Palestine Polytechnic University

College of Engineering

Civil Engineering

Graduation Project

Structural Design for the

" Hayat Hospital project specialized in treating cancerous tumors"

in Yatta City

Project Team

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Supervisor:

Dr. Maher Amro.

This project submitted to the College of Engineering in partial fulfillment of requirements of the Bachelor degree of Civil Engineering

Hebron - Palestine

December 2021

The undersigned hereby certify that they have read, examined, and recommended to the Department of Civil Engineering and Architecture in the College of Engineering at Palestine Polytechnic University the approval of a project entitled:

Structural Design for the

" Hayat Hospital project specialized in treating cancerous tumors"in Yatta City

submitted by

Abdullah Qufaisha & Hasan Abu Tuhfa

for partial fulfillment of the requirements for the bachelor's degree.

Dr. Maher Amro. (Supervisor) :

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ABSTRACT

Hayat Hospital project specialized in treating cancerous tumors . This hospital is located in the Yatta area on a plot of 15000m 2 , with an area of approximately 10,000 square meters. It consists of 7floors, 4 floors over the ground floor and two underground floors.

The hospital has a main entrance and an emergency entrance on the western side . The ground floor consists of an emergency place, a reception area, a radiology place, and an outpatient place, with an area of 1293 square meters. The first floor consists of an early detection place, a review place for cancer patients, and a laboratory place, with an area of 1293 square meters. And the second floor consists of the operations place, the special care place, and the sleep place, and its area is 1380 square meters. And the third floor is the bedroom section, the administration area, the laundry section , and its area is 1293 square meters. As for the fourth floor, it is a complete bedroom section, with an area of 1293 square meters .

There are two underground floors that are used as a car park and complement the health and medical services. The area of each floor is 1678 square meters .

التصميم الإنشائي لِ

"مستشفى حياة المتخصص بعلاج الأورام السرطانية"

في مدينه يطا

فريق العمل:

عبد الله قفيشة . **حسن أبو تحفة .**

إشراف:

د.ماهر عمرو

الملخص

يهدف المشروع الى عمل تصميم انشائي لجميع العناصر الانشائية المكونة لمستشفى مكون من 7 طوابق تقدر مساحتها الاجمالية بـ 10000 م^2. وذلك لما للتصميم الانشائي من اهمية فهو من اهم المراحل التي يمر بها المبنى والتي يتم فيها تحديد اماكن الاعمدة و الانظمة الانشائية لمختلف عناصر المبنى وبذلك يتم تحويل المخططات المعمارية الاولية الى مخططات قابلة للتنفيذ .

وتحقيقا لهدف المشروع تم في البداية دراسة المخططات المعمارية و اختيار انسب الية لتوزيع العناصر الانشائية بما لا يتعارض مع التصميم المعماري للمبنى , ثم تم عمل دراسة انشائية مفصلة تم فيها تقدير الاحمال المتوقعة على جميع العناصر الانشائية بالاعتماد على الكود الاردني والكود الامريكي ASCE-16 والكود UBC-97 لتقدير احمال الزلازل.بعد ذلك تم تحليل وتصميم جميع تلك العناصر بالاعتماد على الكود الامريكي ACI318-14 وباستخدام مجموعة من البرامج الهندسية . وفي النهاية تم إعداد المخططات التنفيذية لجميع العناصر الانشائية المكونة لهيكل المبنى ليصبح المبنى قابلاً للتنفيذ .

DEDICATION

To those who have always believed in us ...

To those who have been our source of inspiration ...

To those who gave us strength ...

To those who provide us their endless support and encouragement ...

To our families ...

Project Team

ACKNOWLEDGEMENT

It has been a great opportunity for us to gain a lot of knowledge through working on this project ,but the successful completion of any task would be incomplete without mention of the people who made it possible.

For that we would like to thank everyone who helped, supported and encouraged us :

Palestine Polytechnic University, Engineering Collage, Civil Engineering Department, including all members of the helpful and reverend staff.

Special thanks to our supervisor **Dr. Maher Amro**, who was the guiding light every step of the way as we worked for this project .

Thanks for all instructors for all efforts they did to provide us with all useful information and sharing their knowledge and experience to make from us successful engineers .

Finally, our deep gratitude and sincere thanks to our parents, brothers and sisters for their patience, for everyone who tried to help us during our work and gave us strength to complete this task.

Project Team

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LIST OF ABBREVIATIONS

As	Area Of Non-Prestressed Tension Reinforcement.
As′	Area Of Non-Prestressed Compression Reinforcement.
Ag	Gross Area Of Section.
Av	Area Of Shear Reinforcement Within A Distance (S).
At	Area Of One Leg Of A Closed Stirrup Resisting Tension Within A (S).
b	Width Of Compression Face Of Member.
bw	Web Width, Or Diameter Of Circular Section.
d	Distance From Extreme Compression Fiber To Centroid Of Tension Reinforcement.
Ec	Modulus Of Elasticity Of Concrete.
fy	Specified Yield Strength Of Non-Prestressed Reinforcement.
h	Overall Thickness Of Member.
I	Moment Of Inertia Of Section Resisting Externally Applied Factored Loads.
ln	Length Of Clear Span , Measured Face-To-Face Of Supports In Slabs Without Beams And Face To Face Of Beam Or Other Supports In Other Cases.
M	Bending Moment.
Mu	Factored Moment At Section.
Mn	Nominal Moment.
S	Spacing Of Shear Or In Direction Parallel To Longitudinal Reinforcement.
Vc	Nominal Shear Strength Provided By Concrete.
Vn	Nominal Shear Stress.
Vs	Nominal Shear Strength Provided By Shear Reinforcement.
ρ	Ratio Of Steel Area.
εc	Compression Strain Of Concrete=0.003mm /Mm
Fsd,r	Total Additional Tension Force Above The Support.
Ved,0	Shear Force At Critical Section.
Vu	Factored Shear Force At Section.
Wu	Factored Load Per Unit Length.
Φ	Strength Reduction Factor.

CHAPTER 1

INTRODUCTION

1.1. Introduction
1.2. General Overview
1.3. Project Problem
1.4. Project Objectives
1.5. Work Procedure
1.6. Project Scope
1.7. Project Timeline
1.8. Programs Used In The Project

1.1 Introduction

Civil engineering affects many of our daily activities: the buildings we live in and work in, the transportation facilities we use, the water we drink, and the drainage and sewage systems that are necessary to our health and well-being.

Civil engineers:

1. Measure and map the earth's surface.
2. Design and supervise the construction of bridges, tunnels, large buildings, dams, and coastal structures.

Plan, layout, construct, and maintain railroads, highways, and airports.

1.2 General Overview

We chose one of the hospitals in Yatta , a hospital that specializes in treating cancerous tumors, to provide to the graduation project and to conduct an integrated structural study that includes structural analysis and design of building elements so that they can withstand loads that affect the building.

1.3 Project Problem

The problem of this project in the work of the structural design of the building that was chosen to be the field of this research, where the study was done in the work of a study of the work of equilibrium of the entire building on implementation to avoid any risk to users of this building, and in this project will be analyzed each of the elements of construction such as : beams, columns, foundations, and other structural elements, and determine the loads located on the structural elements of the loads of live or dead loads resulting from the node and the entire elements built in the structure.

why this project was chosen?

There are many reasons that led to the selection of this project, including the reasons as follows :

1 - The project is a specialized hospital that enables us to study and analyze the structural elements in line with the scientific qualifications and skills that we gained through studying in the field of engineering professions.

2- Because this project is widely implemented in our society and the need to implement buildings in an engineering manner.

3- The need to increase the experience and skill of structural design, which we studied .

1.4 Project Objectives

This Project was chosen to achieve the following galls:

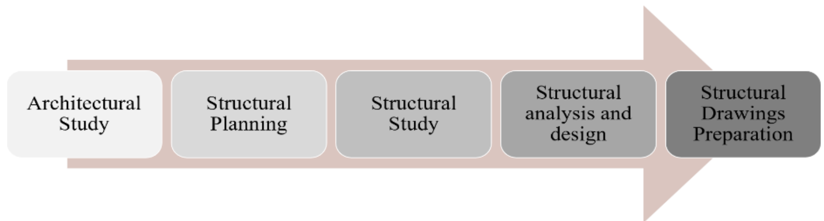
- Correlate the theory that has been gained in the design courses with practical life.
- Get experience in dealing with different problems encountered in the design process.
- Practice the structural analysis and design programs as well as theoretical knowledge.

- 1.5 Work Procedure

To achieve the objectives of the project following steps were followed :

1. Architectural study in which the site, building plans, and elevations were been studied.
2. Structural planning of the building, in which the location of columns, beams, and shear walls was determined to fit with architectural design.
3. Structural study in which all structural members were identified and different loads were been estimated.
4. Starting analysis and design for elements according to the ACI Code.

5. Preparation of Structural drawings of all existing elements in the building.



Figure(1- 1) :Work Procedure

1.6 Project Scope

This Project contains the following Chapters :

CHAPTER 1: Introduction.

CHAPTER 2: Architectural description of the project.

CHAPTER 3: Description of the structural elements.

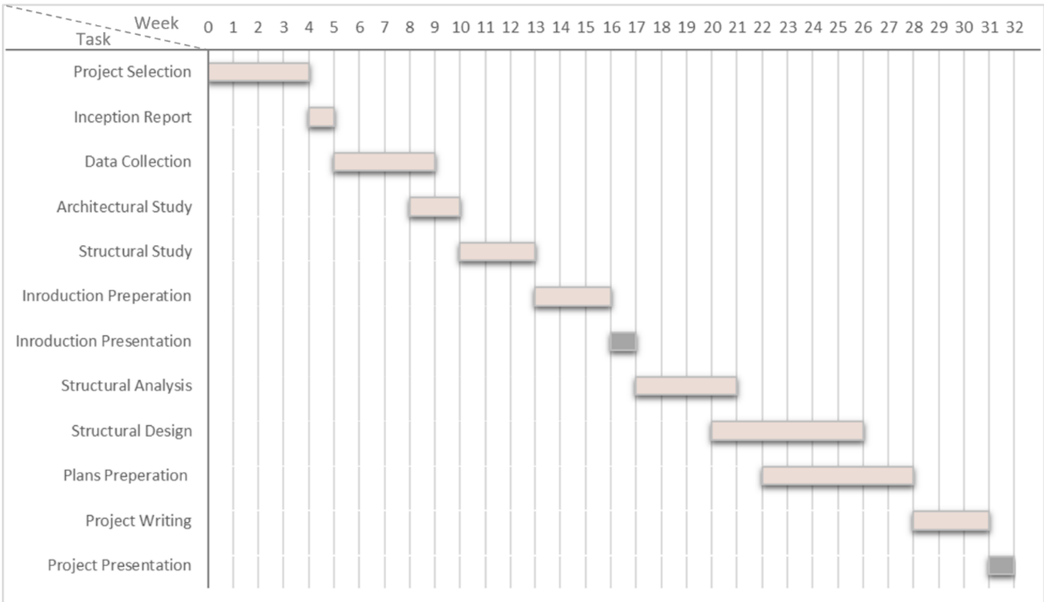
CHAPTER 4: Structural design According Earthquake and Wind (By Etabs& Safe).

CHAPTER 5: Structural analysis and design of all structural elements (By Calculations).

CHAPTER 6: Results and Recommendations.

