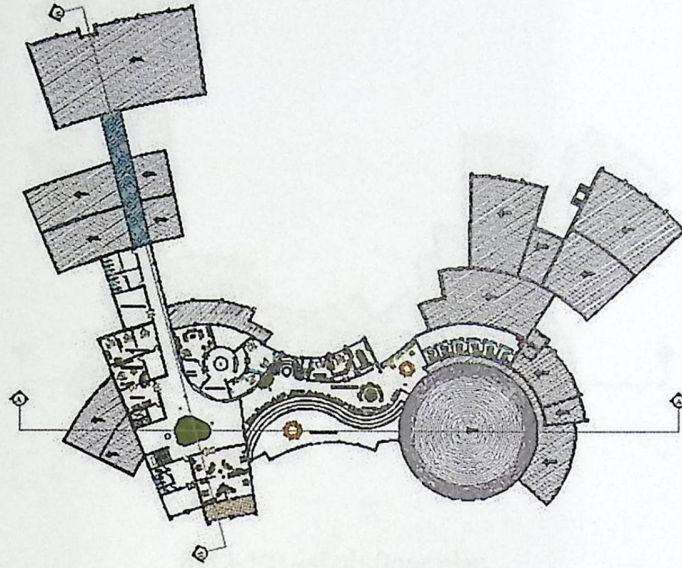
The area of this floor is 3635 m² , and its consist makeup & clothes ,Public Relations , Sound Engineers offices , video conference , Stores , Production department, marketing department and all services needed .



fig(2-9) –third floor plan

6) Fourth Floor Plan

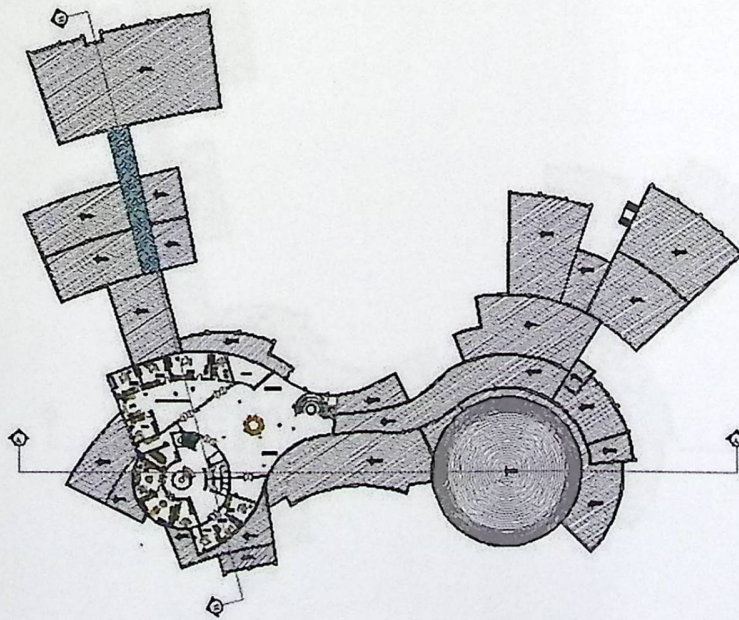
this floor has an area of 2500 m² , and its consist of makeup & clothes rooms, Photography rooms , Public Relations , Financial department , Personnel department and all services needed .



fig(2-10) –forth floor plan

7) Fifth Floor Plan

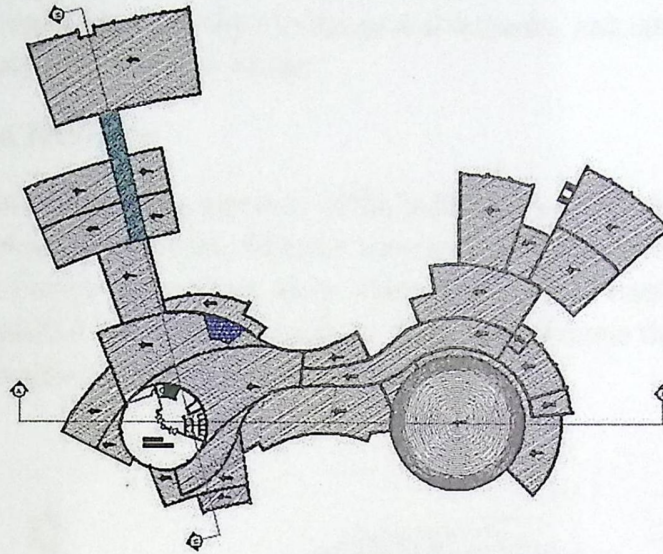
this floor has an area of 1265 m² , full floor for the Central Administration with all services needed .



fig(2-11) –fifth floor plan

8) Sixth Floor Plan

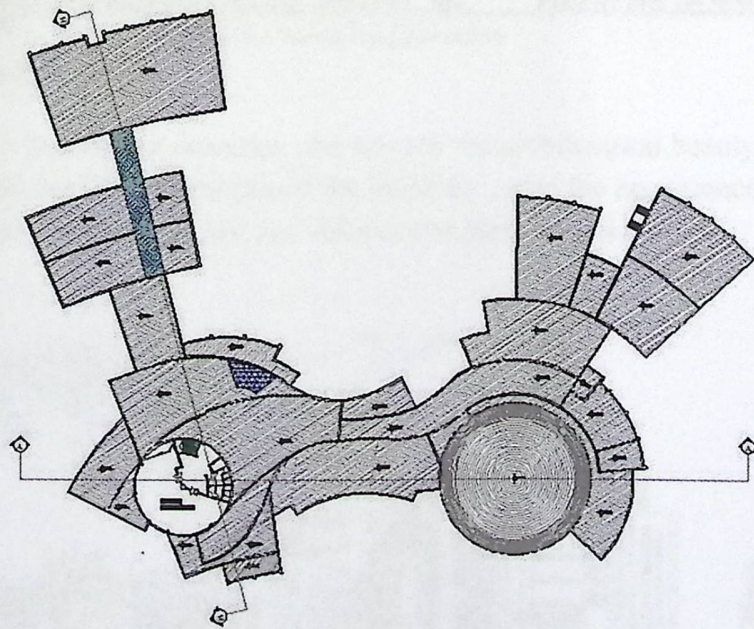
The area of this floor is 350 m², full floor for a major conference hall with all services needed .



fig(2-12) –sixth floor plan

9) Seventh Floor Plan

The area of this floor is 350 m² ,full floor for a major conference hall with all services needed .



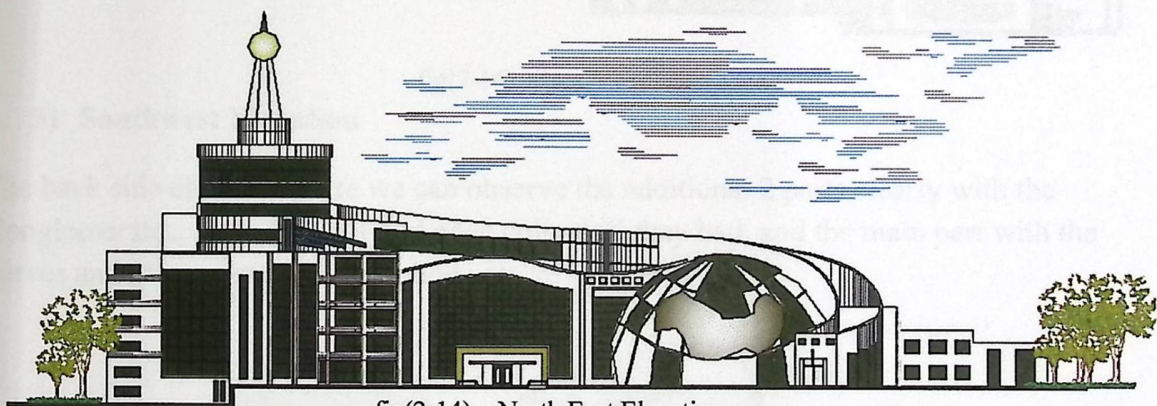
fig(2-13) –seventh floor plan

2.4.2 Project Elevations description

The interest of elevations for any architect is great as the elevations appearance must be suitable with the kind of the building and its uses, so it's a duty of the engineer to consider every detail of the elevations details in terms of materials used, the distribution of the openings, varying elevations and setbacks, and other factors that highlight the beauty of Elevations design.

1) North East Elevation

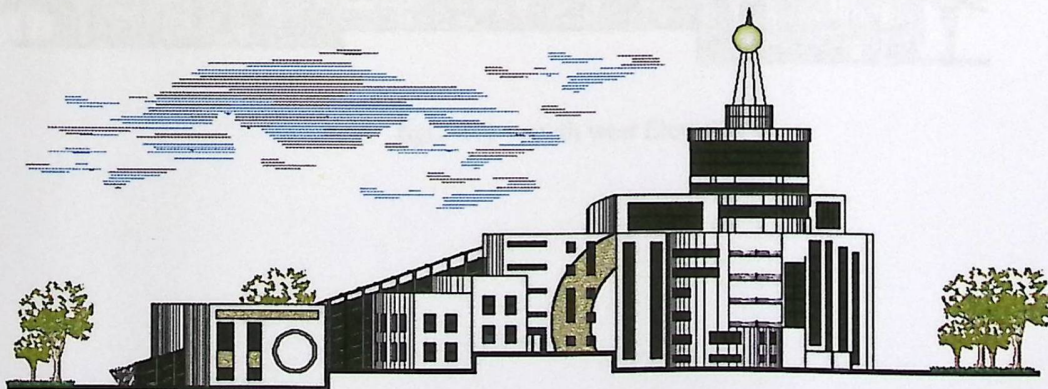
The main elevation and the main entrance of the building, we can observe the Creativity in this elevation from the different waves of setbacks and curves, also we can clearly see the beauty of the Glass spear which represent the earth and the media broadcast tower cuddled that sphere to indicate that media domain the word, finally we can notice the appearance of the edge column.



fig(2-14) – North East Elevation

1) South East Elevation

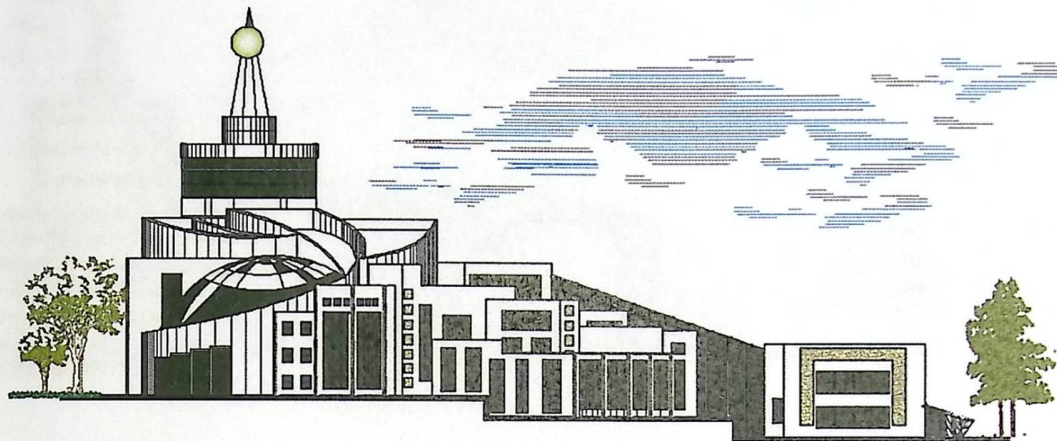
This elevation has a Secondary entrance, we can see the architectural beauty from the regular setbacks and the Conglomerates of the building, also the appearance of the Curves and edge column, finally we can observe the glass corridor.



fig(2-15) – south East Elevation

2) North West Elevation

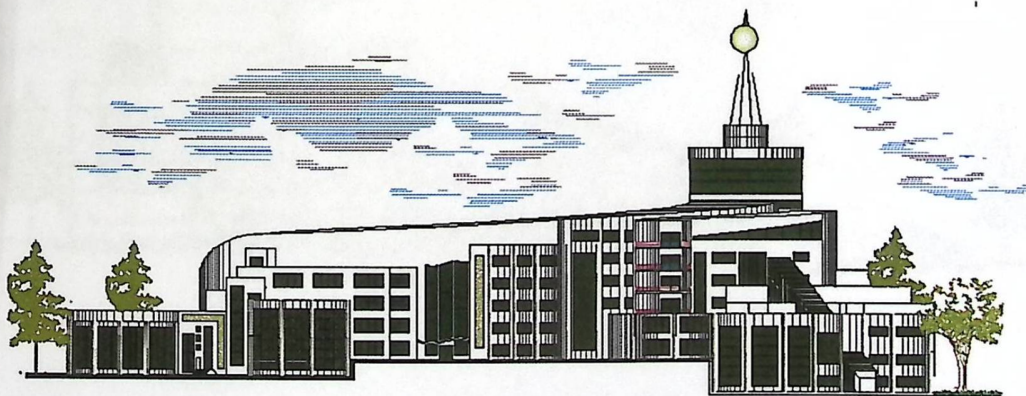
A wonderful Elevation , we can observe the Creativity in this elevation from how the media broadcast tower curves cuddled the glass earth also we can clearly see the beauty Conglomerates and setbacks with edge column , finally we can observe the glass corridor .



fig(2-16) – North west Elevation

3) Southwest Elevation

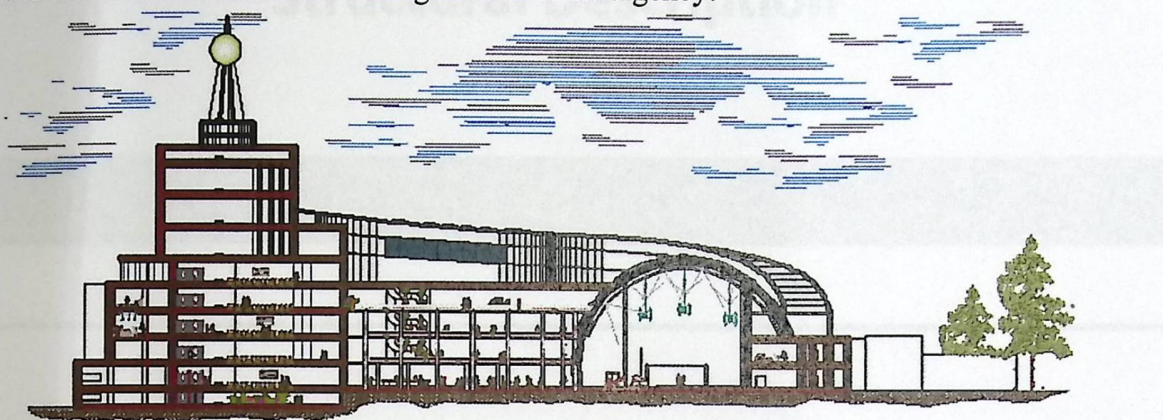
The back side elevation, here we can observe the additional 2 parts clearly with the Conglomerates, glass corridor and edge column it they had, and the main part with the curves and glass wonderful elevation .



fig(2-17) – south west Elevation

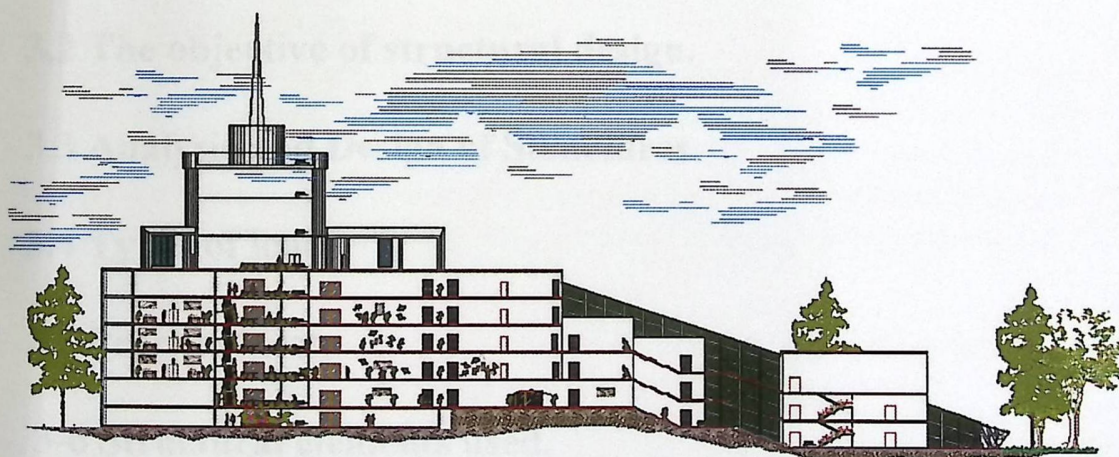
2.4.3 Vertical movement description

The designer distributed the movement through the horizontal and vertical axes through stairs and corridors with the provision of elevators, according to the number of users and the allowable distance between each vertical axis for easy contacts between the floors and to facilitate exiting in case of emergency.



Section A-A

fig(2-18) – section A-A



Section B_B

fig(2-19) – section B-B

Chapter Three

Structural Description

3

3.1 Introduction.

3.2 The objective of structural design.

3.3 Analysis and Design of Structures.

3.4 Types of loads.

3.5 Practical tests.

3-6 Structural elements used.

3.1 introduction :

After the completion of the process of the project architect explanation of all the details, we must move to the construction phase of the study for the project, in order to choose the appropriate structural system for each element in the building so that it provides all the requirements and work on the design of the elements necessary for that system. So that it is taking into account the loads affecting the types of elements, and show how to deal with them and work to resist, so we must know these structural elements in detail, in order to be customized and analyzed accurately.

In generally, the physical systems is the basic system broken down in the building .So, they have the following type:

- STRUCTURAL SYSTEM
- EXTERIOR SYSTEM
- INTERIOR SUBDIVISIONS OF SPACE

Each of these, in turn, can be seen to be made up of linear and planar assemblies.

3.2 The purpose of structural design :

The purpose of structural design is the work to find the building is available where all safety requirements, so be it by working on the Aegean all the structural elements of the crisis, in order to resist all the forces that affect the building in different forms, whether powers internal, such as loads of dead and live or external forces such as earthquakes, wind and landing in the soil. When designing any element of these structural elements should be taken into consideration the following standers :

1-Safety: is the essential element that must be provided in the design, so choosing the appropriate element of each region so resist load that affect them.

2-Cost: must be supplied when working on the selection of appropriate materials, and sufficient for its desired purpose and appropriate quantity.

3- Serviceability : Work to avoid any external failures, such as the decline in soil or any cracks in the external shape, or anything that works to increase this failure.

4- Architectural side : Work to take into account the distinctive architectural elements in the building and try to keep it as much as possible.

3.3 Theoretical studies of the structural elements of the building:

The most important basic steps that should work out of the project before starting the structural design, you must work on a comprehensive study of the project in terms of its size and the nature of his work, and how to work to estimate the loads that strike building and selected in a corrective, and choose items that are exposed to these loads, and identify systems construction, which used to resist those loads.

3.4 Types of loads :

Loads are basically in the design process, so you must give them great importance, so it must work to identify quality

Accurately, so as to differ from the building to another depends on the architectural design and Materials used in construction and other influences, therefore divided loads to:

1- Basic loads:

It loads which must be taken into account in the structural design of the building in all cases, It includes:

Dead load , Live load and Environmental loads .

2- Secondary loads:

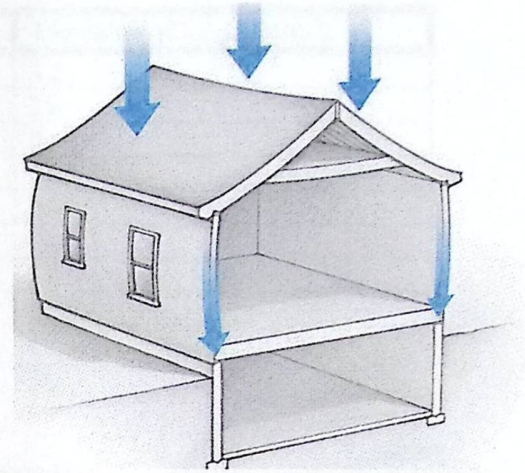
Are the loads that take them into account in the design in some buildings, depending on the nature of the building and other influences , It includes:

Shrinkage load, Thermal load, snows load, Dynamic load, seismic load.

3.4.1 Dead load :

The dead load includes loads that are relatively constant over time, including the weight of the structure itself, and immovable fixtures such as walls, plasterboard or carpet. Roof is also a dead load. Dead loads are also known as Permanent loads.

The designer can also be relatively sure of the magnitude of dead loads as they are closely linked to density and quantity of the construction materials. These have a low variance, and the designer is normally responsible for specifying these components.



fig(3-1) –dead loads

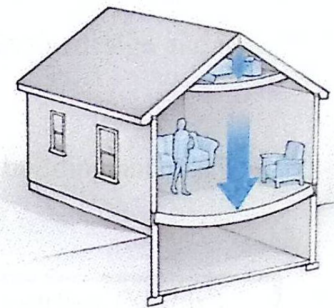
Table (3-1) Specific density of the materials used

Number	The type of material	The value of load(KN/m ³)
1	Furniture	3.2
2	Reinforcement concrete	25
3	normal concrete	24
4	Sand	14-17
5	Tiles	24-25
6	Granite stone	28
7	Plaster	22
8	Hollow block	9-12

3.4.2 Live load :

Live load is imposed loads, are temporary, of short duration, or moving. These dynamic loads may involve considerations such as impact, momentum, vibration, slosh dynamics of fluids, fatigue, etc.

Live loads, sometimes also referred to as probabilistic loads include all the forces that are variable within the object's normal operation cycle not including construction or environmental loads.



fig(3-2) –live load

Table (3-2) : Live loads

Number	The type of material	The value of Live load (KN/m ³)
1	Kitchens	3
2	Offices	3
3	Corridors and Stairs	5
4	stories	5
5	Roof	1.5
6	Hall Equipment	3

3.4.3 Environmental loads:

Are loads arising from the changes in the environment such as seismic, wind and snow.

3.4.3.1 Seismic load :

Are loads caused by earthquakes. Buildings should be designed to withstand minor earthquakes because they can occur almost anywhere. During an earthquake the ground can move both horizontally and vertically in any direction. This exerts tremendous horizontal loads onto members.

The methods of protecting against high winds can also be used for protecting against earthquakes. To avoid collapse, oscillations are dampened to prevent damage to both structural and non-structural members.

3.4.3.2 Winds load :

Is a forces that affect horizontally on the building, appear especially in high-rise buildings, and are designed on the basis of wind speed and height of the building, and the amount of buildings surrounding the building.

This has increased in the last few days and therefore to resort to high-altitude construction.

3.4.3.3 Snows load:

The building must be designed to resist and to be taken into account in the design and it depends on the height of the building and the area of this building.

The following table shows the relationship between the height of the building and carry snow that we take him in the case of design:

Table(3-3) : Loads of snow by sea level

Building height above sea level	The value of load in surface (KN/m ²)
$250 > h$	0
$500 > h > 250$	$(h-250)/800$
$1500 > h > 500$	$(h-400)/320$

Note: The building that we design located in masioon area in Ramallah, so it do take snow in the accounts.

3.5 Practical tests :

Before you begin the process of design and construction, must be the work of some necessary tests at the site, especially on the soil, and work to see the quality of the rocks in the region, and work to deviate place waterfalls groundwater and its impact on the building, and work to resolve the problems if available of those problems.

3.6 Structural elements used :

There are many structural elements used in the building as the slabs , Beam , Column , Stairs , The Shear wall and Foundation .

3.6.1 The slabs :

Is an element which transfers the loads that are exposed to other structural elements such as column, beam ,wall and are selected based on several factors:

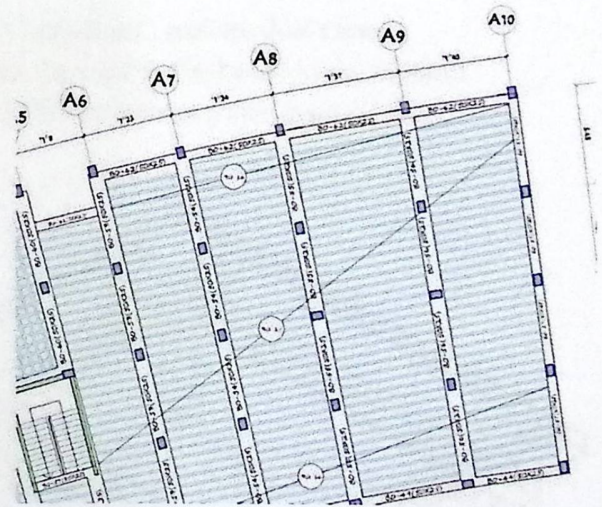
- 1) The distance between the spaces and columns
- 2) The desired function of the vacuum
- 3) Cost
- 4) Ease of implementation and duration available for building

Through our project this slabs using different types including:

3.6.1.1 The Rib:

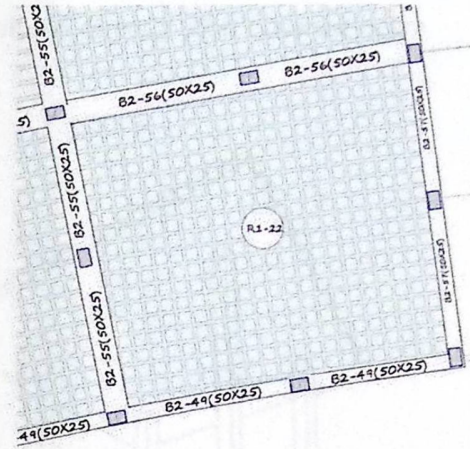
In general, this type is most commonly used in our ,this contain the pipe steel use to transfer the loads , and Block and The concrete between this block and the topping of all , and we have two types of this : one way rib an Two way rib ,this dependent of the load and how you can transfer it.

The one way rib slab contain the bar steel in one direction as you see in the next figure :



fig(3-3) –real one way rib & sample in the project

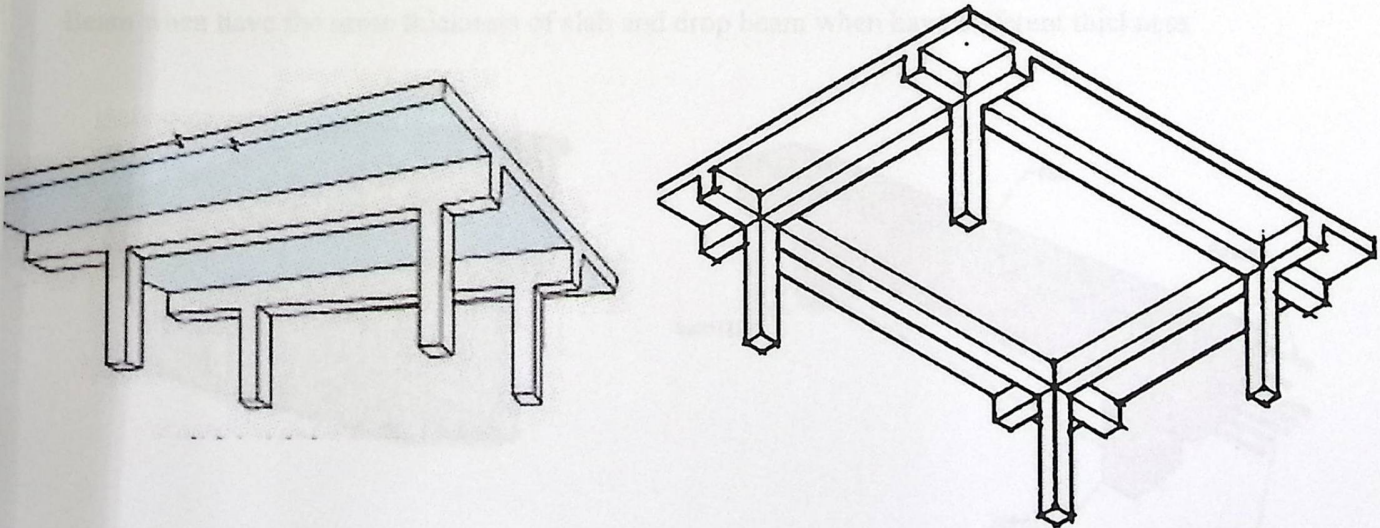
The two way rib slab is the type use when the length of the two direction in the space approximately is equals , and we used in this type bar of steel in two direction to transfer the load in this direction as you show in the next figure :



fig(3-4) –real two way rib & sample in the project

3.6.1.2 Solid slab:

We use this method when the height of the spaces is important , and we don't have problem when show the drop beam, and this transfer the load to the beam to the column ,and we have two types one way and two way and different between two types is the direction of transfer load:

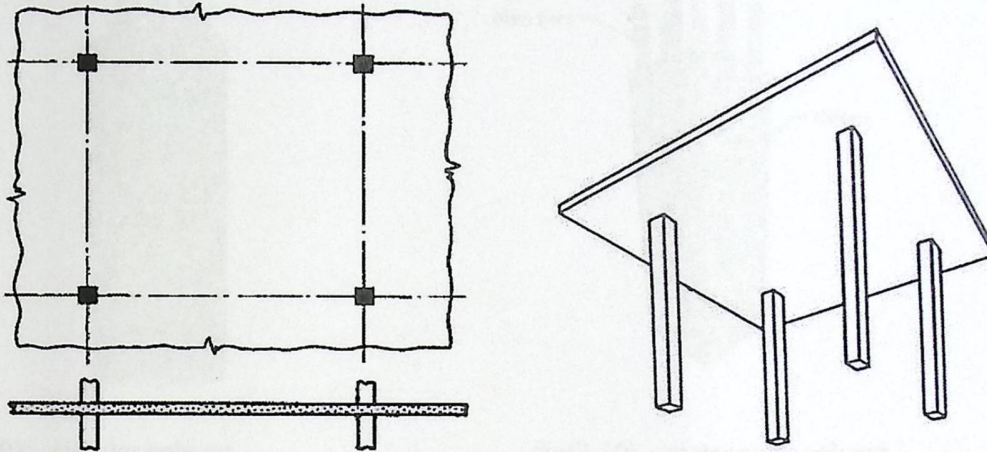


fig(3-5) – solid slabs

❖ **Note:** This type has not been used during this project .

3.6.1.3 Flat plate:

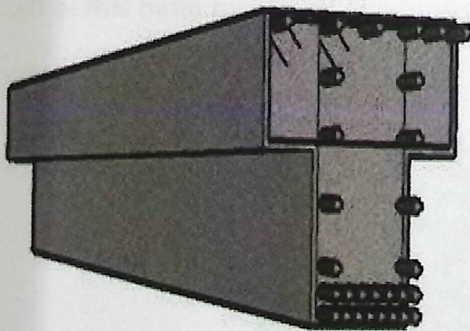
Use this type two transfer the load through it to the column directly ,don't use the beam ,as show :



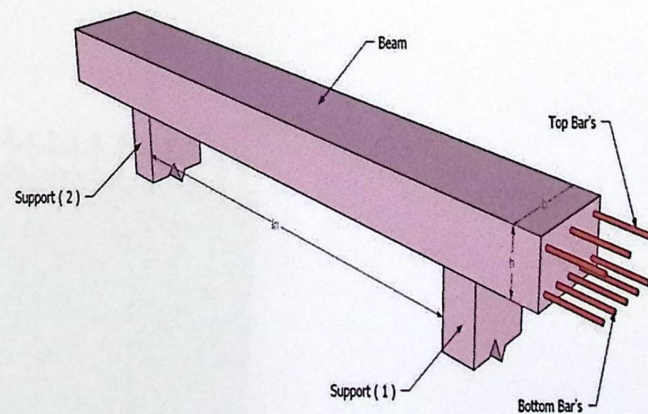
fig(3-6) –flat slabs

3.6.2 Beams:

Use this element to transfer the load from the slab to the column, and have the type as Hidden Beam when have the same thickness of slab and drop beam when have different thickness.



fig(3-7) – drop beam



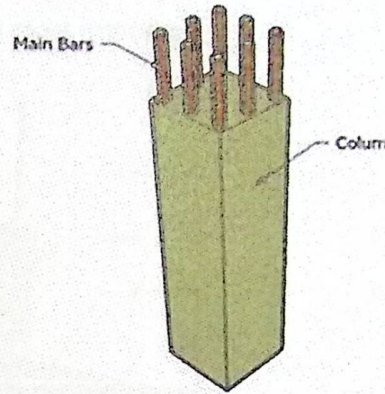
fig(3-8) – hidden beam

3.6.3 column :

The element is used to transfer the load from the slab to the foundation, and it helps in the stability of the building, and when design we will know the type design if short column or slender column.



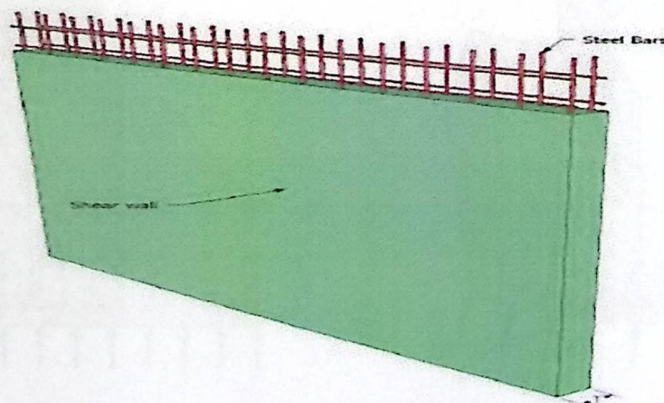
fig(3-9) – circular column



fig(3-10) – rectangular column

3.6.4 Shear wall:

Shear wall is the important of the element structures because use to resist the vertical and horizontal load; Shear walls are a type of structural system that provides lateral resistance to a building or structure. They resist in-plane loads that are applied along its height, these loads as the wind and earthquake. When design this wall, we use to layer steel to give it more strength, and this type use in stairs I generally and sometimes use in other place depend of the required of design of this building .

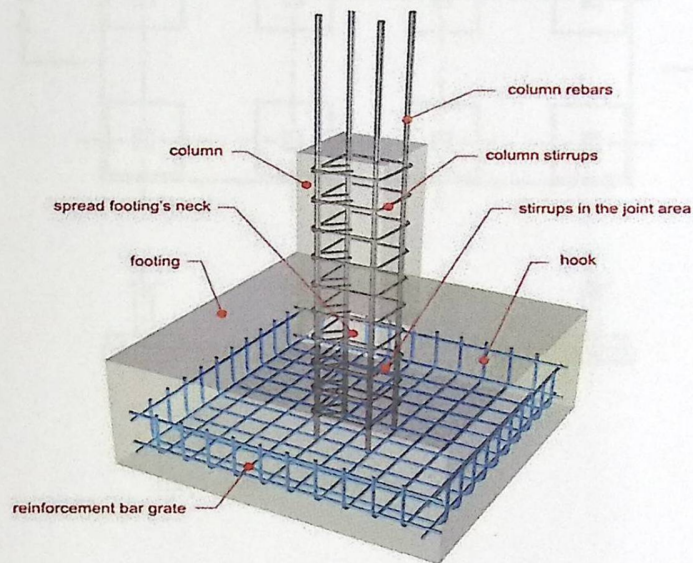


fig(3-11) – shear wall

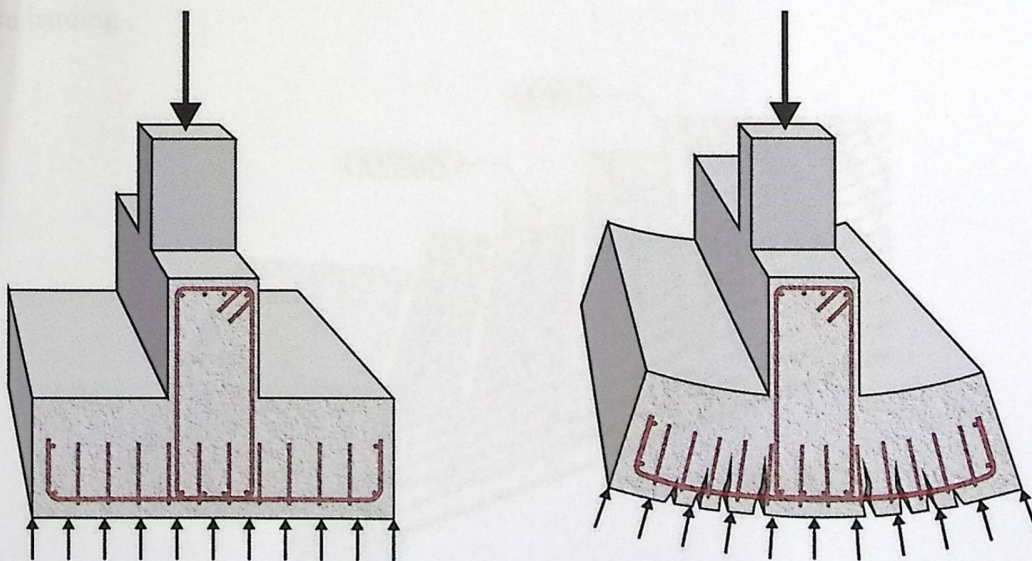
3.6.5 The Foundation :

The first element we implemented on the ground, but is the last element we design, because all loads are transmitted to him whether the basic load as dead or live load or secondary load . So is the basic element, which receives all the loads and distributed it to the soil, so when you take into account the design of soil that we created it, and we designed after identifying all loads transmitted to him.