1.7 Project Timeline

The following chart shows the project plan and timeline :



Figure(1- 2):Project Timeline

1.8 programs used

There are several computer programs used in this project:

1. Microsoft Office: text writing and project output.
2. AUTOCAD 2019: for detailed drawings of structural elements.
3. ATIR18: Structural design and analysis of structural elements.
4. Etabs17:design of structural elements.
5. Safe16: design of slabs & footings .

CHAPTER 2

ARCHITECTURAL DESCRIPTION

2.1. Introduction
2.2. General Identification of the project
2.3. General site description
2.4. Floors Description
2.5. Elevations Description
2.6. Sections of the building

2.1 Introduction

Building any structure is an integrative process between several engineering specializations and the design process for any building takes place through several stages until it is fully accomplished.

An architectural study that must precede the start of architectural design must be easy to handle and understand different events that it contains building and functional relations among them, and the nature of the association movement and using these parts, and other things of importance that give a clear view of the project and therefore it will be possible to locate the columns and other structural elements to suit architectural design.

2.2 General Identification of the project

This hospital is located in the Yatta area on a plot of 15000 m 2 , with an area of approximately 10,000 square meters. It consists of 7floors ; 4 floors over the ground and two underground floors.

The hospital has a main entrance and an emergency entrance on the western side .

The ground floor consists of an emergency place, a reception area, a radiology place, and an outpatient place, with an area of 1293 square meters.

The first floor consists of an early detection place, a review place for cancer patients, and a laboratory place, with an area of 1293 square meters. And the second floor consists of the operations place, the special care place, and the sleep place, and its area is 1380 square meters. And the third floor is the bedroom section, the administration area, the laundry section , and its area is 1293 square meters. As for the fourth floor, it is a complete bedroom section, with an area of 1293 square meters .

There are two underground floors that are used as a car park and complement the health and medical services. The area of each floor is 1678 square meters .

2.3 General site description

The hospital is located in the area of Ziv in the city of Yatta, south of Hebron. It is located about 13 kilometers from the center of Hebron governorate. The area of the hospital is 15 dunums
The hospital is located in an excellent location in this area and is accessible by public transport and several streets connected to it.
The chosen piece is situated at two ends of the city of Yatta, which is surrounded by two streets. The first is directly connected to the center of the main city, and the second is a side street that extends along the length of the ground to reach the piece of land and connects to the main street shown in Figure (2-1).

Figure (2- 1): Site Location

2.4 Floors Description

The project has three types of floors: Two basement, ground floor, and 4 residential floors with a total area of 10000 m 2. The following is a brief description of each floor.

2.4.1 SECOND BASEMENT FLOOR :

The Second basement floor level is 7.5 m below the level of Main Street with an area of 1678 m2. It is used as a parking lot and contains an entrance and exit for cars, a public services department, and an elevator and stairway for transport into the building.

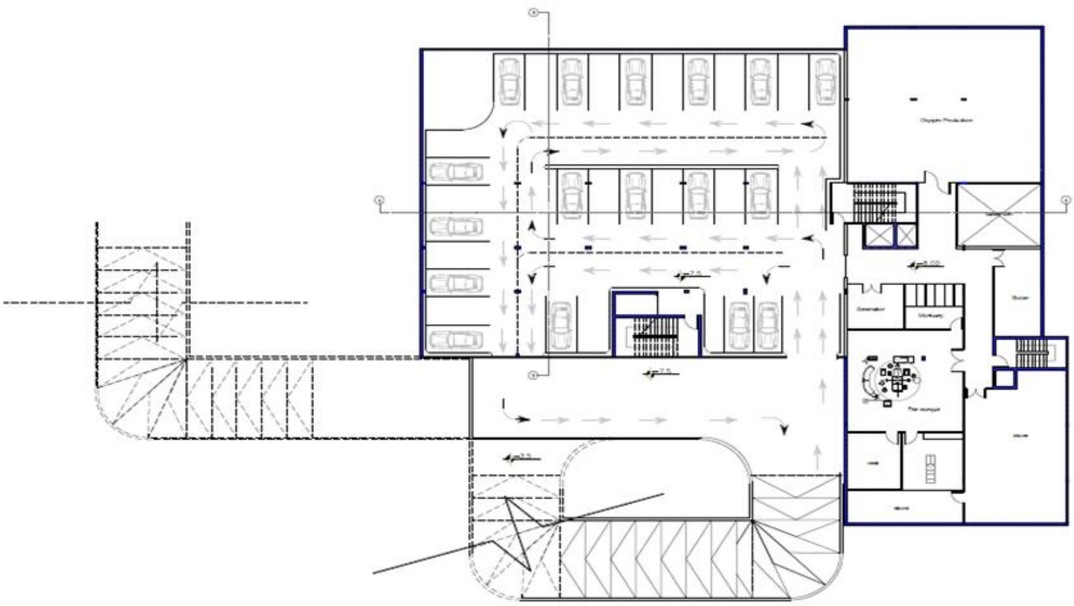


Figure (2- 2): Second basement floor Plan

2.4.2 FIRST BASEMENT FLOOR:

The First basement floor level is 3.5 m below the level of Main Street with an area of 1678 m2. It is used as a parking lot and contains an entrance and exit for cars, a public services section, and an elevator and stairway for transport into the building.

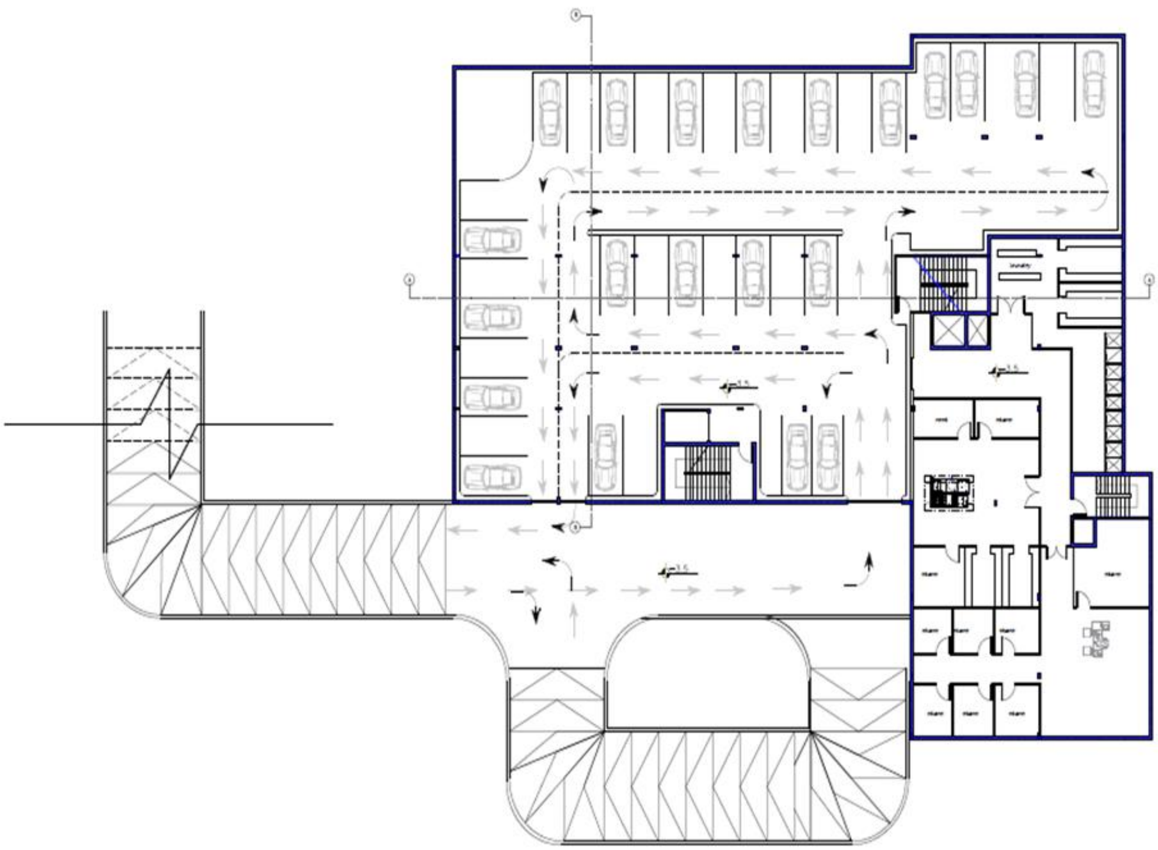


Figure (2- 3): First basement floor Plan

2.4.3 GROUND FLOOR

The ground floor consists of an emergency place, a reception area, a radiology place, and an outpatient place, with an area of 1293 square meters..

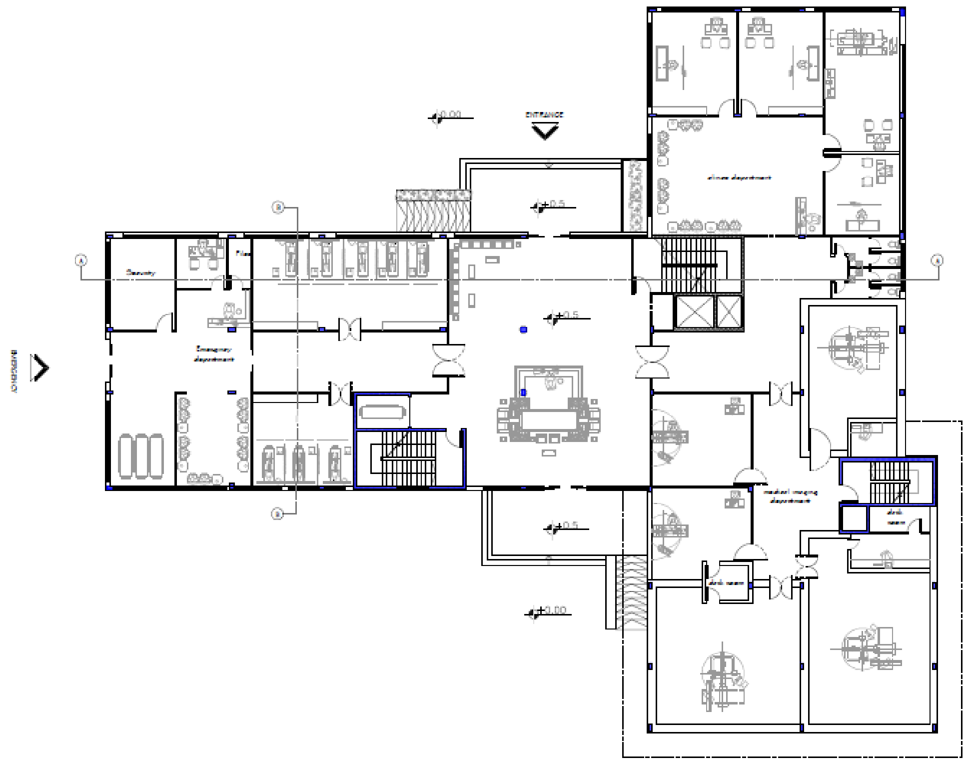


Figure (2- 4):Ground floor Plan

2.4.4 FIRST FLOOR:

The first floor consists of an early detection place, a review place for cancer patients, and a laboratory place, with an area of 1293 square meters.

Figure (2- 5): First floor Plan

2.4.5 SECOND FLOOR:

The second floor consists of the operations place, the special care place, and the sleep place, and its area is 1380 square meters.

Figure (2- 6):Second floor Plan

2.4.6 THIRD FLOOR :

And the third floor is the bedroom section, the administration area, the laundry section , and its area is 1293 square meters.

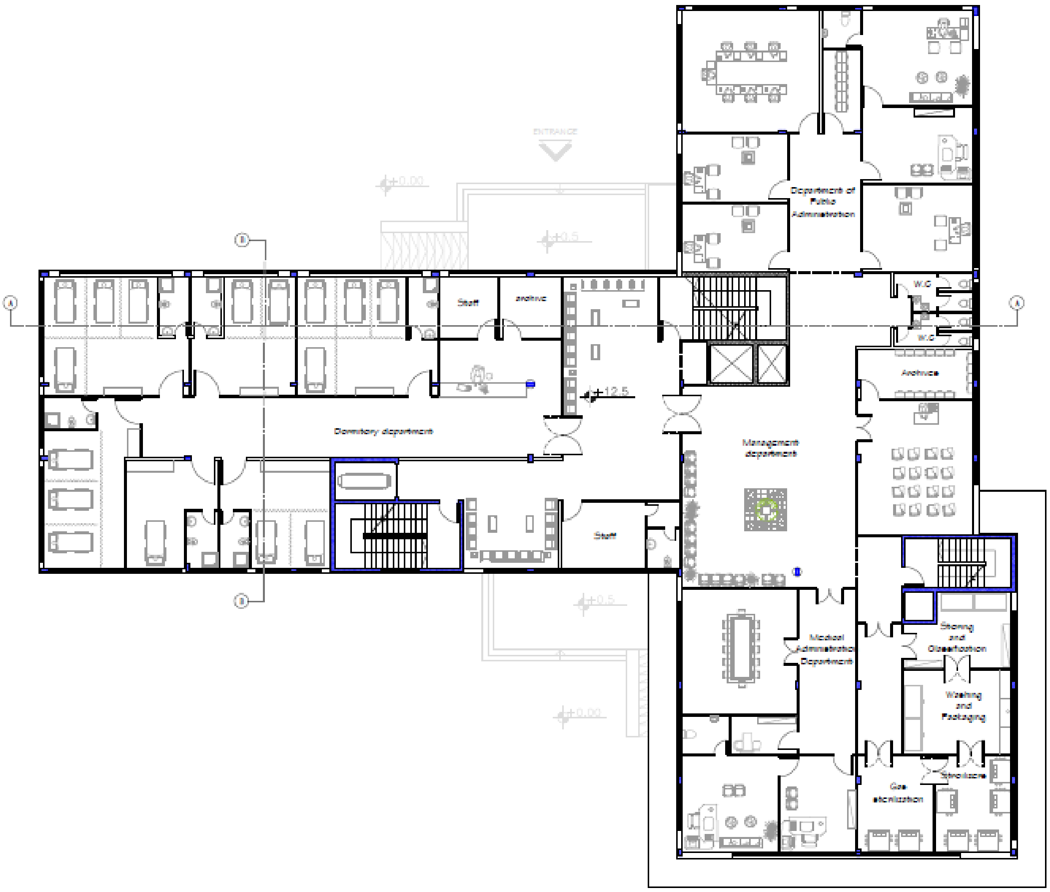


Figure (2- 7):Third floor Plan

2.4.7 FOURTH FLOOR :

The fourth floor, it is a complete bedroom section, with an area of 1293 square meters .

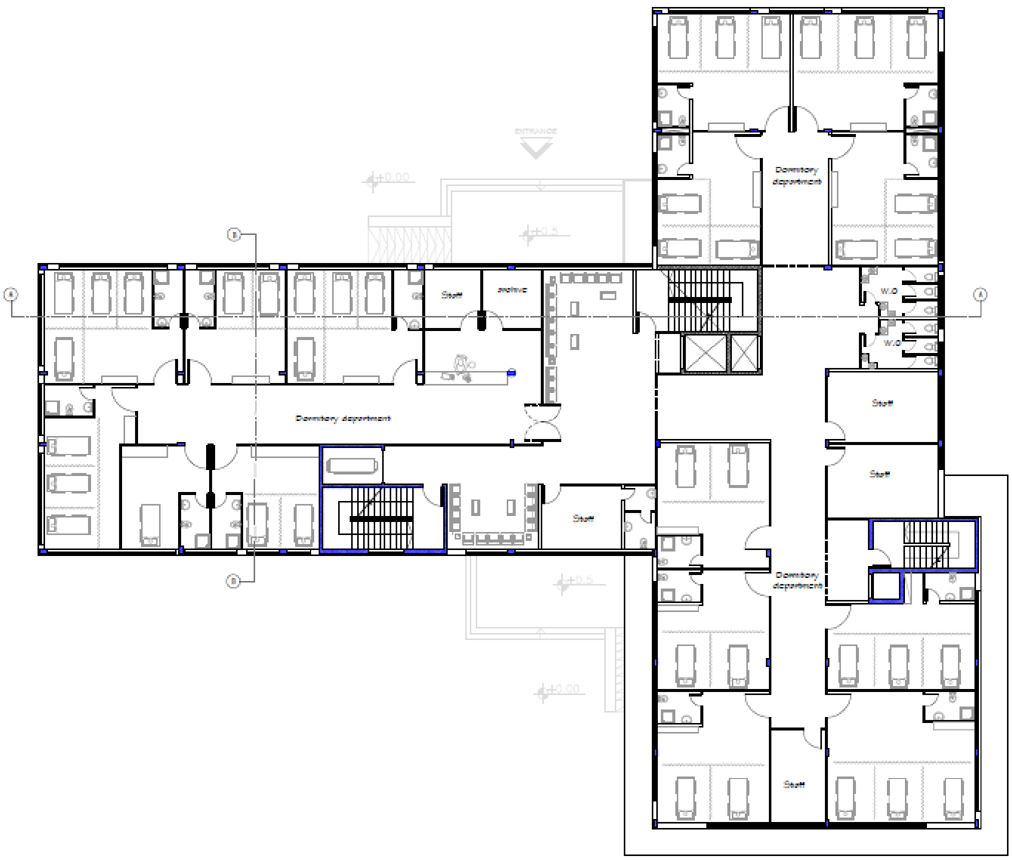


Figure (2- 8):Fourth floor Plan

2.5 Elevations Description

The following is a description of different elements and components of the project elevations :

2.5.1 North elevation :

The northern elevation shows the main entrance to the hospital, and it also shows a repeating staircase facade, in addition to some architectural protrusions.

Figure (2- 9):North elevation

2.5.2 South elevation:

The southern elevation, the second entrance to the hospital appears, in addition to some architectural protrusions and consistency between the openings to give a beautiful appearance to the building.

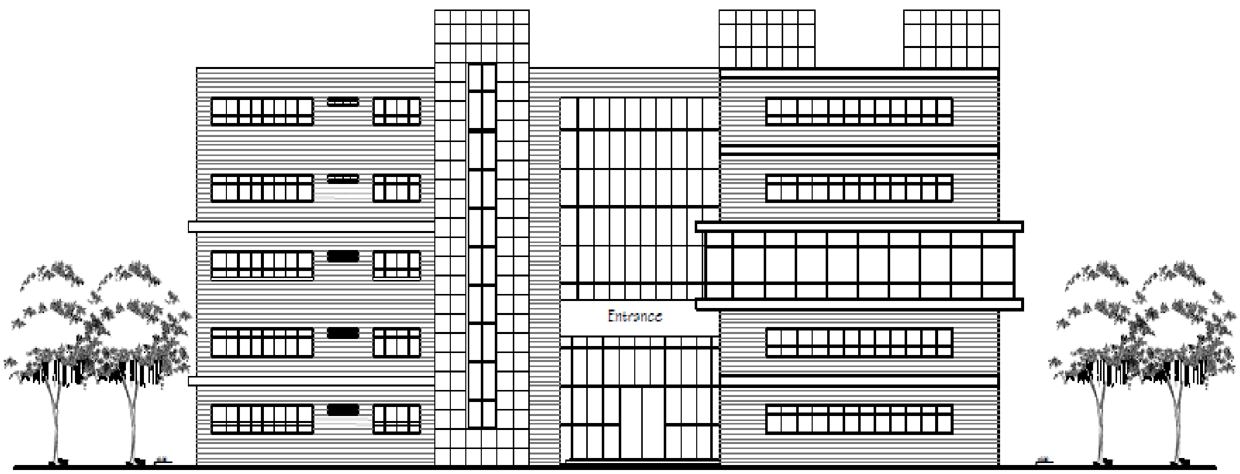


Figure (2- 10):South elevation

2.5.3 East elevation:

The eastern elevation shows a staircase. The staircase shows the architectural protrusion on the second floor clearly with symmetry in the openings and protrusions.

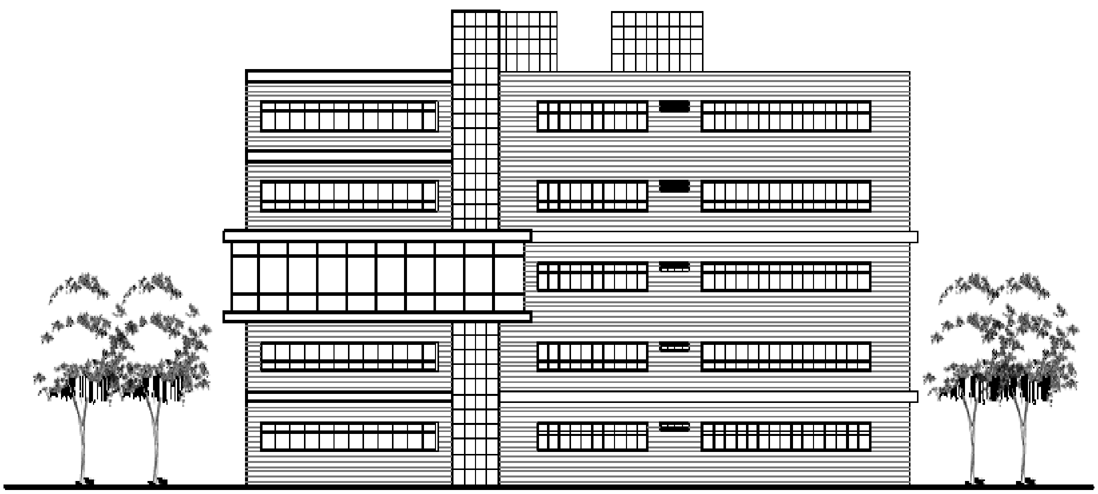


Figure (2- 11): East elevation

2.5.3.1.1.1 West elevation:

The western elevation, the entrance to the emergency department appears with some architectural protrusions and symmetry in the openings to give a harmonious and integrated appearance of the building.