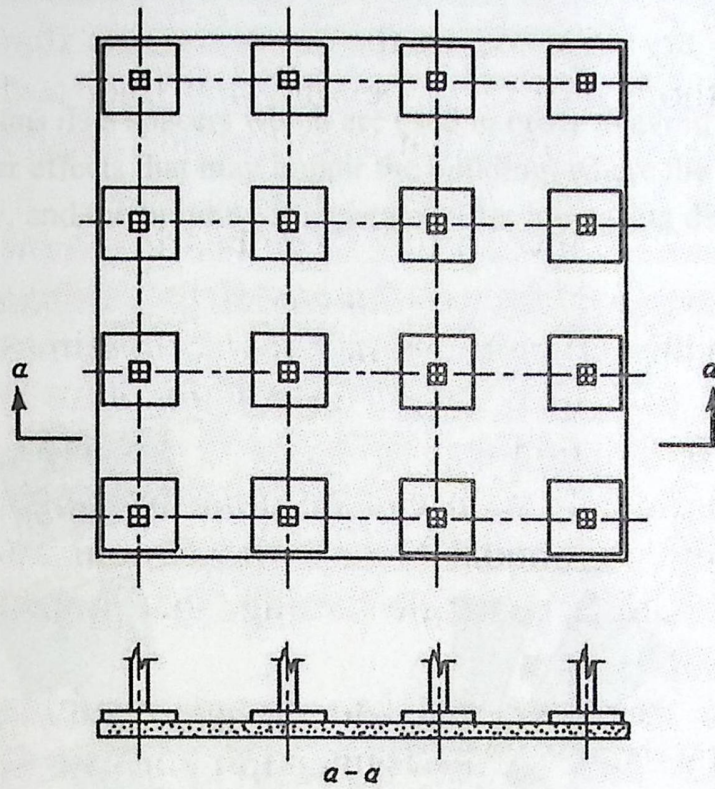
There are many types of foundations and this is the most important :



fig(3-12) – isolated footing



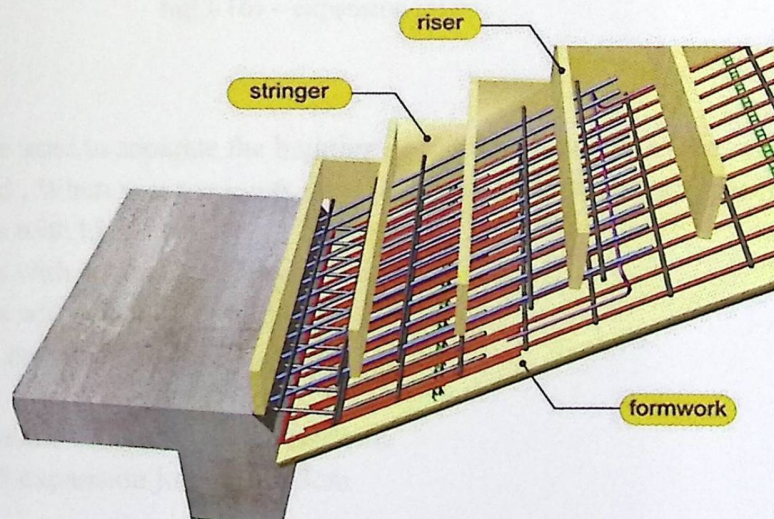
fig(3-13) - strip footing



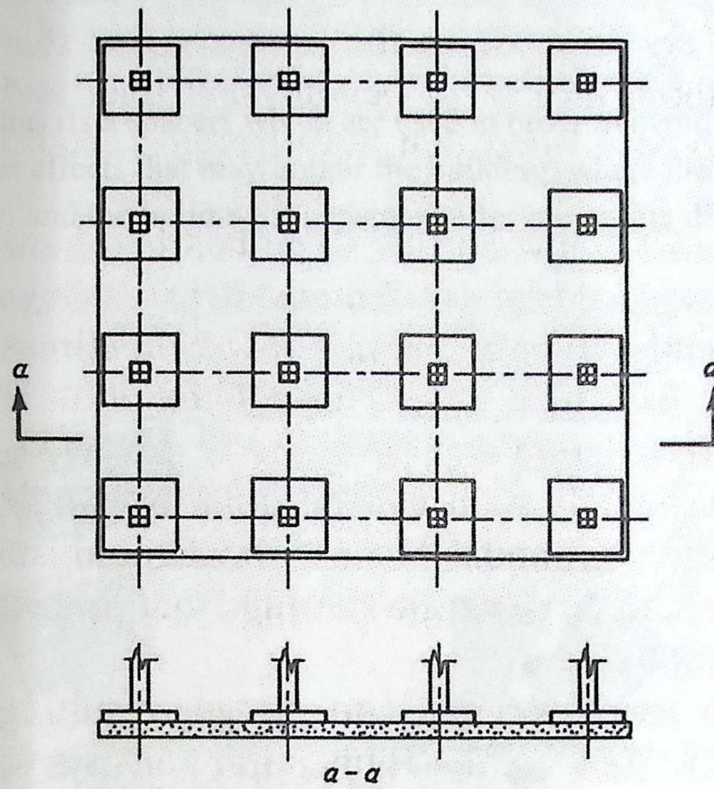
fig(3-14) - Mat footing

3.6.6 Stairs:

The stairs is a vertical transmission elements between the layers, and we used the one way solid slab in the landing .



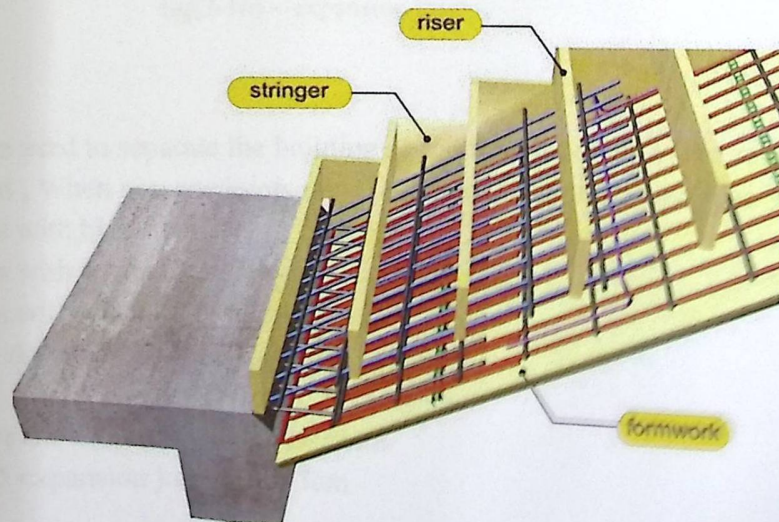
fig(3-15) - stairs



fig(3-14) - Mat footing

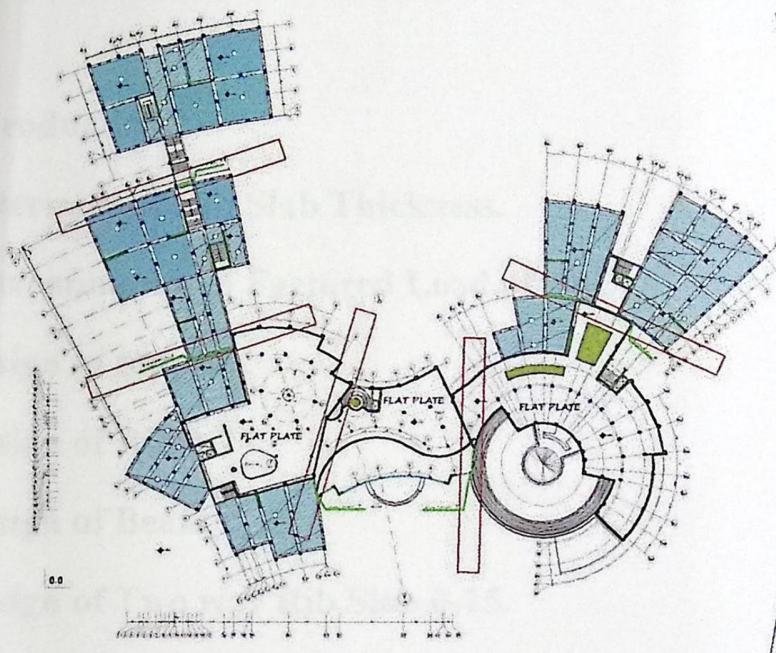
6.6 Stairs:

The stairs is a vertical transmission elements between the layers, and we used the one way solid slab in the landing .



fig(3-15) - stairs

3.6.7 Expansion Joints :Is a spacers which are used in order to avoid getting any expansion or other effects that may impair the building, where the building is separated entirely, and the building is separated after increasing distance (35-45 m)



fig(3-16) – expansion joints

- joints are used to separate the building so the foundation up to the roof without a separated , When you use joints must take into account the vast spaces of the building:
 - 1) 40m Areas with high humidity
 - 2) 36m Areas with normal humidity
 - 3) 32m Areas with Medium humidity
 - 4) 28 m with dry areas
- Will have the minimum thickness 3cm.
- We use 5 expansion joints with 3cm

Chapter Four

Structural Analysis & Design

4

- 4.1 Introduction.
- 4.2 Determination of Slab Thickness.
- 4.3 Determination of Factored Load of ribs.
- 4.4 Design of topping.
- 4.5 Design of Rib 0-17.
- 4.6 Design of Beam 0-55.
- 4.7 Design of Two way Rib Slab 0-15.
- 4.8 Design of flat plate.
- 4.9 Design of Stair.
- 4.10 Design of short Column.
- 4.11 Design of Long Column.
- 4.12 Design of tow way punching shear .
- 4.13. Design of Shear wall.
- 4.14 Design of Isolated Footing.
- 4.15 Design of strip Footing.
- 4.16 Design of steel.

1 Introduction:-

The project consists of several structural elements that will be designed according to the ACI code and by using the finite element method using much of computer software such as "ETABS, STAADpro, Safe And Etabs to find the internal forces, deflections, Shear and moments for the all structural element in order to design them.

1.2 Determination of Slab Thickness:-

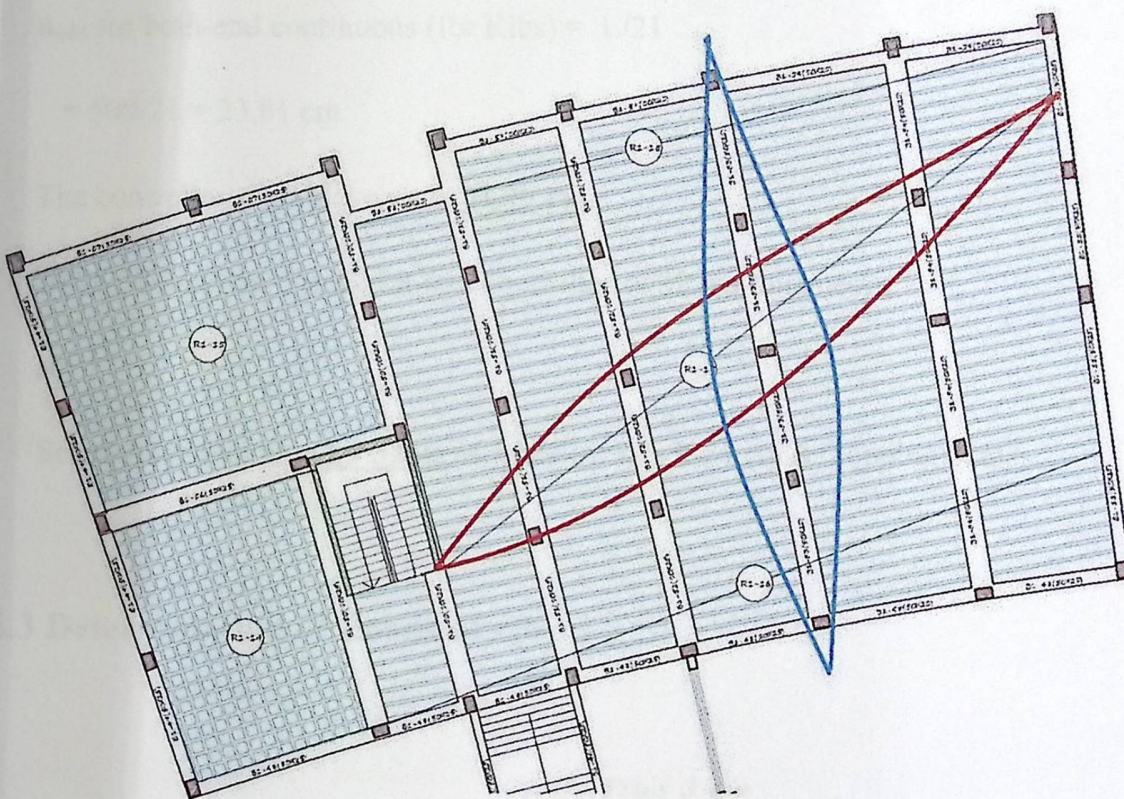


Figure (4-1): Ground Floor Slab.

According to ACI-Code-318-05, the minimum thickness of nonprestressed beams or one way slabs unless deflections are computed as follow:

h_{\min} for one-end continuous (for Ribs) = $L/18.5$

$$= 567.8/18.5 = 30.69 \text{ cm.}$$

h_{\min} for both-end continuous (for Ribs) = $L/21$

$$= 635/21 = 31.09 \text{ cm}$$

h_{\min} for one-end continuous (for Beams) = $L/18.5$

$$= 551/18.5 = 29.78 \text{ cm.}$$

h_{\min} for both-end continuous (for Ribs) = $L/21$

$$= 500/21 = 23.81 \text{ cm}$$

The controller slab thickness is **31.09 cm**.

But by deflection checked it was controlled at 32 cm thickness.

so

Select Slab thickness **h = 32 cm** with hollow block 24 cm & Topping 8 cm.

4.3 Determination of Loads of ribs :-

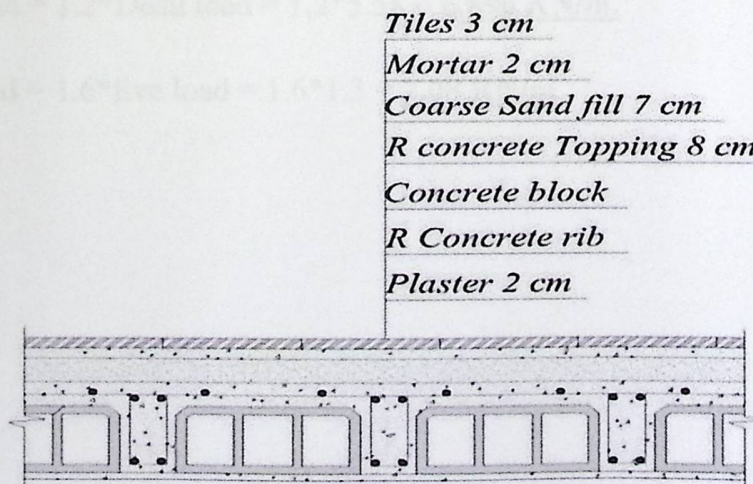


Figure (4-2): Typical Section in Ribbed slab

3.1 Determination of Dead load:-

Table(4-1) : Dead loads of the one way rib slab

Type	$\gamma b h$	KN/m/rib
Tiles	0.03*0.52*23	0.359
Mortar	0.03*0.52*22	0.343
Course Sand	0.07*0.52*17	0.619
Topping	0.08*0.52*25	1.04
Hollow block	0.4*0.24*10	0.96
Plaster	0.03*0.52*22	0.343
R.C rib	0.12*0.24*25	0.72
Partitions	2.3*0.52	1.196
Sum		5.58

4.3.2 Determination of live load:-

Nominal Total live load for studies = $2.5 * 0.52 = \underline{1.3 \text{ kN/m/ rib}}$

4.3.3 Determination of factored dead & live load :-

Factored dead load = $1.2 * \text{Dead load} = 1.2 * 5.58 = \underline{6.696 \text{ KN/m.}}$

Factored Live load = $1.6 * \text{live load} = 1.6 * 1.3 = \underline{2.08 \text{ KN/m}}$

4 Design of Topping:-

4.1 Determination of dead load of topping

Table(4-2) : Dead loads of the topping

Type	$\gamma b h$	KN/m
Tiles	0.03*1*23	0.69
Mortar	0.02*1*22	0.44
Sand	0.07*17*1	1.19
Topping	0.08*1*25	2
Partitions	2.3*1	2.3
Sum		6.84

Live Load = **2.5KN/m.** (for Studies)

$$w_u = 1.2 \text{ DL} + 1.6 \text{ LL}$$

$$= 1.2 * 6.84 + 1.6 * 2.5 = \underline{\underline{12.208 \text{ KN/m.}}} \text{ (Total Factored Load).}$$

$$\begin{aligned} \rightarrow M_u &= \frac{w_u * l^2}{12} = \frac{12.208 * 0.4^2}{12} \\ &= 0.1628 \text{ KN.m.} \end{aligned}$$

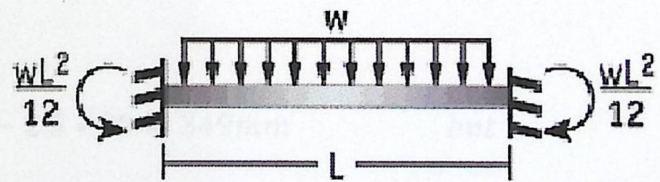


Figure (4-3): Both fixed end static system

$\phi M_n > M_u$ — Strength condition, where $\phi = 0.55$ —for plane concrete.

$$\rightarrow M_n = 0.42 \sqrt{f_c'} * \frac{bh^2}{6}$$

Where S_m for rectangular section of the slab :

$$S_m = \frac{bh^2}{6} = \frac{1000 * 80^2}{6} = 1066666.67 \text{ mm}^3$$

$$I_n = 0.42 * \sqrt{24} * \frac{1000 * 80^2}{6} = 2.19 \text{ KN.m.}$$

$$\phi * Mn = 0.55 * 2.19 = 1.2 \text{ KN.m.}$$

$$\phi * Mn = 1.2 > Mu = 0.1628 \text{ KN.m. OK!}$$

No Reinforcement is required by analysis . According to ACI 10.5.4 , provide $A_{s,min}$ for slabs as shrinkage and temperature reinforcement.

According to ACI 7.12.2.1 , $\rho_{shrinkage} = 0.0018$

$$A_s = \rho * b * t = 0.0018 * 1000 * 80 = 144 \text{ mm}^2 / \text{m strip}$$

Try bars $\phi 8$ with $A_s = 50.27 \text{ mm}^2$

$$n = A_s / 100 = 144 / 50.27 = 2.87 \text{ bars}$$

Take $3\phi 8$ with $A_s = 150.8 \text{ mm}^2 / \text{m strip}$ or $\phi 8 @ 300 \text{ mm}$ in both directions.

Step(s) is the smallest of :

$$1. S = 3 h = 3 * 80 = 240 \text{ mm (control)}$$

$$2. S = 450 \text{ mm}$$

$$3. s = 380 \left(\frac{280}{f_s} \right) - 2.5 C_c = 380 \left(\frac{280}{\frac{2}{3} * 400} \right) - 2.5 * 20 = 349 \text{ mm} \quad \text{but}$$

$$s \leq 300 \left(\frac{280}{f_s} \right) = 300 \left(\frac{280}{\frac{2}{3} * 400} \right) = 315 \text{ mm}$$

Take $\phi 8 @ 200 \text{ mm}$ in both direction .

$$s = 200 \text{ mm} < S_{max} = 240 \text{ mm} - \text{ok}$$

4.5 Design of Rib1-17 :-

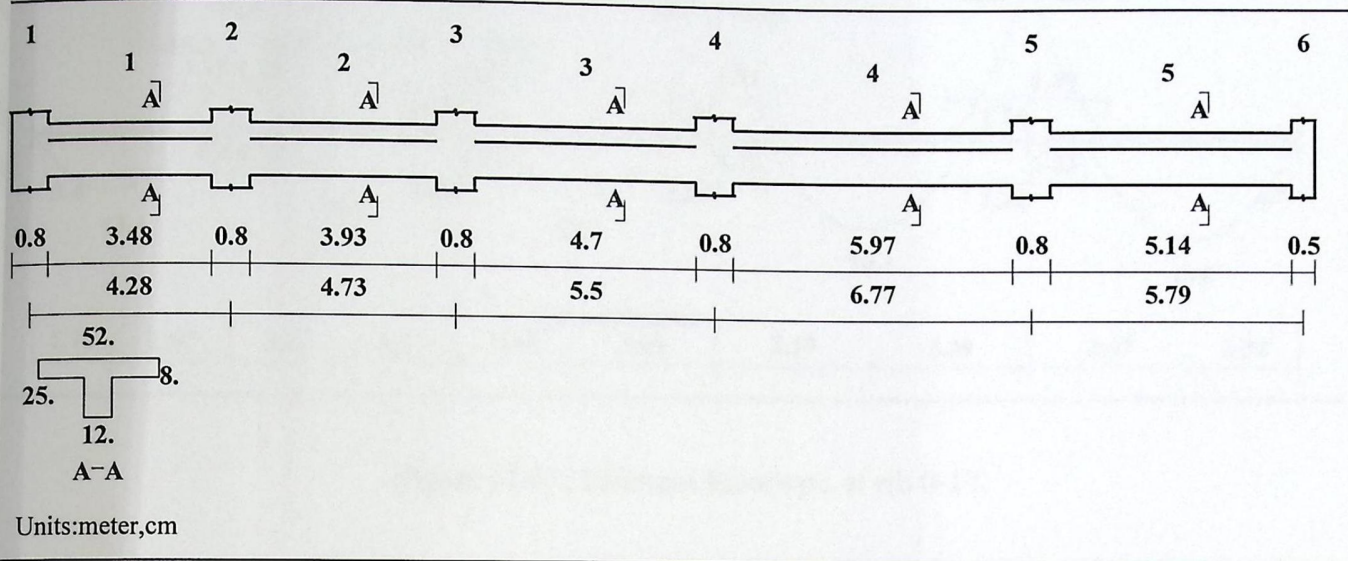
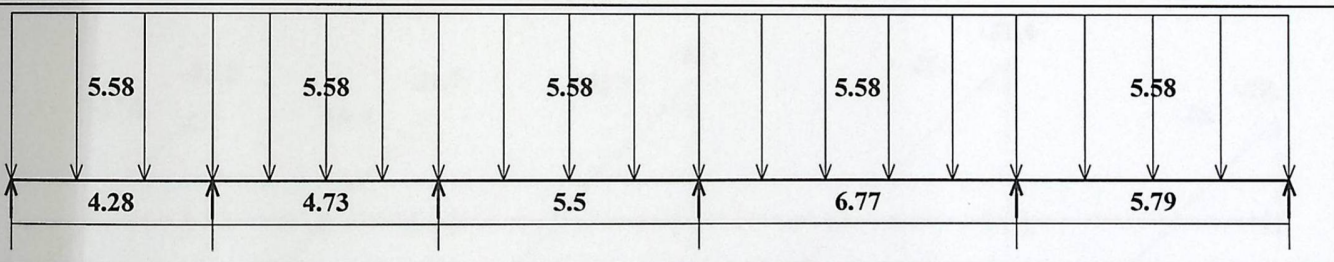


Figure (4-4): Rib 0-17 geometry.

Dead load - Service

Units: kN, meter



Live load - Service

Load factors: 1.20, 1.20/1.60, 0.00

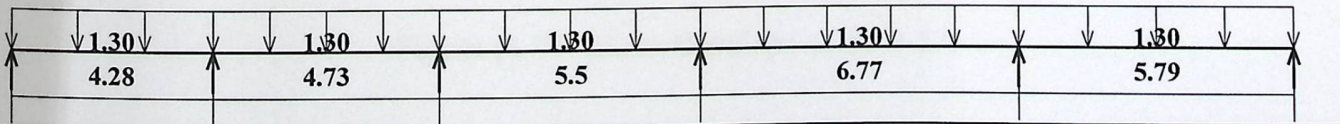


Figure (4-5) : loading of Rib 0-17

ments: spans 1 to 5

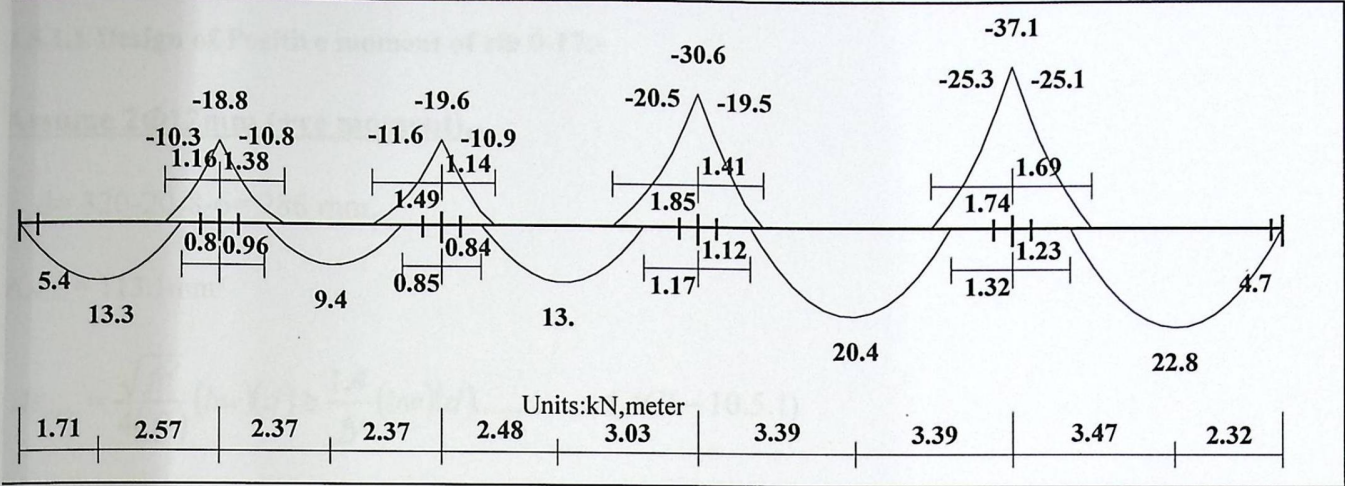


Figure (4-6) : Moment Envelope of rib 0-17.

ear

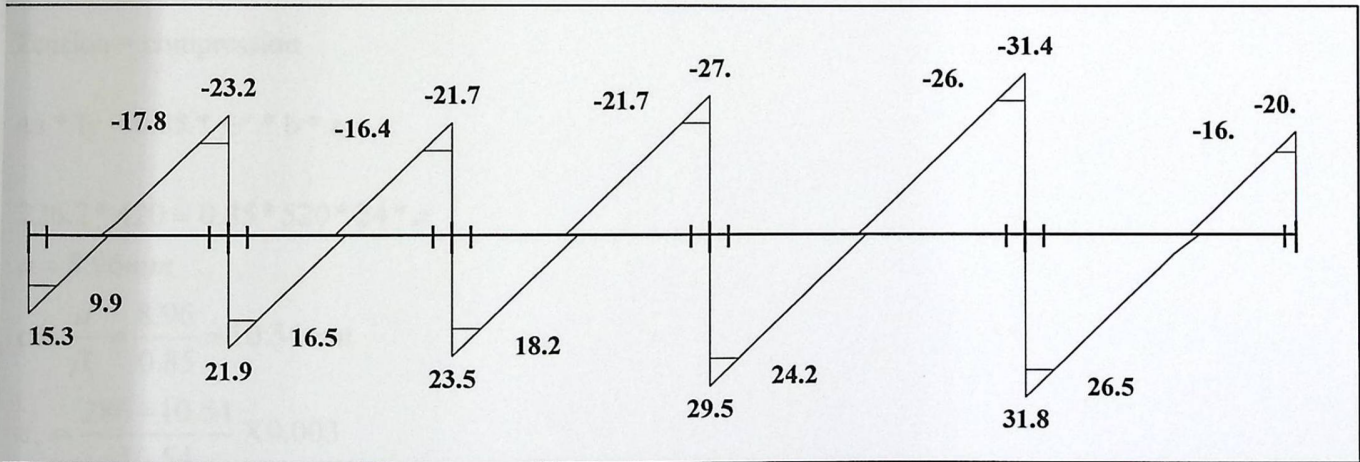


Figure (4-7) : Shear Envelope of rib 0-17.

1.5.1 Design of flexure:**1.5.1.1 Design of Positive moment of rib 0-17:-****Assume 2Φ12mm (+ve moment),...**

$$d = 320 - 20 - 8 - 6 = 286 \text{ mm.}$$

$$A_{2\Phi 12} = 113.1 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots \dots \dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (120)(286) \geq \frac{1.4}{420} (120)(286)$$

$$A_{s_{\min}} = 100.08 < 114 \quad A_{s_{\min}} = 114.4 \text{ mm}^2 \dots \text{Control}$$

$$226.2 \text{ mm}^2 > A_{s_{\min}} = 114.4 \text{ mm}^2$$

Tension = compression

$$A_s \cdot f_y = 0.85 \cdot f_c' \cdot b \cdot a$$

$$226.2 \cdot 420 = 0.85 \cdot 24 \cdot 420 \cdot a$$

$$a = 8.96 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{8.96}{0.85} = 10.54 \text{ mm}$$

$$\epsilon_s = \frac{286 - 10.54}{10.54} \times 0.003$$

$$\epsilon_s = 0.0784 > 0.005$$

$$\phi = 0.9.$$

$$M_n = A_s \cdot f_y \left(d - \frac{a}{2} \right) = 226.2 \cdot 420 \left(286 - \frac{8.96}{2} \right) = 26.745 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 \cdot 26.745 = 24.071 \text{ kN.m.}$$

Select 2Φ12mm At Spans (4&5).

Assume 2Φ10mm (+ve moment),...

$$d = 320 - 20 - 8 - 5 = 287 \text{ mm.}$$

$$2\Phi_{10} = 157.08 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots \dots \dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (120)(287) \geq \frac{1.4}{420} (120)(287)$$

$$A_{s_{\min}} = 100.43 < 114 \quad A_{s_{\min}} = 114.8 \text{ mm}^2 \dots \text{ control}$$

$$157.08 \text{ mm}^2 > A_{s_{\min}} = 114.8 \text{ mm}^2$$

Tension = compression

$$A_s \cdot f_y = 0.85 \cdot f_c' \cdot b \cdot a$$

$$157.08 \cdot 420 = 0.85 \cdot 520 \cdot 24 \cdot a$$

$$a = 6.22 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{6.22}{0.85} = 7.32 \text{ mm}$$

$$\epsilon_s = \frac{287 - 7.32}{7.32} \times 0.003$$

$$\epsilon_s = 0.115 > 0.005$$

$$\phi = 0.9.$$

$$M_n = A_s \cdot f_y \left(d - \frac{a}{2} \right) = 157.08 \cdot 420 \left(287 - \frac{6.22}{2} \right) = 18.73 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 \cdot 18.73 = 16.86 \text{ kN.m.}$$

Select 2Φ10mm At Spans (1&2&3).

5.1.2 Design of negative moment of rib 0-17:-

Assume bars diameter of 14mm(-ve Moment),

$$d = 320 - 20 - 8 - 6 = 285 \text{ mm.}$$

$$A_{2\phi 14} = 307.88 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots \dots \dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (120)(285) \geq \frac{1.4}{420} (120)(285)$$

$$A_{s_{\min}} = 99.73 < 114 \quad A_{s_{\min}} = 114 \text{ mm}^2 \dots \text{control}$$

$$307.88 \text{ mm}^2 > A_{s_{\min}} = 114 \text{ mm}^2$$

Q

Tension = compression

$$A_s * f_y = 0.85 * f_c' * b * a$$

$$307.88 * 420 = 0.85 * 120 * 24 * a$$

$$a = 52.87 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{52.87}{0.85} = 62.2 \text{ mm}$$

$$\epsilon_s = \frac{285 - 62.2}{62.2} * 0.003$$

$$\epsilon_s = 0.01075 > 0.005$$

$$\phi = 0.9.$$

$$M_n = A_s * f_y \left(d - \frac{a}{2} \right) = 307.8 * 420 \left(285 - \frac{52.87}{2} \right) = 33.43 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 * 33.43 = 30.084 \text{ kN.m.}$$

Select 2 ϕ 14mm at supports (5).

Assume bars diameter of 12mm(-ve moment),

$$d = 320 - 20 - 8 - 6 = 286 \text{ mm.}$$

$$A_{2\Phi 12} = 113.1 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (bw)(d) \geq \frac{1.4}{f_y} (bw)(d) \dots \dots \dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (120)(286) \geq \frac{1.4}{420} (120)(286)$$

$$A_{s_{\min}} = 100.08 < 114 \quad A_{s_{\min}} = 114.4 \text{ mm}^2 \dots \text{ control}$$

$$226.2 \text{ mm}^2 > A_{s_{\min}} = 114.4 \text{ mm}^2$$

Tension = compression

$$A_s * f_y = 0.85 * f_c' * b * a$$

$$226.2 * 420 = 0.85 * 120 * 24 * a$$

$$a = 38.81 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{38.81}{0.85} = 45.66 \text{ mm}$$

$$\epsilon_s = \frac{286 - 45.66}{45.66} \times 0.003$$

$$\epsilon_s = 0.0158 > 0.005$$

$$\phi = 0.9.$$

$$M_n = A_s \cdot f_y \left(d - \frac{a}{2} \right) = 226.2 * 420 \left(286 - \frac{38.81}{2} \right) = 25.33 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 * 25.33 = 22.797 \text{ kN.m.}$$

Select 2Φ12mm At Support (4)

Assume bars diameter of 10mm(-ve moment),

$$d = 320 - 20 - 8 - 5 = 287 \text{ mm.}$$

$$A_{2\Phi 10} = 157.08 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots \dots \dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (120)(287) \geq \frac{1.4}{420} (120)(287)$$

$$A_{s_{\min}} = 100.43 < 114 \quad A_{s_{\min}} = 114.8 \text{ mm}^2 \dots \text{ control}$$

$$157.08 \text{ mm}^2 > A_{s_{\min}} = 114.8 \text{ mm}^2$$

Tension = compression

$$A_s * f_y = 0.85 * f_c' * b * a$$

$$157.08 * 420 = 0.85 * 120 * 24 * a$$

$$a = 26.95 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{26.95}{0.85} = 31.71 \text{ mm}$$

$$\epsilon_s = \frac{287 - 31.71}{31.71} \times 0.003$$

$$\epsilon_s = 0.02415 > 0.005$$

$$\phi = 0.9.$$

$$M_n = A_s * f_y \left(d - \frac{a}{2} \right) = 157.08 * 420 \left(287 - \frac{26.95}{2} \right) = 18.045 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 * 18.045 = 16.241 \text{ kN.m.}$$

Select 2Φ10mm At Supports (2&3).