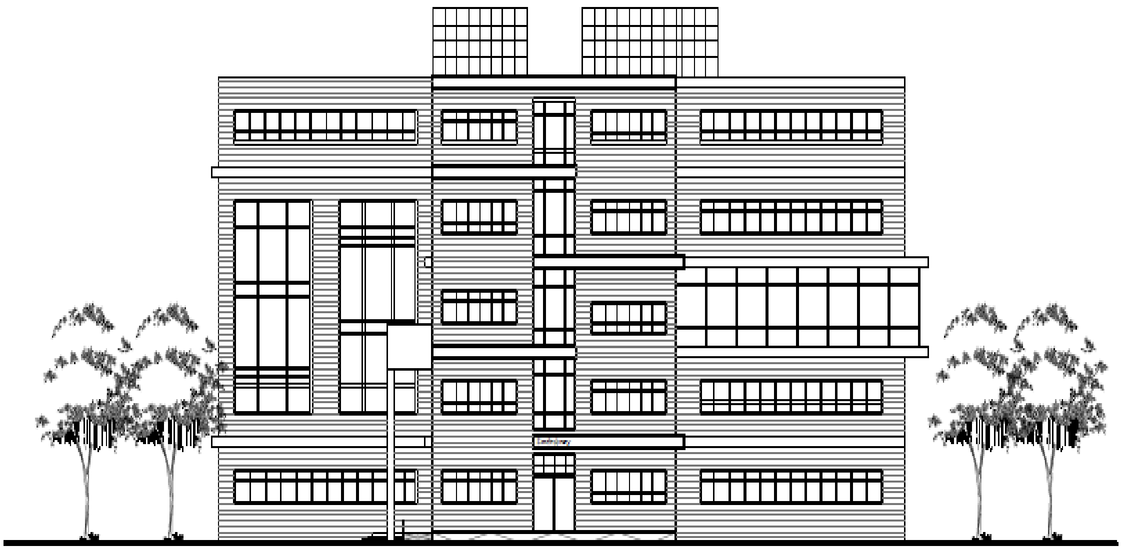


Figure (2- 12): West elevation

2.6 Sections of the building

These sections explain the movement inside the building through the stairs and elevator. It also shows more details for the heights and levels for slabs, windows, and doors.

Figures (2-12) and (2-13) shows two sections of the building.

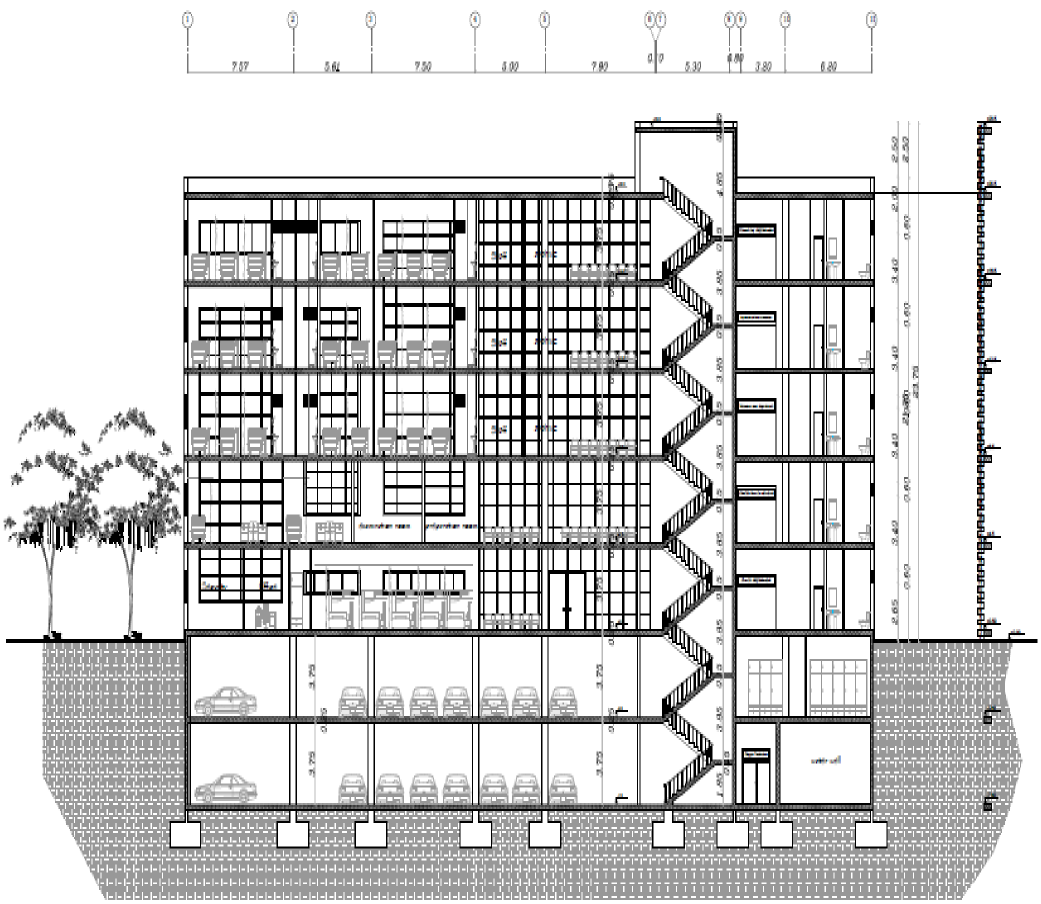


Figure (2- 2):Section A-A

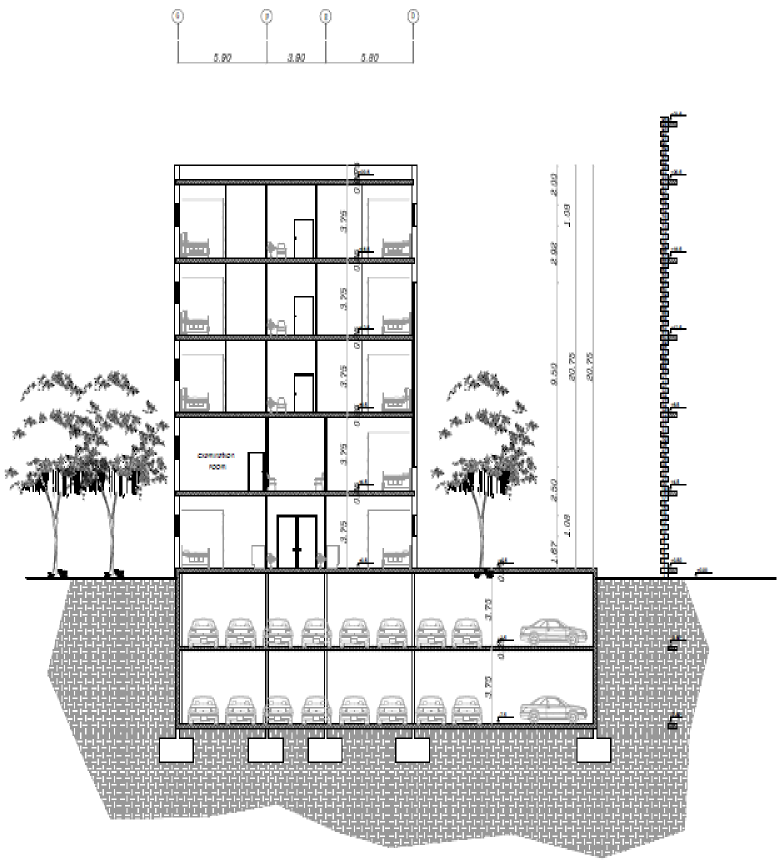
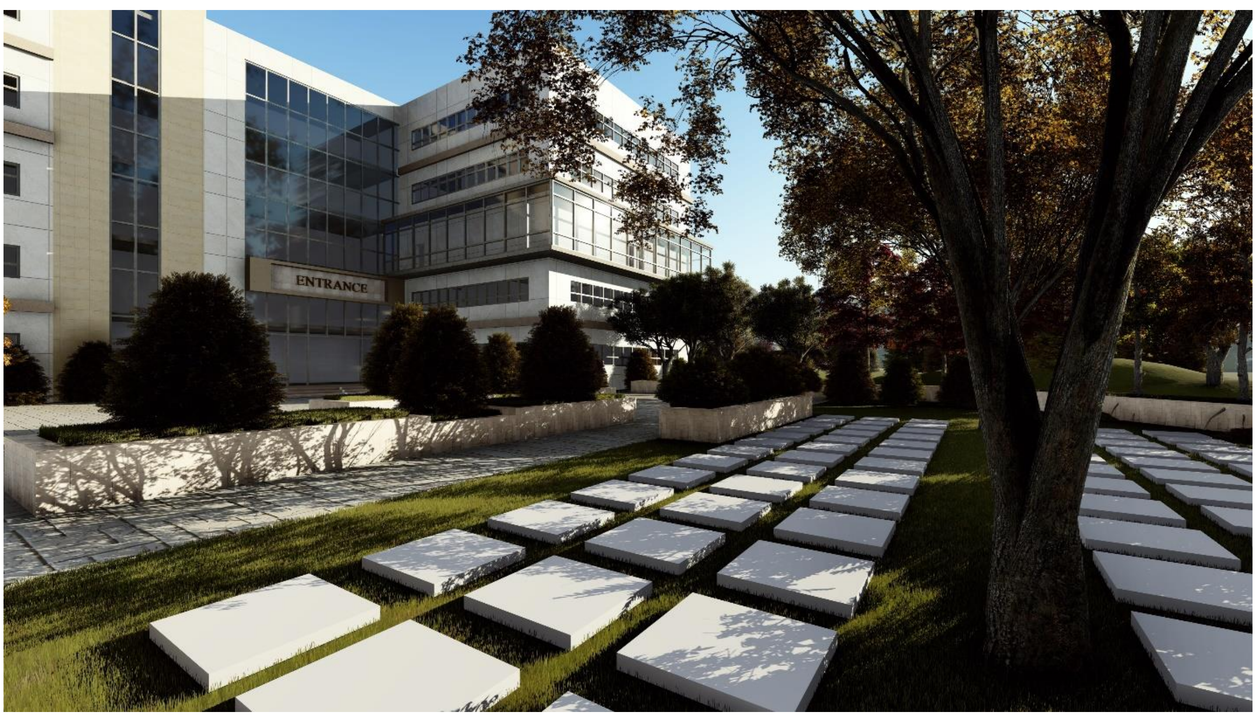


Figure (2- 3):Section B-B

CHAPTER 3

STRUCTURAL DESCRIPTION

3.1 Introduction
3.2 The Aim of the Structural Design
3.3 Scientific Tests
3.4 Loads Acting on the Building
3.5 Structural Elements of the Building (beams and slabs)



3.1 Introduction :

After completion of the architectural study of the building, A study of the structural elements was done to determine the optimal structural system for the building to make the best design of all structural elements.

After the human known the structural design, it was necessary to evolve its structural

design to provide two basic factors, namely safety and economy.

Therefore, it is necessary to identify the structural structures that make up the project

in order to choose the best and optimal elements so as to achieve safety and economy,

in addition to not to conflict with the architectural plans laid down, and the purpose

of the process of structural design is to ensure that the necessary operating

advantages, while preserving as much as possible On the economic factor.

So In this chapter, the structural elements of the project will be identified and explained.

3.2 The Aim of the Structural Design

The main purpose of structural design is to make a safe, economic, and serviceable design, so In designing a structure the following objectives must be taken into consideration :

1- **Safety**: The structure should be able to carry all expected loads safely.
2- **Durability:** The structure should last for a reasonable period of time.
3- **Stability:** to prevent overturning, sliding, or buckling of the structure .
4- **Strength:** to resist safely the stresses induced by the loads in the various structural members.
5- **Serviceability**: To ensure satisfactory performance under service load conditions .

3.3 Scientific Tests

Before the structural study of any building, there is the work of geotechnical studies of the site, which means all work related to exploring the site and studying soil, rocks, and groundwater, then analyzing information and translating it to predict the way the soil behaves when building on it, and the most important thing is to obtaining soil durability (Bearing Capacity) required to design the building's foundations.

3.4 Loads Acting on the Building

3.4.1 dead loads

Dead loads consist of the weight of all materials of construction incorporated into the building .