.5.2 Design of shear of rib 0-17 :

$$V_{ud} = 25.9 \text{ KN} \rightarrow \text{at Support 5}$$

$$V_c = \Phi * \frac{\sqrt{f_c'}}{6} b_w * d$$

$$= 0.75 * \frac{\sqrt{24}}{6} * 120 * 286$$

$$= 21.015 \text{ KN}$$

$$1.1 * \Phi V_c = 1.1 * 28.02 = 23.118 \text{ KN.}$$

$$V_{s,max} = \frac{2}{3} * \sqrt{24} * 120 * 286 * 10^{-3} = 112.088 \text{ KN}$$

$$V_s = V_n - V_c = \frac{25.9}{0.75} - 30.824 = 3.7$$

The Section is large enough

Check for items:-

Case I:

$$V_u \leq \Phi V_c / 2$$

$$25.9 > 11.559 \quad (\text{Not Ok})$$

Case II:

$$\Phi V_c / 2 \leq V_u \leq \Phi V_c$$

$$11.559 < 25.9 > 23.118 \quad (\text{not ok})$$

Case III:

$$V_{s,min} = \frac{1}{16} * \sqrt{f_c'} * b_w * d$$

$$= \frac{1}{16} * \sqrt{24} * 120 * 286 * 10^{-3}$$

$$= 10.508 \text{ KN}$$

Or:

$$V_{s,\min} = \frac{1}{3} * b_w * d = \frac{1}{3} * 120 * 286 * 10^{-3} = 11.44 \text{ KN} \quad \dots \dots \text{control}$$

$$\Phi(V_c + V_{s,\min}) = 31.668 \text{ KN}$$

$$\phi V_c = 23.118 \leq V_u = 25.9 \leq \Phi(V_c + V_{s,\min}) = 31.668 \text{ KN} \quad \dots (\text{OK})$$

$$\frac{A_v}{s} = \frac{1}{16} * \sqrt{f'_c} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \sqrt{24} * \frac{120}{420} = 0.087$$

Or:

$$\frac{A_v}{s} = \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{3} * \frac{120}{420} = 0.095 \quad \dots \dots \text{control}$$

Use stirrups 2U-shape (2-lg stirrups) Φ 8 mm with $A_v = 100.53$

$$S = 1055.65 \text{ mm}$$

$$S \leq 600 \quad \text{or} \quad S \leq \frac{d}{2} = \frac{286}{2} = 143 \text{ mm}$$

Take $s = 125 \text{ mm}$

Use stirrups 2U-shape (2-lg stirrups) Φ 8 mm @ 125 mm

2) $V_{ud} = 23.6 \text{ KN} \rightarrow$ at Support 4

Case I:

$$V_u \leq \Phi V_c / 2$$

$$23.6 > 11.559 \quad (\text{Not Ok})$$

Case II:

$$\Phi V_c / 2 \leq V_u \leq \Phi V_c$$

$$11.559 < 23.6 > 23.118 \quad (\text{not ok})$$

Case III:

$$V_{s,\min} = \frac{1}{16} * \sqrt{f_c'} * b_w * d$$

$$= \frac{1}{16} * \sqrt{24} * 120 * 286 * 10^{-3}$$

$$= 10.508 \text{ KN}$$

Or:

$$V_{s,\min} = \frac{1}{3} * b_w * d = \frac{1}{3} * 120 * 286 * 10^{-3} = 11.44 \text{ KN} \quad \dots \dots \text{control}$$

$$\Phi(V_c + V_{s,\min}) = 31.668 \text{ KN}$$

$$\phi V_c = 23.118 \leq V_u = 23.6 \leq \Phi(V_c + V_{s,\min}) = 31.668 \text{ KN} \quad \dots (\text{OK})$$

$$\frac{A_v}{s} = \frac{1}{16} * \sqrt{f_c'} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \sqrt{24} * \frac{120}{420} = 0.087$$

Or:

$$\frac{A_v}{s} = \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{3} * \frac{120}{420} = 0.095 \quad \dots \dots \text{control}$$

Use stirrups 2U-shape (2-lg stirrups) Φ 8 mm with $A_v = 100.53$

$$S = 1055.65 \text{ mm}$$

$$S \leq 600 \quad \text{or} \quad S \leq \frac{d}{2} = \frac{286}{2} = 143 \text{ mm}$$

Take $s = 125 \text{ mm}$ **Use stirrups 2U-shape (2-lg stirrups) Φ 8 mm @ 125 mm**3) $V_{ud} = 17.6 \text{ KN} \rightarrow$ at Support 3

Case I:

$$V_u \leq \Phi V_c / 2$$

$$23.6 > 11.559 \quad (\text{Not Ok})$$

Case II:

$$\Phi V_c/2 \leq V_u \leq \Phi V_c$$

$$11.559 < 23.6 < 23.118 \quad (\text{ok})$$

Minimum shear reinforcement is required except for concrete joist construction. So, No shear reinforcement is provided.

4.6 Design of Beam: → Beam (0-55)

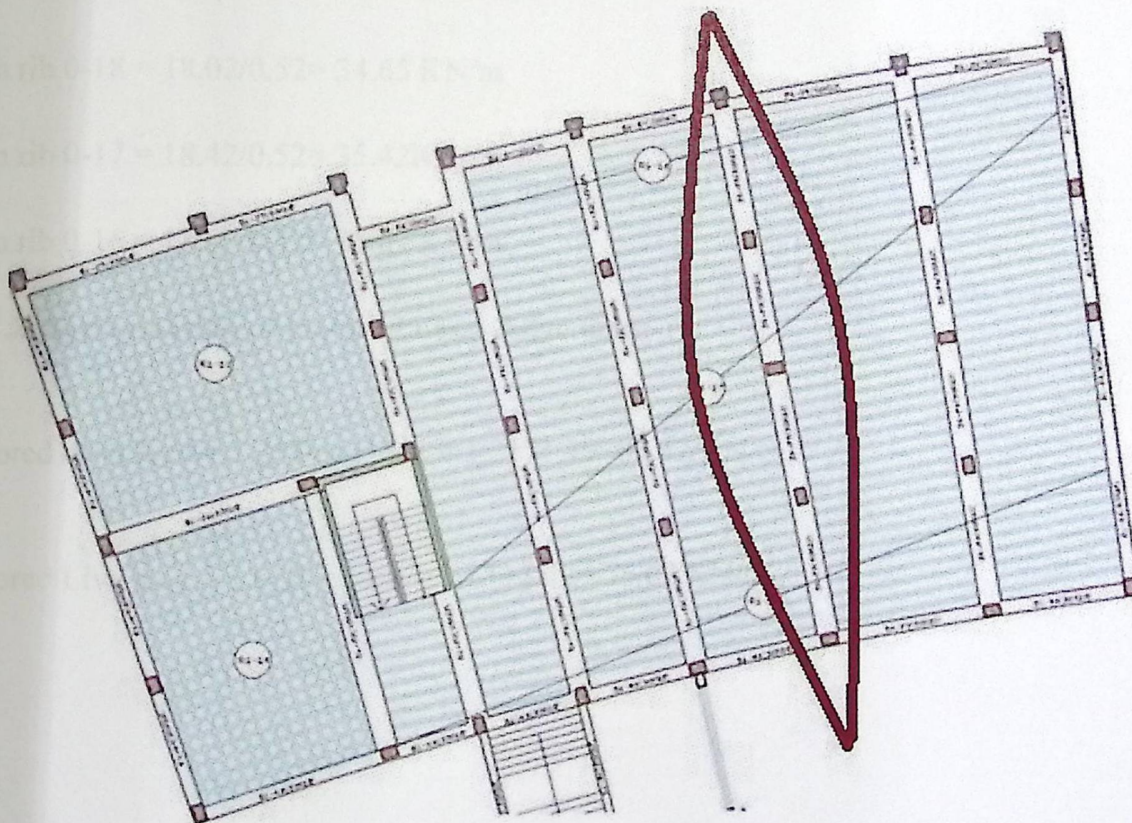


Figure (4-8) : Beam 0-55 Plan

4.6.1 Determination of Dead load of beam:-

Table(4-3) : Dead load of beam

Type	$\gamma b h$	KN/m
Reinforcement concrete	$25 * 0.80 * 0.52$	10.4

$$\text{From rib 0-18} = 36.5/0.52 = 70.19 \text{ KN/m}$$

$$\text{From rib 0-17} = 37.02/0.52 = 71.19 \text{ KN/m}$$

$$\text{From rib 0-16} = 37.36/0.52 = 71.84 \text{ KN/m}$$

4.6.1.1 Determination of live load of beam:-

$$\text{From rib 0-18} = 18.02/0.52 = 34.65 \text{ KN/m}$$

$$\text{From rib 0-17} = 18.42/0.52 = 35.42 \text{ KN/m}$$

$$\text{From rib 0-16} = 18.55/0.52 = 35.67 \text{ KN/m}$$

4.6.1.2 Determination of factored dead & live load:-

$$\text{Factored dead load} = 1.2 * \text{Dead load} = 1.2 * 71.19 = 85.42 \text{ KN/m.}$$

$$\text{Factored Live load} = 1.6 * \text{live load} = 1.6 * 35.42 = 56.67 \text{ KN/m}$$

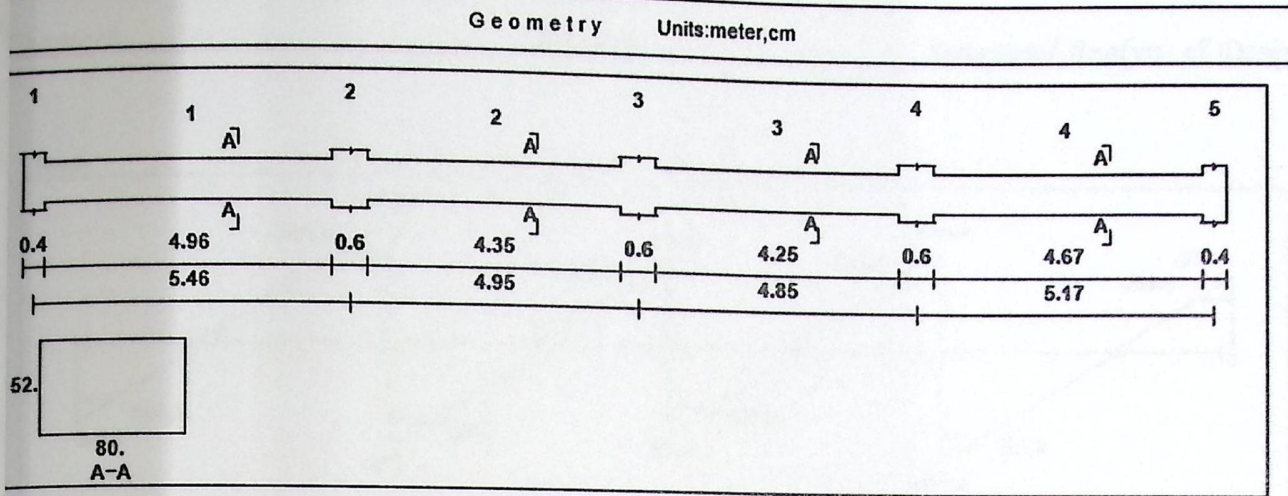


Figure (4-9): Beam (0-55) geometry

Loading

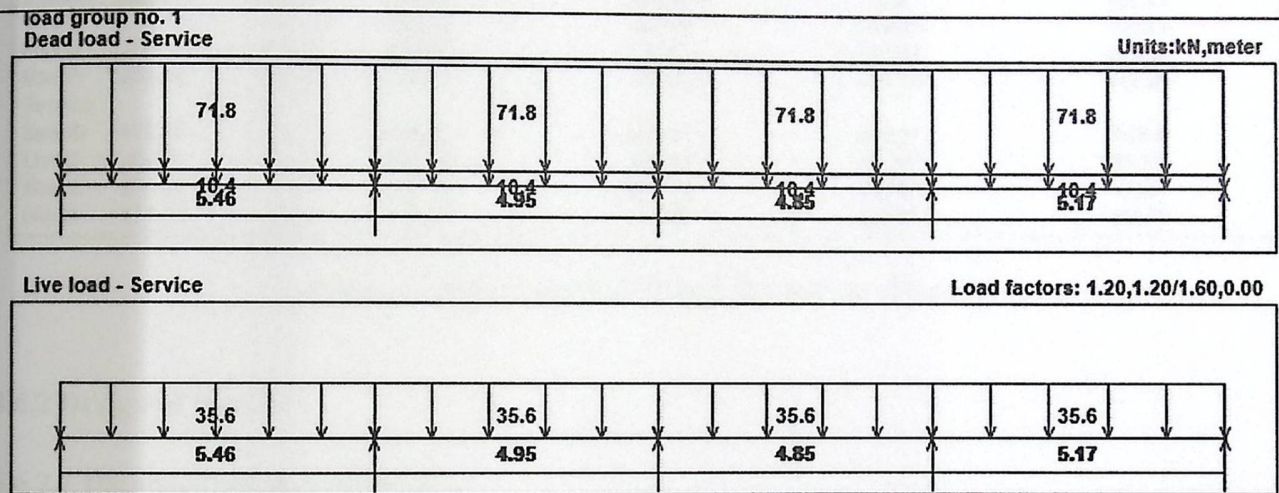


Figure (4-10): Beam (0-55) loading

Moment/Shear Envelope (Factored) Units: kN, meter

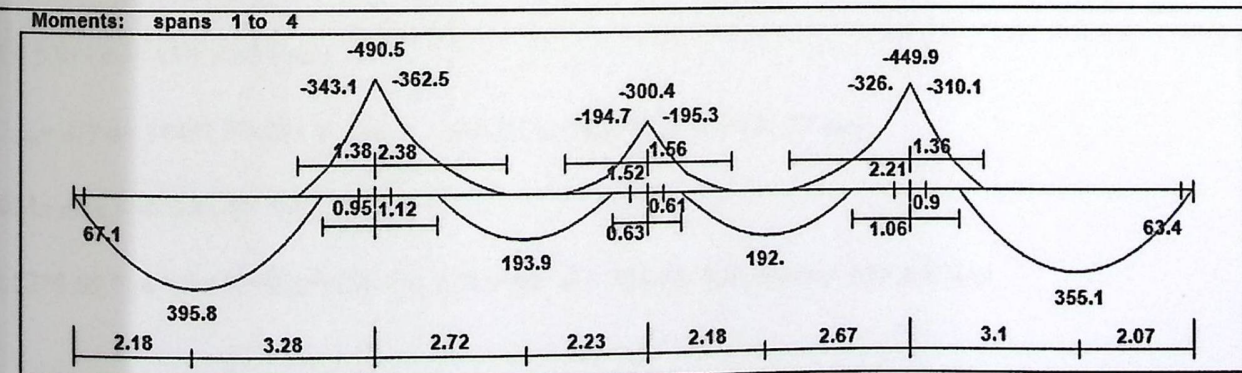


Figure (4-11): Beam (0-55) Moment diagram

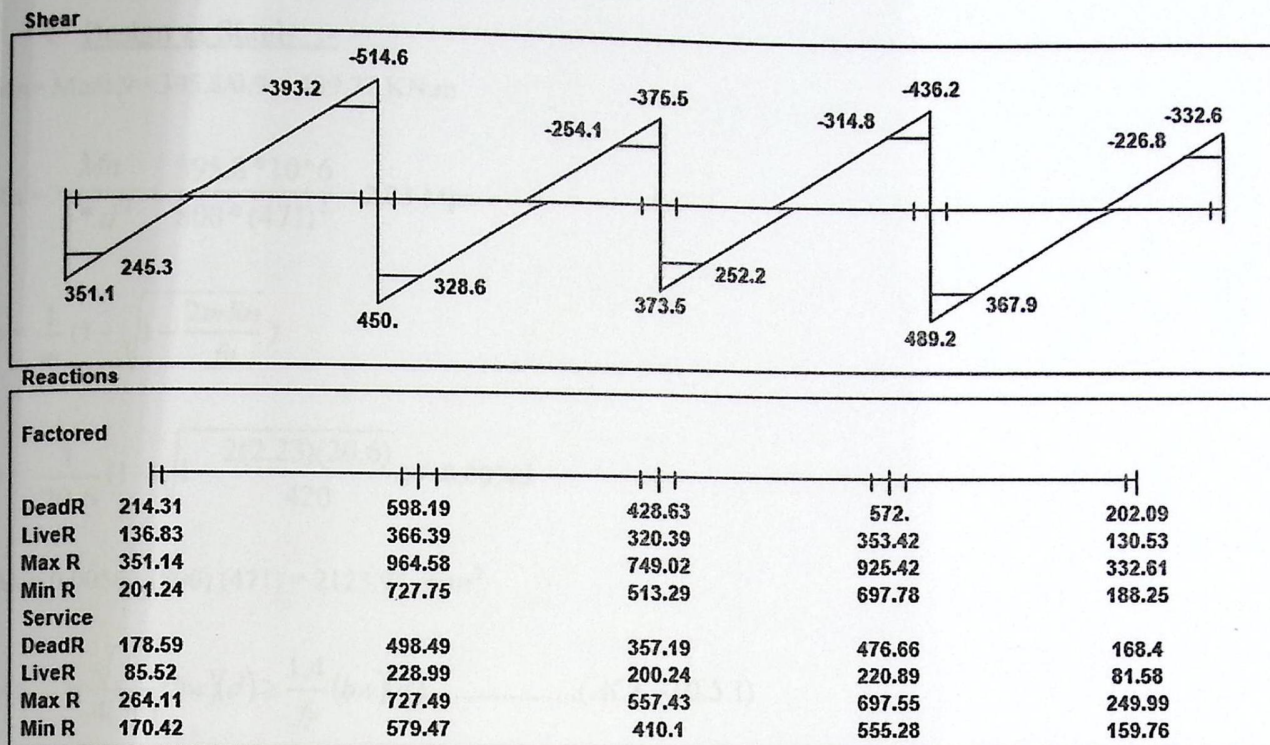


Figure (4-12): Beam (0-55) shear & reactions diagram

4.6.2 Design of flexure:-

4.6.2.1 Design of positive moment:-

Assume bars of $\Phi 18$ $b_w=80\text{cm}$ $h=32\text{ cm}$

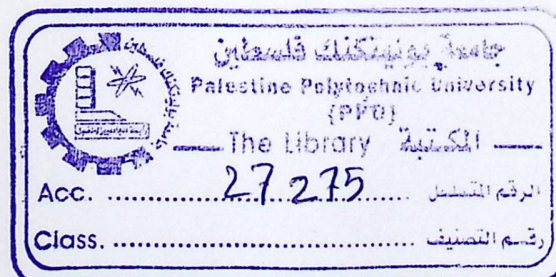
1) $M_u=395.8\text{ KN.m}$

$$d = 520 - 40 - 18/2 = 471\text{ mm}$$

$$C_{\max} = 3/7 d = 3 \cdot 471 / 7 = 201.85\text{ mm} \quad a = 0.85 C = 0.85 \cdot 201.85 = 171.57\text{ mm}$$

$$\Phi M_n \max = \Phi 0.85 f_c' \cdot a \cdot b (d - a/2)$$

$$0.82 \cdot 0.85 \cdot 24 \cdot 171.57 \cdot 800 \cdot (471 - 171.57/2) \cdot 10^{-6} = 884.46\text{ KN.m} > M_u = 395.8\text{ KN.m.}$$



❖ **Design as Singly :-**

$$M_n = Mu/0.9 = 395.8/0.9 = 439.77 \text{ KN.m}$$

$$R_n = \frac{M_n}{b * d^2} = \frac{395.8 * 10^6}{800 * (471)^2} = 2.23 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(2.23)(20.6)}{420}} \right) = 0.00563$$

$$A_s = 0.00563 (800) (471) = 2123.94 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d) \dots \dots \dots (ACI - 10.5.1)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (800)(471) \leq \frac{1.4}{420} (800)(471)$$

$$A_{s_{\min}} = 1256 \text{ mm}^2$$

$$2123.94 \text{ mm}^2 > A_{s_{\min}} = 1256 \text{ mm}^2 \quad \text{OK.}$$

$$\# \text{ of bars} = A_s / A_{s_{\text{bar}}} = 2123.94 / 254.34 = 9 \text{ bars.}$$

$$* \text{ Note } A_{\Phi 18} = 256.34 \text{ mm}^2$$

Select 9 Φ 18mm with $A_s = 2289.06 > A_s \text{ req ok.}$

❖ **Check for strain:**

Tension = compression

$$A_s * f_y = 0.85 * f_c' * b * a$$

$$a = \frac{2123.94 * 420}{0.85 * 24 * 800} = 54.66 \text{ mm}$$

$$c = 64.31 \text{ mm}$$

$$\epsilon = 0.003 * \left(\frac{d-c}{c} \right) = 0.003 * \left(\frac{471-64.31}{64.31} \right) = 0.00632 > 0.005$$

$$\phi = 0.9.$$

$$\begin{aligned} M_n &= 0.85 f_c' a b \left(d - \frac{a}{2} \right) = 0.85 * 24 * 64.31 * 800 * \left(471 - \frac{64.31}{2} \right) \\ &= 460.58 \text{ KN.m} \end{aligned}$$

$$\phi M_n = 0.9 * 460.58 = 414.52 \text{ KN.m} > 395.8 \text{ KN.m}$$

❖ Check for spacing

$$= (800 - 40 * 2 - 2 * 10 - 9 * 18) / 8 = 67.25 \text{ mm} > 25 \text{ mm} \quad \text{Ok}$$

2) $M_u = 193.9 \text{ KN.m}$

$$d = 520 - 40 - 18/2 = 471 \text{ mm}$$

$$c_{\max} = 3/7 d = 3 * 471 / 7 = 201.85 \text{ mm} \quad a = 0.85 C = 0.85 * 201.85 = 171.57 \text{ mm}$$

$$\phi M_n \max = \phi 0.85 f_c' a b (d - a/2)$$

$$0.82 * 0.85 * 24 * 171.57 * 800 * (471 - 171.57/2) * 10^{-6} = 884.46 \text{ KN.m} > M_u = 193.9 \text{ KN.m}$$

❖ Design as Singly :-

$$M_n = M_u / 0.9 = 193.9 / 0.9 = 215.44 \text{ KN.m}$$

$$R_n = \frac{M_n}{b * d^2} = \frac{215.44 * 10^6}{800 * (471)^2} = 1.214 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(1.214)(20.6)}{420}} \right) = 0.00298$$

$$s = 0.00298(800)(471) = 1123.64 \text{ mm}^2$$

$$s_{\min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) \geq \frac{1.4}{fy}(bw)(d) \dots \dots \dots (ACI-10.5.1)$$

$$s_{\min} = \frac{\sqrt{24}}{4(420)}(800)(471) \leq \frac{1.4}{420}(800)(471)$$

$$s_{\min} = 1256 \text{ mm}^2$$

$$1123.64 \text{ mm}^2 > s_{\min} = 1256 \text{ mm}^2 \quad \text{OK.}$$

$$\text{of bars} = As / A_{s \text{ bar}} = 1123.64 / 254.34 = 5 \text{ bars.}$$

$$* \text{ Note } A_{\Phi 18} = 256.34 \text{ mm}^2$$

Select 5 Φ 18 mm with $As = 1271.7 > As \text{ req ok.}$

❖ **Check for strain:**

Tension = compression

$$As * fy = 0.85 * fc' * b * a$$

$$a = \frac{1271.7 * 420}{0.85 * 24 * 800} = 32.72 \text{ mm}$$

$$c = 38.5 \text{ mm}$$

$$\epsilon = 0.003 * \left(\frac{d-c}{c} \right) = 0.003 * \left(\frac{471-38.5}{38.5} \right) = 0.0336 > 0.005$$

$$\phi = 0.9.$$

$$M_n = 0.85 f_c' a b \left(d - \frac{a}{2} \right) = 0.85 * 24 * 32.72 * 800 * \left(471 - \frac{32.72}{2} \right)$$

$$= 242.773 \text{ KN.m}$$

$$\phi M_n = 0.9 * 242.77 = 218.5 \text{ KN.m} > 193.9 \text{ KN.m}$$

❖ Check for spacing

$$= (800 - 40 \times 2 - 2 \times 10 - 5 \times 18) / 4 = 152.5 \text{ mm} > 25 \text{ mm} \quad \text{Ok}$$

$$1) M_u = 192 \text{ KN.m}$$

Select 5 Φ 18 mm with $A_s = 1271.7 > A_s \text{ req. like the previous.}$

$$2) M_u = 355.1 \text{ KN.m}$$

Select 9 Φ 18mm with $A_s = 2289.06 > A_s \text{ req. like the previous.}$

.6.2.2 Design of negative moment:

Assume bars of Φ 18

$$b_w = 110 \text{ cm} \quad h = 32 \text{ cm}$$

$$d = 520 - 40 - 18/2 = 471 \text{ mm}$$

$$C_{\max} = 3/7 d = 3 \times 471 / 7 = 201.85 \text{ mm} \quad a = 0.85 C = 0.85 \times 201.85 = 171.57 \text{ mm}$$

$$\Phi M_n \max = \Phi 0.85 f_c' * a * b (d - a/2)$$

$$0.82 * 0.85 * 24 * 171.57 * 800 * (471 - 171.57/2) * 10^{-6} = 884.46 \text{ KN.m} > M_u = 395.8 \text{ KN.m.}$$

❖ Design as Singly :-

$$1) M_u = -362.5 \text{ KN.m}$$

Select 9 Φ 18mm with $A_s = 2289.06 > A_s \text{ req. like the previous.}$

$$2) M_u = 195.3 \text{ KN.m}$$

Select 5 Φ 18 mm with $A_s = 1271.7 > A_s \text{ req. like the previous.}$

$$3) M_u = 326.0 \text{ KN.m}$$

select 9 Φ 18mm with $A_s=2289.06$ > As req. like the previous.

6.2.3 Design of shear :

1) $V_u = 245.3 \text{ KN} \rightarrow$ support (1)

$$V_c = \frac{1}{6} * 1 * \sqrt{24} * 800 * 471 * 10^{-3} = 307.65 \text{ KN}$$

$$\phi * V_c = \frac{1}{2} * 0.75 * 307.65 = 115.37 \text{ KN}$$

Case I:

$$V_u \leq \frac{1}{2} * \phi * V_c \quad (\text{Not Ok})$$

Case II:

$$\phi * V_c = 0.75 * 307.65 = 230.73 \text{ KN}$$

$$\frac{1}{2} * \phi * V_c < V_u \leq \phi * V_c \quad (\text{Not Ok})$$

Case III:

$$V_{s,min} = \frac{1}{16} * \sqrt{24} * 800 * 471 * 10^{-3} = 115.37 \text{ KN}$$

$$V_{s,min} = \frac{1}{3} * 800 * 471 * 10^{-3} = 125.6 \text{ KN}$$

$$\phi(V_c + V_{s,min}) = 0.75(307.65 + 125.6) = 324.93$$

$$\phi V_c < V_u \leq \phi(V_c + V_{s,min})$$

$$230.73 < 245.3 \leq 324.93 \quad (\text{Ok})$$

Use stirrups U - shape (4 - leg stirrups) $\Phi 10$ with $A_{v,min} = 4 * 78.54 = 314.159 \text{ mm}^2$

$$\frac{A_{v,min}}{s} = \frac{1}{16} * \sqrt{f_c'} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \sqrt{24} * \frac{800}{420} \rightarrow \frac{A_{v,min}}{s} = 0.583$$

$$\frac{min}{s} = \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \frac{800}{420} \rightarrow \frac{A_{v,min}}{s} = 0.635 \dots \text{control}$$

$$\frac{14.159}{s} = 0.635 \rightarrow s = 494.8 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} \rightarrow \frac{471}{2} = 235.5 \quad \text{or} \quad s_{max} \leq 600 \text{ mm}$$

Take $s = 15 \text{ cm}$.

2) $V_u = 393.2 \text{ KN} \rightarrow$ support (2)

$$V_c = \frac{1}{6} * 1 * \sqrt{24} * 800 * 471 * 10^{-3} = 307.65 \text{ KN}$$

$$\phi * V_c = \frac{1}{2} * 0.75 * 307.65 = 115.37 \text{ KN}$$

Case I:

$$V_u \leq \frac{1}{2} * \phi * V_c \quad (\text{Not Ok})$$

Case II:

$$\phi * V_c = 0.75 * 307.65 = 230.73 \text{ KN}$$

$$\frac{1}{2} * \phi * V_c < V_u \leq \phi * V_c \quad (\text{Not Ok})$$

Case III:

$$V_{s,min} = \frac{1}{16} * \sqrt{24} * 800 * 471 * 10^{-3} = 115.37 \text{ KN}$$

$$V_{s,min} = \frac{1}{3} * 800 * 471 * 10^{-3} = 125.6 \text{ KN}$$

$$\phi(V_c + V_{s,min}) = 0.75(307.65 + 125.6) = 324.93$$

$$\phi V_c < V_u \leq \phi(V_c + V_{s,min})$$

$$230.73 < 393.2 \leq 324.93 \quad (\text{Not Ok})$$

Case IV:

$$V_s' = \frac{1}{3} * \sqrt{f_c'} * b_w * d$$

$$V_s' = \frac{1}{3} * \sqrt{24} * 800 * 471 * 10^{-3} = 615.30 \text{ KN}$$

$$(V_c + V_s') = 0.75(307.65 + 615.30) = 692.21 \text{ KN}$$

$$307.65 < 393.2 \leq 692.21 \quad \text{Ok}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} * d}$$

Use stirrups U - shape (4 - leg stirrups) $\phi 10$ with $A_v = 4 * 78.54 = 314.159 \text{ mm}^2$

$$V_s = V_n - V_c = \frac{393.2}{0.75} - 307.65 = 216.61 \text{ KN}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} * d} \rightarrow \frac{314.159 * 10^{-3}}{s} = \frac{216.61}{420 * 471} \rightarrow s = 286.89 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} \rightarrow \frac{471}{2} = 232.5 \quad \text{or} \quad s_{max} \leq 600 \text{ mm}$$

Take $s = 15 \text{ cm}$

3) $V_u = 254.1 \text{ KN} \rightarrow$ support (3)

$$V_c = \frac{1}{6} * 1 * \sqrt{24} * 800 * 471 * 10^{-3} = 307.65 \text{ KN}$$

$$\frac{1}{2} * \phi * V_c = \frac{1}{2} * 0.75 * 307.65 = 115.37 \text{ KN}$$

Case I:

$$V_u \leq \frac{1}{2} * \phi * V_c \quad (\text{Not Ok})$$

Case II:

$$* V_c = 0.75 * 307.65 = 230.73 \text{ KN}$$

$$* \phi * V_c < V_u \leq * \phi * V_c \quad (\text{Not Ok})$$

Case III:

$$V_{s,min} = \frac{1}{16} * \sqrt{24} * 800 * 471 * 10^{-3} = 115.37 \text{ KN}$$

$$V_{s,min} = \frac{1}{3} * 800 * 471 * 10^{-3} = 125.6 \text{ KN}$$

$$\phi(V_c + V_{s,min}) = 0.75(307.65 + 125.6) = 324.93$$

$$\phi V_c < V_u \leq \phi(V_c + V_{s,min})$$

$$230.73 < 254.1 \leq 324.93 \quad (\text{Ok})$$

Use stirrups U - shape (4 - leg stirrups) $\phi 10$ with $A_{v,min} = 4 * 78.54 = 314.159 \text{ mm}^2$

$$\frac{A_{v,min}}{s} = \frac{1}{16} * \sqrt{f_c'} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \sqrt{24} * \frac{800}{420} \rightarrow \frac{A_{v,min}}{s} = 0.583$$

$$\frac{A_{v,min}}{s} = \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \frac{800}{420} \rightarrow \frac{A_{v,min}}{s} = 0.635 \dots \dots \dots \text{control}$$

$$\frac{314.159}{s} = 0.635 \rightarrow s = 494.8 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} \Rightarrow \frac{471}{2} = 235.5 \quad \text{or} \quad s_{max} \leq 600 \text{ mm}$$

Take $s=15\text{cm}$.

4) $V_u = 367.9 \text{ KN} \rightarrow$ support (4)

$$V_c = \frac{1}{6} * 1 * \sqrt{24} * 800 * 471 * 10^{-3} = 307.65 \text{ KN}$$

$$\frac{1}{2} * \phi * V_c = \frac{1}{2} * 0.75 * 307.65 = 115.37 \text{ KN}$$

Case I:

$$\leq \frac{1}{2} * \phi * V_c \quad (\text{Not Ok})$$

Case II:

$$* V_c = 0.75 * 307.65 = 230.73 \text{ KN}$$

$$* \phi * V_c < V_u \leq * \phi * V_c \quad (\text{Not Ok})$$

Case III:

$$V_{s,min} = \frac{1}{16} * \sqrt{24} * 800 * 471 * 10^{-3} = 115.37 \text{ KN}$$

$$V_{s,min} = \frac{1}{3} * 800 * 471 * 10^{-3} = 125.6 \text{ KN}$$

$$\phi(V_c + V_{s,min}) = 0.75(307.65 + 125.6) = 324.93$$

$$\phi V_c < V_u \leq \phi(V_c + V_{s,min})$$

$$230.73 < 393.2 \leq 324.93 \quad (\text{Not Ok})$$

Case IV:

$$V_s' = \frac{1}{3} * \sqrt{f_c'} * b_w * d$$

$$V_s' = \frac{1}{3} * \sqrt{24} * 800 * 471 * 10^{-3} = 615.30 \text{ KN}$$

$$\phi(V_c + V_s') = 0.75(307.65 + 615.30) = 692.21 \text{ KN}$$

$$307.65 < 367.9 \leq 692.21 \quad \text{Ok}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} * d}$$

Use stirrups U - shape (4 - leg stirrups) $\phi 10$ with $A_v = 4 * 78.54 = 314.159 \text{ mm}^2$

$$V_s = V_n - V_c = \frac{367.9}{0.75} - 307.65 = 182.88 \text{ KN}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} * d} \rightarrow \frac{314.159 * 10^{-3}}{s} = \frac{182.88}{420 * 471} \rightarrow s = 339.81 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} \rightarrow \frac{471}{2} = 232.5 \quad \text{or} \quad s_{max} \leq 600 \text{ mm}$$

Take $s = 15 \text{ cm}$

5) $V_u = 226.8 \text{ KN} \rightarrow$ support (5)

$$V_c = \frac{1}{6} * 1 * \sqrt{24} * 800 * 471 * 10^{-3} = 307.65 \text{ KN}$$

$$\frac{1}{2} * \phi * V_c = \frac{1}{2} * 0.75 * 307.65 = 115.37 \text{ KN}$$

Case I:

$$V_u \leq \frac{1}{2} * \phi * V_c \quad (\text{Not Ok})$$

Case II:

$$\phi * V_c = 0.75 * 307.65 = 230.73 \text{ KN}$$

$$\frac{1}{2} * \phi * V_c < V_u \leq \phi * V_c \quad (\text{Not Ok})$$

Case III:

$$V_{s,min} = \frac{1}{16} * \sqrt{24} * 800 * 471 * 10^{-3} = 115.37 \text{ KN}$$

$$V_{s,min} = \frac{1}{3} * 800 * 471 * 10^{-3} = 125.6 \text{ KN}$$

$$\phi(V_c + V_{s,min}) = 0.75(307.65 + 125.6) = 324.93$$

$$\phi V_c < V_u \leq \phi(V_c + V_{s,min})$$

$$230.73 < 226.8 \leq 324.93 \quad (Ok)$$

Use stirrups U - shape (4 - leg stirrups) $\phi 10$ with $A_{v,min} = 4 * 78.54 = 314.159 \text{ mm}^2$

$$\frac{A_{v,min}}{s} = \frac{1}{16} * \sqrt{f_c'} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \sqrt{24} * \frac{800}{420} \rightarrow \frac{A_{v,min}}{s} = 0.583$$

$$\frac{A_{v,min}}{s} = \frac{1}{3} * \frac{b_w}{f_{yt}} = \frac{1}{16} * \frac{800}{420} \rightarrow \frac{A_{v,min}}{s} = 0.635 \dots \dots \dots \text{control}$$

$$\frac{314.159}{s} = 0.635 \rightarrow s = 494.8 \text{ mm}$$

$$s_{max} \leq \frac{d}{2} \rightarrow \frac{471}{2} = 235.5 \quad \text{or} \quad s_{max} \leq 600 \text{ mm}$$

Take $s = 15 \text{ cm}$.