3.4.2 live load

Live loads are those loads produced by the use and occupancy of the building

3.4.3 wind load

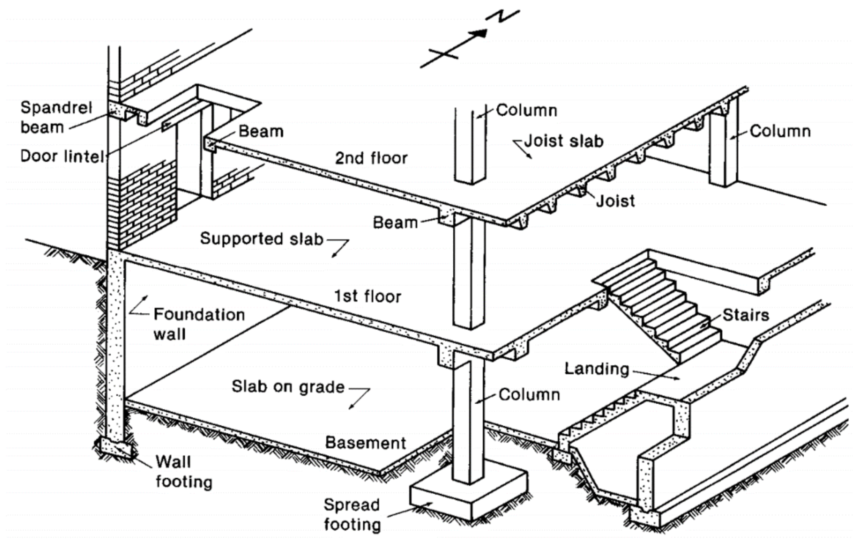
They are horizontal forces that affect the building and their effect appears in tall buildings.

3.4.4 sesimic load

horizontal and vertical forces that generate torque, and can be resisted by using shear walls designed with thicknesses and sufficient reinforcement to ensure the safety of the building.

3.5 Structural Elements of the Building

All buildings usually consist of a set of structural elements that work together to maintain the continuity of the building and its suitability for human use, The most important of these slabs, beams, columns, and load-bearing walls, etc. are being defined.

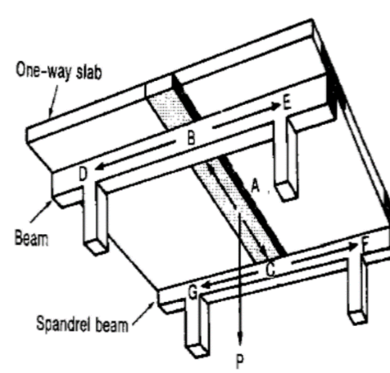


3.5.1 Slabs

Structural elements are capable of delivering vertical forces due to the loads affecting the building's load-bearing structural elements such as beams, columns, and walls, without distortions.
There are many different Structural systems of reinforced concrete slabs, including the following:

3.5.1.1 Solid slab (one or two way)

Solid Slabs are fully customizable concrete slabs of varying width, length, and thickness. They can be used in a variety of applications such as bridges, piers, and building floors. It is known that solid slabs should be supported by drop beams.

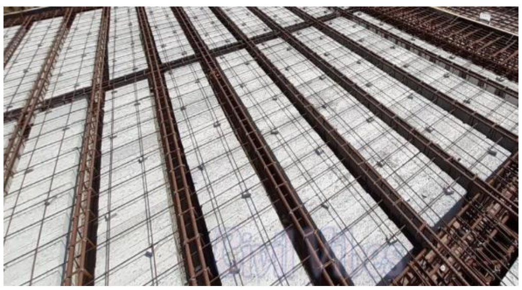


Figure(3- 2): Solid slab

3.5.1.2 Ribbed slab (one or two way)

It’s the most common system used in Palestine. They are made up of wide band beams running between columns with narrow ribs spanning the orthogonal direction. Normally the ribs and the beams are the same depth. A thin topping slab completes the system. It can be designed to carry loads either in one direction only, or in two directions.

Figures (3-5),(3-6) describe one-way and two-way ribbed slabs respectively.

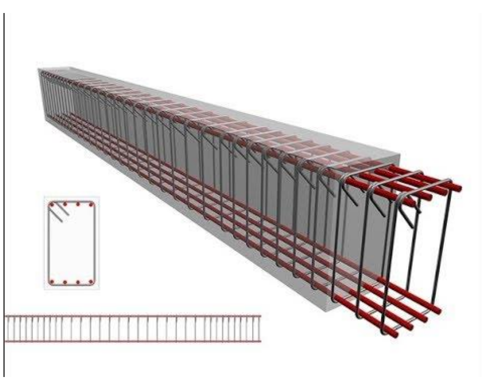


Figure(3- 3): One way ribbed slab

Figure(3- 4):Two way ribbed slab

3.5.2 Beams

They are basic structural elements in transferring loads from slabs to the columns, and they are of two types, hidden inside the slab and Dropped Beams that emerge from the slab from the bottom.



Figure(3- 5):Beams

3.5.3 Columns

Columns are the main member in transporting loads from slabs and beams to foundations, and as such, they are a necessary structural component for conveying loads and building stability. Therefore, they must be designed to be able to carry and distribute the loads on them.

Figure(3- 6):Different types of Columns

3.5.4 Shear walls

They are structural load-bearing elements that resist vertical and horizontal forces located on them and are mainly used to resist horizontal loads such as wind and earthquake forces.

These walls are armed with two layers of steel to increase their efficiency to resist the horizontal forces. The two directions taking into consideration that the distance between the center of resistance formed by the shear walls in each direction and the center of gravity of the building is minimal. And that these walls are sufficient to prevent or reduce the generation of torque waves and their effects on the walls of the building resisting horizontal forces.

3.5.5 Foundations

Loads act on foundations came from the loads on the slabs which transferred to the beams, then to columns, and finally to foundations. and these loads are the design loads for the foundations.

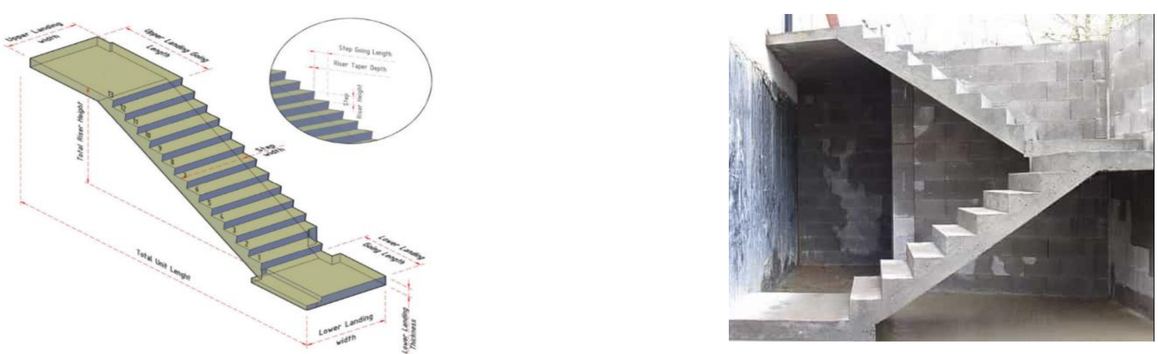
There a many types of foundations that can be used in each project it depends on the type of loads and the nature of the soi in the site.

Figure(3- 7): Isolated Footing

3.5.6 Stairs

Stairs must be provided in almost all buildings. It consists of rises, runs, and landings. The total steps and landings are called a staircase.

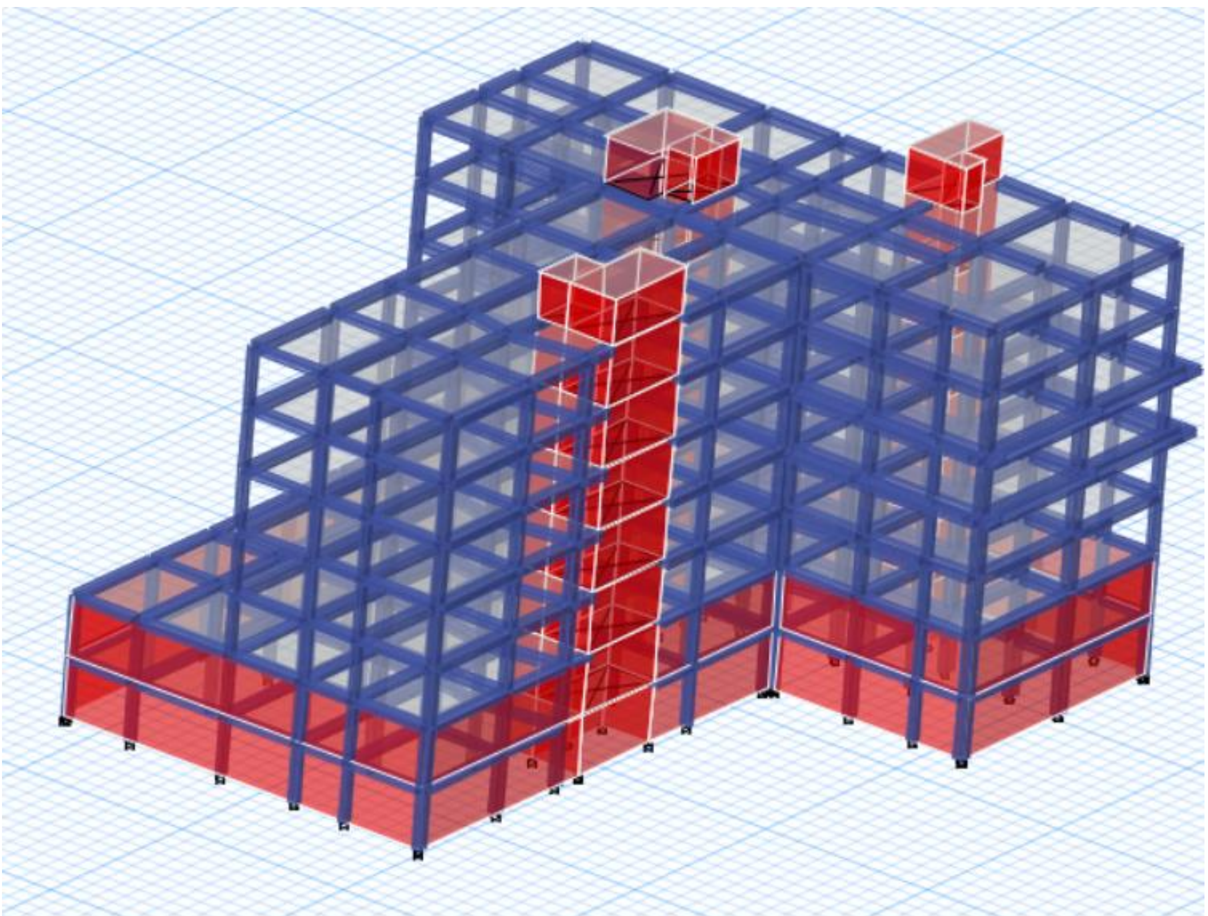
There are different types of stairs, which depend mainly on the type and function of the building and the architectural requirements.



Figure(3- 8): Stairs

CHAPTER 4

Design Of Structural Elements According Earthquake and Wind



4.1 Structural System :

- Structural system : Building Frame Systems .

One way ribbed slabs = 27 Block +8 Topping =35 cm thickness

Method of design : Ultimate design method per ACI 318M-11 code.

- The building will be analyzed using **ETABS & SAFE** Program.

4.2 Codes of practice :

- For estimating dead loads (DL): Jordanian Code.
- For estimating live loads (LL): Jordanian Code.
- For the design and construction requirements of structural concrete ACI 318M-11 Code.
- For the design and construction requirements of earthquake loads UBC -97 code .

4.3 Structural materials :

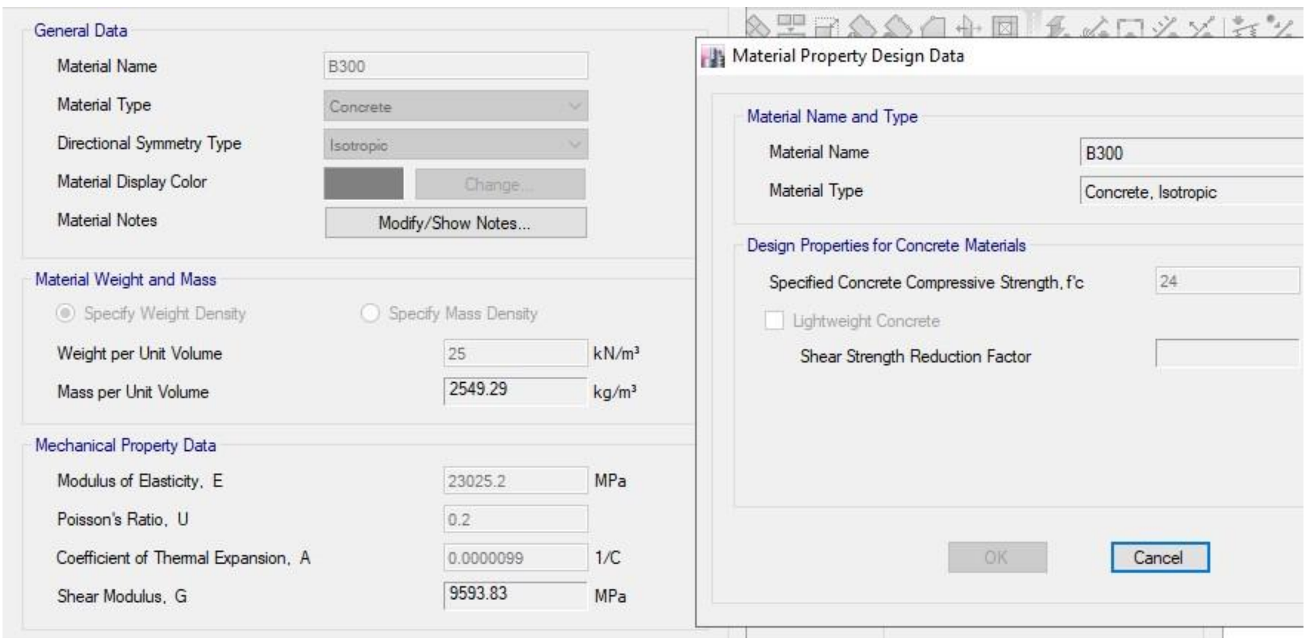
- **Concrete :**

B300

Concrete compressive strength (fc’) : 24 MPa

Modulus of Elasticity = $4700\sqrt{fc'}$ = $4700\sqrt{24}$ = 23025 MPa

Weight per unit volume : 25 KN/m3



- **Steel :**

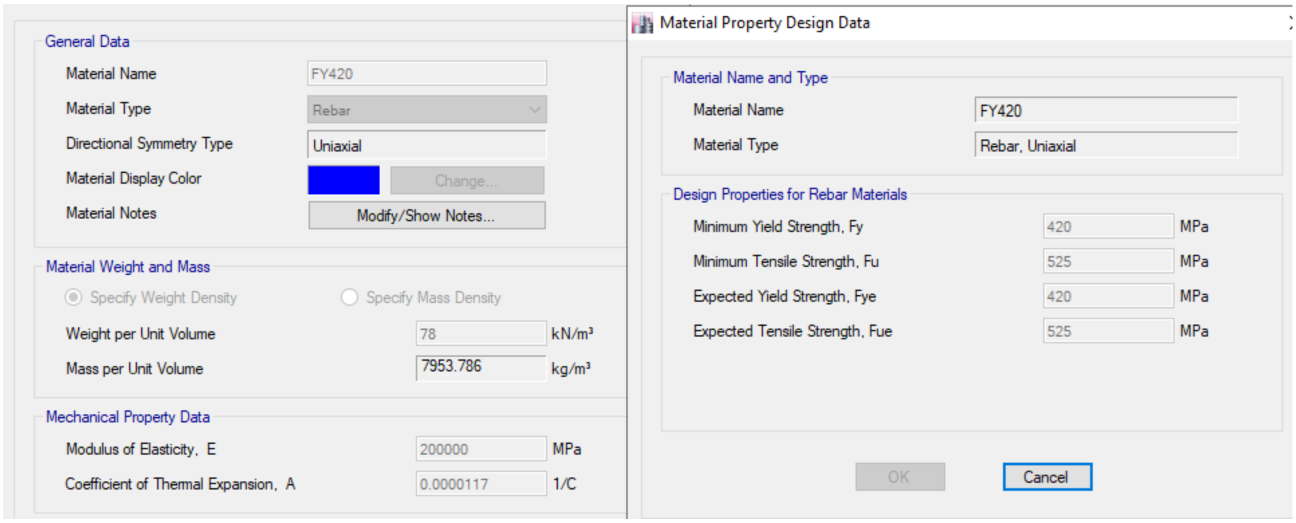
Fy420

Yield strength (fy) : 420 MPa

Tensile strength (fu): 1.25 X fy = 1.25 X 420 = 525 MPa

Modulus of Elasticity = 200000 MPa

Weight per unit volume : 78 KN/m3



4.4 Member properties used for analysis :

- The member section properties used for an analysis should take in to account then fluency of cracking and duration of loads. To do so, ACI code permits use of the following properties for the members of the structure:

Beams :(EI)b=0.35 EcIg

Columns :(EI)c=0.70 EcIg

Shear walls (EI)s.w=0.70 EcIg

Slabs (EI)s=0.25 EcIg

- Where,

Ig: moment of inertia of gross section.

Ec: modulus of elasticity of concrete.

4.5 Soil Bearing Capacity :

- The foundations are designed with **allowable bearing capacity** equal to **<u>450</u>**

  <u>**Kg/cm²** for isolated & continuous foundations .</u>

4.6 Earthquake Data

Seismic zone	**SA**
Seismic zone factor (Z)	**0.15**
Soil profile type	**SA**
Seismic coefficient (ca)	**0.15**
Seismic coefficient (cv)	**0.15**
Importance factor (I)	**1**
Response modification factor (R)	**5.5**

- **SUPERIMPOSED DEAD LOAD**

According Jordanian Code the density and thickness of material :

Material	**Density (**KN/ m3)	**Thickness** (mm)
Tiles	**23**	**30**
Mortar	**22**	**30**
Sand	**17**	**70**
Plaster	**22**	**30**
Hollow Block	**10**	**27**

- **Live LOAD**

According Jordanian Code :

Live load **5** KN/ m2

Applied Loads :

- **Superimposed dead load:**

Material	Density (KN/ cm3)	Thickness (m)	Flange width(m)	Σ(KN/ rib)
Tiles	**23**	**0.03**	**0.54**	**0.3726**
Mortar	**22**	**0.03**	**0.54**	**0.3564**
sand	**17**	**0.07**	**0.54**	**0.6426**
Block	**10**	**0.27**	**0.40**	**1.08**
Plaster	**22**	**0.03**	**0.54**	**0.3464**
Partions		**2.3**	**0.54**	**1.242**
Sum				***=4.05/0.54***
SUPERIMPOSED DEAD LOAD				***=7.50/M2***

- **LIVE LOAD = 5 KN/m2**

- **Self weight calculate by program :**

Tooping = 25 X 0.08 X0.54 = 1.08

Rib =25X0.14X0.27 =0.945

SUM = 2.025/0.54 =3.75 KN/M2

Primary load

- SW : Self weight dead load
- SD : Superimposed Dead Load
- L : Live load
- EQX : Earthquake Load in (X)direction
- EQY : Earthquake Load in (Y)direction
- EQX±e: Earthquake Load in (X)direction± Eccentricity
- EQY±e: Earthquake Load in (Y)direction± Eccentricity

LOAD COMBINATION

- **For Analysis :**
  - D±0.72EQ
  - D±L±0.72EQ
- **For Design (Ultimate load combination) :**
  - 1.4D
  - 1.2D+1.6L
  - 0.9D± EQ
  - 1.2D±0.5L±1.5EQ

4.7 Analysis element structure

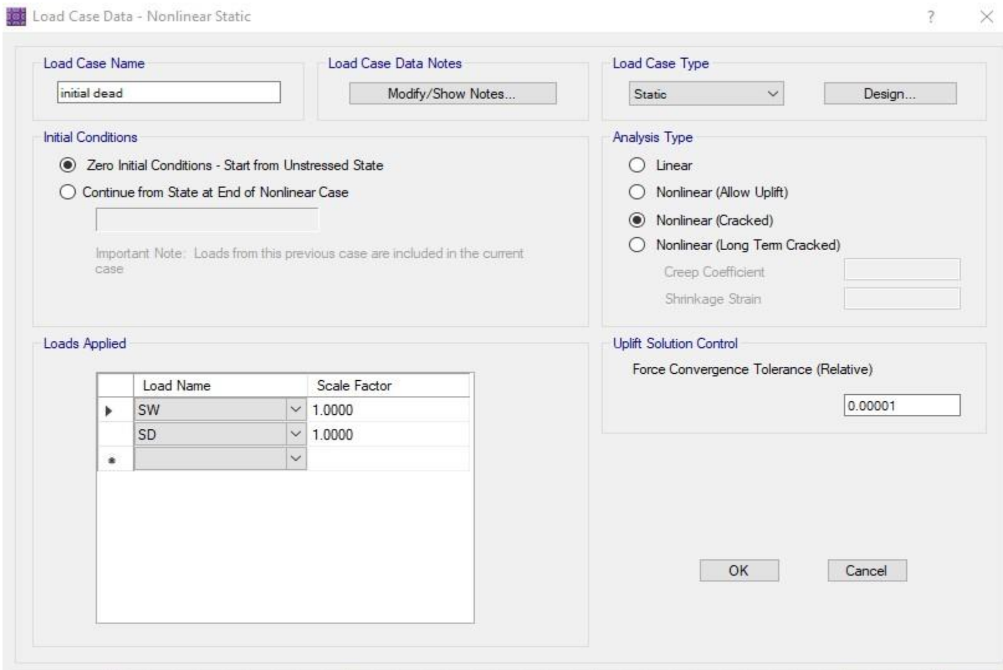
Long-term deflections of reinforced concrete structures