4.6.2.4 Cut of Bars Calculations :-

According to German code (DIN 1045-1) :-

Required Reinforcement At End Supports:

❖ The first & final support :-

$$F_{sd,R} = V_{Ed,0} * \frac{a_1}{z} + N_{Ed,0} \geq \frac{V_{Ed,0}}{2}$$

$$V_{Ed,0} = 245.3 \text{ KN.}$$

$$a_1 = 0.54 * d = 0.54 * 245.3 = 132.46 \text{ mm.}$$

$$z = 0.9 * 245.3 = 220.77 \text{ mm.}$$

$$F_{sd,R} = 245.31 * \frac{132.46}{220.77} + 0.0 \geq \frac{245.3}{2}$$

$$F_{sd,R} = 147.17 \geq 122.65.$$

$$A_{s,req} = \frac{F_{sd,R}}{0.75 \times f_y} = \frac{147.17}{0.75 \times 42} = 4.672 \text{ cm}^2.$$

$$A_{s,prov} = 9\phi 18 = 9 \times 2.54 = 22.86 > 4.67 > 0.25 \times A_{s,Pos} = 0.25 \times 22.86 = 5.715 \text{ cm}^2$$

$$L_{b,dir} = \frac{2}{3} \times L_{b,net} \times \alpha > 15 \text{ cm} > 6 \times db$$

$$L_{b,net} = L_{dt}$$

$$L_{dt} = \frac{9}{10} \times \frac{f_y}{\sqrt{f_c'}} \times \frac{\psi_t \times \psi_e \times \psi_s}{\frac{c_b + K_{tr}}{d_b}} \times db$$

$$c_b = \frac{1}{2} \times 67.25 = 33.625 \text{ mm. - control.}$$

$$\text{or, } = 40 + 10 + \frac{20}{2} = 60 \text{ mm}$$

$$L_{dt} = \frac{9}{10} \times \frac{420}{\sqrt{24}} \times \frac{1 \times 1 \times 0.8}{2.5} \times 20 = 493.82 \text{ mm.}$$

$$\alpha = \frac{A_{s,req}}{A_{s,prov.}} = \frac{4.67}{22.86} = 0.204$$

$$L_{b,dir} = \frac{2}{3} \times 493.82 \times 0.204 = 67.16 \text{ mm} > 150 \text{ mm.} > 6 \times db = 108 \text{ mm.}$$

$$L_{available} = 600 - 40 = 560 \text{ mm} > L_{b,dir} \dots \text{Ok.}$$

❖ Lab Splices :-

$$L_{dt} = \frac{9}{10} \times \frac{f_y}{\sqrt{f_c'}} \times \frac{\psi_t \times \psi_e \times \psi_s}{\frac{c_b + K_{tr}}{d_b}} \times db$$

$$c_b = \frac{1}{2} \times 306.7 = 153.33 \text{ mm - control.}$$

$$\text{or, } = 40 + 10 + \frac{20}{2} = 60 \text{ mm.}$$

$$L_{dt} = \frac{9}{10} \times \frac{420}{\sqrt{24}} \times \frac{1 \times 1 \times 0.8}{2.5} \times 20 = 493.82 \text{ mm.}$$

$$L_{s,t} = 1.30 \times L_{dt} = 641.966 \text{ mm.}$$

7 Design of Two Way Rib Slab (R0-15):

4.7.1 Determination of Thickness for Two Way Rib Slab:

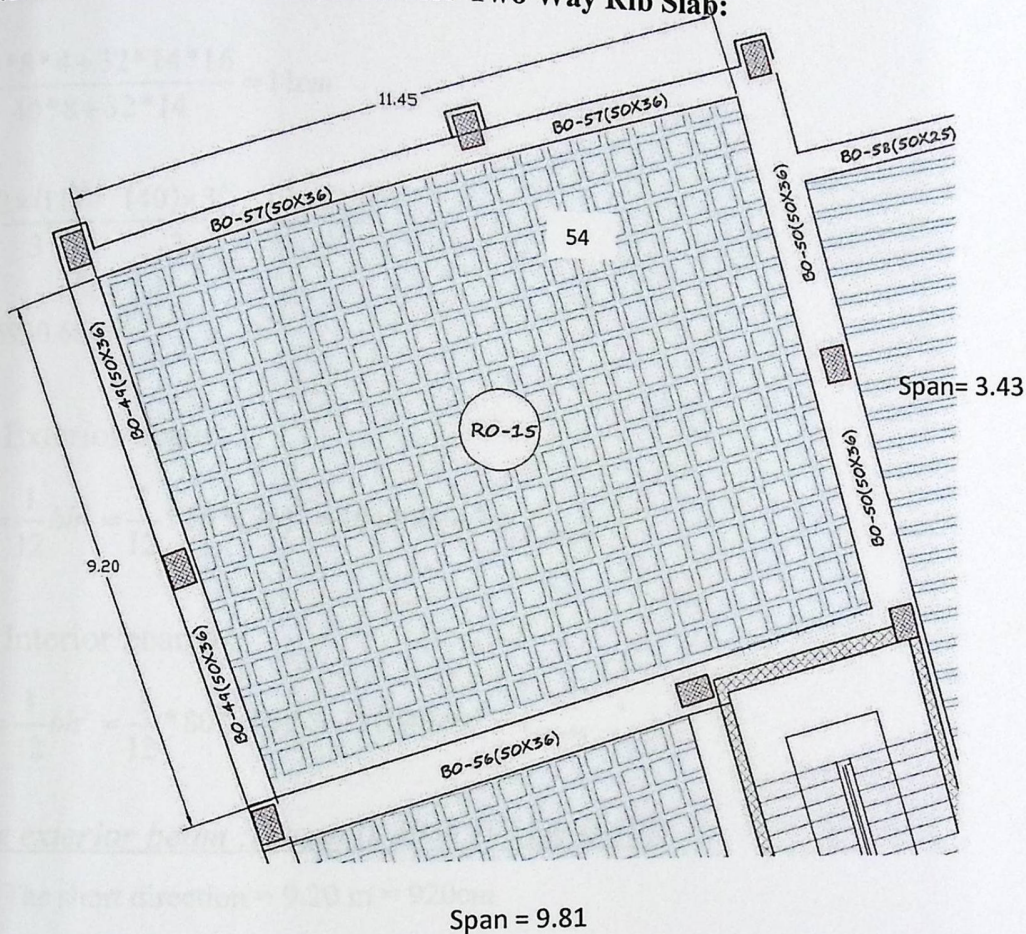
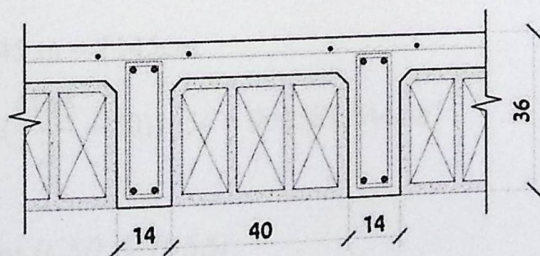


Figure (4-13): two-way rib slab(0-15).



Two Way Ribbed Slab
Figure (4-14): T Section .

$$\bar{Y} = \frac{\sum AY}{\sum A}$$

$$\bar{Y} = \frac{40 \cdot 8 \cdot 4 + 32 \cdot 14 \cdot 16}{40 \cdot 8 + 32 \cdot 14} = 11 \text{ cm}$$

$$I_{rib} = \frac{52 \times (11)^3}{3} - \frac{(40) \times 3^3}{3} + \frac{12 \times (25)^3}{3}$$

$$I_{rib} = 85930.66 \text{ cm}^4$$

- Exterior beams :

$$I_{B0-49} = \frac{1}{12} bh^3 = \frac{1}{12} * 50 * (36)^3 = 194400 \text{ cm}^4 = I_{B0-51}$$

- Interior beams :

$$I_{B0-50} = \frac{1}{12} bh^3 = \frac{1}{12} * 80 * (56)^3 = 311040 \text{ cm}^4 = I_{B0-56}$$

1. The exterior beam : (beam 0-49 & 0-51 & 0-50):

- ✓ The short direction = 9.20 m = 920 cm

$$I_{s1} = \frac{I_{RB} (l/2 + b_w)}{b_F} = (85930 \times (\frac{920}{2} + 50)) / 54 = 811561.1 \text{ cm}^4$$

- ✓ The long direction = 11.45 m = 1145 cm

$$I_{s2} = \frac{I_{RB} (l/2 + b_w)}{b_F} = (85930 \times (\frac{1145}{2} + 50)) / 54 = 990581.944 \text{ cm}^4$$

2. The interior beam : (beam 0-50 & 0-56):

- ✓ The long direction = 11.45 m = 1145 cm

$$I_{s2} = \frac{I_{RB} * (\frac{I_{Right}}{2} + \frac{I_{left}}{2} + b_w)}{b_F} = (85930 \times (\frac{920}{2} + \frac{943}{2} + 80)) / 54 = 1609596.2 \text{ cm}^4$$

$$\alpha_1 = \frac{I_{b1}}{I_{s1}} = \frac{194400}{811561} = 0.239$$

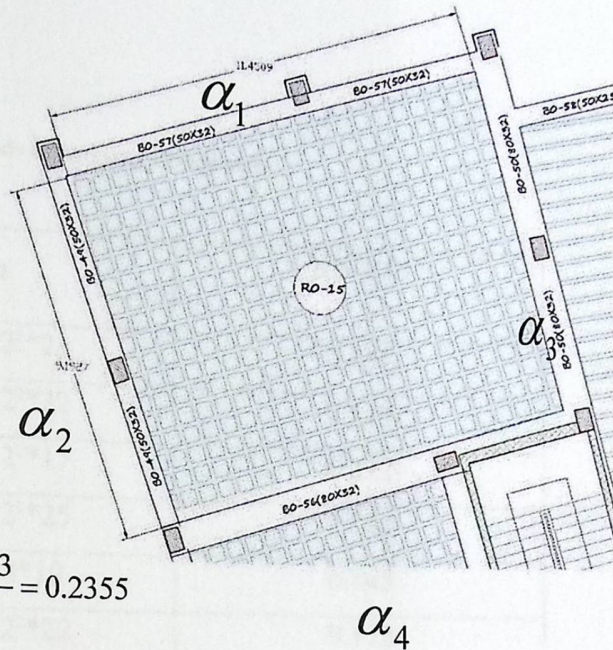
$$\alpha_2 = \frac{I_{b2}}{I_{s2}} = \frac{194400}{990581} = 0.196$$

$$\alpha_3 = \frac{I_{b1}}{I_{s1}} = \frac{311040}{990581} = 0.314$$

$$\alpha_4 = \frac{I_{b2}}{I_{s2}} = \frac{311040}{1609596} = 0.193$$

$$\alpha_{fm} = \frac{\alpha_1 + \alpha_2 + \alpha_3 + \alpha_4}{4} = \frac{0.239 + 0.196 + 0.314 + 0.193}{4} = 0.2355$$

$$0.2 < \alpha < 2 \implies 0.2 < 0.2355 < 2$$



According to ACI-code:

$$h_m = \frac{l(0.8 + f_y/1400)}{36 + 5\beta(\alpha_{fm} - 0.2)}$$

$$\beta = \frac{L_a}{L_b} = \frac{1145}{920} = 1.244$$

$$h_m = \frac{1145 * (0.8 + 420/1400)}{36 + 5 * 1.244 * (0.2355 - 0.2)} = 347.75 \text{ mm}$$

347 mm > 125 mm ...ok

take First trial thickness $h=360 \text{ mm} > 347.75 \text{ mm}$ -ok

Take slab thickness $h=360$, 80mm - topping, 280mm concrete block.

4.7.2 Load Calculation:-

4.7.2.1 Determination of Dead load:-

Table(4-4) : Dead loads of two way ribbed slab

Type	$\gamma b h$	KN/Rib
Tiles	$0.03 \times 0.52^2 \times 23$	0.187
Mortar	$0.02 \times 0.52^2 \times 22$	0.119
Sand	$0.07 \times 0.52^2 \times 17$	0.322
Topping	$0.08 \times 0.52^2 \times 25$	0.541
Hollow block	$0.4^2 \times 0.28 \times 10$	0.448
Plaster	$0.03 \times 0.52^2 \times 22$	0.178
R.C rib	$0.12 \times 0.28 \times 25 \times (0.52 + 0.4)$	0.7728
Partitions	2.3×0.52^2	0.621
Sum		3.19

Nominal Total Dead Load = **3.19 KN/Rib**

$$3.19 / (0.52^2) = \mathbf{11.8 \text{ KN/m}^2}$$

Nominal Total live load for studios = 2.5 KN/m^2

4-7.2.2 Determination of factored dead & live load

$$\text{Factored dead load} = 1.2 \times \text{Dead load} = 1.2 \times 11.8 = \mathbf{14.16 \text{ KN/m}^2}.$$

$$\text{Factored Live load} = 1.6 \times \text{live load} = 1.6 \times 2.5 = \mathbf{4 \text{ KN/m}^2}.$$

$$\text{Total factored load} = 14.16 + 4 = \mathbf{18.16 \text{ KN/m}^2}.$$

4.7.3 Design for moment:

- The slab is discontinuous from two sides, so it will be assumed as : (case 6 in analysis for moments)
- The moment at discontinuous edges will be taken as $\frac{1}{3} * M +ve$ in each direction.

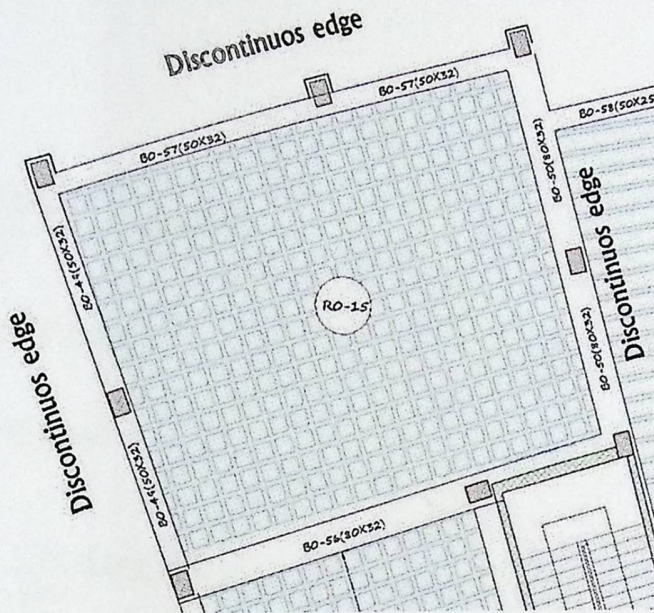


Figure (4-15): Moment Slab.

➤ Short - direction (a) case 4 :

Max Positive Moment :-

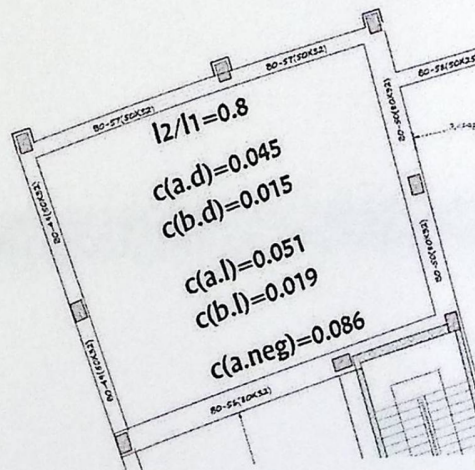
Assume $\Phi 14$

$d = 360 - 20 - 10 - 14/2 = 323 \text{ mm}$

$L_a/L_b = 9.20/11.45 = 0.8 \dots\dots\dots \text{Case 6}$

$C_{a, \text{pos/dl}} = 0.045$

$C_{a, \text{pos/l}} = 0.051$



$M_a(+ve) = (C_{ad} * W_{dl} * L_a^2 * 0.52) + (C_{al} * W_{ll} * L_a^2 * 0.52)$ Figure (4-16): coefficient of Slab.

$= (0.045 * 14.16 * 9.20^2 * 0.52) + (0.051 * 4 * 9.20^2 * 0.52)$

$= 37.02 \text{ KN.m/Rib}$

Positive moment for a-direction = +31.91KN.m/Rib.

$$m = \frac{f_y}{0.85 * f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$R_n = \frac{M_n}{b * d^2} = \frac{(37.02 / 0.9) * 10^6}{540 * (323)^2} = 0.731 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(0.731)(20.6)}{420}} \right) = 0.00177$$

$$A_s = \rho b d = 0.00177 * 540 * 323 = 309 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (140)(323) \geq \frac{1.4}{420} (140)(323)$$

$$A_{s_{\min}} = 131.9 < 150.8$$

$$\text{So: } A_{s_{\min}} = 150.8 \text{ mm}^2$$

$$A_{s_{\min}} < A_s = 309 \text{ mm}^2$$

select (2) bars Φ 14 with area = 401.92 mm²

❖ **Check for strain:**

$$a = \frac{A_s * f_y}{0.85 * f_c' * b} = \frac{401.92 * 420}{0.85 * 24 * 540} = 15.32 \text{ mm}$$

$$c = \frac{15.32}{0.85} = 18.02 \text{ mm}$$

$$\epsilon_s = \frac{323 - 18.02}{18.02} * 0.003 = 0.0507 > 0.005 \dots \text{ok}$$

The negative moment for a-direction at the discontinues edge

$$= (1/3) * Ma \text{ (+ve)} = (1/3) * 37.02 = -12.34 \text{ KN.m/Rib.}$$

Assume $\Phi 12$

$$d = 360 - 20 - 10 - 12/2 = 324 \text{ mm}$$

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 24} = 20.6$$

$$Rn = \frac{Mn}{b * d^2} = \frac{(12.34 / 0.9) * 10^6}{140 * (324)^2} = 0.932 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mRn}{fy}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(0.932)(20.6)}{420}} \right) = 0.002274$$

$$As = \rho bd = .002274 \times 140 \times 324 = 103.17 \text{ mm}^2$$

$$As_{\min} = \frac{\sqrt{fc'}}{4(fy)} (bw)(d) \geq \frac{1.4}{fy} (bw)(d)$$

$$As_{\min} = \frac{\sqrt{24}}{4(420)} (140)(324) \geq \frac{1.4}{420} (140)(324)$$

$$As_{\min} = 132.27 < 151.2$$

$$As_{\min} = 151.2 \text{ mm}^2 \text{ -control}$$

$$As_{\min} = 151.2 < As = 103.17 \text{ mm}^2$$

Take the minimum.

Select (2) bars $\Phi 10$ with area = 157.1 mm²

❖ Check for strain:

$$a = \frac{A_s \times f_y}{0.85 \times f_c \times b} = \frac{157.1 \times 420}{0.85 \times 24 \times 140} = 23.1 \text{ mm}$$

$$c = \frac{23.1}{0.85} = 27.18 \text{ mm}$$

$$\epsilon_s = \frac{324 - 27.18}{27.18} \times 0.003 = 0.0327 > 0.005 \dots \text{ok}$$

The negative moment for a-direction a continuous edge:-

$$d = 360 - 20 - 10 - 14/2 = 323 \text{ mm}$$

$$L_a/L_b = 9.20/11.45 = 0.8 \dots \dots \dots \text{Case 6}$$

$$C_{a,\text{neg}} = 0.086$$

$$M_{a(+ve)} = (C_{a,\text{neg}} \times W_{\text{total}} \times L_a^2 \times 0.52)$$

$$= (0.086 \times 18.16 \times 9.20^2 \times 0.52) = \underline{\underline{68.73 \text{ KN.m/Rib}}}$$

Negative moment for a-direction at continues edge = -68.73KN.m/Rib.

$$m = \frac{f_y}{0.85 \times f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$R_n = \frac{M_n}{b \times d^2} = \frac{(68.73 / 0.9) \times 10^6}{140 \times (324)^2} = 5.19 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(5.19)(20.6)}{420}} \right) = 0.01455$$

$$A_s = \rho b d = 0.01455 \times 140 \times 324 = 660.23 \text{ mm}^2$$

$$A_{s_{min}} = \frac{\sqrt{f_c'}}{4(f_y)}(bw)(d) \geq \frac{1.4}{f_y}(bw)(d)$$

$$A_{s_{min}} = \frac{\sqrt{24}}{4(420)}(140)(324) \geq \frac{1.4}{420}(140)(324)$$

$$A_{s_{min}} = 132.27 < 151.2$$

$$A_{s_{min}} = 151.2 \text{ mm}^2 - \text{control}$$

$$A_{s_{min}} = 151.2 < A_s = 660.23 \text{ mm}^2$$

Select (2) bars $\Phi 22$ with area = 759.88 mm²

❖ Check for strain:

$$a = \frac{A_s \times f_y}{0.85 \times f_c' \times b} = \frac{157.1 \times 420}{0.85 \times 24 \times 140} = 23.1 \text{ mm}$$

$$c = \frac{23.1}{0.85} = 27.18 \text{ mm}$$

$$\epsilon_s = \frac{324 - 27.18}{27.18} \times 0.003 = 0.0327 > 0.005 \dots \text{ok}$$

➤ Long - direction:(case4)

❖ Max positive moment :

Assume $\Phi 12$

$$d = 360 - 20 - 10 - 12/2 = 324 \text{ mm}$$

$$L_a/L_b = 9.20/11.45 = 0.8 \dots \dots \dots \text{Case 4}$$

$$C_{b_{pos/dl}} = 0.015$$

$$C_{b_{pos/l}} = 0.019$$

$$M_{a(+ve)} = (C_{b_{dl}} * W_{dl} * L_b^2 * 0.52) + (C_{b_{ll}} * W_{ll} * L_b^2 * 0.52)$$

$$= (0.015 * 14.16 * 11.45^2 * 0.52) + (0.019 * 4 * 11.45^2 * 0.52) = \underline{\underline{19.66 \text{ KN.m/Rib}}}$$

Positive moment for b-direction = + 19.66 kN.m/Rib.

$$m = \frac{fy}{0.85 * fc'} = \frac{420}{0.85 * 24} = 20.6$$

$$Rn = \frac{Mn}{b * d^2} = \frac{(19.66 / 0.9) * 10^6}{540 * (324)^2} = 0.385 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mRn}{fy}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(0.385)(20.6)}{420}} \right) = 0.000926$$

$$As = \rho b d = 0.000926 * 540 * 324 = 162.1 \text{ mm}^2$$

$$As_{\min} = \frac{\sqrt{fc'}}{4(fy)} (bw)(d) \geq \frac{1.4}{fy} (bw)(d)$$

$$As_{\min} = \frac{\sqrt{24}}{4(420)} (120)(324) \geq \frac{1.4}{420} (120)(324)$$

$$As_{\min} = 113.4 < 129.6$$

$$As_{\min} = 129.6 \text{ mm}^2 \text{ -control}$$

$$As_{\min} = 129.6 < As = 172.3 \text{ mm}^2$$

Select (2) bars $\Phi 12$ with area = 226 mm²

❖ Check for strain:

$$a = \frac{As * fy}{0.85 * fc' * b} = \frac{226 * 420}{0.85 * 24 * 540} = 8.61 \text{ mm}$$

$$c = \frac{8.61}{0.85} = 10.13 \text{ mm}$$

$$\epsilon_s = \frac{324 - 10.13}{10.13} \times 0.003 = 0.092 > 0.005 \dots \text{ok}$$

The negative moment for b-direction at the discontinuous edge = $(1/3) * 19.66 = -6.55 \text{KN.m/Rib}$

Assume $\Phi 12$

$$d = 360 - 20 - 10 - 12/2 = 324 \text{ mm}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$R_n = \frac{M_n}{b * d^2} = \frac{(6.55 / 0.9) * 10^6}{140 * (324)^2} = 0.495 \text{ Mpa}$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right)$$

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(0.495)(20.6)}{420}} \right) = 0.001194$$

$$A_s = \rho b d = 0.001194 * 140 * 324 = 54.17 \text{ mm}^2$$

$$A_{s_{\min}} = \frac{\sqrt{f_c'}}{4(f_y)} (b_w)(d) \geq \frac{1.4}{f_y} (b_w)(d)$$

$$A_{s_{\min}} = \frac{\sqrt{24}}{4(420)} (120)(324) \geq \frac{1.4}{420} (120)(324)$$

$$A_{s_{\min}} = 113.4 < 129.6$$

$$A_{s_{\min}} = 129.6 \text{ mm}^2 \text{ -control}$$

$$A_{s_{\min}} = 129.6 < A_s = 54.17 \text{ mm}^2$$

select (2) bars $\Phi 10$ with area = 157.1 mm²

❖ Check for strain:

$$a = \frac{A_s \times f_y}{0.85 \times f_c' \times b} = \frac{157.1 \times 420}{0.85 \times 24 \times 120} = 26.95 \text{ mm}$$

$$c = \frac{26.95}{0.85} = 31.71$$

$$\epsilon_s = \frac{324 - 31.71}{31.71} \times 0.003 = 0.0276 > 0.005 \dots \text{ok}$$

4.7.4 Design for shear:

The shear in the slab calculated by using tributary area for shear:

$$w_u = 1.2 \text{ DL} + 1.6 \text{ LL} = 1.2 \times 11.88 + 1.6 \times 2.5 = 18.256 \text{ KN/m}^2$$

$$V_{ud} = w_u \times b_f (\ln/2 - d)$$

$$V_{ud} = 18.256 \times 0.52 \times (9.2/2 - 0.324) = 40.6 \text{ KN}$$

$$V_c = \frac{1.1}{6} \sqrt{f_c'} \times b_w \times d$$

$$V_c = \frac{1.1}{6} \sqrt{24} \times 140 \times 324 \times 10^{-3} = 40.74 \text{ KN}$$

$$\phi V_c = 0.75 \times 40.74 = 30.55 \text{ KN}$$

$$\phi V_c = 30.55 < V_{ud} = 40.6$$

$$V_{s \text{ min}} = \frac{1}{3} b_w \times d \geq \frac{1}{16} \times \sqrt{f_c'} \times b_w \times d$$

$$V_{s \text{ min}} = \frac{1}{3} \times 140 \times 324 \times 10^{-3} = 15.11 \text{ KN}$$

$$\geq \frac{1}{16} \times \sqrt{24} \times 140 \times 324 \times 10^{-3} = 10.03$$

$$\phi V_c = 30.55 < V_u = 40.6 \leq \phi (V_c + V_{s \text{ min}}) = 45.65$$

Provide minimum shear reinforcement... use 2 Φ 8 for stirrups $A_v = 2 \times 50 = 100 \text{ mm}^2$

$$\frac{A_{v(\min)}}{s} = \frac{b_w}{3F_y t} = \frac{1}{3} * \frac{140}{420} = 0.1111$$

$$\frac{100}{s} = 0.1111, \dots, s = 900 \text{ mm}$$

$$s \leq d/2 = 324/2 = 162 \text{ mm -control}$$

$$\leq 600 \text{ mm.}$$

Use 2Φ 8 @ 150 mm.

4.8 : Design Flat plate.

We use SAFE Program to design the flat plate in the project. The plan of ground floor was divided into 6 parts by expansion joints three of them was flat plate .

We will show you one of them as sample of solution , the first flat plate entered into the SAFE Program and this is the procedure .

4.8.1 Drawing the Plan as DXF :

We draw the plan on the AUTOCAD as DXF format . After entering the drawing into SAFE Program , we define all materials (slab , column , beams , shear wall ,.....) , members and combinations , and redraw it there .

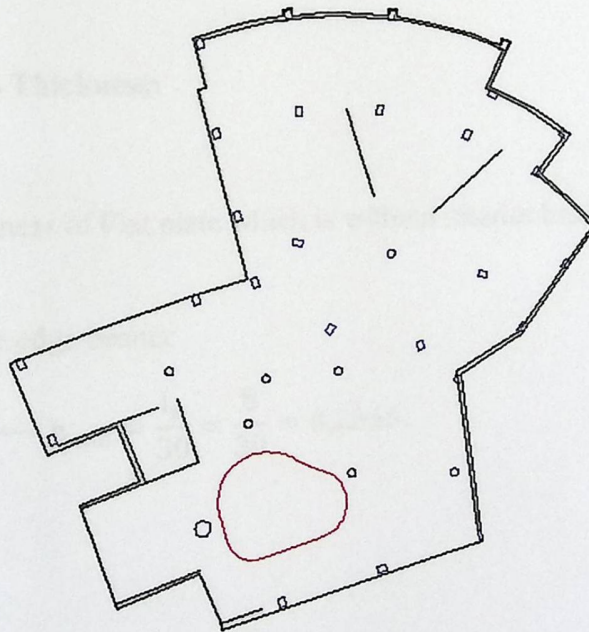


Fig (4-16):plan on AutoCAD as DXF

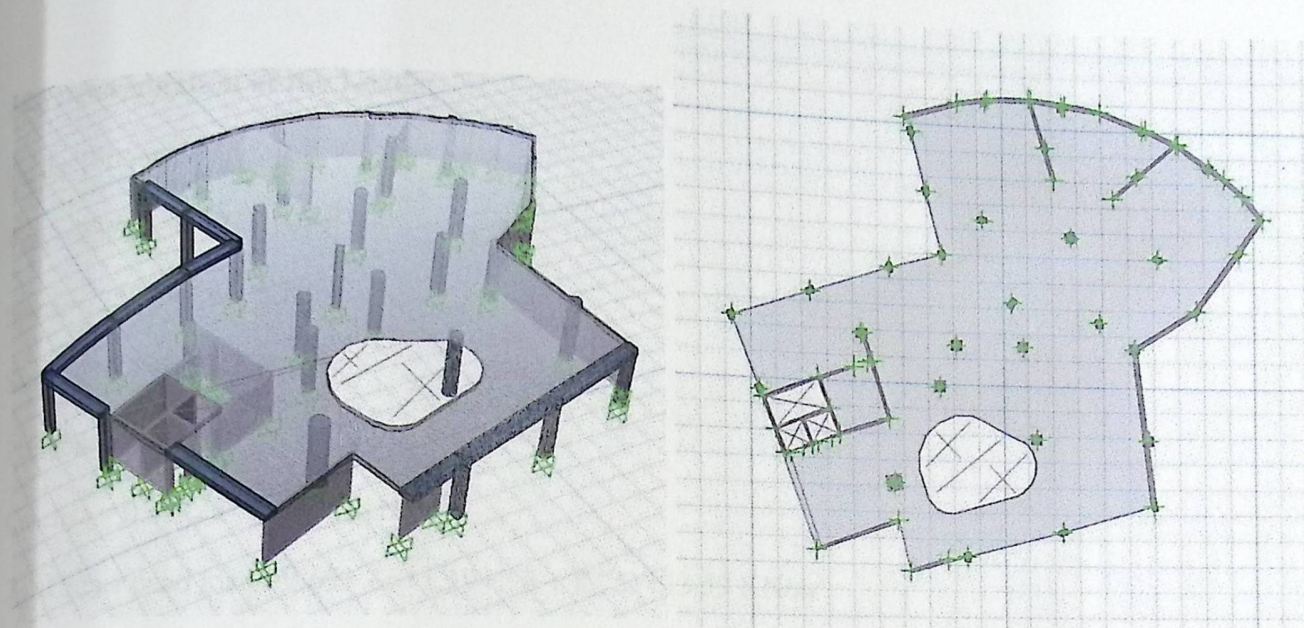


Fig (4-17):plan on SAFE Program

4.8.2 Determination of Slab Thickness:

We can determine min. thickness of Flat plate which is without interior beams and drop panel (with $f_y = 420 \text{ Mpa}$), as following :

1. for Exterior panels without edge beams:

$$\text{span length (L = 8.0m)} \rightarrow \rightarrow \rightarrow h_{\min} = \frac{l_n}{30} = \frac{8}{30} = 0.2666$$

2. for Interior panels:

$$\text{span length (L = 8.0m)} \rightarrow \rightarrow \rightarrow h_{\min} = \frac{l_n}{33} = \frac{8}{33} = 0.2424$$

Use slab thickness = 30 cm .

4.8.3 Determination of the Loads:

For the 1st floor :

$$\text{Tiles} = 23 * 0.03 = 0.69 \text{ KN/m}^2$$

$$\text{Mortar} = 22 * 0.3 = 0.66 \text{ KN/m}^2$$

$$\text{Sand} = 17 * 0.7 = 1.19 \text{ KN/m}^2$$

$$\text{Partition} = 2.3 \text{ KN/m}^2$$

$$\text{Concrete} = 0.35 * 25 * 1 = 8.75 \text{ KN/m}^2$$

$$\text{Total Super dead load} = 0.69 + 0.66 + 1.19 + 2.3 + 8.75 = 13.59 \text{ KN/m}^2$$

$$\text{Live load} = 5 \text{ KN/m}^2$$

Then , we enter these values into the SAFE drawing and apply the self weight as service loads .

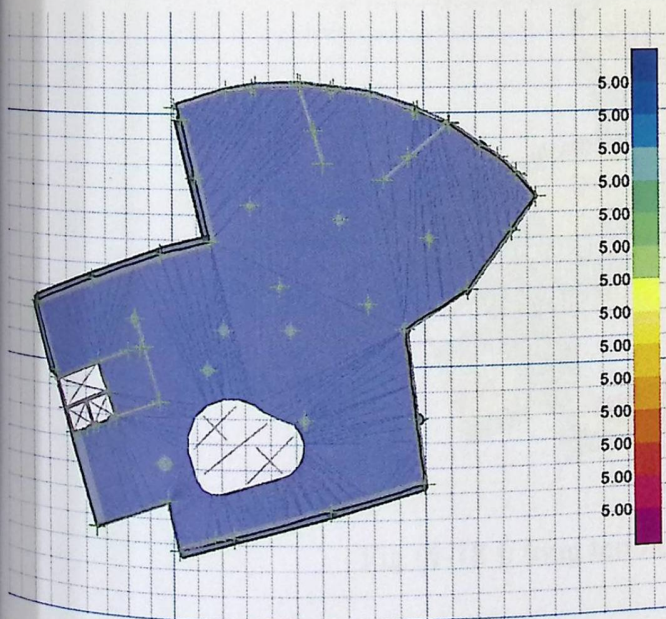


Fig (4-18): dead load

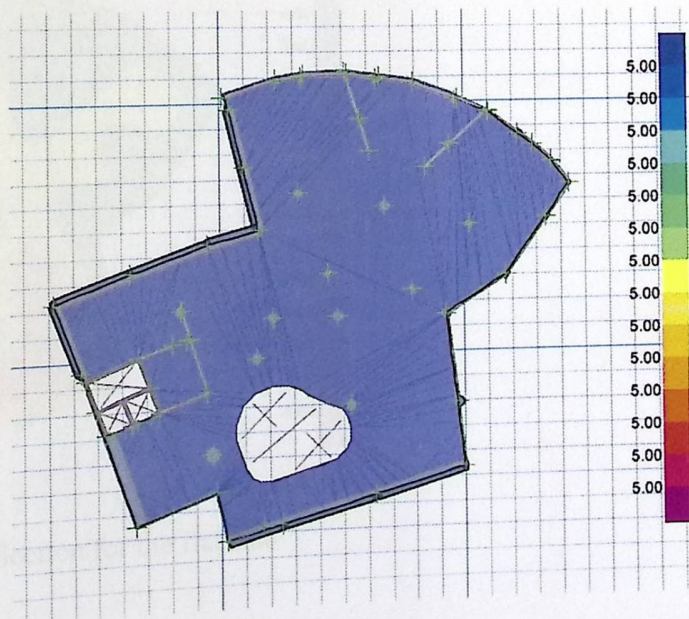


Fig (4-19):live load

4.8.4 Check the Allowable deflection:

After finishing the whole procedure (initial dead load deflection, initial dead + live load deflection, long term deflection, and the load compensation required), when we running the program take the following result.

we should compare the deflection with the maximum one.

$$\text{max. def. } (\Delta) = \frac{L}{360}$$

$$\text{for max. span } L = 8.0\text{m} \rightarrow \rightarrow \rightarrow \text{max. } \Delta = \frac{8.0}{360} = 22.22 \text{ mm}$$

$$\Delta = -8.0\text{mm} < \text{max. } \Delta = -27.8$$

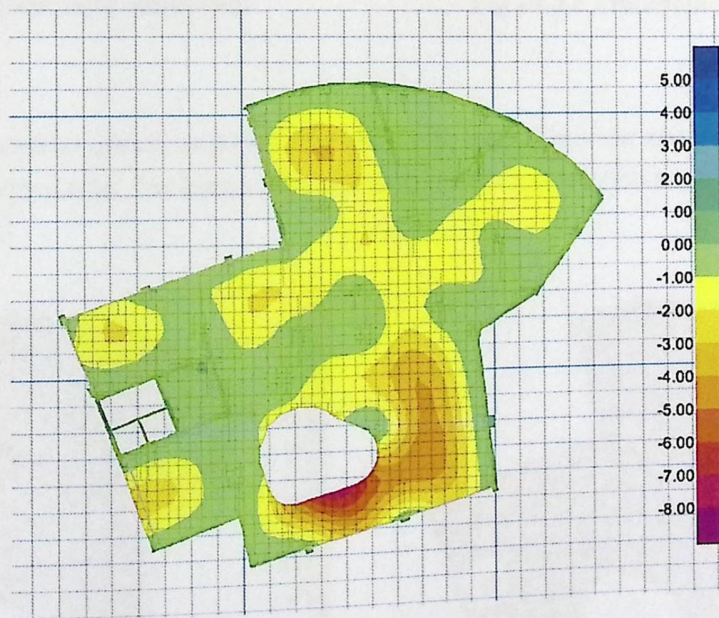
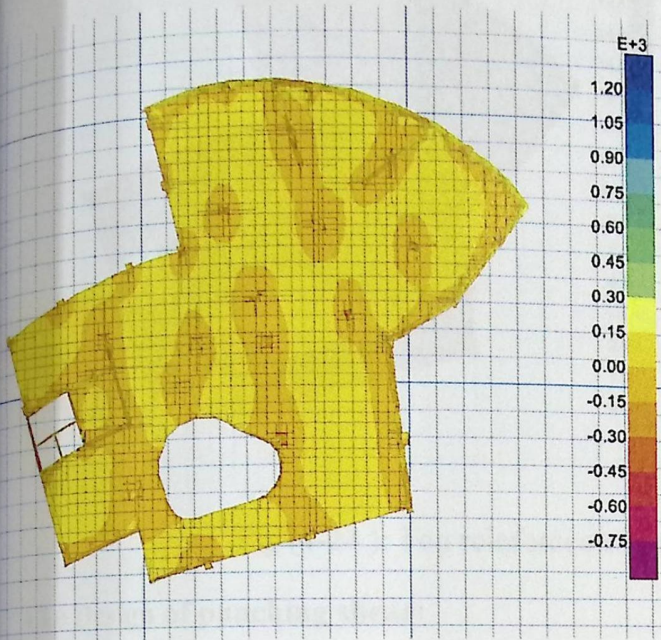


Fig (4-18): long term deflection for the flat slab

4.8.5 Design of Moment:



Fig(4-21): moment in X-direction

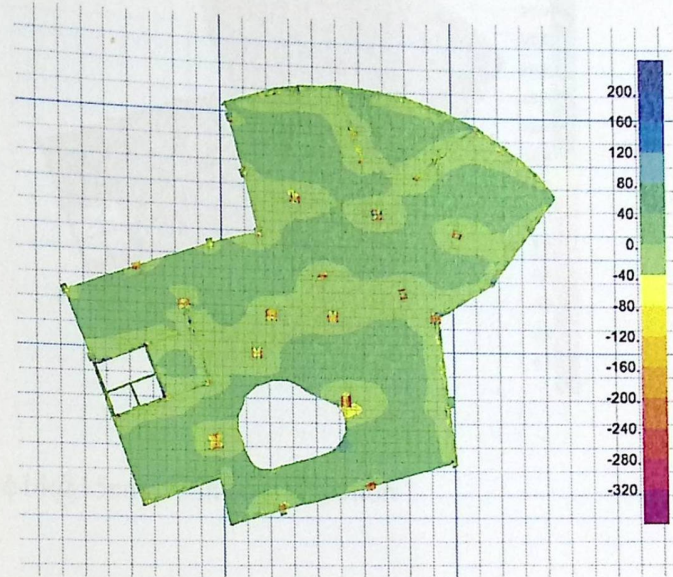


Fig (4-22):moment in Y-direction

Therefore , the Bottom reinforcement is $\phi 12@150$ cm c/c in both direction and the Top reinforcement is $\phi 14@15$ cm c/c in both direction . as Basic Meshes with additional reinforcement in some areas as shown in the next figures .