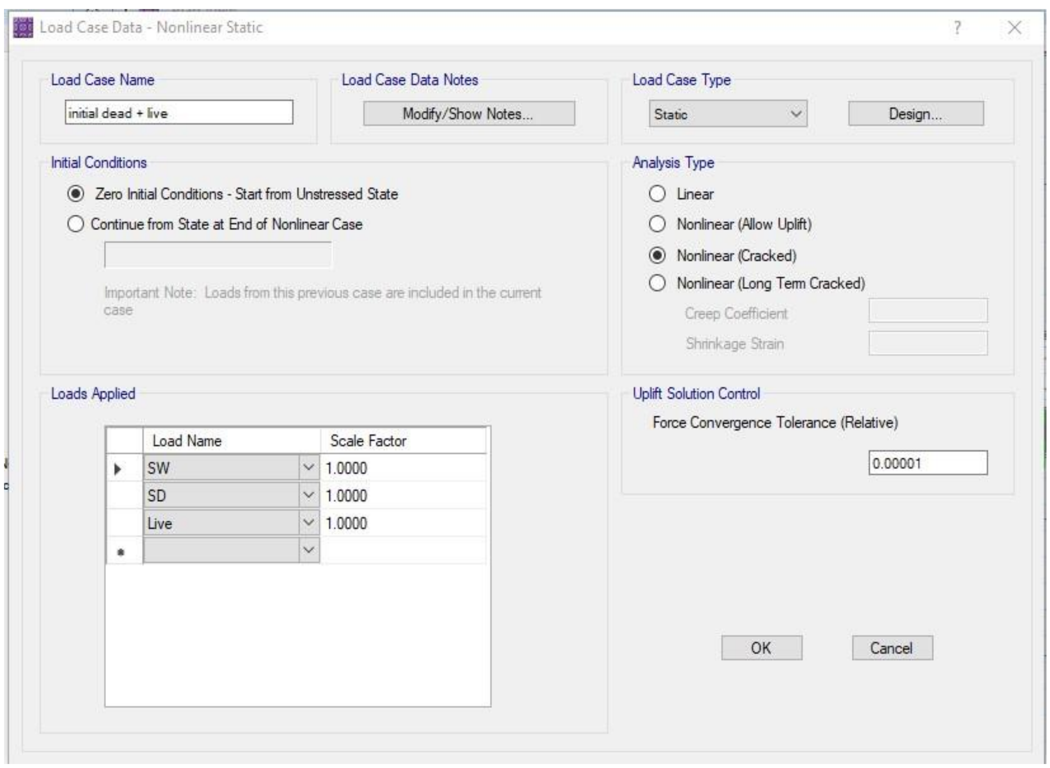
Simplified methods are available for prediction of long-term deflection of beams and slabs. Long-term deflection is a combination of initial elastic deflection ,long-term elastic deflection, deflection due to cracking ,deflection due to shrinkage, and deflection due to creep of concrete

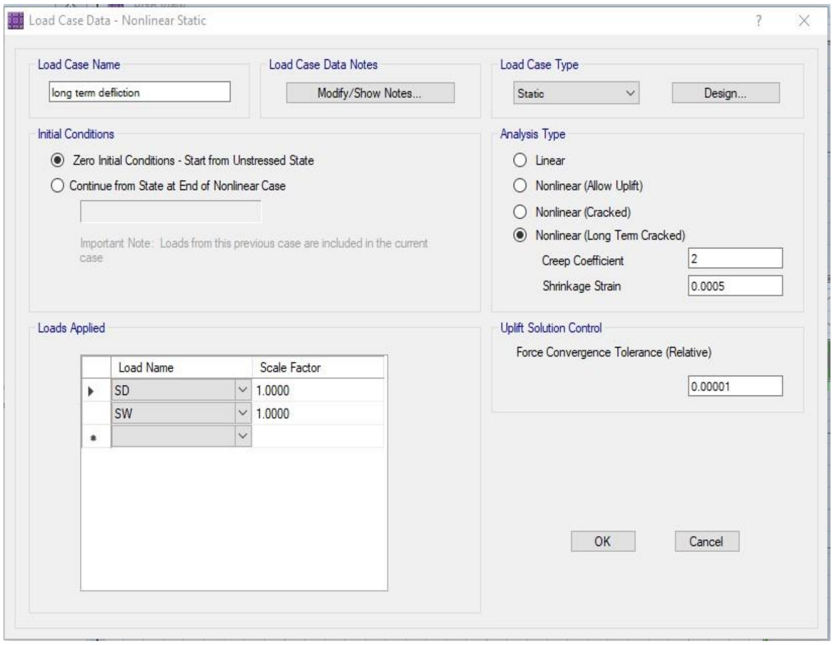
The initial elastic deflection is the deflection that occurs under the slab dead load when formwork is removed.

The long-term elastic deflection is the deflection that occurs under live load and the finishing or construction loading ,without producing a cracked section. The deflection due to shrinkage occurs when one surface of the concrete slab/beam is reinforced and does not shorten appreciably and the opposite surface is essentially unreinforced and does shorten due to concrete shrinkage. The deflection due to creep occurs as the concrete is stressed under sustained loading. It increases with time and also with amount of loading.

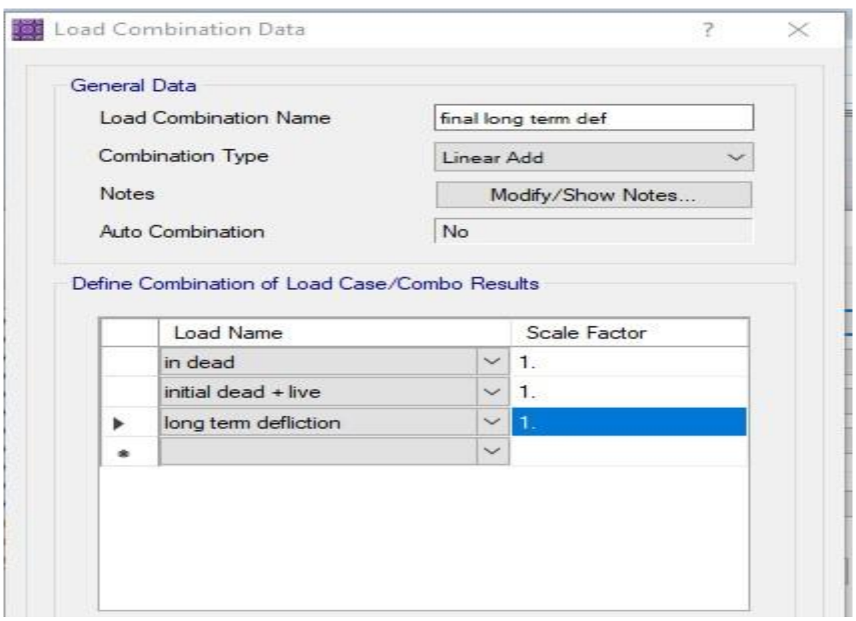
Using safe program : Define load Cases







Using safe program : Define load Combination



After have the result for program ,
the deflections are to be computed, deflections that occur immediately on application of load shall be computed by formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness

TABLE 9.5(b) — MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/180^{*}$
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load ***L***	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell/480^{\ddagger}$
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240^{\S}$

*Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

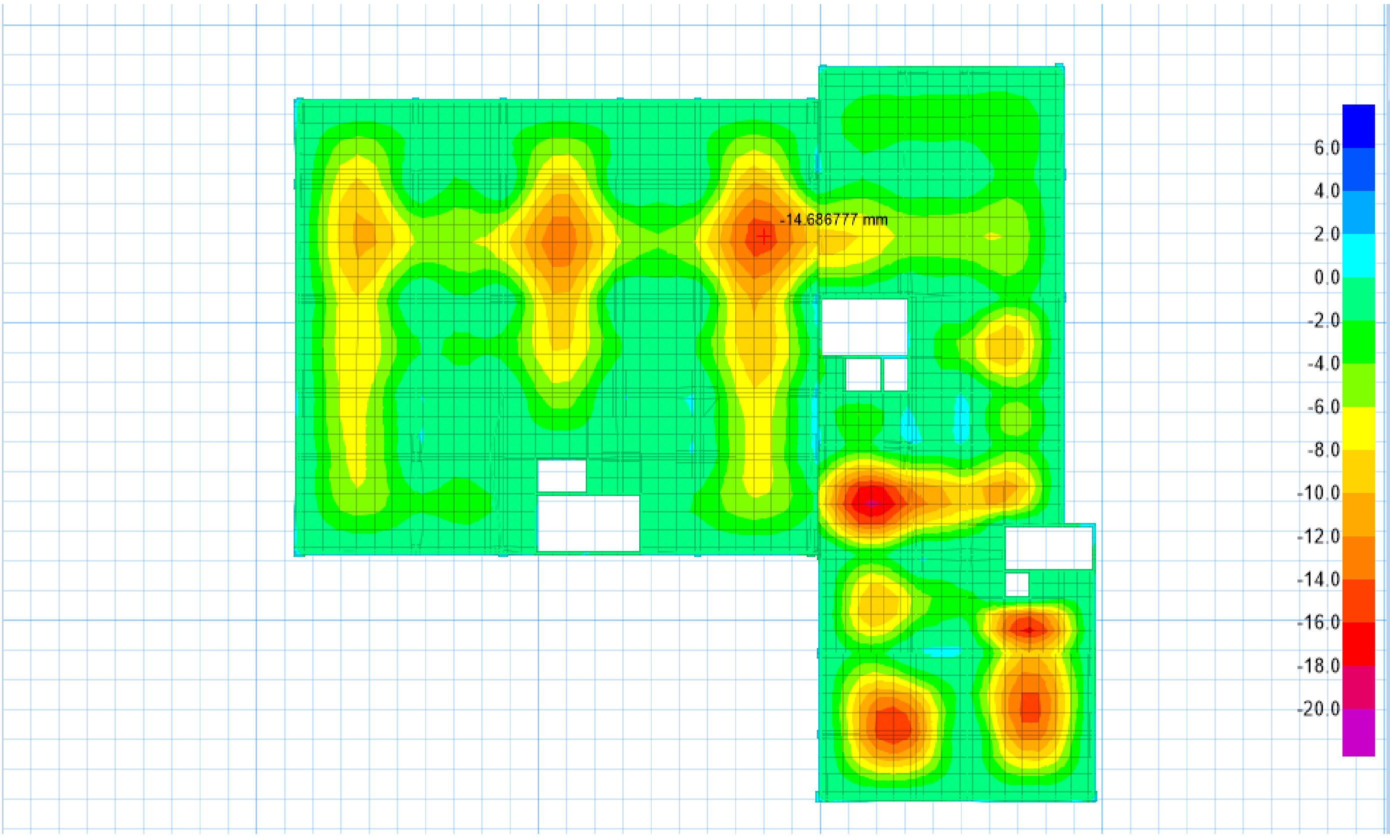
†Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

Check Long term deflection , basement 2 floor slab :

USING SAFE PROGRAM :