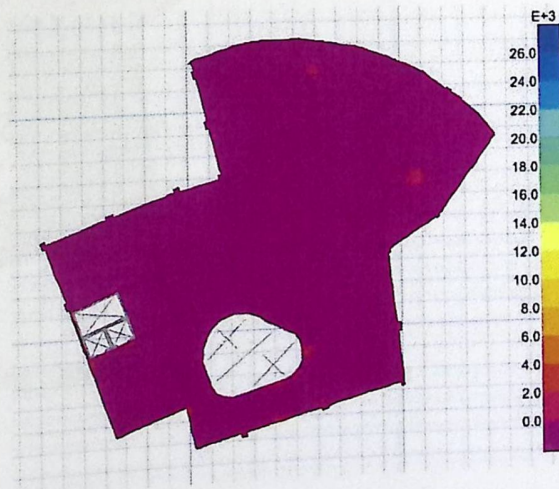
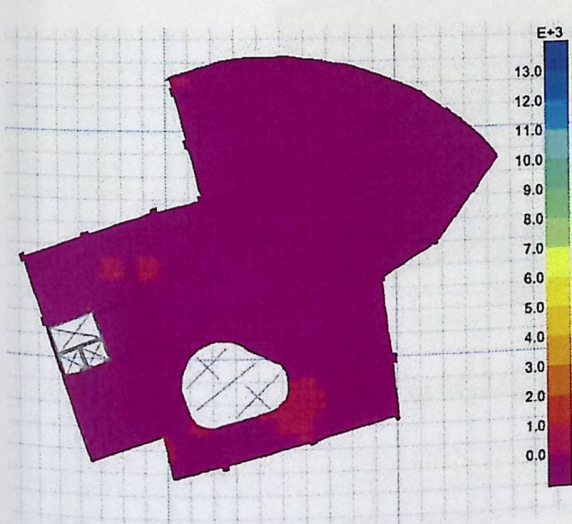


Fig (4-23):Bottom reinforcement of $\phi 12@20$ cm c/c in both direction

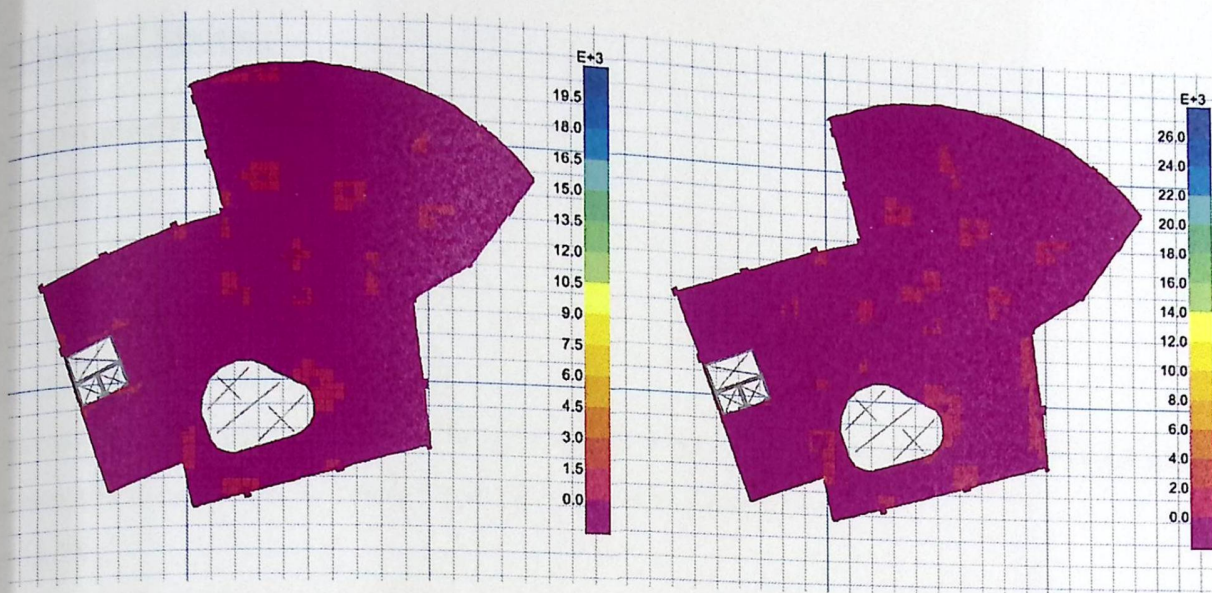


Fig (4-24): Top reinforcement of $\phi 14@15\text{cm c/c}$ in both direction

4.8.6 Design of punching shear:

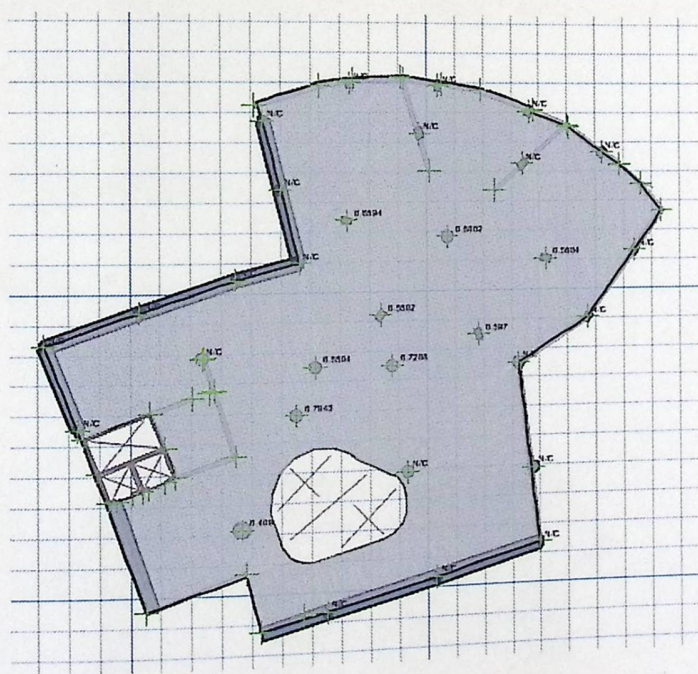


Fig (4-25): punching shear of slab

There is NO problem in punching shear.

4.9 Design of Stairs :

4.9.1 Determination of Slab Thickness:

For Flight:

$$L = (5.353) = 7.4 \text{ m.}$$

$$h_{req} = L/20.$$

$$h_{req} = 7.4 / 20 = 0.3665 \text{ m} = 36.65 \text{ cm.}$$

In the case presented here, where the slab end is cast with the supporting beams and additional negative reinforcement is provided, minimum thickness can be assumed to be:

$$h_{req} = L/28.$$

$$h_{req} = (7.40/28) = 0.264 \text{ m.} = 26.4 \text{ cm}$$

Take $h = 30 \text{ cm.}$

❖ Minimum slab thickness for deflection is (for a simply supported one-way solid slab).

$$\text{Min } h = (L/20) = \frac{5.353}{20} = 0.2677 \text{ m} = 26.77 \text{ cm.}$$

$$\text{And } h = (L/28) = \frac{5.353}{28} = 0.1912 \text{ m} = 19.12 \text{ cm.}$$

Take $\text{Min } h = 250 \text{ mm.}$

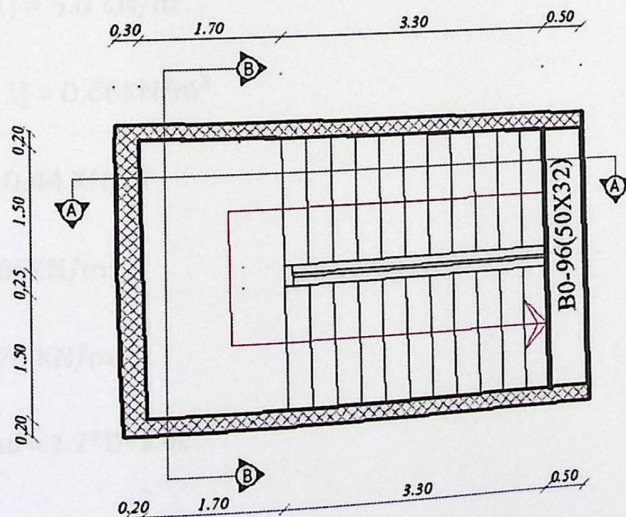


Fig (4-26): stairs details

4.9.2 Load Determination:**Flight dead load computation:**

$$\alpha = \tan^{-1}\left(\frac{\text{rise}}{\text{run}}\right) = \tan^{-1}\left(\frac{166.7}{300}\right) = 29.059.$$

$$\text{Concrete} = \frac{(20 \times 0.25 \times 1)}{\cos 29.059} = 5.72 \text{ KN/m.}$$

$$\text{Plastering} = \frac{(22 \times 0.03 \times 1)}{\cos 29.059} = 0.755 \text{ KN/m.}$$

$$\text{Stair steps} = \left(\frac{25}{0.3}\right) * \left(\frac{(0.1667 \times 0.3)}{2}\right) = 2.0838 \text{ KN/m.}$$

$$\text{Mortar} = 22 * \left(\frac{(0.1667 + 0.3)}{0.3}\right) * 0.02 * 1 = 0.6845 \text{ KN/m.}$$

$$\text{Tiles} = 27 * \left(\frac{(0.1667 + 0.35)}{0.3}\right) * 0.03 * 1 = 1.3951 \text{ KN/m}$$

$$\text{Total Dead Load} = 10.61 \text{ KN/m}$$

Landing Dead load computation:

$$\text{Concrete} = (20 \times 0.25 \times 1) = 5.0 \text{ KN/m}$$

$$\text{Plastering} = (0.03 \times 22 \times 1) = 0.66 \text{ KN/m}^2$$

$$\text{Mortar} = 0.02 \times 22 \times 1 = 0.44 \text{ KN/m}$$

$$\text{Tiles} = 0.03 \times 22 \times 1 = 0.66 \text{ KN/m}^2$$

$$\text{Total Dead Load} = 6.76 \text{ KN/m}$$

$$\text{Factored Total Dead Load} = 1.2 \times D + 1.6L$$

$$\text{Live load} = 5 \text{ KN/m.}$$

$$\text{For flight: } w = 1.2 \times 10.61 + 1.6 \times 5 \times 1 = 20.732 \text{ kN/m}$$

$$\text{For landing: } w = 1.2 \times 6.76 + 1.6 \times 5 \times 1 = 16.112 \text{ kN/m}$$

4.9.2 Load Determination:**Flight dead load computation:**

$$\alpha = \tan^{-1}\left(\frac{\text{rise}}{\text{run}}\right) = \tan^{-1}\left(\frac{166.7}{300}\right) = 29.059.$$

$$\text{Concrete} = \frac{(20 \times 0.25 \times 1)}{\cos 29.059} = 5.72 \text{ KN/m.}$$

$$\text{Plastering} = \frac{(22 \times 0.03 \times 1)}{\cos 29.059} = 0.755 \text{ KN/m.}$$

$$\text{Stair steps} = \left(\frac{25}{0.3}\right) * \left(\frac{(0.1667 \times 0.3)}{2}\right) = 2.0838 \text{ KN/m.}$$

$$\text{Mortar} = 22 * \left(\frac{(0.1667 + 0.3)}{0.3}\right) * 0.02 * 1 = 0.6845 \text{ KN/m.}$$

$$\text{Tiles} = 27 * \left(\frac{(0.1667 + 0.35)}{0.3}\right) * 0.03 * 1 = 1.3951 \text{ KN/m}$$

$$\text{Total Dead Load} = 10.61 \text{ KN/m}$$

Landing Dead load computation:

$$\text{Concrete} = (20 \times 0.25 \times 1) = 5.0 \text{ KN/m}$$

$$\text{Plastering} = (0.03 \times 22 \times 1) = 0.66 \text{ KN/m}^2$$

$$\text{Mortar} = 0.02 \times 22 \times 1 = 0.44 \text{ KN/m}$$

$$\text{Tiles} = 0.03 \times 22 \times 1 = 0.66 \text{ KN/m}^2$$

$$\text{Total Dead Load} = 6.76 \text{ KN/m}$$

$$\text{Factored Total Dead Load} = 1.2 \times D + 1.6L$$

$$\text{Live load} = 5 \text{ KN/m.}$$

$$\text{For flight: } w = 1.2 \times 10.61 + 1.6 \times 5 \times 1 = 20.732 \text{ kN/m}$$

$$\text{For landing: } w = 1.2 \times 6.76 + 1.6 \times 5 \times 1 = 16.112 \text{ kN/m}$$

4.9.3 Design of flight :

Total dead load for flight = $1.2 \times 10.61 = 12.73 \text{ KN/m}$.

Total dead load for landing = $1.2 \times 6.76 \times 0.5 = 4.056 \text{ KN/m}$.

By using ATIR program use $b = 100 \text{ cm}$.

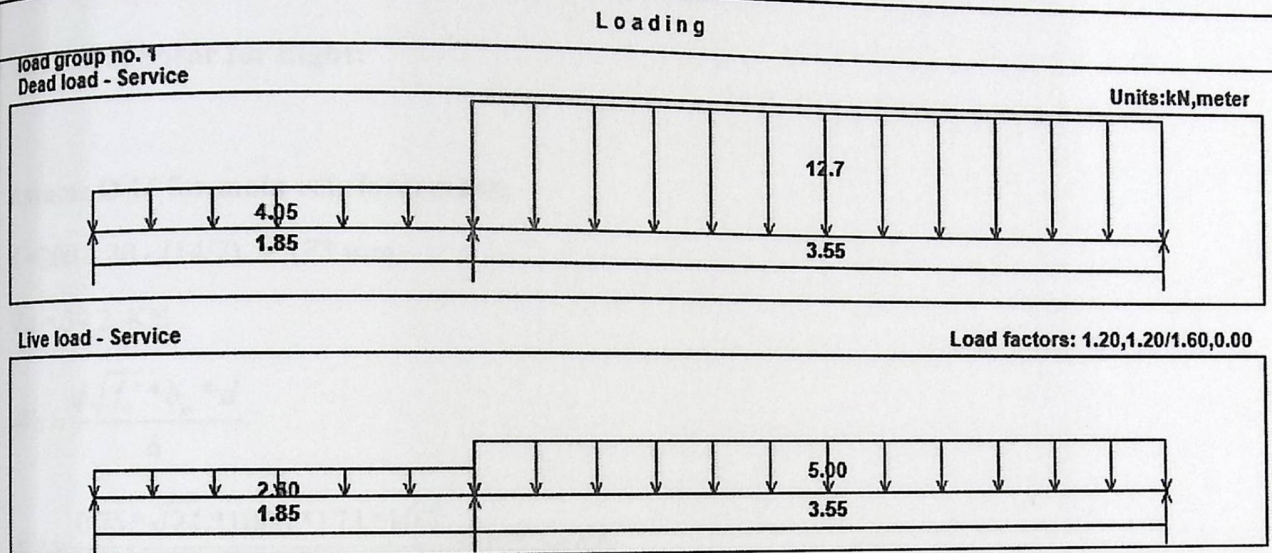


Fig (4-27): loads on stairs

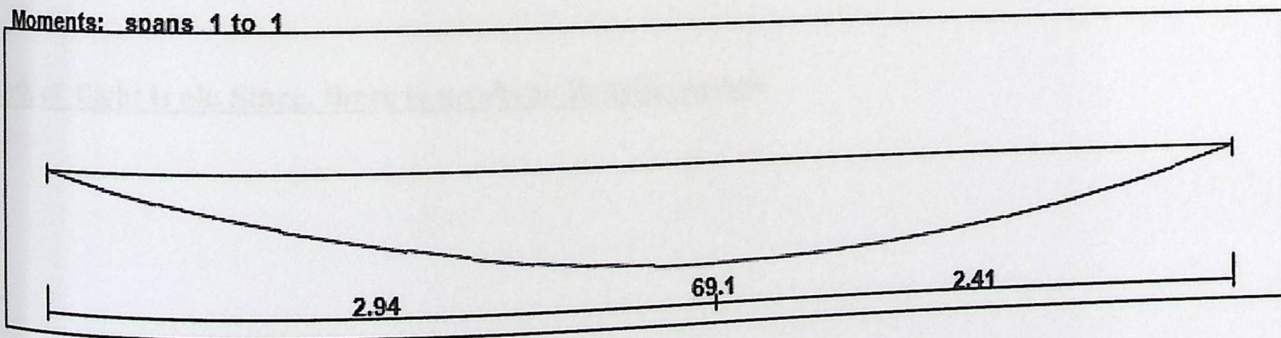


Fig (4-28): Moment on stairs

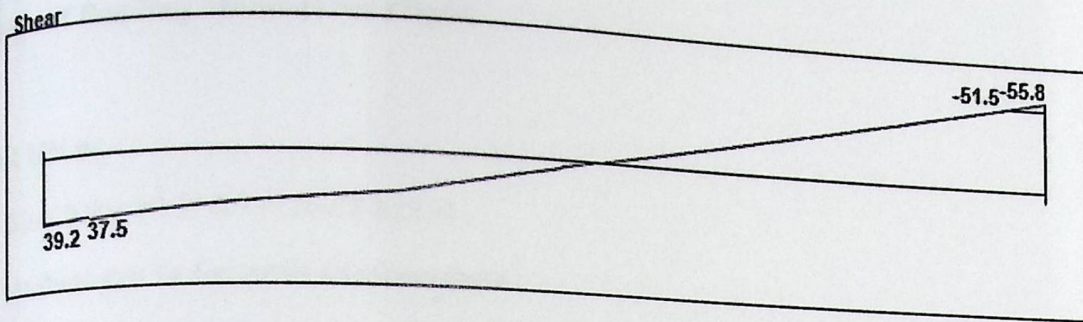


Fig (4-29): Shear on stairs

4.9.4 Design of Shear for flight:

- Assume $\emptyset 14$ for main reinforcement:

$$\text{So, } d = 200 - 20 - (14/2) = 173 \text{ mm..}$$

$$V_u = 39.2 \text{ KN.}$$

$$\phi V_c = \frac{\phi \sqrt{f_c'} * b_w * d}{6}$$

$$\phi V_c = \frac{0.75 * \sqrt{24} * 1000 * 173 * 10^{-3}}{6} = 105.94 \text{ KN}$$

$$V_u = 39.2 \text{ KN} < \phi V_c = 105.94 \text{ KN.}$$

Depth of flight is ok. Since, there is no shear Reinforcement.

4.9.5 Design of Bending Moment for Flight:

$$M_u = 69.1 \text{ KN.m.}$$

$$M_{req} = M_u / 0.9 = 69.1 / 0.9 = 76.78 \text{ KN.m.}$$

Assume bar diameter 14 for main reinforcement.

$$d = 200 - 20 - (14/2) = 173 \text{ mm.}$$

$$R_n = \frac{M_n}{b \cdot d^2}$$

$$R_n = \frac{76.78 \cdot 10^6}{1000 \cdot (173)^2} = 2.56 \text{ MPa.}$$

$$m = \frac{f_y}{0.85 \times f_c'}$$

$$m = \frac{420}{0.85 \times 24} = 20.588$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right) = \frac{1}{20.588} \left(1 - \sqrt{1 - \frac{2 \cdot 20.588 \cdot 2.56}{420}} \right) = 0.00653$$

$$A_{s \text{ req}} = 0.00653 \times 1000 \times 173 = 1130.52 \text{ mm}^2 \dots \dots \dots \text{control}$$

$$A_{s \text{ min}} = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2$$

$$S = 1130.5 / 113.1 = 0.10 \text{ m}$$

Use 11 Φ 12 /m OR use Φ 12 @ 10 cm.

❖ **Check for spacing:**

$$3h = 3 \times 250 = 750 \text{ mm.}$$

$$S = 450.$$

$$s = 380 \left(\frac{280}{0.667 \cdot 420} \right) - 2.5 \cdot 20 = 330 \text{ mm}$$

$$s = 300 \left(\frac{280}{0.667 \times 420} \right) = 300 \text{mm} \dots \text{control.}$$

❖ **Secondary Reinforcement:**

For shrinkage & Temperature A_s provide equal :

$$A_{s \text{ min}} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2$$

Use 8 $\Phi 8$ /m or 1 $\Phi 8$ @ 12.50 cm.

❖ **Check for strain:**

Tension = Compression

$$A_s \cdot f_y = 0.85 \cdot f_c' \cdot b \cdot a$$

$$1130.5 \cdot 420 = 0.85 \cdot 24 \cdot 1000 \cdot a$$

$$a = 23.275 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{23.275}{0.85} = 27.38 \text{ mm}$$

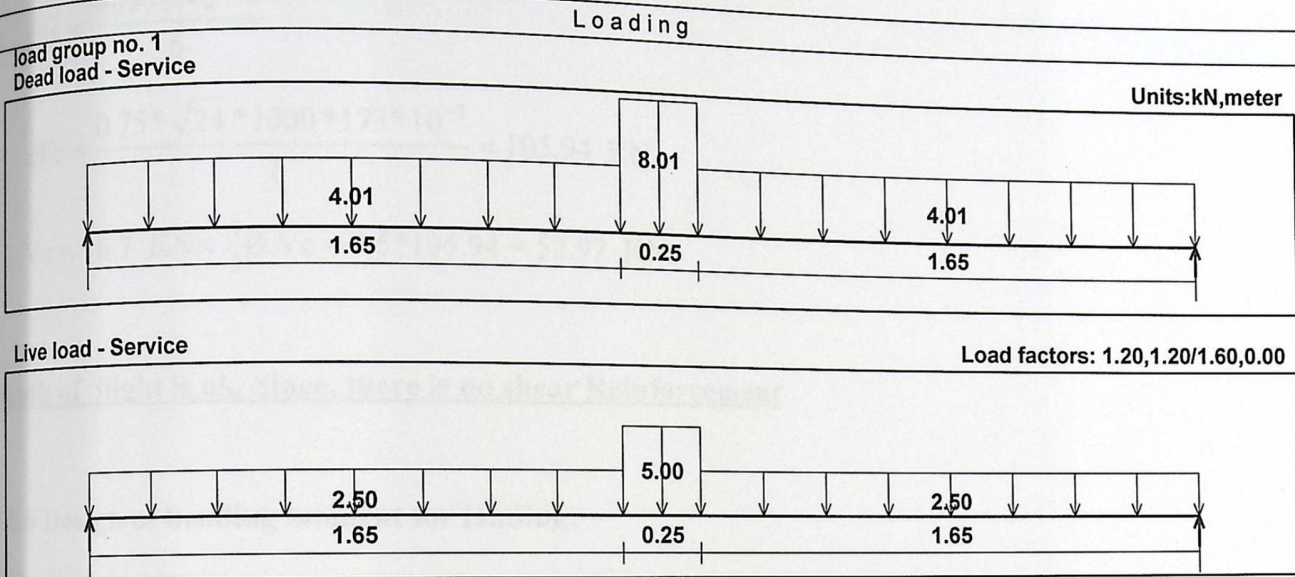
$$\epsilon_s = \frac{173 - 27.38}{27.38} \cdot 0.003$$

$$\epsilon_s = 0.0159 > 0.005 \longrightarrow \text{ok}$$

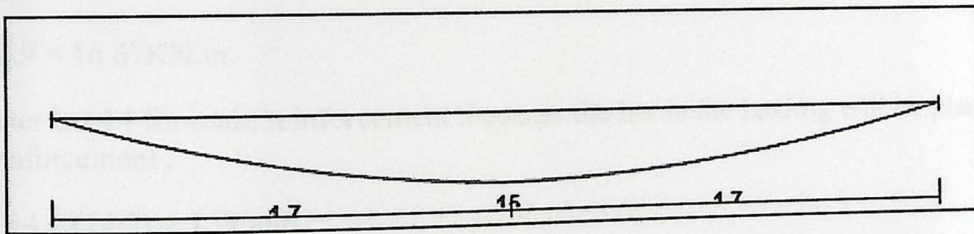
4.9.6 Design of landing:

Same thickness = 20 cm.

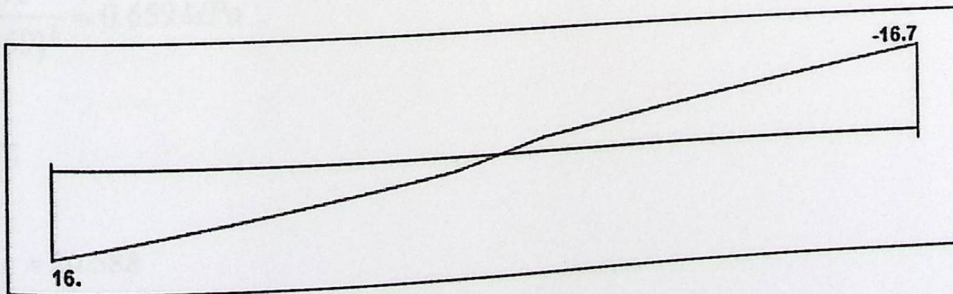
Total Dead Load For Flight = $1.2 \times 8.01 + 1.6 \times 5 = 17.612 \text{ KN/m}$.



Fig(4-30) Loads on Landing.



Fig(4-31) Moment Diagram for landing



Fig(4-32) Shear Diagram for landing.

4.9.7 Design of Shear for Landing:

$$V_u = 16.7 \text{ KN.}$$

$$\phi V_c = \frac{\phi \sqrt{f_c'} * b_w * d}{6}$$

$$\phi V_c = \frac{0.75 * \sqrt{24} * 1000 * 173 * 10^{-3}}{6} = 105.94 \text{ KN}$$

$$V_u = 16.7 \text{ KN} < \frac{1}{2} \phi V_c = 0.5 * 105.94 = 52.97 \text{ KN.}$$

Depth of flight is ok. Since, there is no shear Reinforcement

4.9.8 Design of bending moment for landing:

$$M_u = 15 \text{ KN.m.}$$

$$M_{nreq} = 15 / 0.9 = 16.67 \text{ KN.m.}$$

Assume diameter bar 14 for main reinforcement .because the bar in the landing will be placed on the top of the main reinforcement .

$$d = 200 - 20 - 14 - (14/2) = 159 \text{ mm.}$$

$$R_n = \frac{M_n}{b \cdot d^2}$$

$$R_n = \frac{16.67 * 10^6}{1000 * (159)^2} = 0.659 \text{ MPa .}$$

$$m = \frac{f_y}{0.85 * f_c'}$$

$$m = \frac{420}{0.85 * 24} = 20.588$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right) = \frac{1}{20.588} \left(1 - \sqrt{1 - \frac{2 * 20.588 * 0.659}{420}} \right) = 0.001595$$

$$A_{s \text{ req}} = 0.001595 \times 1000 \times 159 = 253.64 \text{ mm}^2$$

$$A_{s \text{ min}} = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2 \dots \text{ control}$$

$$A_s < A_{s \text{ min}} \dots$$

Use $6\Phi 10 / \text{m}$ or $1\Phi 10 @ 15 \text{ cm}$.

❖ Check for spacing:

$$3h = 3 \times 250 = 750 \text{ mm}$$

$$s = 450$$

$$s = 380 \left(\frac{280}{0.667 \times 420} \right) - 2.5 \times 20 = 330 \text{ mm}$$

$$s = 300 \left(\frac{280}{0.667 \times 420} \right) = 300 \text{ mm} \dots \dots \text{ control .}$$

Use $1\Phi 10 @ 15 \text{ cm}$.

❖ Secondary Reinforcement :

For shrinkage & Temperature A_s provide equal :

$$A_{s \text{ min}} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2$$

Use $4\Phi 14 / \text{m}$ or $1\Phi 8 @ 12.5 \text{ cm}$.

❖ Check for strain:

Tension = Compression

$$A_s \times f_y = 0.85 \times f_c' \times b \times a$$

$$360 \times 420 = 0.85 \times 24 \times 1000 \times a$$

$$a = 7.41 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{7.41}{0.85} = 8.72 \text{ mm}$$

$$\epsilon_s = \left(\frac{159 - 8.72}{8.72} \right) \times 0.003$$

$$\epsilon_s = 0.051 > 0.005 \longrightarrow \text{ok}$$

$$A_s \text{ req} = 0.001595 \times 1000 \times 159 = 253.64 \text{ mm}^2$$

$$A_s \text{ min} = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2 \dots \text{ control}$$

$$A_s < A_s \text{ min} ..$$

Use 6Φ10 /m or 1 Φ 10 @ 15 cm.

❖ Check for spacing:

$$3h = 3 \times 250 = 750 \text{ mm}$$

$$S = 450$$

$$s = 380 \left(\frac{280}{0.667 \times 420} \right) - 2.5 \times 20 = 330 \text{ mm}$$

$$s = 300 \left(\frac{280}{0.667 \times 420} \right) = 300 \text{ mm} \dots \dots \text{ control .}$$

Use 1Φ 10 @ 15 cm.

❖ Secondary Reinforcement :

For shrinkage & Temperature A_s provide equal :

$$A_s \text{ min} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2$$

Use 4Φ14 /m or 1Φ 8 @ 12.5 cm.

❖ Check for strain:

Tension = Compression

$$A_s \times f_y = 0.85 \times f_c' \times b \times a$$

$$360 \times 420 = 0.85 \times 24 \times 1000 \times a$$

$$a = 7.411 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{7.411}{0.85} = 8.72 \text{ mm}$$

$$\epsilon_s = \left(\frac{159 - 8.72}{8.72} \right) \times 0.003$$

$$\epsilon_s = 0.051 > 0.005 \longrightarrow \text{ok}$$

$$A_s \text{ req} = 0.001595 \times 1000 \times 159 = 253.64 \text{ mm}^2$$

$$A_s \text{ min} = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2 \dots \text{control}$$

$$A_s < A_s \text{ min} \dots$$

Use $6\Phi 10 / \text{m}$ or $1\Phi 10 @ 15 \text{ cm}$.

❖ Check for spacing:

$$3h = 3 \times 250 = 750 \text{ mm}$$

$$S = 450$$

$$s = 380 \left(\frac{280}{0.667 \times 420} \right) - 2.5 \times 20 = 330 \text{ mm}$$

$$s = 300 \left(\frac{280}{0.667 \times 420} \right) = 300 \text{ mm} \dots \dots \text{control.}$$

Use $1\Phi 10 @ 15 \text{ cm}$.

❖ Secondary Reinforcement :

For shrinkage & Temperature A_s provide equal :

$$A_s \text{ min} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 200 = 360 \text{ mm}^2$$

Use $4\Phi 14 / \text{m}$ or $1\Phi 8 @ 12.5 \text{ cm}$.

❖ Check for strain:

Tension = Compression

$$A_s \times f_y = 0.85 \times f_c' \times b \times a$$

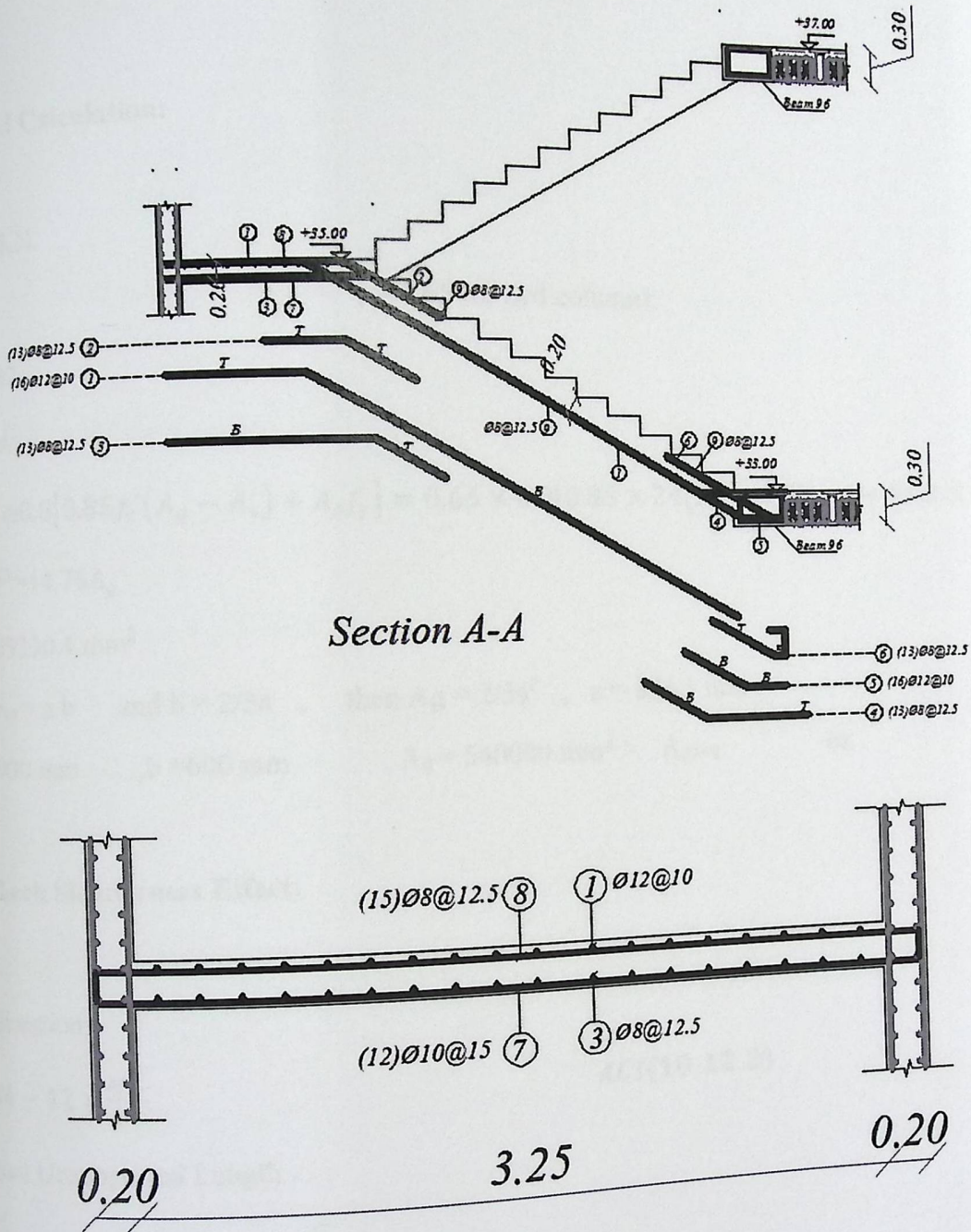
$$360 \times 420 = 0.85 \times 24 \times 1000 \times a$$

$$a = 7.411 \text{ mm}$$

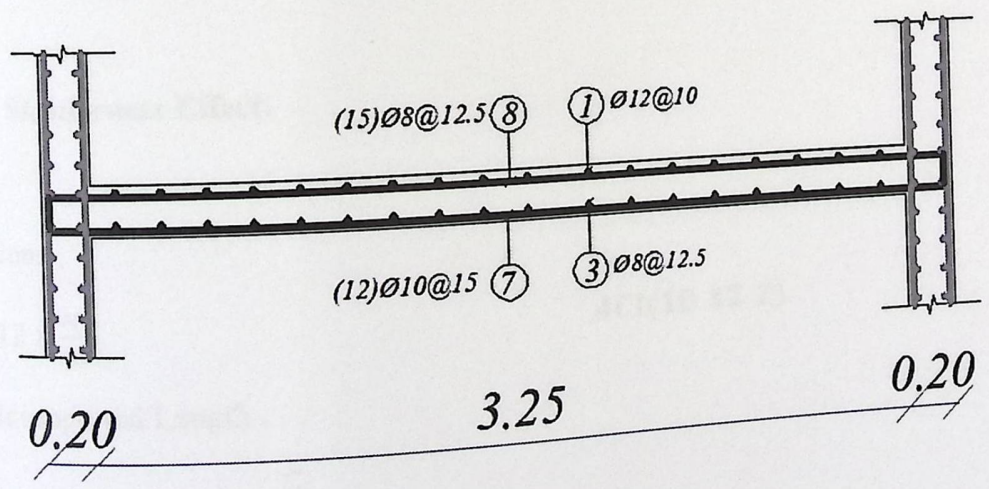
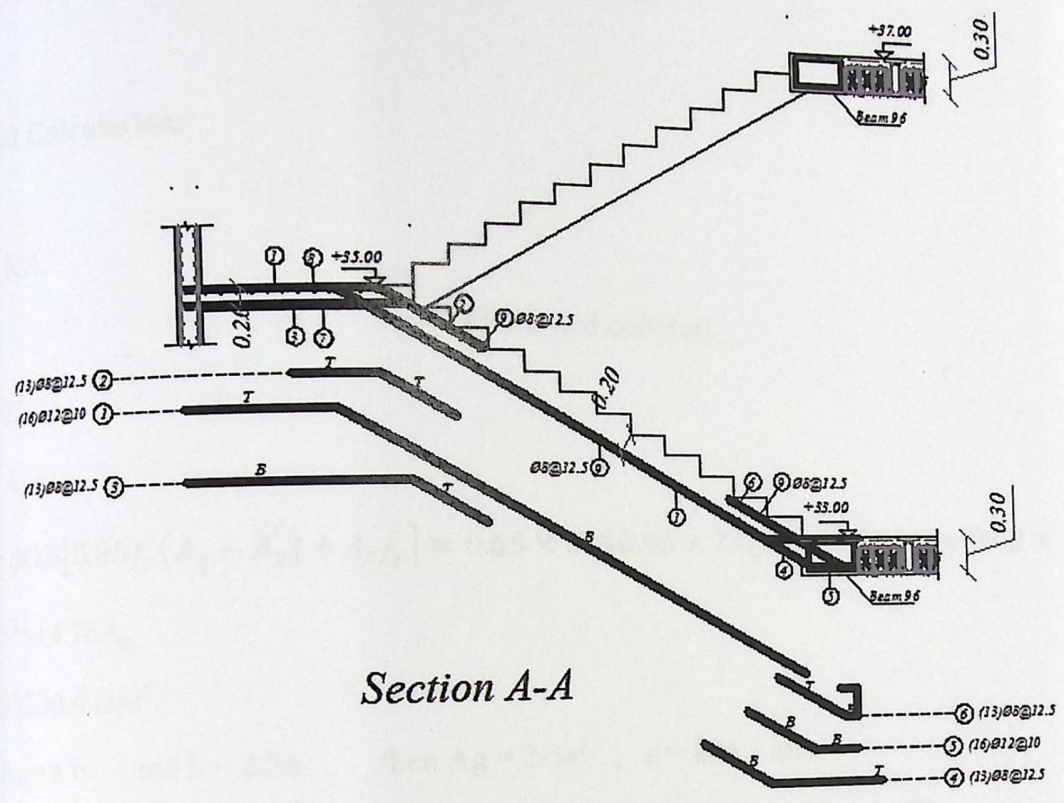
$$c = \frac{a}{\beta_1} = \frac{7.411}{0.85} = 8.72 \text{ mm}$$

$$\epsilon_s = \left(\frac{159 - 8.72}{8.72} \right) \times 0.003$$

$$\epsilon_s = 0.051 > 0.005 \longrightarrow \text{ok}$$



Fig(4-33) Sections Details for stairs.



Fig(4-33) Sections Details for stairs.

4.10 Design of Short Column:

Column (C-1-60) in the Ground Floor of the Medical College.

4.10.1 Load Calculation:

$$P_u = 7900 \text{ KN.}$$

$$\phi P_n = P_u \quad (\phi = 0.65 \text{ for tied column}).$$

$$\text{Let } \rho = 0.02$$

$$A_s = 0.02 A_g$$

$$\phi P_{n,\max} = \phi 0.8 [0.85 f'_c (A_g - A_s) + A_s f_y] = 0.65 \times 0.8 [0.85 \times 24 (A_g - 0.02 A_g) + 0.02 \times 420 A_g]$$

$$7900 \times 10^3 = 14.76 A_g$$

$$A_{g,\text{req}} = 535230.4 \text{ mm}^2.$$

$$\text{Assume } A_g = a b \quad \text{and } b = 2/3 a, \quad \text{then } A_g = 2/3 a^2, \quad a = 896.1 \text{ mm}$$

$$\text{Take } a = 900 \text{ mm}, \quad b = 600 \text{ mm}, \quad A_g = 540000 \text{ mm}^2 > A_{g,\text{req}} \quad \text{ok.}$$

4.10.2 Check Slenderness Effect:

In both direction:

$$\frac{K l_u}{r} \leq 34 - 12 \times \frac{M_1}{M_2} \quad \text{ACI(10.12.2)}$$

L_u : Actual Unsupported Length .

$$r: \text{ Radius Of Gyration} = 0.3 \times h = \sqrt{\frac{I}{A}}$$

$K = 1.0$, According to ACI 318-08 (10.12.2) the effective length factor (K) shall be permitted to be taken as 1.0

$$L_u = 3.0 - 0.3 = 2.7 \text{ m.}$$

$$\frac{M_1}{M_2} = 1.0$$

For the width $b=0.9$ m

$$r = 0.3 \times 0.9 = 0.27$$

$$\frac{1.0 \times 2.7}{0.27} \leq 34 - 12 \times 1.0$$

$10 < 22$, short column in first direction direction.

For the width $b=0.6$ m

$$r = 0.3 \times 0.6 = 0.18$$

$$\frac{1.0 \times 2.7}{0.18} \leq 34 - 12 \times 1.0$$

$15 < 22$, short column in both directions.

4.10.3 Select the longitudinal bars:

$$\phi P_{n,max} = 0.65 \times 0.8 [0.85 \times 24(540000 - A_s) + 420A_s]$$

$$A_{s,req} = 10451.22 \text{ mm}^2 \quad , \text{ try } \phi 25 \text{ with } A_s = 490.9 \text{ mm}^2$$

Use 22 $\phi 25$ with $A_s = 10800 \text{ mm}^2 > A_{s,req}$ ok

$$\rho = \frac{A_s}{A_g} = \frac{10800}{540000} = 0.02$$

4.10.4 Design for ties:

use ties $\phi 10$ with spacing shall not exceed the smallest of :

- $48 \times d_s = 48 \times 10 = 480 \text{ mm} .$
- $16 \times d_b = 16 \times 25 = 400 \text{ mm} .$
- The least dim. Of the column = 600 mm.

Cont.

Use $\phi 10 @ 250 \text{ mm} .$

4.10.5 Check for code requirements:

- clear spacing between longitudinal bars = $\frac{900 - 40 \times 2 - 10 \times 2 - 25 \times 8}{7} = 85.7 \text{ mm}$

85.71 mm > 40mm

> 1.5 d_b = 37.5 mm.

Ok

- Gross reinforcement ratio = 0.02, 0.01 < 0.0215 < 0.08 ok

- NO of bars = 28 > 4 bars for square columns.

- Min ties diameter: ϕ10 for ϕ32 longitudinal bars and smaller.

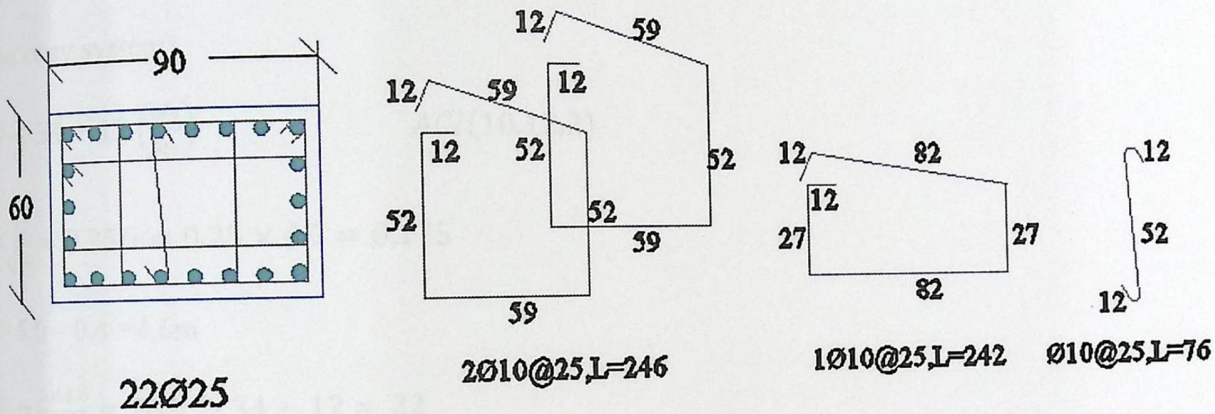


Fig (4-34): C-1-60 detail

4.11 Design of long column (C6-58):

Circular Column.

$$P_u = 5000 \text{ KN}$$

$$D_u = 3900 \text{ KN}$$

$$L_u = 1100 \text{ KN}$$

4.11.1 Check the slenderness effect:

(Non sway system)

$$\frac{kL_u}{r} < 34 - 12 \left(\frac{M_1}{M_2} \right) \quad \text{ACI(10.12.2)}$$

$$r = \sqrt{\frac{I}{A}} \approx 0.25D = 0.25 \times 0.7 = 0.175$$

$$L_u = 5.0 - 0.4 = 4.6 \text{ m}$$

$$\frac{kL_u}{r} = \frac{1 \times 4.6}{0.175} = 26.3 > 34 - 12 = 22$$

So the column is long.

4.11.2 Calculate e_{\min} , M_{\min} :

$$e_{\min} = 15 + 0.03h = 15 + 0.03 \times 700 = 36 \text{ mm.}$$

$$M_{\min} = P_u \times e_{\min} = 5000 \times 0.033 = 180 \text{ KN.m}$$

$$E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{24} = 23025.2 \text{ Mpa.}$$

$$I_g = \frac{\pi \cdot D^4}{64} = 1.18 \times 10^{10} \text{ mm}^4.$$

$$\beta_{\text{dns}} = \frac{D_u}{P_u} = \frac{3900}{5000} = 0.78 < 1.$$

$$E.I = \frac{0.4 E_c I_g}{1 + \beta_{\text{dns}}} = \frac{0.4 \times 23025.2 \times 11.8}{1.78} = 61055.6 \text{ KN.m}^2$$

4.11.3 Determine of Euler buckling load:

$$P_e = \frac{\pi^2 EI}{(Kl_u)^2} = \frac{\pi^2 \times 61055.6}{(4.6)^2} = 28478 \text{ KN}$$

4.11.4 Calculate the moment magnifier factor:

$$C_m = 0.6 + 0.4 \left(\frac{M_1}{M_2} \right) = 1$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} = \frac{1}{1 - \frac{5000}{0.75 \times 28478}} = 1.31 > 1$$

The magnified (e) and (M):

$$e = \delta_{ns} e_{min} = 1.31 \times 36 = 47.2$$

$$M = \delta_{ns} M_{min} = 1.31 \times 180 = 235.8 \text{ KN.m}$$

From the interaction diagram constructed in PCA_COLUMN program: $\rho = 0.0114\%$

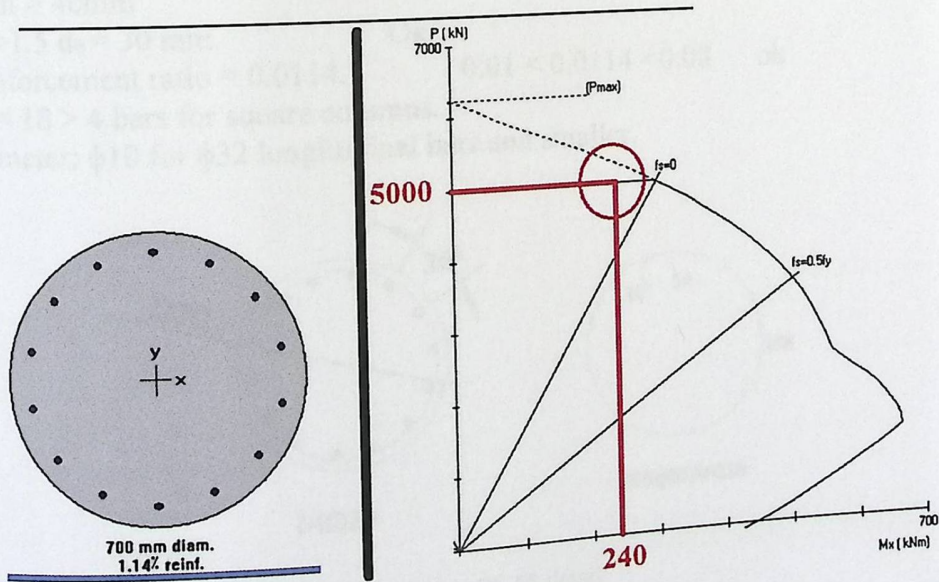


Fig (4-35): the results from PCA_COLUMN program.

4.11.5 Select the longitudinal bars:

$$A_s = \rho \times A_g = 0.0114 \times \pi \times (700)^2 / 4 = 4387.23 \text{ mm}^2$$

$$n_{\phi 20} = \frac{4387.23}{314.16} = 13.96$$

Use 14 ϕ 20

4.11.6 Design the stirrups:

The spacing of ties shall not exceed the smallest of:

- $16 \times d_b = 16 \times 20 = 320 \text{ mm}$ cont.
- $48 \times d_s = 48 \times 10 = 480 \text{ mm}$
- Least diminution of the column = 700 mm

Use $\phi 10 @ 250 \text{ mm}$.

4.11.7 Check for code requirements:

- clear spacing between longitudinal bars = $\frac{\pi \times 580 - 14 \times 20}{14} = 110.15 \text{ mm}$
 $110.15 \text{ mm} > 40 \text{ mm}$
 $> 1.5 d_b = 30 \text{ mm}$. Ok
- Gross reinforcement ratio = 0.0114, $0.01 < 0.0114 < 0.08$ ok
- NO of bars = 18 > 4 bars for square columns.
- Min ties diameter: $\phi 10$ for $\phi 32$ longitudinal bars and smaller.

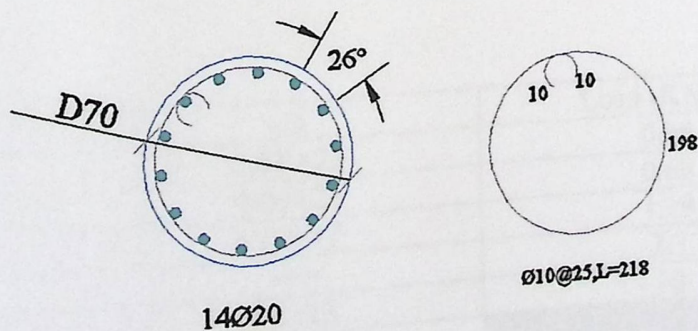


Fig (4-36): C6-58 detail

4.12 Design of punching (two way shear) at interior column in flat plate :

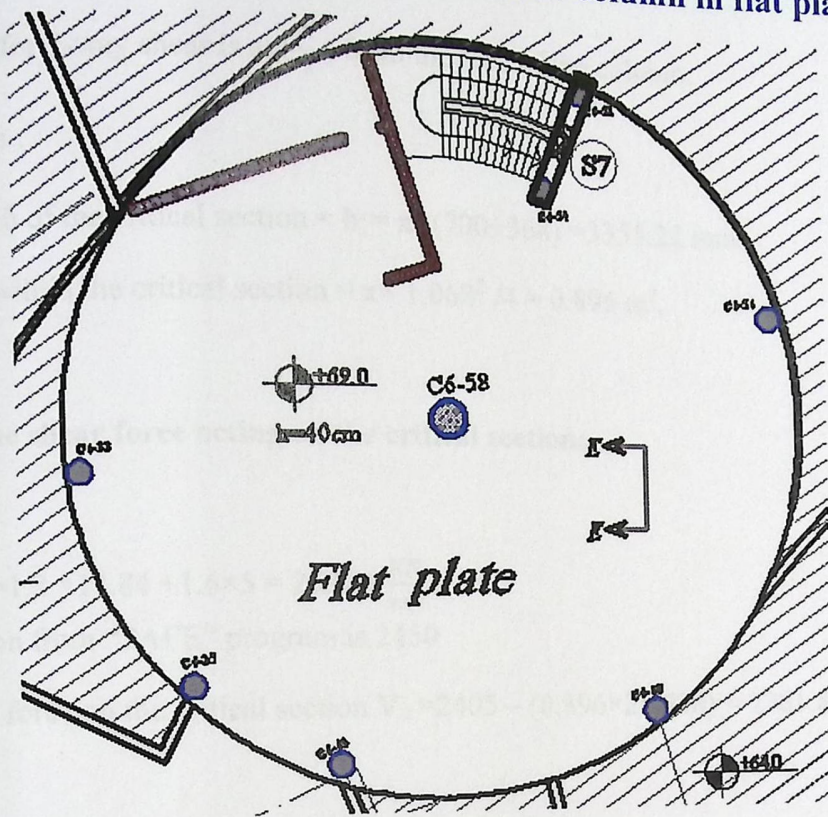


Fig (4-37) : C6-58.

Column section is 700mm circular

$h_{slab} = 400\text{mm}$.

4.12.1 Load calculations:

Dead load of the slab:

material	$\gamma \cdot \Delta$	Load (KN/m ²)
Tile	0.03×23	0.69
Mortar	0.03×22	0.66
Sand	0.07×17	1.19
Partitions	—	2.3
Self weight	0.4×25	10
Σ		14.84

Live load = $5 \frac{\text{KN}}{\text{m}^2}$

$$d_{avg} = h_{slab} - cover - d_b = 400 - 20 - 12 = 368 \text{ mm.}$$

The critical section for 2-way shear is at $(\frac{d}{2})$ from the face of the column.

$$\frac{d}{2} = \frac{368}{2} = 184 \text{ mm.}$$

The perimeter length of the critical section = $b^{\circ} = \pi \times (700 + 368) = 3355.22 \text{ mm.}$

The area enclosed within the critical section = $\pi \times 1.068^2 / 4 = 0.896 \text{ m}^2.$

4.12.2 Compute the shear force acting on the critical section:

$$W_u = 1.2 D + 1.6 L = 1.2 \times 14.84 + 1.6 \times 5 = 25.808 \frac{\text{KN}}{\text{m}^2}.$$

The column reaction from "SAFE" program is 2450

The factored shear force on the critical section $V_u = 2405 - (0.896 \times 25.808) = 2381.88 \text{ KN}$

4.12.3 Compute ϕV_c :

$$B = \frac{700}{700} = 1.$$

$\alpha = 40$, for interior column.

V_c is the smallest of :

- $\frac{1}{6} \left(1 + \frac{2}{B}\right) \sqrt{f'_c} \cdot b \cdot d = 0.5 \times \sqrt{f'_c} \cdot b \cdot d$
- $\frac{1}{12} \left(2 + \frac{\alpha \cdot d}{b}\right) \sqrt{f'_c} \cdot b \cdot d = 0.366 \times \sqrt{f'_c} \cdot b \cdot d$
- $\frac{1}{3} \sqrt{f'_c} \cdot b \cdot d = 0.333 \times \sqrt{f'_c} \cdot b \cdot d$ cont.

$$V_c = 0.3333 \sqrt{24} \times 3355.22 \times 368 / 1000 = 2016.3 \text{ KN}$$

$$\phi V_c = 0.75 \times 2016.3 = 1512.22 \text{ KN.}$$

$\phi V_c < V_u$ Shear reinforcement is required

4.12.4 Using shear stirrups as shear reinforcement:

For this $V_c = \frac{1}{6} \sqrt{f'_c} \cdot b \cdot d = \frac{1}{6} \sqrt{24} \times 3355.22 \times 368/1000 = 1008.15 \text{ KN}$

$V_s \geq V_n - V_c = \frac{V_u}{\phi} - V_c = \frac{2381.88}{0.75} - 1008.15 = 2167.69 \text{ KN}$

So $V_{s,req} = 2167.69 \text{ KN}$.

The max value of ϕV_n allowed with headed shear ties is :

$\phi V_{n,max} = \phi \frac{1}{2} \sqrt{f'_c} \cdot b \cdot d = 0.75 \times \frac{1}{2} \sqrt{24} \times 3355.22 \times \frac{368}{1000}$
 $= 2419.55 \text{ KN} > V_u = 2381.88 \text{ KN}$ ok.

To determine the properties of shear ties reinforcement the trial and error process will be used.

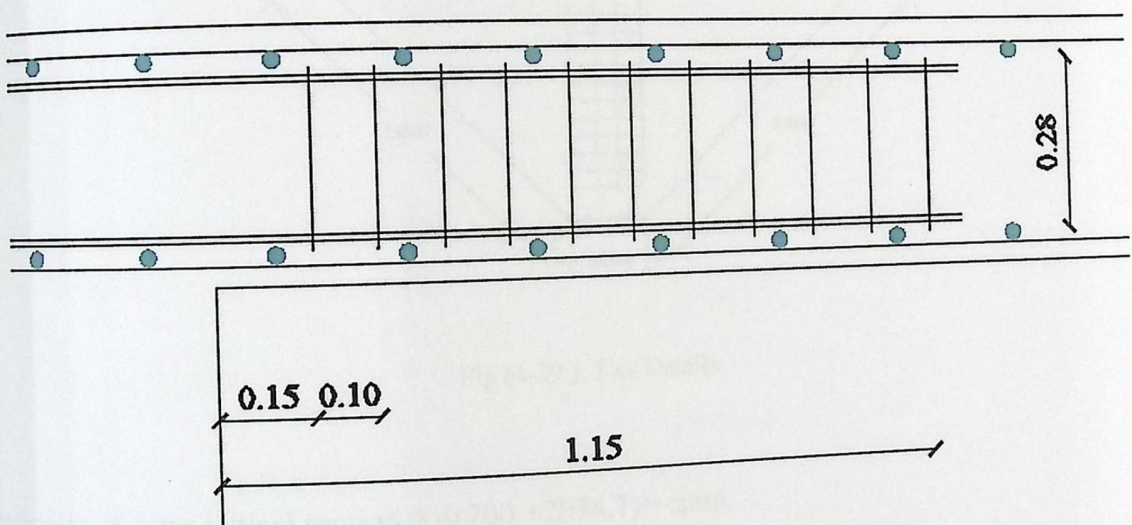


Fig (4-38) : detail of stirrups.

and $f_{yt} =$

Try a 16 stirrups, each with nine 12mm diameter ties ($A_b = 113.1 \text{ mm}^2$), 420 Mpa.

The spacing to the first set of shear studs (s_o) is to be taken $\leq \frac{d}{2} = 184 \text{ mm}$.

Take it = 150 mm.

The area provided by the inner row of shear studs is:

$A_v = 16 \times 113.1 = 1809.6 \text{ mm}^2$

$$V_s = \frac{Av_f y_t d}{s} = \frac{1809.6 \times 420 \times 368}{100} / 1000 = 2796.92 \text{ KN} \geq V_{s,req}$$

The outer most studs at: $10 \times 100 + 150 = 1150 \text{ mm}$, from the face of the column.

And the outer critical section at $1150 + \frac{d}{2} = 1334 \text{ mm}$, from the face of column.

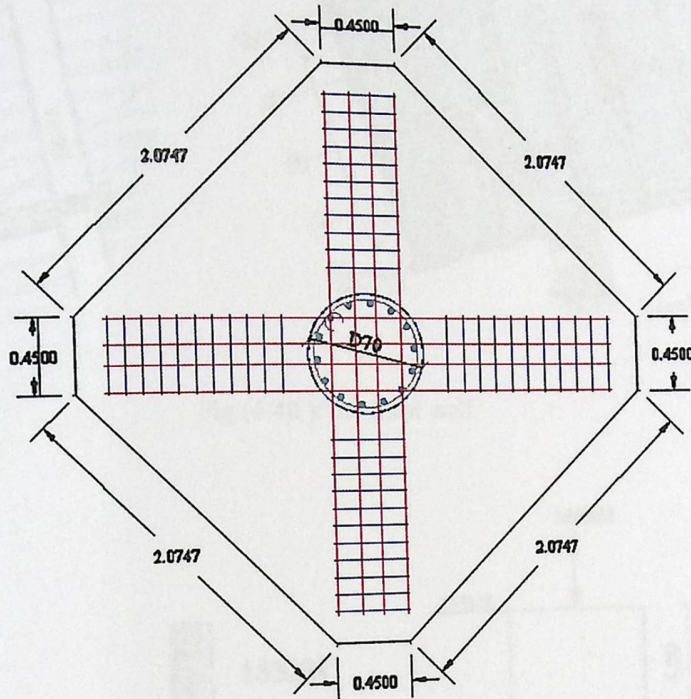


Fig (4-39): Ties Details

The perimeter of outer critical section = $4(700 + 2074.7) = \text{cmm}$.

The area inside the outer perimeter = 7.09

4.12.5 Now check the force stresses at the outer critical section:

$$V_u = 2405 - (7.09 \times 25.808) = 2222.02 \text{ KN}$$

$$\phi V_c = 0.75 \times \frac{1}{3} \sqrt{24} \times 11098.8 \times 368 / 1000 = 5002.3 \text{ KN}$$

$\phi V_c > V_u$ the assumption is correct.

Also Check

$$\phi V_c = 0.75 \times \frac{1}{6} \sqrt{24} \times 11098.8 \times 368 / 1000 = 2501.15 \text{ KN OK.}$$

13 Shear wall design:

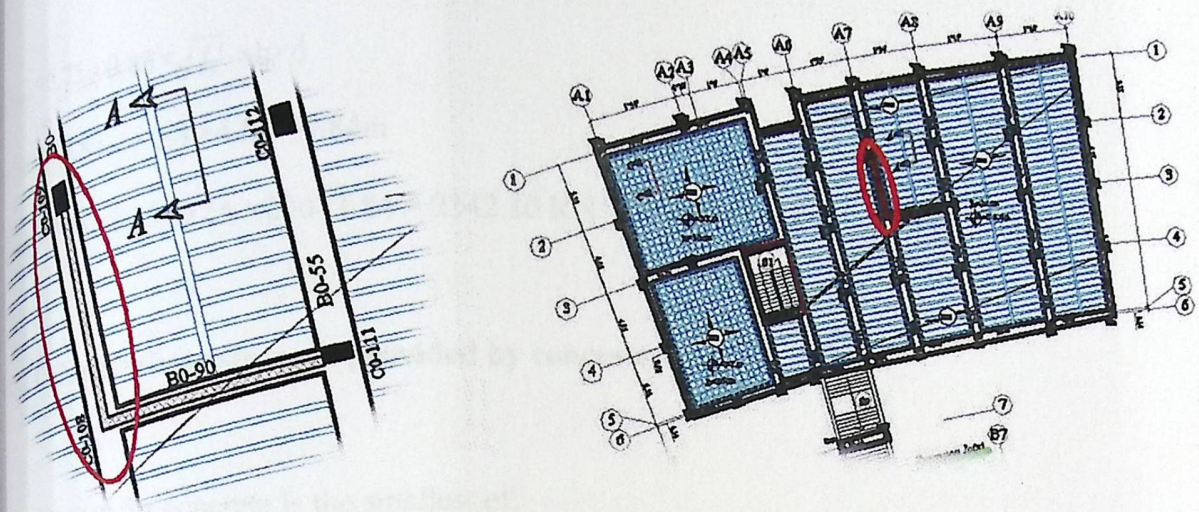


Fig (4-40): the shear wall.

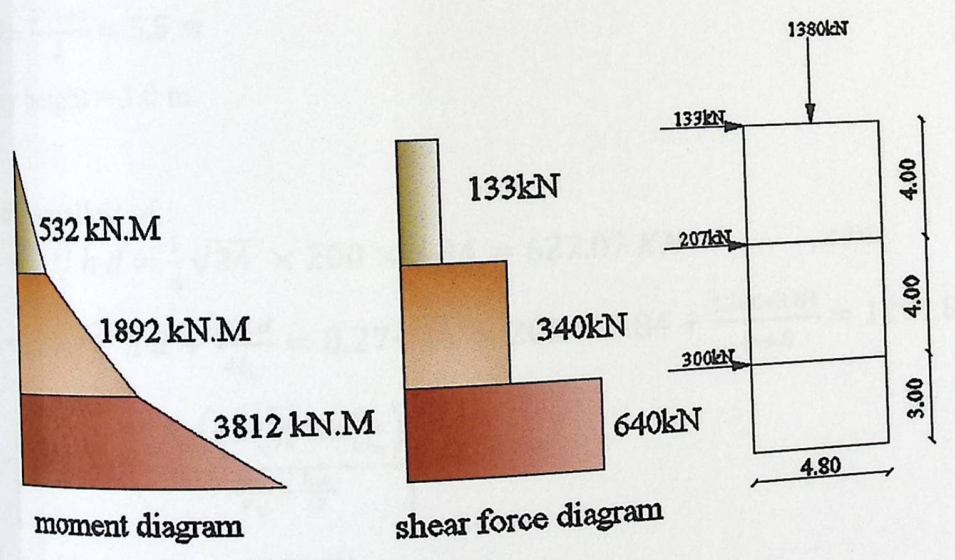


Fig (4-41): shear and moment for the shear wall.

Shear wall thickness, $h = 20 \text{ cm}$.

Story height, $h_w = 4.0 \text{ m}$

4.13.1 Check max shear strength permitted:

$$\phi V_{n,max} = 0.75 \times 0.83 \times \sqrt{f'_c} \times h \times d$$

$$\text{let that } d = 0.8 L_w = 0.8 \times 4.80 = 3.84 \text{ m}$$

$$\phi V_{n,max} = 0.75 \times 0.83 \times \sqrt{24} \times 200 \times 3.84 = 2342.10 \text{ KN} > V_u = 640 \text{ KN} \quad \text{ok.}$$

4.13.2 Calculate shear strength provided by concrete:

Critical section for concrete is the smallest of:

$$- \frac{L_w}{2} = \frac{4.80}{2} = 2.40 \text{ m} \dots \dots \text{cont.}$$

$$- \frac{\sum h_w}{2} = \frac{2 \times 4 + 3}{2} = 5.5 \text{ m}$$

$$- \text{Story height} = 3.0 \text{ m.}$$

V_c is the smallest of :

$$- V_c = \frac{1}{6} \sqrt{f'_c} h d = \frac{1}{6} \sqrt{24} \times 200 \times 3.84 = 627.07 \text{ KN} \quad , \text{cont.}$$

$$- V_c = 0.27 \sqrt{f'_c} h d + \frac{N_u \cdot d}{4 l_w} = 0.27 \sqrt{24} \times 200 \times 3.84 + \frac{1380 \times 3.84}{4 \times 4.8} = 1291.85 \text{ KN}$$

$$- V_c = \left[0.05 \sqrt{f'_c} + \frac{L_w \left(0.1 \sqrt{f'_c} + 0.2 \frac{N_u}{h l_w} \right)}{\frac{M_u}{V_u} - \frac{L_w}{2}} \right] h d$$

M_u at critical section = 2276 KN.m

$$\frac{2276}{640} - \frac{4.80}{2} = 1.156 > 0.0 \quad \text{ok.}$$

$$V_c = \left[0.05 \sqrt{24} + \frac{4.80 \left(0.1 \sqrt{24} + 0.2 \frac{1380}{3 \times 4.8} \right)}{1.156} \right] 200 \times 3.84 = 1811.5 \text{ KN}$$

4.13.3 Determine required horizontal reinforcement:

$$V_u = 640 \text{ KN.}$$

$$0.5\phi V_c = 0.5 \times 0.75 \times 627.07 = 235.15 \text{ KN.}$$

$V_u > 0.5\phi V_c$ need reinforcement.

$$V_s = V_u - V_c = \frac{V_u}{\phi} - V_c = \frac{640}{0.75} - 627.07 = 226.26 \text{ KN.}$$

$$\rho = \frac{A_s}{s \cdot h}, \quad \frac{A_s}{s} = \frac{A_s}{f_y \cdot d} = \frac{226.26 \times 10^3}{420 \times 3800} = 0.141 \frac{\text{mm}^2}{\text{mm}}$$

$$\rho = \frac{0.141}{200} = 0.0007 < 0.0025$$

by using $\phi 10$

$$\rho = \frac{2 \times 78.5}{s \times 200} = 0.0025 \longrightarrow s = 314 \text{ mm.}$$

max. spacing is the smallest of:

$$\frac{L_w}{5} = \frac{4.80}{5} = 0.96 \text{ m}$$

$$- 3h = 0.6 \text{ m.}$$

$$- 450 \text{ mm} \quad \text{cont.}$$

For horizontal reinforcement use $\phi 10 @ 300 \text{ mm}$.

4.13.4 Determine required vertical reinforcement:

$$\frac{\sum h_w}{L_w} = \frac{2 \times 4 + 3}{4.8} = 2.29 \text{ m.}$$

$$\rho_{v, \min} > 0.0025 + 0.5 \left(2.5 - \frac{\sum h_w}{L_w} \right) (\rho_t - 0.0025) \geq 0.0025.$$

take $\rho_v = 0.0025$.

max spacing is the least of,

$$- \frac{L_w}{3} = \frac{4.80}{3} = 1.60 \text{ m}$$

$$- 3h = 0.6 \text{ m.}$$

$$- 450 \text{ mm} \quad \text{cont.}$$

Use $\phi 10 @ 300 \text{ mm}$.

4.13.5 Design for flexure (uniformly distributed flexure reinforcement):

Check moment strength based on required vertical reinforcement for shear,
The uniformly distributed vertical reinforcement $\phi 10@300\text{mm}$.

$$A_{st} = \frac{4800}{300} \times 2 \times 78.5 = 2512 \text{ mm}^2$$

$$\omega = \left(\frac{A_{st}}{L_w h} \right) \frac{f_y}{f_c'} = \left(\frac{2512}{4800 \times 200} \right) \frac{420}{24} = 0.0457$$

$$\alpha = \frac{P_u}{L_w f_c' h} = \frac{1380000}{4800 \times 24 \times 200} = 0.059$$

$$\frac{c}{L_w} = \frac{\omega + \alpha}{2\omega + 0.85\beta_1} = \frac{0.0457 + 0.059}{2 \times 0.0457 + 0.85 \times 0.85} = 0.1297$$

$$\phi M_n = \phi \left[0.5 A_{st} f_y L_w \left(1 + \frac{P_u}{A_s f_y} \right) \left(1 - \frac{c}{L_w} \right) \right]$$

$$= 0.9 [0.5 \times 2590.5 \times 420 \times 4800 (1 - 0.1297)] \times 10^{-6} = 2272.54 \text{ KN.m}$$

$$\phi M_n < M_u,$$

The uniformly distributed vertical reinforcement $\phi 10@300$, is not adequate for flexure therefore the amount of vertical reinforcement must be increased.

Try $\phi 12@200$,

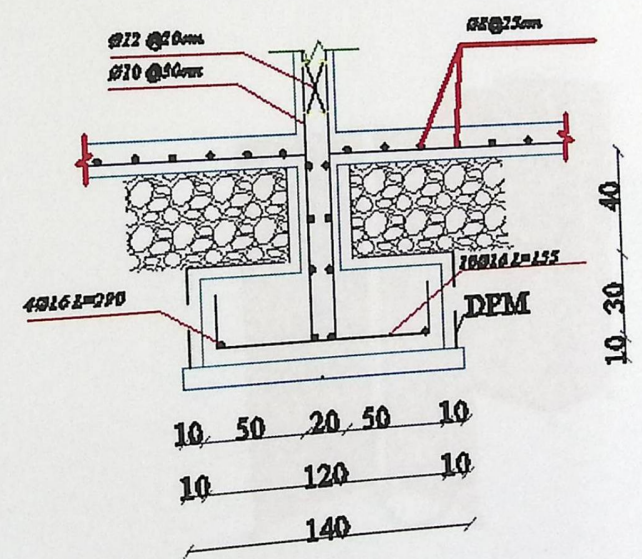
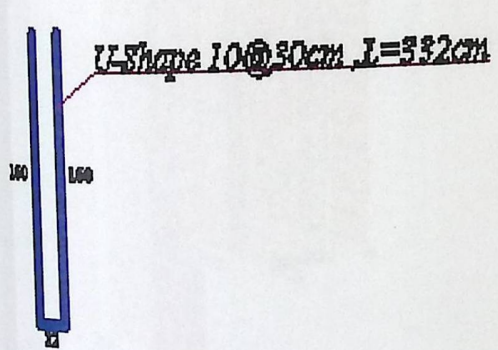
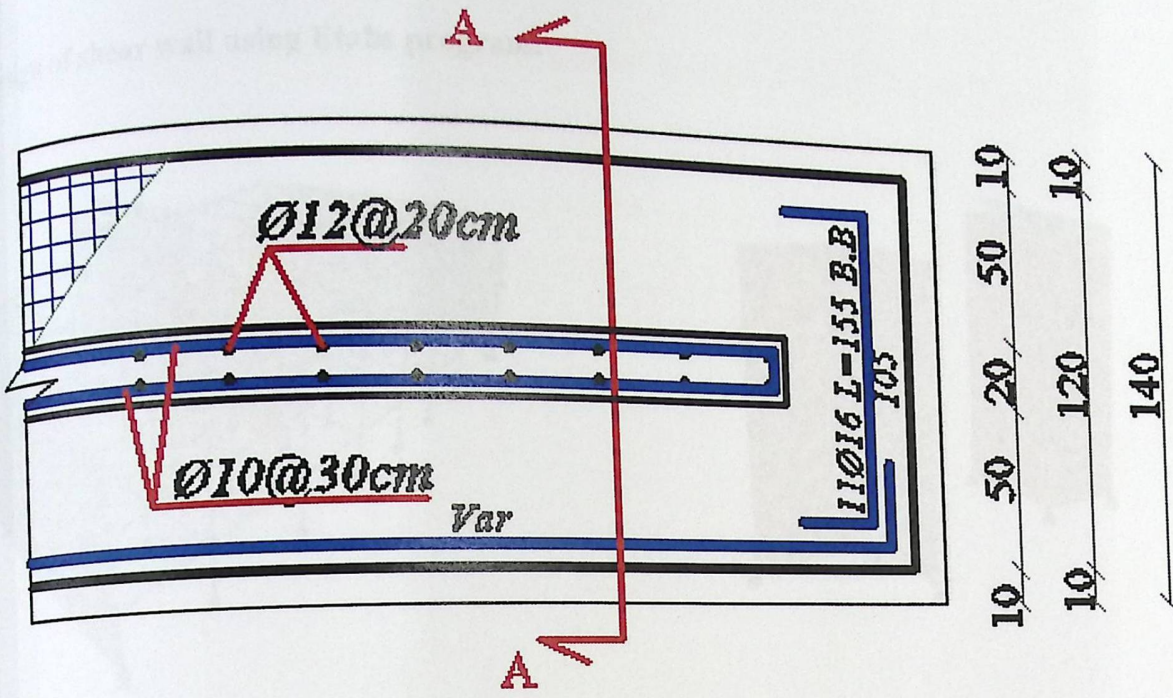
$$A_{st} = \frac{4800}{200} \times 2 \times 113.1 = 5428.8 \text{ mm}^2$$

$$\omega = \left(\frac{5428.8}{4800 \times 200} \right) \frac{420}{24} = 0.0989$$

$$\frac{c}{L_w} = \frac{0.0989 + 0.059}{2 \times 0.0989 + 0.85 \times 0.85} = 0.171$$

$$\phi M_n = 0.9 [0.5 \times 5428.8 \times 420 \times 4800 (1 - 0.0989)] \times 10^{-6} = 4437.92 \text{ KN.m} > M_u$$

Use $\phi 12@200$ for vertical reinforcement.



Section A-A

Fig (4-42): top view & section for the wall

Design of shear wall using Etabs program:

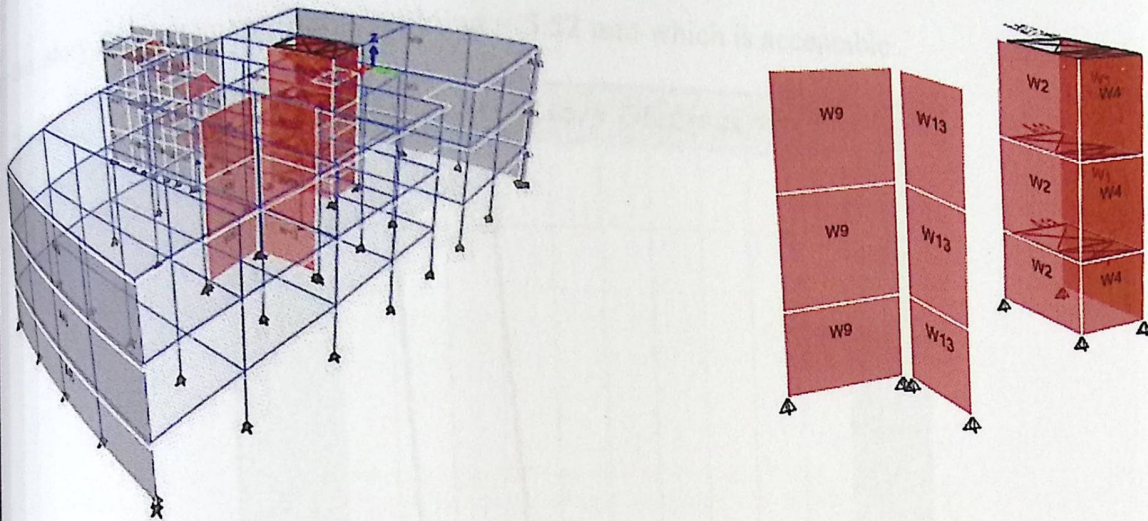
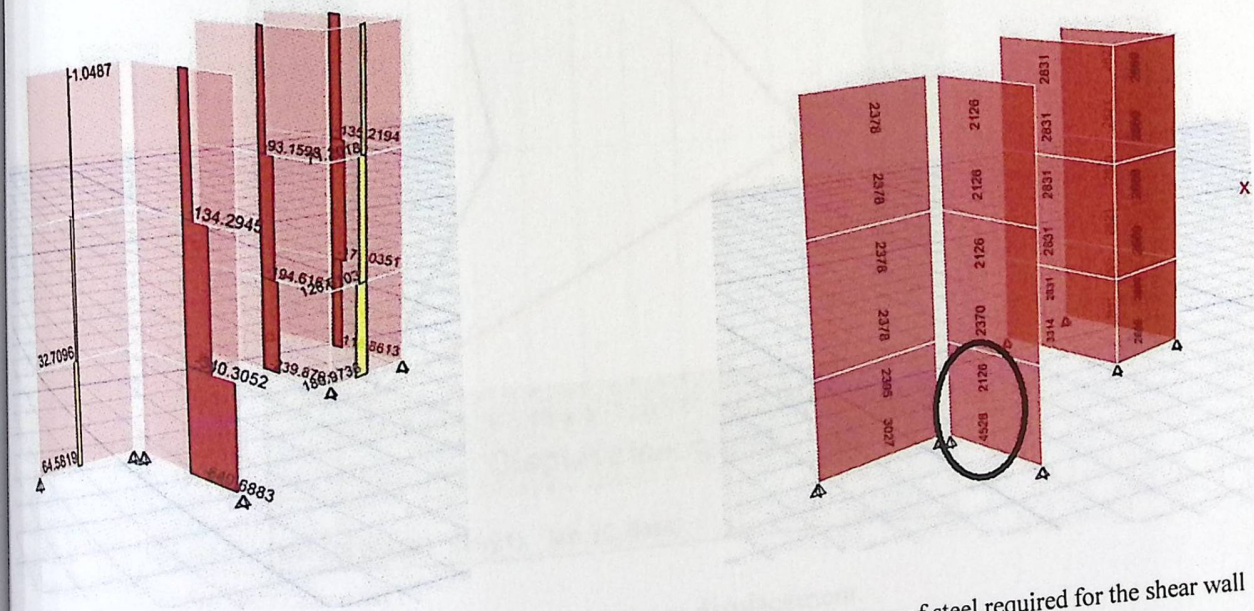


Fig (4-43): the shear wall (w13).



(4-44): shear force for the shear wall

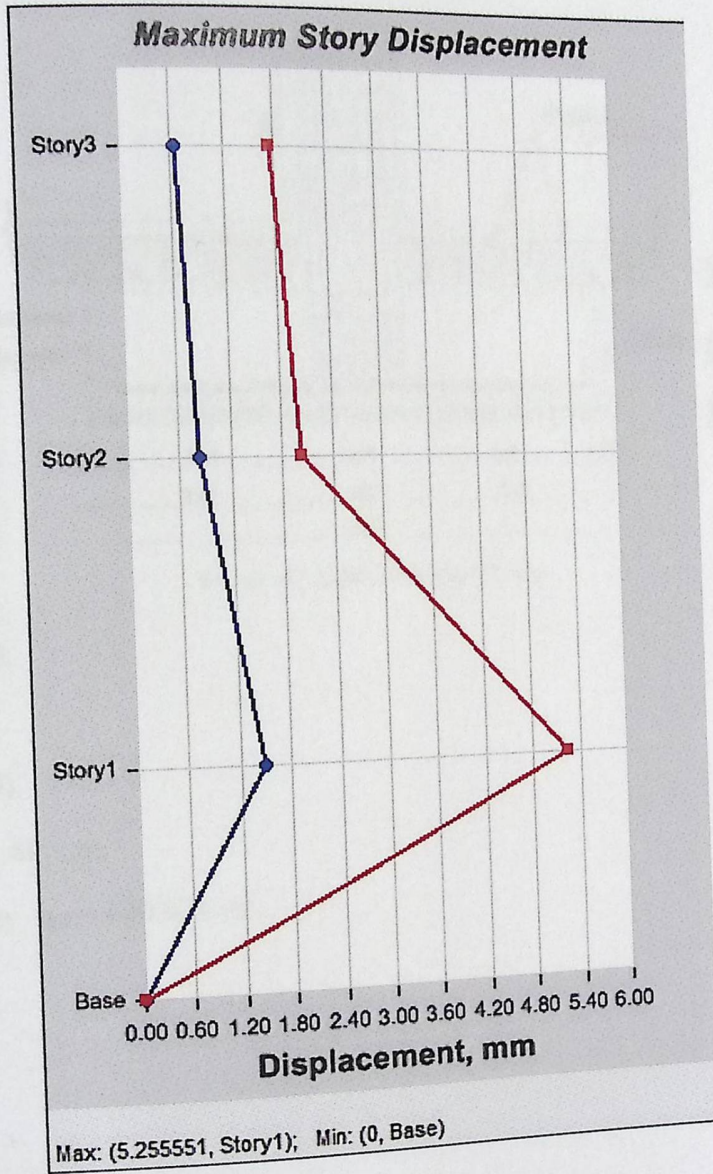
(4-45): Area of steel required for the shear wall

use $\phi 12@200$ for vertical reinforcement will gives

$$A_{st} = \frac{4800}{200} \times 2 \times 113.1 = 5428.8 \text{ mm}^2 > 4528 \dots OK$$

The maximum story displacement allowable for earthquake design as in the ACI code is equal to $\frac{L}{50} = \frac{3000}{50} = 60mm$.

The max. story Displacement in this building = 5.52 mm which is acceptable .



(4-46): maximum story displacement.

Depth of footing:

assume $h = 95\text{cm}$.

Check one-way shear:

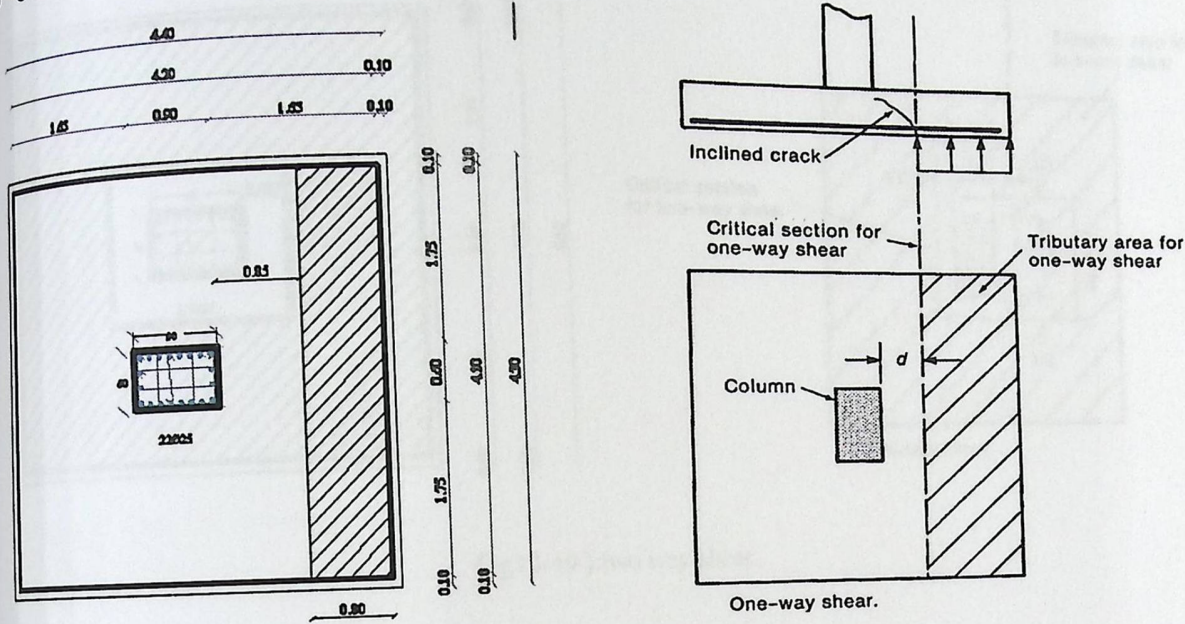


Fig (4-48):one way shear.

$$q_u = \frac{P_u}{\text{Area}} = \frac{7882.1}{17.22} = 457.72 \text{KN/m}^2$$

$$d = 950 - 75 - 20 = 855 \text{ mm}$$

$$V_c = \Phi \frac{1}{6} \sqrt{f_c'} b_w d = \frac{0.75}{6} * \sqrt{24} * 4100 * 0.855 * 1000 = 2146.67 \text{KN}$$

$$V_u = q_{ult} * \left(\frac{B-a}{2} - d \right) * L$$

$$V_{ud} = 457.73 * \left(\frac{4.10 - 0.9}{2} - 0.855 \right) * 4.2 = 1432.23 \text{KN}$$

$$\phi V_c = 0.75 * 2146.67 = 1610.00 \text{KN} > V_{ud} = 1432.23 \dots \dots \text{ok}$$

Check two-way shear:

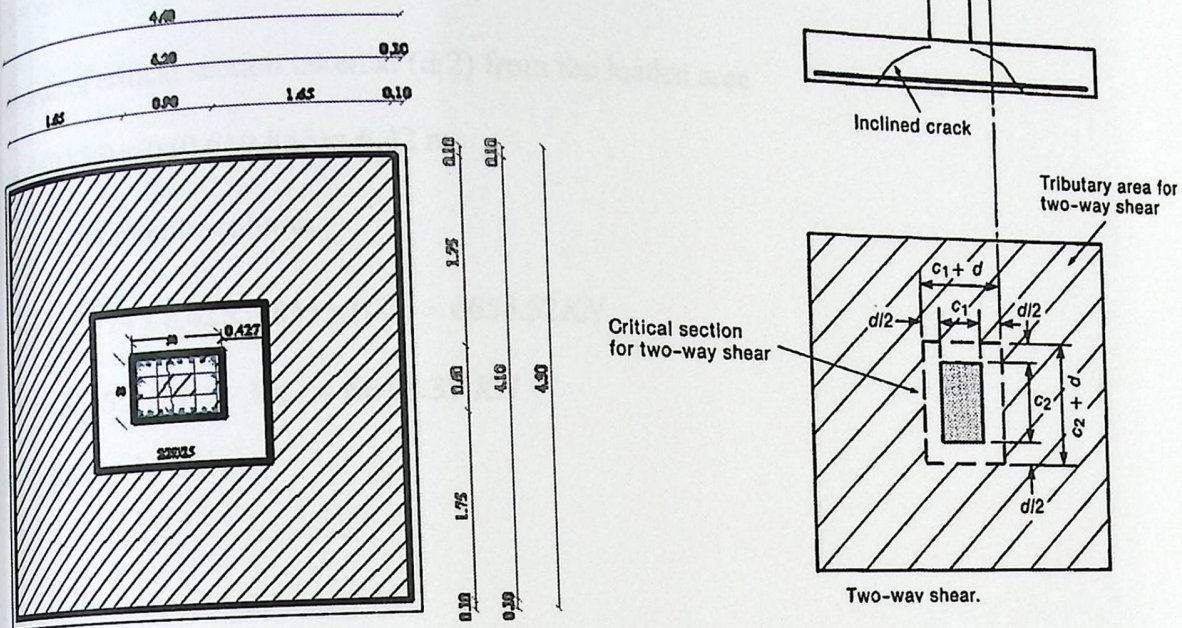


Fig (4-49):two way shear.

$$\frac{0.855}{2} = 0.427 \text{ m}$$

calculate V_u at the critical section which take rectangular shape as shown in the last fig.

$$\text{inner area} = (0.9 + 0.855) \times (0.6 + 0.855) = 2.553 \text{ m}^2$$

$$\text{outer area} = \text{area of the footing} = 17.22 \text{ m}^2$$

$$V_u = q_u (\text{outer area} - \text{inner area}) = 457.72 \times (17.22 - 2.553) = 6613.37 \text{ KN}$$

According to ACI, V_c shall be the smallest of :

$$V_c = \frac{1}{6} \left(1 + \frac{2}{\beta_c} \right) \sqrt{f'_c} b_o d = 0.5 \sqrt{f'_c} b_o d$$

$$V_c = \frac{1}{12} \left(\frac{\alpha_s}{b_o/d} + 2 \right) \sqrt{f'_c} b_o d = 0.585 \sqrt{f'_c} b_o d$$

$$V_c = \frac{1}{3} \sqrt{f'_c} b_o d \dots \text{Control}$$

Where:

$$\beta_c = a/b = 90/60 = 1.5$$

b_o = Perimeter of critical section taken at $(d/2)$ from the loaded area

$$= 2 \times (0.9 + 0.855) + 2 \times (0.6 + 0.855) = 6.42 \text{ m.}$$

$\alpha_c = 40$ for interior column.

$$\phi V_c = 0.75 \times 0.33 \sqrt{24} \times 6.42 \times 0.855 \times 1000 = 6655.52 \text{ KN}$$

$$\phi V_c = 6655.2 \text{ KN} > V_u = 6613.37 \text{ KN}$$

SO $h = 95 \text{ cm}$ Is OK.

4.14.3 Design of flexural reinforcement:

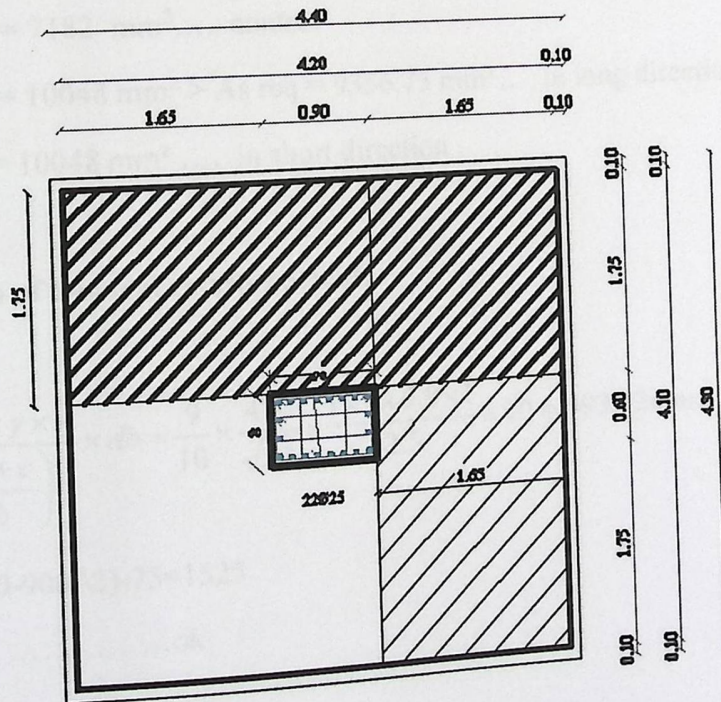


Fig (4-50): Max. moment.

$$= 187.72 \times 4.2 \times 1.75 \times \frac{1.75}{2} = 2943.71 \text{ KN.m.}$$

$$= 2943.71/0.9 = 3270.8 \text{ KN.m.}$$

$$\frac{M_u}{b \times d^2} = \frac{3270.8 \times 10^{-3}}{4200 \times (0.855)^2} = 1.065 \text{ Mpa}$$

$$\frac{f_y}{0.85 \times f_c'} = \frac{420}{0.85 \times 24} = 20.59$$

$$\frac{1}{m} \left(1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right)$$

$$\frac{1}{20.59} \left(1 - \sqrt{1 - \frac{2(20.59)(1.065)}{420}} \right) = 2.605 \times 10^{-3}$$

$$A_{s, \text{req}} = \rho \times b \times d = 2.605 \times 10^{-3} \times 4200 \times 855 = 9356.75 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.0018 \times 4200 \times 950 = 7182 \text{ mm}^2 \dots \text{control}$$

Use 32 Φ 20 with $A_s = 10048 \text{ mm}^2 > A_{s, \text{req}} = 9356.75 \text{ mm}^2 \dots$ in long direction .

Use 30 Φ 20 with $A_s = 10048 \text{ mm}^2 \dots$ in short direction .

14.4 Development length of flexural reinforcement:

Ld for Φ 20:

$$L_d = \frac{9}{10} \times \frac{f_y}{\sqrt{f_c'}} \times \frac{\alpha \times \beta \times \gamma \times \lambda}{\left(\frac{k_{tr} + c}{db} \right)} \times db = \frac{9}{10} \times \frac{420}{\sqrt{24}} \times \frac{1 \times 1 \times 0.8 \times 1}{2.5} \times 20 = 493.82 \text{ mm}$$

$$\text{Available length} = ((4100 - 900)/2) - 75 = 1525$$

$$= 1525 \text{ mm} > 493.82 \text{ mm} \dots \text{ok}$$

14.5 Development length of column reinforcement:

for $\phi 25$:

$$\frac{f_y}{4\sqrt{f_c'}} db = \frac{420}{4\sqrt{24}} \times 25 = 535.82 \text{ mm}$$

$$0.043 \times db \times f_y = 0.043 \times 25 \times 420 = 451.5 \text{ mm}$$

535.8 mm

14.6 Load transfer at the column-foundation interface (Dowels design):

$$\phi P_{n,b} = \phi(0.85 f_c' A_1 \times \sqrt{\frac{A_2}{A_1}})$$

$$A_1 = 0.9 \times 0.6 = 0.54 \text{ m}^2$$

$$A_2 = 4.2 \times 4.1 = 17.22 \text{ m}^2$$

$$\frac{\sqrt{A_2}}{\sqrt{A_1}} = \sqrt{\frac{17.22}{0.54}} = 5.64 > 2$$

$$\frac{\sqrt{A_2}}{\sqrt{A_1}} = 2$$

$$\phi P_{n,b} = 0.65 \times (0.85 \times 24 \times 0.54 \times 2) \times 1000 = 14320.8 \text{ KN}$$

$$\phi P_n = 14320 > P_u = 7882.1 \dots \dots \text{ok}$$

In column:

$$\phi P_{n,b} = \phi(0.85 f_c' A_1)$$

$$\phi P_{n,b} = 0.65(0.85 \times 24 \times 0.54 \times 1000) = 7160.4 \text{ KN}$$

$$\phi P_{n,b} = 7160 \text{ KN} < p_u = 7882.1 \text{ KN}$$

$$A_{\text{dowels}} = \frac{p_u - \phi P_{n,b}}{\phi f_y} = \frac{7882.1 - 7160}{0.65 \times 420} = 2652 \text{ mm}^2$$

and the minimum reinforcement of dowels:

$$A_s = 0.005 \times 0.54 \times 1000000 = 2700 \text{ mm}^2$$

take minimum Ok

Lap splice of column:

$l_d = 0.071 \cdot f_y \cdot d_b = 0.071 \times 420 \times 25 = 745.5 \text{ mm}$. Use 750 mm.

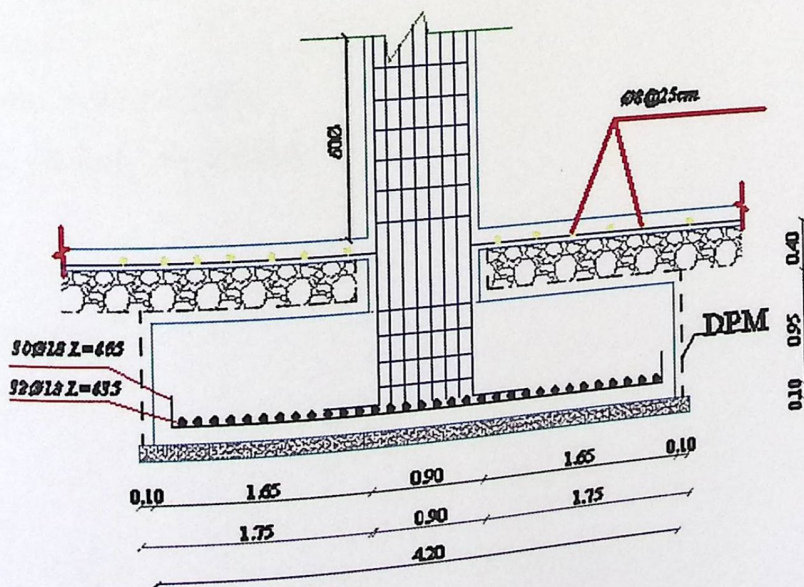
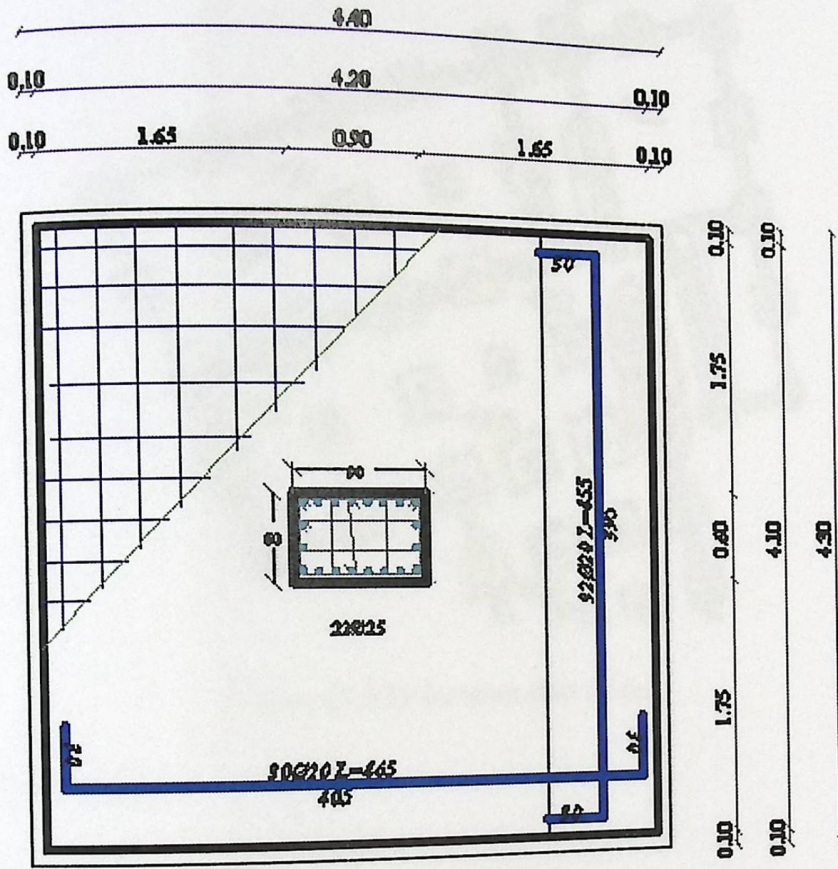


Fig (4-51):details of the footing

15 Design of strip Footing:

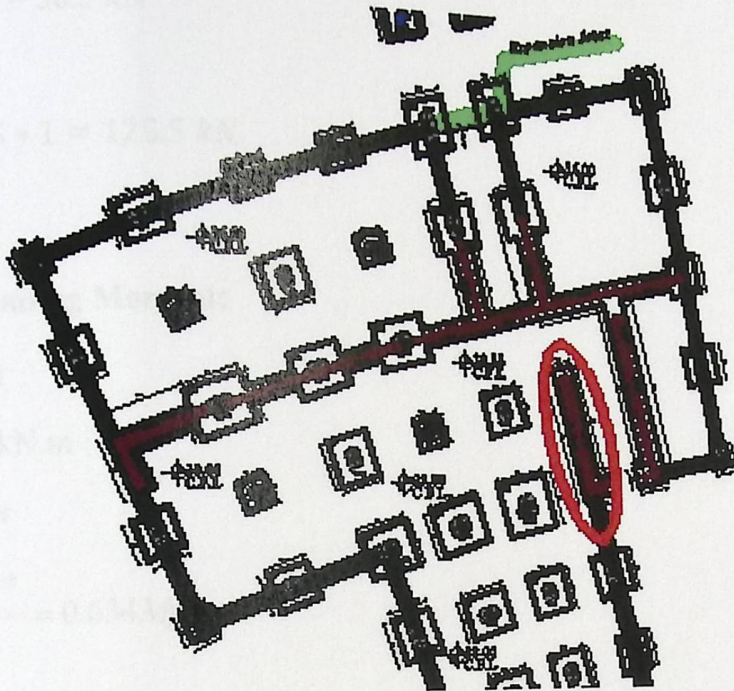


Figure (4-52): location strip footing

4.15.1 Determination of load:

From slab and Wight wall

Total factored load = 300 kN/m.

Soil density = 18 Kg/cm³.

Allowable soil Pressure = 400 kN/m².

Assume footing to be about (30 cm) thick.

live load = 5 kN/m²

$$Q_{allow} = 400 - 5 - 0.4 \cdot 18 - 0.3 \cdot 25 = 380.3 \text{ kN/m}^2$$

⇒ For one meter strip

$$A = \frac{300}{380.3} = 0.79 \text{ m}^2$$

B = 1 m, h = 30 cm

$$d = 300 - 75 - 20 = 205 \text{ mm}$$

$$Q_{ult} = 300 / 1 \cdot 1 = 300 \text{ kN/m}^2.$$

4.15.2 Check of One Way Shear:

$$V_u = 1 * (0.40 - 0.205) * 300 = 58.5 \text{ kN}$$

$$\phi V_c = \frac{\phi}{6} \sqrt{f_c} * d * b$$

$$\phi V_c = \frac{0.75}{6} \sqrt{24} * 0.205 * 1 = 125.5 \text{ kN}$$

$$\phi V_c > V_u$$

4.15.3 Design of Bending Moment:

In longitudinal direction

$$M_u = 300 * 0.4^2 / 2 = 24.0 \text{ kN.m}$$

$$M_n = \frac{24.0}{0.9} = 26.66 \text{ KN.m}$$

$$K_n = \frac{M_n}{bd^2} = \frac{26.666 * 10^{-3}}{1 * 0.205^2} = 0.634 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 * f_c'} = \frac{420}{0.85 * 24} = 20.588$$

$$\rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 * m * K_n}{f_y}} \right)$$

$$\rho = \frac{1}{20.588} \left(1 - \sqrt{1 - \frac{2 * 20.588 * 0.634}{420}} \right) = 1.533 * 10^{-3}$$

$$A_{s_{Req.}} = \rho * b * d = 0.0001533 * 205 * 1000 = 314.416 \text{ mm}^2$$

$$A_{s_{Shrinkage}} = 0.0018 * b * h = 0.0018 * 300 * 1000 = 540 \text{ mm}^2$$

$$A_{s_{Req.}} = 314.416 < A_{s_{Shrinkage}} = 540 \text{ mm}^2$$

Use ϕ 12

$$\text{No.} = 540 / 113.1 = 4.77 \quad , \text{ Use 5 bars}$$

ϕ 12 at 20 cm c/c

Check of strain:

$$\epsilon_s \cdot f_y = 0.85 \cdot f_c \cdot b \cdot a$$

$$300 \cdot 420 = 0.85 \cdot 24 \cdot 1000 \cdot a$$

$$a = 11.11 \text{ mm}$$

$$\beta_1 = \frac{a}{c} = \frac{11.11}{0.85} = 13.08 \text{ mm}$$

$$\epsilon_s = \frac{205 - 13.08}{13.08} \times 0.003$$

$$\epsilon_s = 0.044 > 0.005$$

In transverse direction :

$$A_{smin} = 0.0018 \cdot B \cdot h$$

$$A_{smin} = 0.0018 \cdot 800 \cdot 300 = 432 \text{ mm}^2$$

Use ϕ 12

$$\text{No.} = 432 / 113 = 3.82 \quad , \text{ Use 4 bars}$$

Use 4 ϕ 12

4.15.4 Development Length of main Reinforcement:

$$l_{dreq} = \frac{9}{10} \cdot \frac{F_y}{\lambda \sqrt{f_c}} \cdot \frac{\psi_e \psi_s \psi_t}{\frac{ktr+cb}{db}} \cdot db$$

$$l_{dreq} = \frac{9}{10} \cdot \frac{420}{1 \cdot \sqrt{24}} \cdot \frac{1 \cdot 1 \cdot 0.8}{2.5} \cdot 12 = 296.6 \text{ mm}$$

$$l_{davailable} = 500 - (200/2) - 75 = 325 \text{ mm}$$

$$l_{davailable} = 325 \text{ mm} < l_{dreq} = 296.6 \text{ mm}$$

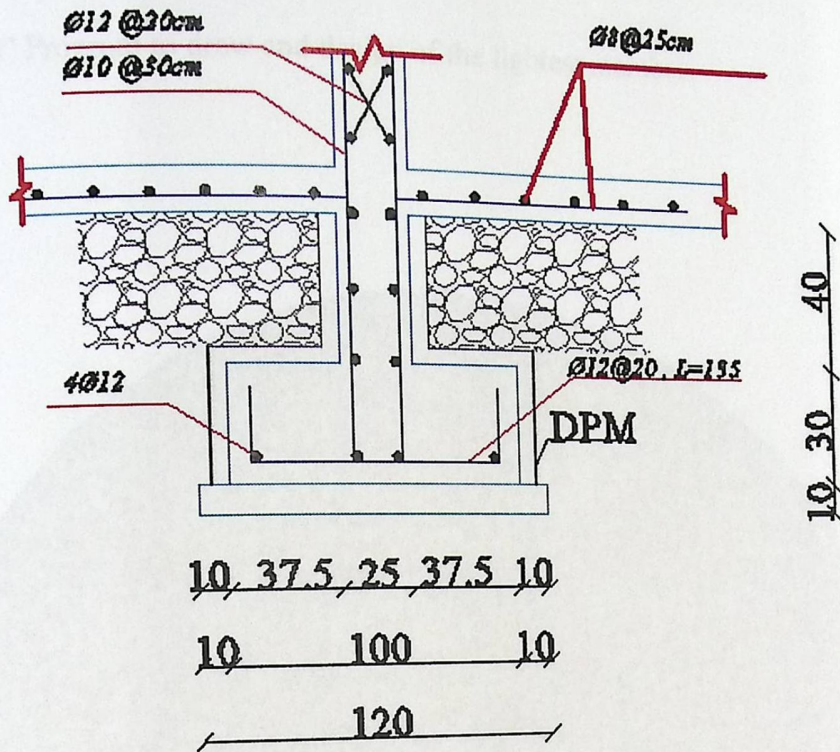


Figure (4-53) Strip Footing Detail

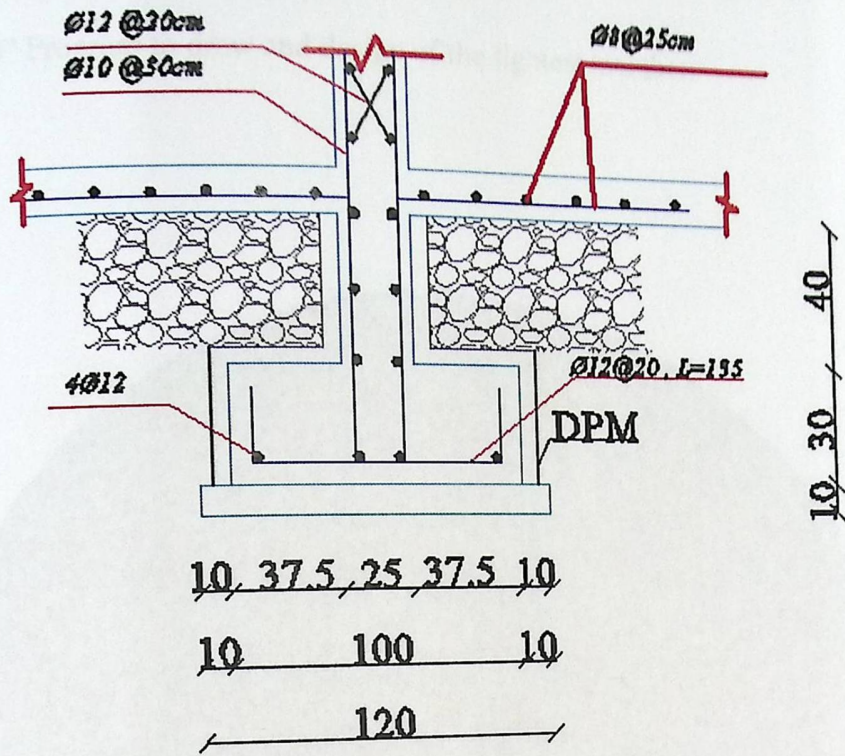


Figure (4-53) Strip Footing Detail

4.16:- Design Of Dom :

Using "SAP2000" Program to draw and design of the lightest member:

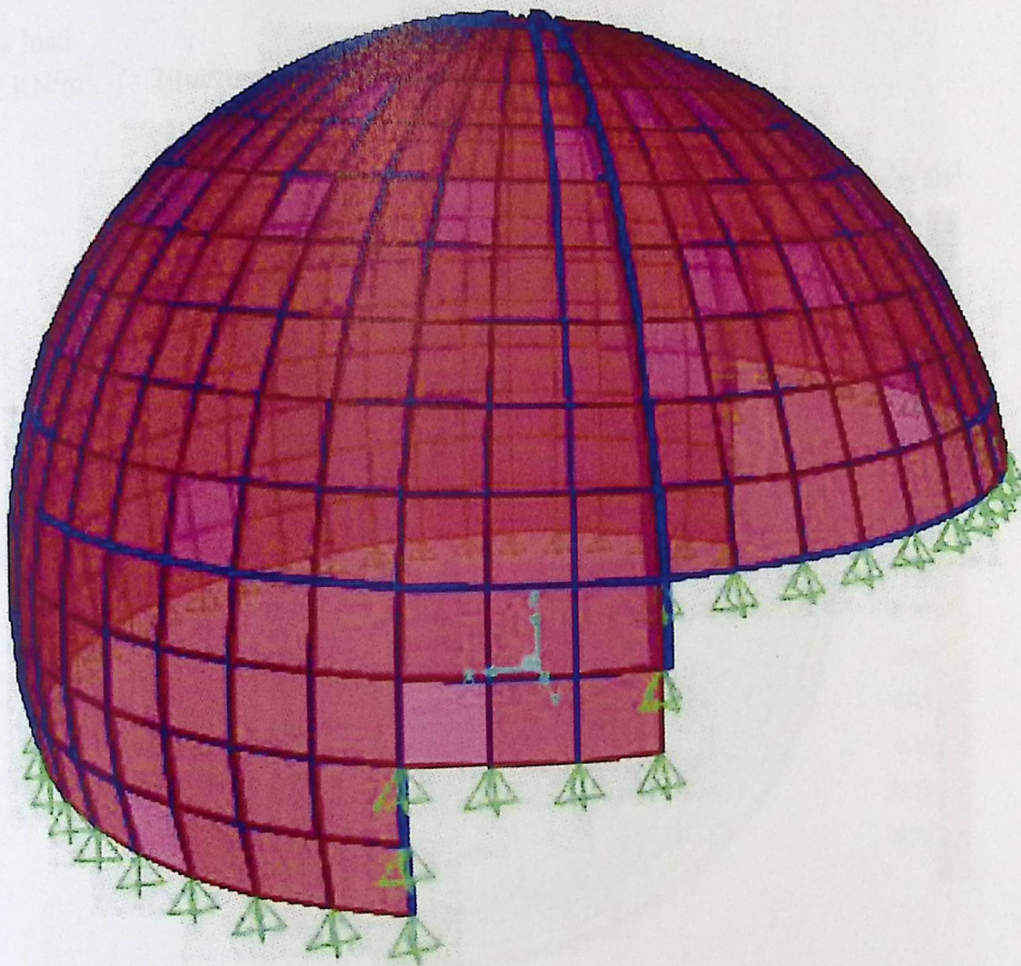


Figure (4-54): SAP Model of the Dom

4.16.1:- Load calculation

1. Dead Load:-

- Surface layer of double glass with thickness of 10mm each. Neglected
- And the self-weight of the steel members.

2. Snow load

$S = 2 \text{ KN/m}^2$ / 2 meters of dry sand.

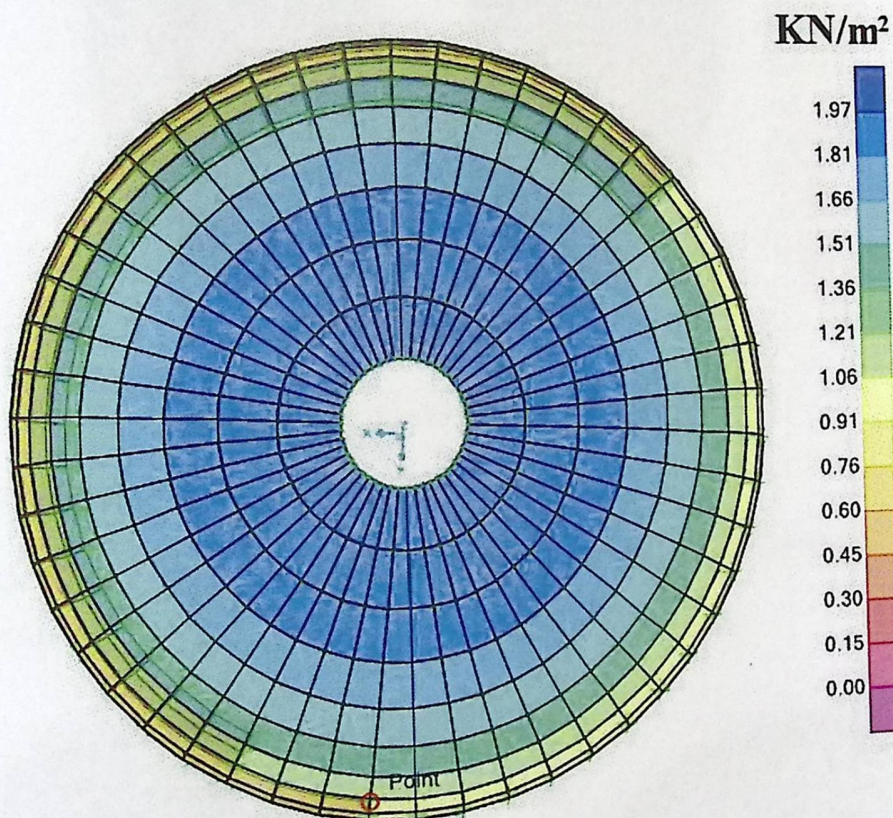


Figure (4-55): Snow load on SAP model

3. Wind load

$$P_{ww} = C_e C_q q_s I_w$$

From the normal force method

Wind speed 70mph, Exposure factor (Ce) B, Pressure factor Cq 0.8 for inward and 0.5 outward and Importance fraction 1.

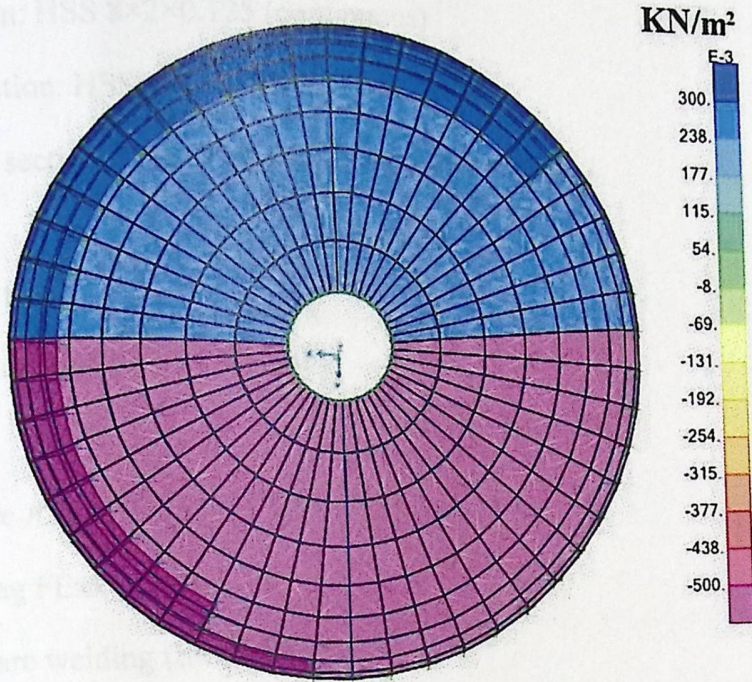


Figure (4-56): Wind load on SAP model

4.16.2:- Steel members design

After analyzing of the model, we check for the lightest weight of all members, till all sections pass the design check.

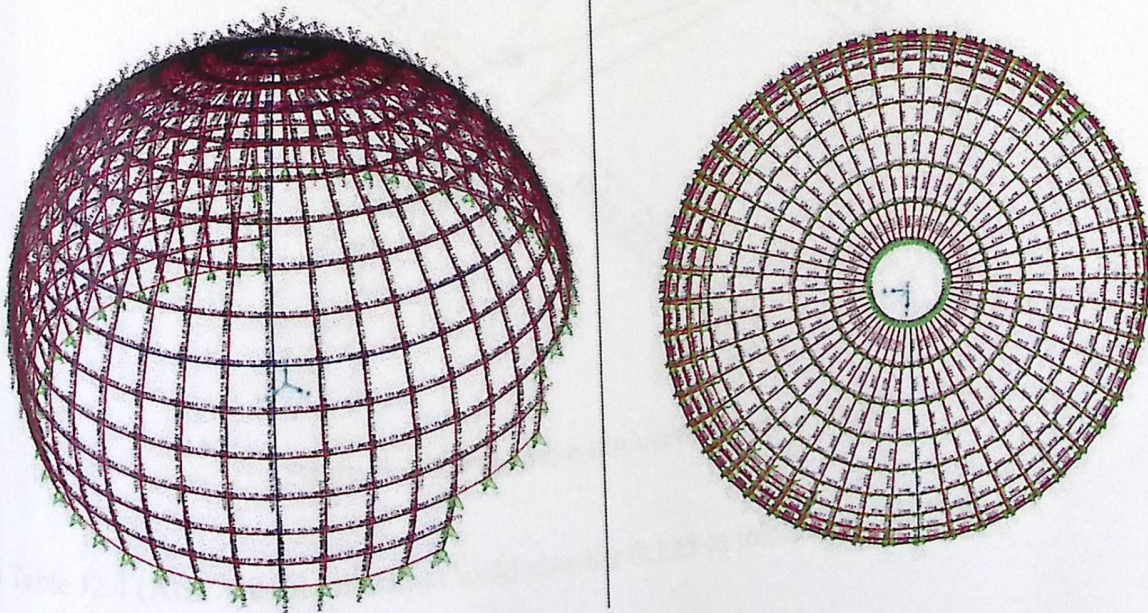


Figure (4-57): Members sections on SAP model

The Chosen sections are as follows:

Vertical members section: HSS 8×2×0.125 (continuous)

Horizontal members section: HSS 2×2×0.125

Top horizontal member section: HSS 10×2×0.125 (continuous circle)

4-13.4:- Design of weld:-

The calculations of weld based on the following:

- 1) Fillet Weld is used.
- 2) The HSS sections are A36 ($F_y = 36$ ksi, $F_u = 58$ ksi).
- 3) The electrodes having $F_{Exx} = 70$ ksi.
- 4) The shielded metal arc welding (SMAW) is used.

A. (1st)- Design of weld between the vertical member HSS 8×2×0.125 and the horizontal members HSS 2×2×0.125 of the Dom:

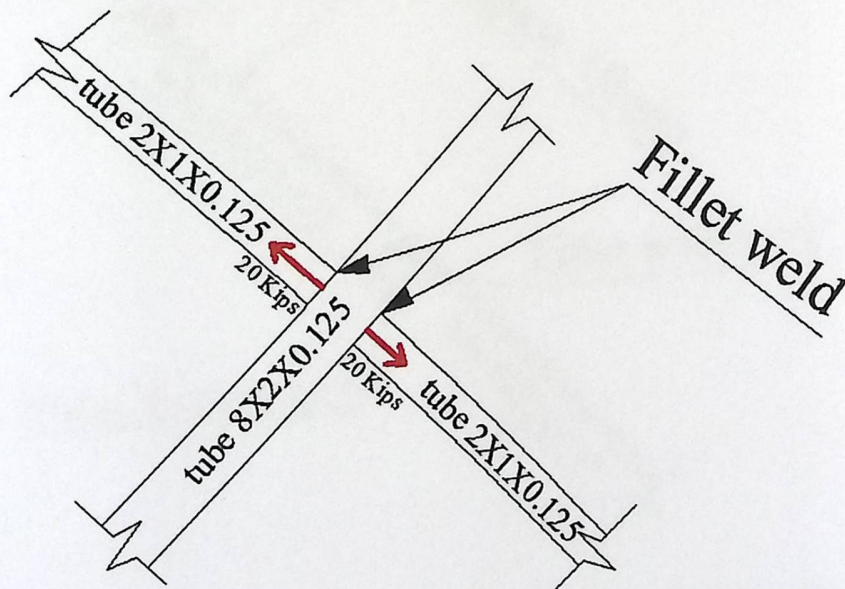


Figure (4-58): Joint between Vertical and horizontal

From Table J2.4 (AISC 2010) minimum weld size for 0.125 in thickness is 0.125 in.

Chapter 4:

Use weld size $(a) = \frac{1}{8}$ in.

Design strength of weld:-

$$\phi R_{nw} = \phi \times t_e \times 0.6 \times F_{Exx}$$

$$\phi R_{nw} = 0.75 \times (0.707 \times \frac{1}{8}) \times 0.6 \times 70 = 2.784 \text{ kips/in}$$

Design strength of material:-

Yielding: $\phi R_n = \phi \times (0.6 F_y) \times t$

$$\phi R_n = 1 \times (0.6 \times 36) \times .0125 = 2.7 \text{ kips/in}$$

Rupture: $\phi R_n = \phi \times (0.6 F_u) \times t$

$$\phi R_n = 0.75 \times (0.6 \times 50) \times .0125 = 2.8125 \text{ kips/in}$$

$$\therefore \phi R_{nw} = 2.7 \text{ kips/in}$$

$$\Rightarrow L_w = \frac{F}{\phi R_{nw}} = \frac{20}{2.7} = 7.407 \text{ in} \dots \dots \dots \text{take } 8 \text{ in (sections premeter)}$$

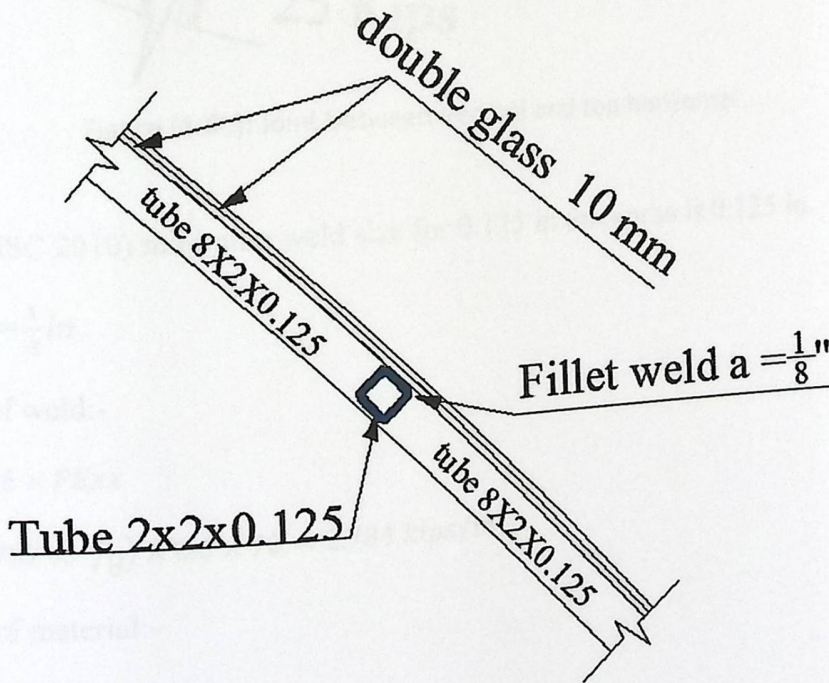


Figure (4-59): Joint between Vertical and horizontal show the welding

B. Design of weld between the vertical member HSS 8x2x0.125 and the top horizontal member HSS 10x2x0.125:

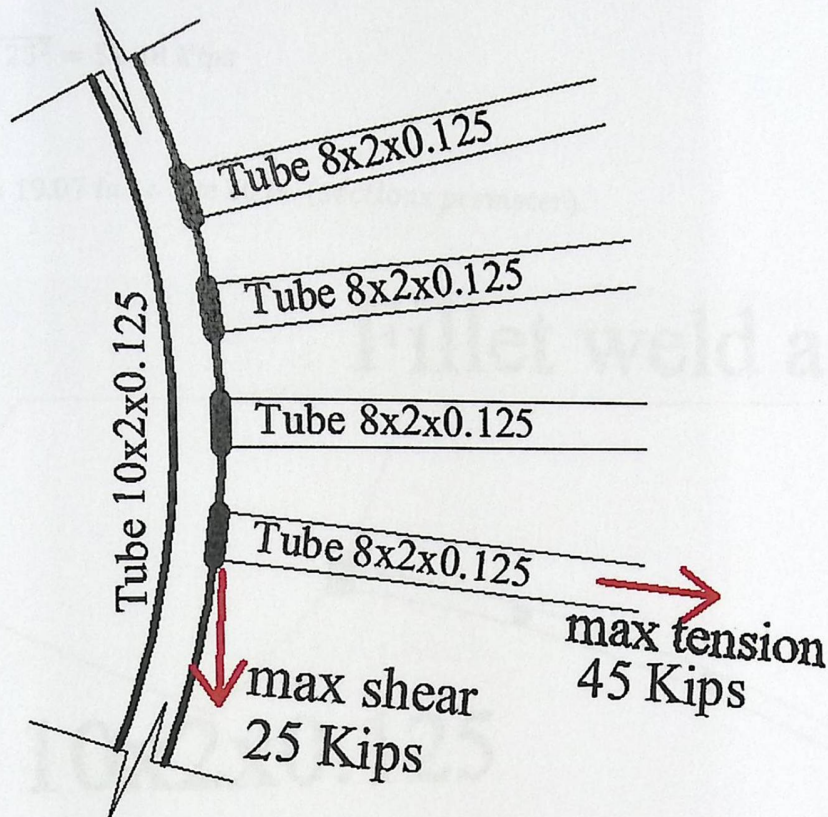


Figure (4-60): Joint between Vertical and top horizontal

From Table J2.4 (AISC 2010) minimum weld size for 0.125 in thickness is 0.125 in.

∴ Use weld size (a) = $\frac{1}{8}$ in.

⇒ Design strength of weld:-

$$\phi R_{nw} = \phi \times t_e \times 0.6 \times F_{Exx}$$

$$\phi R_{nw} = 0.75 \times (0.707 \times \frac{1}{8}) \times 0.6 \times 70 = 2.784 \text{ kips/in}$$

⇒ Design strength of material:-

$$\text{Yielding: } \phi R_n = \phi \times (0.6 F_y) \times t$$

$$\phi R_n = 1 \times (0.6 \times 36) \times 0.125 = 2.7 \text{ kips/in}$$

$$\text{Rupture: } \phi R_n = \phi \times (0.6 F_u) \times t$$

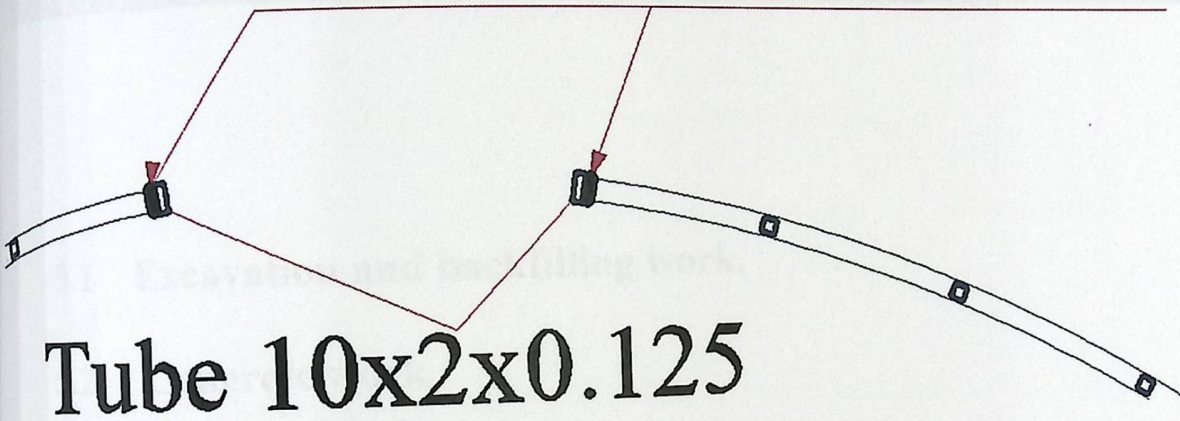
$$\phi R_n = 0.75 \times (0.6 \times 50) \times 0.125 = 2.8125 \text{ kips/in}$$

$$\phi R_{nw} = 2.7 \text{ kips/in}$$

$$\text{Resultant force} = \sqrt{45^2 + 25^2} = 51.48 \text{ Kips}$$

$$L_w = \frac{F}{\phi R_{nw}} = \frac{51.48}{2.7} = 19.07 \text{ in} \therefore \text{use } 20 \text{ in (sections perimeter).}$$

Fillet weld $a = \frac{1}{8}''$



Tube 10x2x0.125

Figure (4-61): Joint between Vertical and top horizontal show welding

Chapter five

Estimated Cost

5

- 5.1 Excavation and backfilling work.**
- 5.2 Concrete work.**
- 5.3 Stone work.**

Excavation and backfilling work :

1.1 General notes for excavation:

1. Excavation for foundation, trenches. Etc. shall be the actual dimensions shown on Drawings including blinding area, and to be measured from Natural Ground Level or Reduced Level (which is lower), i.e.
 1. From Reduced Level when R. L. is lower than N. G. L.
 2. From N. G. L. when N. G. L. is lower than the Reduced Level.
2. Allowance for working space is not applicable, unless it is clearly specified in the Special Conditions.
3. Excavation for foundation of retaining walls is to be measured from Lower Reduced level, or from lower Natural Ground level (as stated in Item 1 above.)
4. Price to include removal of site obstacles, rocks, trees and all debris from site, including any necessary rerouting of infrastructure works, such as piping, conduits, etc. Price also to include maintaining all trees that are not obstructing the buildings, boundary walls, courts and roads.
5. Each layer of back filling shall be tested separately, on the contractor's expense, prior to applying the next layer.
6. Co-ordination with all local authorities to facilitate the works, removal of obstacles or for diversion of roads, services, etc. is the contractor's responsibility and are deemed to be included in his rates.
7. Excavated soil shall be classified and retained, to back fill with suitable fill material and to place agricultural soil as where needed as shown in the

drawings and as instructed by the Engineer. The extra and unsuitable soil shall be removed, on the contractor's expense with no extra cost.

8. Prior to commencement of site leveling or any excavation, the Contractor shall obtain Engineer's written approval on the contour grid of 5m x5m map, checked jointly by the contractor and the Engineer, the excavation profiles and sections showing all levels and dimensions, and the calculations of excavation quantities.

9. Excavation shall be in Cubic Volume, the net measurements of the excavated elements as shown in the drawings, no allowance shall be made for increase in bulk after excavation.

10. Hard Core beds shall be of not less than 15 cm. thick. And measured in Meter Square, (the net measurement of the finished surface). No extra rate shall be paid for Leveling, ramming and filling of voids under the hard core.

11. Rate of Excavation shall include: -

- Excavation to any depth, in any shape, in any type of soil, below water table level and keeping excavation free of water.
- Leveling, ramming, trimming and grading excavation bottoms.
- Planking and strutting.
- Cost of lab. Tests of works and materials.

The estimated quantities and prices:

EXCAVATION AND BACK FILLING (5.1.2)

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE EURO	CENT	AMOUNT EURO	CENT
	Excavation, Back filling and Site Works.						
1.01	<p>Site Leveling to reduced level (Excavation and/or make up of levels for all Site including External Works). Cut in any kind of soil, concrete, or asphalt; to the required Levels (reduced levels as required on drawings).</p> <p>Price to include back filling with excavated suitable soil, or imported approved materials if needed, removal of extra soil, debris, and organic materials to the approved sites specified by authorities. Back filling material shall be applied on layers, 200 mm thick, watered and compacted to min 95% according to modified AASHTO Density Test T-80. Price also includes preparation of agricultural areas according to the levels in the general site layout plan. Price to include removal of any construction elements not needed in the site or filling cavities and wells with suitable materials.</p>	L.S.	1	50000		50000	
1.02	<p>Excavation for foundations; in any kind of soil to the dimensions and levels shown in the drawings.</p> <p>Price to include back filling with excavated soil if suitable or imported approved materials up to bottom of base course, removal of extra and unsuitable soil. Back filling shall be on layers of 200 mm thick with watering and compacting to min 95% to comply with modified AASHTO F.D.T. (T-80)</p>	M.C	1800	8		14400	
1.03	<p>Supply and Well Compact Base Coarse, average 40 cm. Thick, for underground slab. Price includes leveling, watering, compacting to 98%, leveling and compacting of ground (sub-base) under base coarse to min 95% according to Field Density Test.</p>	M.S	7000	7		49000	
	TOTAL					113400	
	TOTAL CARRIED TO SUMMARY.					113400	

Concrete work

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE	AMOUNT	TOTAL
	SKELETON WORKS					
	CONCRETE WORKS					
	<p>Preamble:</p> <p>Rates of Concrete Works shall include:-</p> <ol style="list-style-type: none"> 1. All form works and shuttering in any form, shape and size; Making chamfered and curved edges; allowing for and making grooves and sleeves and the like; removal of forms and cleaning of all exposed tie wires and rods; making good the harmed surfaces and edges and all surface finishing. 2. Supplying, Casting, vibrating and curing as per specifications. 3. Approved additives and admixtures. 4. Incurred costs on Concrete Mix Designs, Sample taking, testing and providing test results certificates, storing and saving of samples, and preparation of bar bending schedules and workshops. 5. Supplying reinforcement of any grade, size and length as detailed in the drawings, storing on site including cutting, bending and fixing in position and providing all tying wires, spacers, shop drawings, testing and bar bending schedules. 6. Painting of exposed surfaces of underground reinforced concrete elements with two coats of hot bituminous paint or special cold bitumen (75/25), the strokes of each layer to be opposite to each other (for the buried parts),. 7. Steel bars used for alignment and fixing of reinforcement works, and extra steel added to assure proper detailing and safety. 8. The minimum cement contents for concrete should be: <ul style="list-style-type: none"> a-B200 — 250kg/m³ b-B250 — 300kg/m³ c-B300 — 350kg/m³ 9. Testing specimens for concrete quality control during construction shall be cubes of size 15x15x15 cm 					

Concrete work :

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE		AMOUNT	
				EURO	CENT	EURO	CENT
	SKELETON WORKS.						
	CONCRETE WORKS.						
	<p>Preamble.</p> <p>Rates of Concrete Works shall include:-</p> <ol style="list-style-type: none"> All form works and shuttering in any form, shape and size. Making chamfered and curved edges, allowing for and making grooves and sleeves and the like; removal of forms and cleaning of all exposed tie wires and rods; making good the harmed surfaces and edges and all surface finishing. Supplying, Casting, vibrating and curing as per specifications. Approved additives and admixtures. Incurring costs on Concrete Mix Designs, Sample taking, testing and providing test results certificates, storing and saving of samples, and preparation of bar bending schedules and workshops. Supplying reinforcement of any grade, size and length as detailed in the drawings, storing on site including cutting, bending and fixing in position and providing all tying wires, spacers, shop drawings, testing and bar bending schedules Painting of exposed surfaces of underground reinforced concrete elements with two coats of hot bituminous paint or special cold bitumen (75/25), the strokes of each layer to be opposite to each other (for the buried parts).. Steel bars used for alignment and fixing of reinforcement works, and extra steel added to assure proper detailing and safety. The minimum cement contents for concrete should be: <ul style="list-style-type: none"> a-B200 ___ 250kg/m3 b-B250 ___ 300kg/m3 c-B300 ___ 350kg/m3 Testing specimens for concrete quality control during construction shall be cubes of size 15x15x15 cm 						

5.2 Concrete work :

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE		AMOUNT	
				EURO	CENT	EURO	CENT
	SKELETON WORKS.						
	CONCRETE WORKS.						
	<p>Preamble.</p> <p>Rates of Concrete Works shall include:-</p> <ol style="list-style-type: none"> 1. All form works and shuttering in any form, shape and size. Making chamfered and curved edges, allowing for and making grooves and sleeves and the like; removal of forms and cleaning of all exposed tie wires and rods; making good the harmed surfaces and edges and all surface finishing. 2. Supplying, Casting, vibrating and curing as per specifications. 3. Approved additives and admixtures. 4. Incurred costs on Concrete Mix Designs, Sample taking, testing and providing test results certificates, storing and saving of samples, and preparation of bar bending schedules and workshops. 5. Supplying reinforcement of any grade, size and length as detailed in the drawings, storing on site including cutting, bending and fixing in position and providing all tying wires, spacers, shop drawings, testing and bar bending schedules 6. Painting of exposed surfaces of underground reinforced concrete elements with two coats of hot bituminous paint or special cold bitumen (75/25), the strokes of each layer to be opposite to each other (for the buried parts).. 7. Steel bars used for alignment and fixing of reinforcement works, and extra steel added to assure proper detailing and safety. 8. The minimum cement contents for concrete should be: <ul style="list-style-type: none"> a-B200 __ 250kg/m3 b-B250 __ 300kg/m3 c-B300 __ 350kg/m3 9. Testing specimens for concrete quality control during construction shall be cubes of size 15x15x15 cm 						

<p>Measurement of Concrete elements shall be the net dimensions as detailed in the drawings. All recesses openings and voids shall be deducted. Measurement shall be only for the seen surfaces of elements. Overlapping and measuring items twice under separate headings shall not be allowed.</p> <p>Slabs measurement should be:</p> <ul style="list-style-type: none">a- from the center to center for external stone wallsb- from the external to the external face of the outer drop beams for stone cladding walls <p>The columns necks shall be measured from the top level of the foundation or footing to the bottom level of the tie beam.</p> <p>(All beams, drop beams, projection including sun breakers and any part casted together with the slab shall not be measured and is deemed to be included in the price of the slab). Columns are to be measured from the top level of the slab to the bottom level of the beam or dropped beam.</p>						
--	--	--	--	--	--	--

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE		AMOUNT	
				EURO	CENT	EURO	CENT
	SKELETON WORKS.						
	CONCRETE WORKS.						
2.01	Supply and Cast Plain Concrete;						
	a) a) Blinding Concrete B150 under foundations, footings & tie beams. Thickness should be 10cm or according to drawings.	M.S	2000	8		16000	
	b) Rubble Concrete B200 with Max. 30% crushed stones for filling cavities and voids, which appears under foundation and according to Project manager	M.C	100	70		7000	
2.02	Supply and Cast Reinforced Concrete Grade 'B 300'						
	a). Foundations and Wall Footings.	M.C	1500	180		270000	
2.03	Supply and Cast Reinforced Concrete Grade 'B 300'.						
	a). Tie Beams.	M.C	80	180		14400	
	b) Reinforced Walls (Shear walls or/and Basement walls) below Ground Slab Level including the columns on these walls.	M.C	50	180		9000	
2.04	Supply and Cast Reinforced Concrete Grade 'B 250'. For Slab on Hard Core. Ten (15) cm. thick Slab Including surface leveling, finishing, polyethylene sheets 125 micron, construction joints and steel reinforcement overlapped to tie beams, and all related works.(measured for projected area)						
		M.S	7000	12		84000	
2.05	Supply and Cast Reinforced Concrete Grade 'B 300' for:						
	a). Stairs, landings, fighting's and where required.	M.C	154	200		30800	

	b). Reinforced Shear Walls above ground slab on hard core including the columns on these walls.	M.C	1460	150	219000	
	c) Basement walls above slab on hard core including the columns on these walls.	M.C	120	170	20400	
2.06	Supply and Cast Reinforced Concrete Grade 'B 300' for the following:					
	a). Column portions. (Necks).	M.C	60	290	17400	
	b). Columns.	M.C	1043	175	182525	
	TOTAL				870725	
	TOTAL CARRIED TO NEXT PAGE				870725	

SKELTON WORKS - CONCRETE WORKS

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE		AMOUNT	
				EURO	CENT	EURO	CENT
	SKELETON WORKS.						
	CONCRETE WORKS.						
	FORWARDED TOTAL.					870725	
2.07	Supply and Cast Reinforced Concrete Grade 'B 300' Price to include projected beams, other beams and projections as per drawings.						
	a- 30 cm thick for flat slabs.	M.S.	9485	50		474250	
	b- 35 cm thick for flat slabs.		1046	60		62760	
	c- 40 cm thick for flat slabs.		1325	70		92750	
2.08	Supply and Cast Reinforced Concrete Grade 'B 300' suspended ribbed slabs. Price to include projected beams, other beams and projections, and polystyrene for beams projections,, Supply, lay and arrange hollow concrete blocks 'Grade 35' for ribbed slabs, using closed end blocks as required by the project manager at all ends, edges, openings, services location etc as per drawings.						
	a. 32 cm one way ribbed slab	M.S.	8865	60		531900	
	b. 36 cm one way ribbed slab		520	65		338000	
	c. 36 cm tow way ribbed slab		8150	70		570500	
	TOTAL					2070160	
						2940885	
	TOTAL CARRIED TO SUMMARY						

5.3 Stone work :**SKELETON WORKS - STONE WORKS**

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE		AMOUNT	
				EURO	CENT	EURO	CENT
	SKELETON WORKS.						
	STONE WORKS.						
	<p>PREAMBLE.</p> <p>All Stone works shall be of local white & yellow stone of First Choice.</p> <p>Stone material, size, and texturing shall be in accordance with detailed drawings and Engineer's instructions and approval.</p> <p>Stones shall be of not less than (5 cm. thick, 50*50 cm. high and of length.)</p> <p>Stones shall be homogeneous in color and texturing and free of any defects.</p> <p>Cutting, shaping and building of stone shall be in straight angles, no twisting, distortion or uneven sizes shall be allowed.</p> <p>Measurement of all types shall be in Meter Square, for the net dimensions of the vertical plan of the elevations excluding openings of any area.</p> <p>No measurement will be made for any recesses, projections or sides of stone pieces.</p> <p>Measurement of door and window jambs and reveals, corner peaces, and the like is not applicable. The stone facades (either built or clad) shall be measured according to their vertical elevations excluding the openings. Neither the projections (including the sides of the stone cladded columns..etc) nor the recesses from the relevant facades shall be subjected to measurement as these are deemed to be included in the unit rates of the relevant stone works. No extra over shall be given to stone angles or any other special stone piece deemed necessary to properly complete the works according to the drawings and specifications.</p>						

<p>No allowances shall be made for stone pieces in any shape.</p> <p>No allowances or extra rates shall be considered for stone courses in lintels, overhanging slabs and roof slab courses.</p> <p>Taking all the above-mentioned into account, measurement for stone cladding works will follow the shape of the clad element.</p> <p>Rates of stone works shall include:-</p> <ul style="list-style-type: none"> • Supplying , storing, protection and removal of rejected and surplus materials. • Shaping, cutting and fitting. • Mortar and backing concrete (plain concrete and/or cement blocks and plain concrete). • Galvanized steel ties, dowels, hangers, steel mesh and all tying tools. • Erecting and dismantling of scaffoldings. • Pointing and pointing materials of any type, and cleaning of walls. • Lintels and other reinforced concrete elements embedded in the stone walls including reinforcement. • 3Cm Polystyrene boards for insulation between concrete and concrete blocks. <p>Note : The stone concrete backing should be cast against proper formwork. After deshuttering the formwork, the polystyrene can be fixed and the 10cm block can be built. Casting directly against the polystyrene and block is NOT allowed.</p>						
--	--	--	--	--	--	--

SKELETON WORKS - STONE WORKS

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	RATE		AMOUNT	
				EURO	CENT	EURO	CENT
	SKELETON WORKS.						
	STONE WORKS.						
3.01	Stone cladding facades over reinforced concrete walls above side walk level (apparent facades), R.C. ground tie beam with its exterior drop recess and slab beams. Price to include concrete filling of plain concrete Grade B-250, steel mesh and all ancillaries. The price include supply and installation of jambs and reveals stone. Price not to include for the reinforced concrete elements to be cladded, which is to be measured under another item. a. white little rough textured "Mofajar" for facades b. yellow smooth "Tafoh stone" for projections .	M.S. M.S.	7205 313	50 60		360250 18780	
	TOTAL					379030	
	TOTAL CARRIED TO SUMMARY					3319915	

Chapter Six

Results and recommendations

6

6.1 Appendix A: Architectural Drawings.

6.2 Appendix B: Structural Drawings.

6.3 results.

6.4 Recommendation. Architectural Drawings

6.5 resources and references.

6.1 Appendix A: Architectural Drawings

6.2 Appendix B: Structural Drawings

6.3 The Results :

1. Each student or structural designer should be able to design manually so he can get the experience and knowledge in using the computerized design programs.
2. One of the factors that must be taken into consideration is the environment factors surrounding the building, the site terrains, and the forces effects on the site.
3. One of the important steps of the structural design is how to connect the structural members to work together as a whole, then to divide these members and design them individually taken the surrounding condition into consideration.
4. The soil strength capacity is 400 KN/m².
5. Various types of slabs have been used: two ways and one way ribbed slabs, in some slabs that have a regular or nearly regular distribution of columns and beams. One way solid slabs mainly in the stairs, because it has high resistance to the concentrated forces. Two ways solid slab in flat plates due to irregularities.
6. The used computerized programs are:
 - a) AUTOCAD 2013/2007 , To draw the detailed drawings for structural drawings.
 - b) ETABS, to analyze and design the structural members.
 - c) STAAD PRO, to analyze and design the structural members.
 - d) ATIR, to analyze and design the structural members.
 - e) SAFE, and design the structural members.
 - f) Microsoft Office, To write , layout and outputting the project text.
7. We have used the live loads using the Jordanian code of loads.
8. One of the characteristics that the designer should have is the engineering feeling that he use it to overcome any problem in a convenient way.

6.4 The recommendations:

This project has an important role in widening and enhancing our understanding to the nature of the structural projects including all the details, analysis, and designs. We want here-through this experience- to introduce a group of recommendations; we hope it to be useful for whom planning to select a structural project.

At the beginning, the architectural drawings have to be prepared and ordered and the construction material and the structural system have to be chosen alongside. And it's essential at this stage to have information about the project site, the soil, the soil strength capacity at the site from the geotechnical report ,after that the bearing walls and the columns is going to be set up alongside the architectural team in a compatible manner. The civil engineer tries at this stage to plant as much as possible the reinforced concrete walls, which should be placed uniformly as possible as it can be all of the building; so it will be used after that in resisting the earthquake loads and the other lateral loads.

6.5 The resources and references:

1. American Concrete Institute (A.C.I), Building code Requirement for structural concrete (ACI-318M-08).
2. National Jordanian Construction codes ,loads and forces code, the National Jordanian construction council , AMMAN , JORDAN, 2006.
3. German code for the design of concrete structures (DIN 1045 – concrete).
4. The Uniform Building Code (UBC) For earthquakes and wind loads.