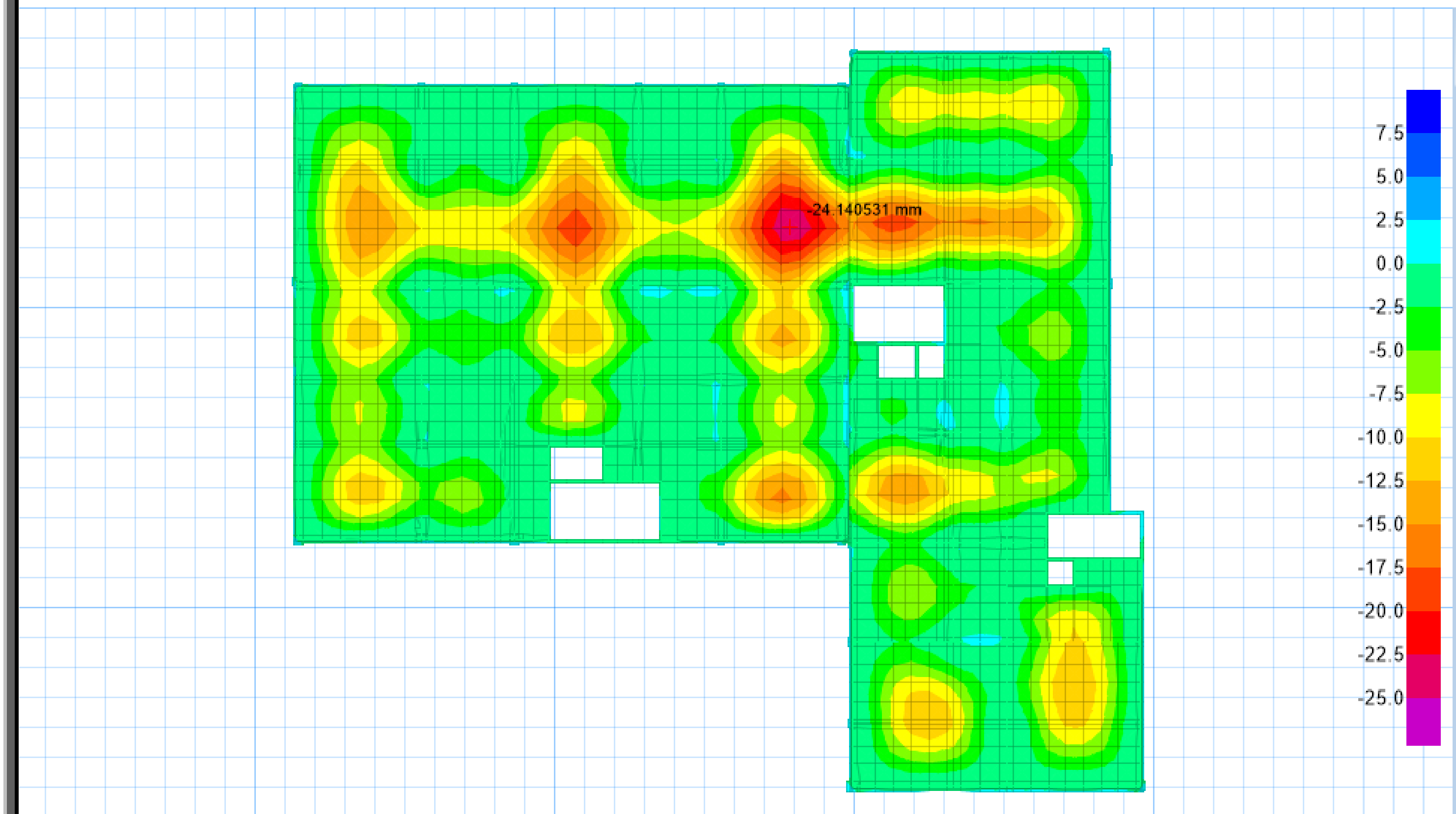
Span on max deflection : 7.50 m

max deflection = $\frac{L}{240} = \frac{7500}{240}$ =31.25mm

31.25 mm> 16.942 mm , the solid slab thickness are enough .

Check Long term deflection , basement 1 floor slab :

USING SAFE PROGRAM :



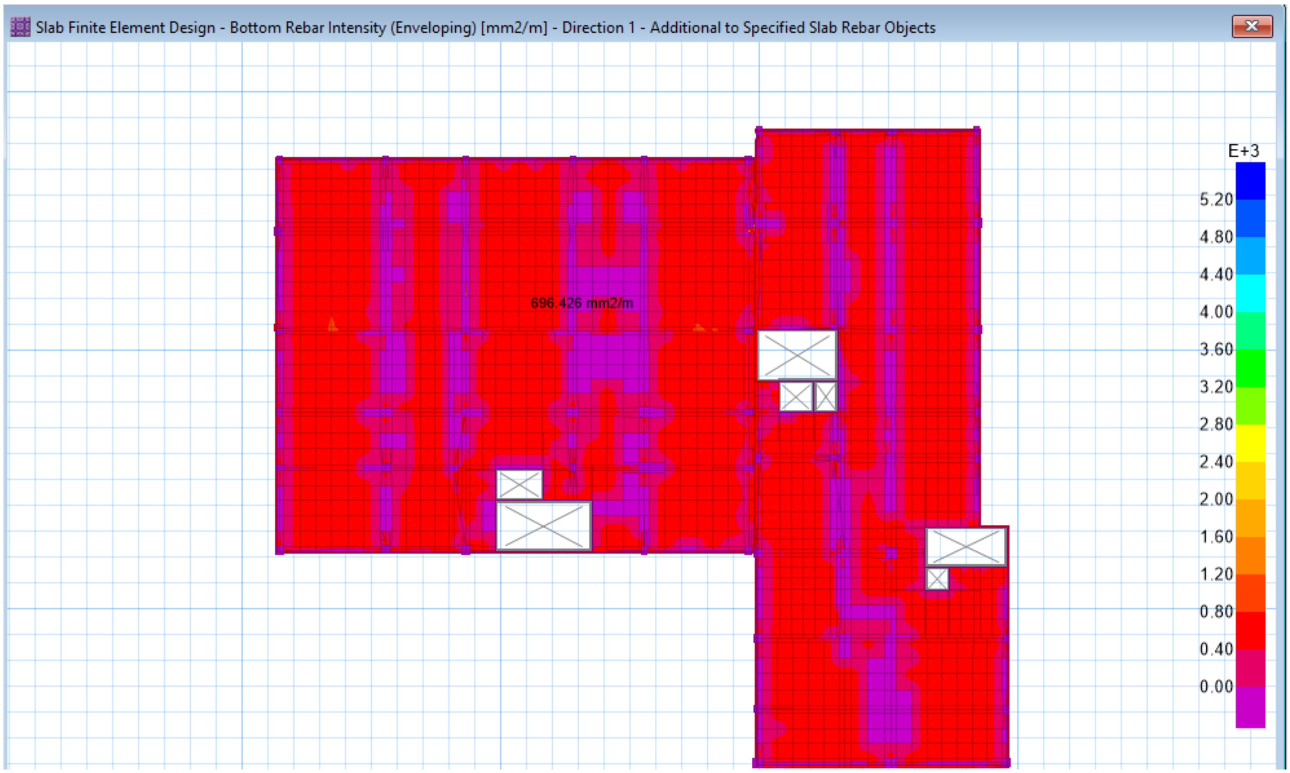
Span on max deflection : 7.50 m

max deflection = $\frac{L}{240}$ =$\frac{7500}{240}$ =31.25mm

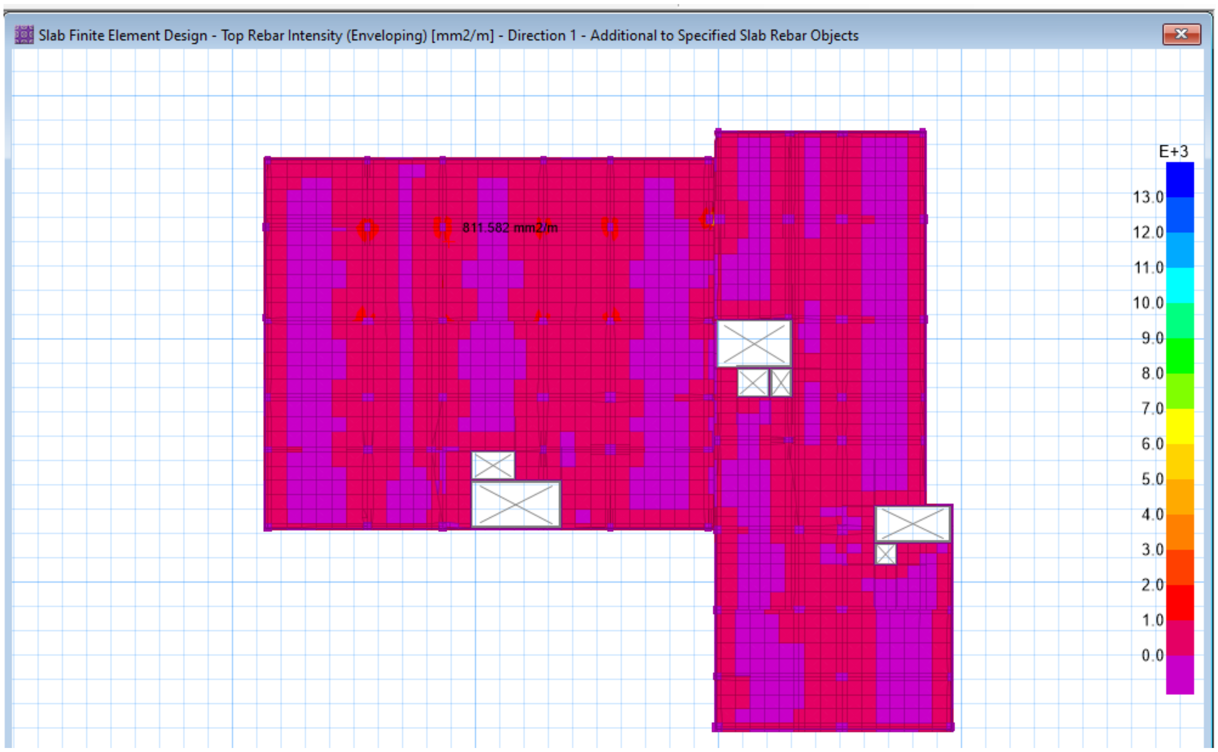
31.25 mm> 16.942 mm , the solid slab thickness are enough .

4.7.1 Design of Solid Slab :

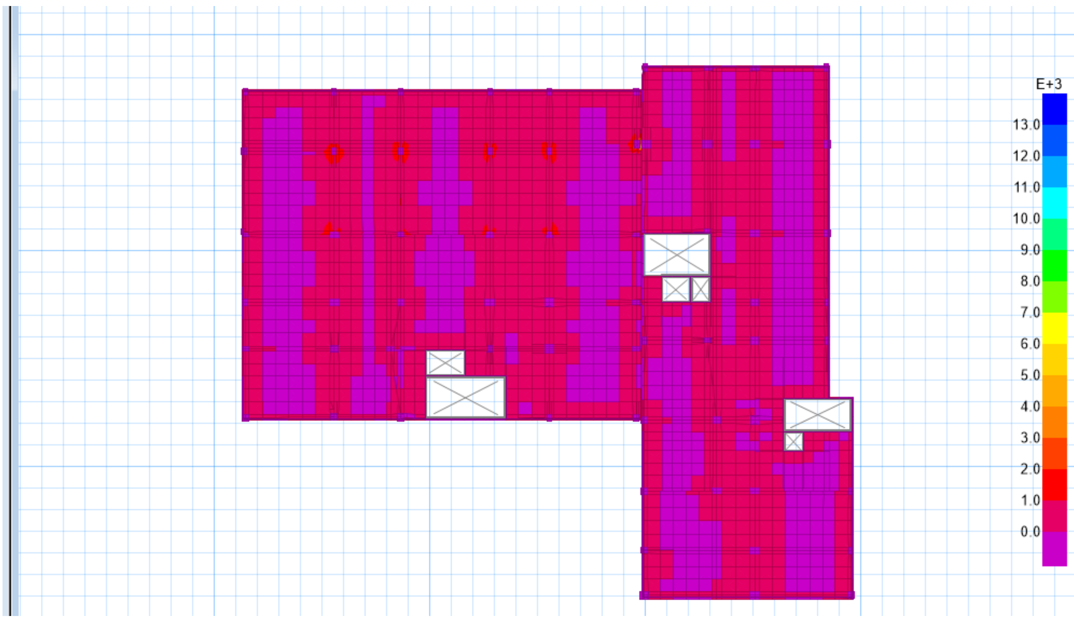
Flexure analysis :
Basic Mesh bottom reinforcement (#ø14@15cm) which equal area / cm 154/0.15 = 1026.67mm2/m



Basic Mesh Top reinforcement (#ø14@15cm) which equal area / cm 154/0.15 = 1026.67 mm2/m
Need additional top reinforcement !

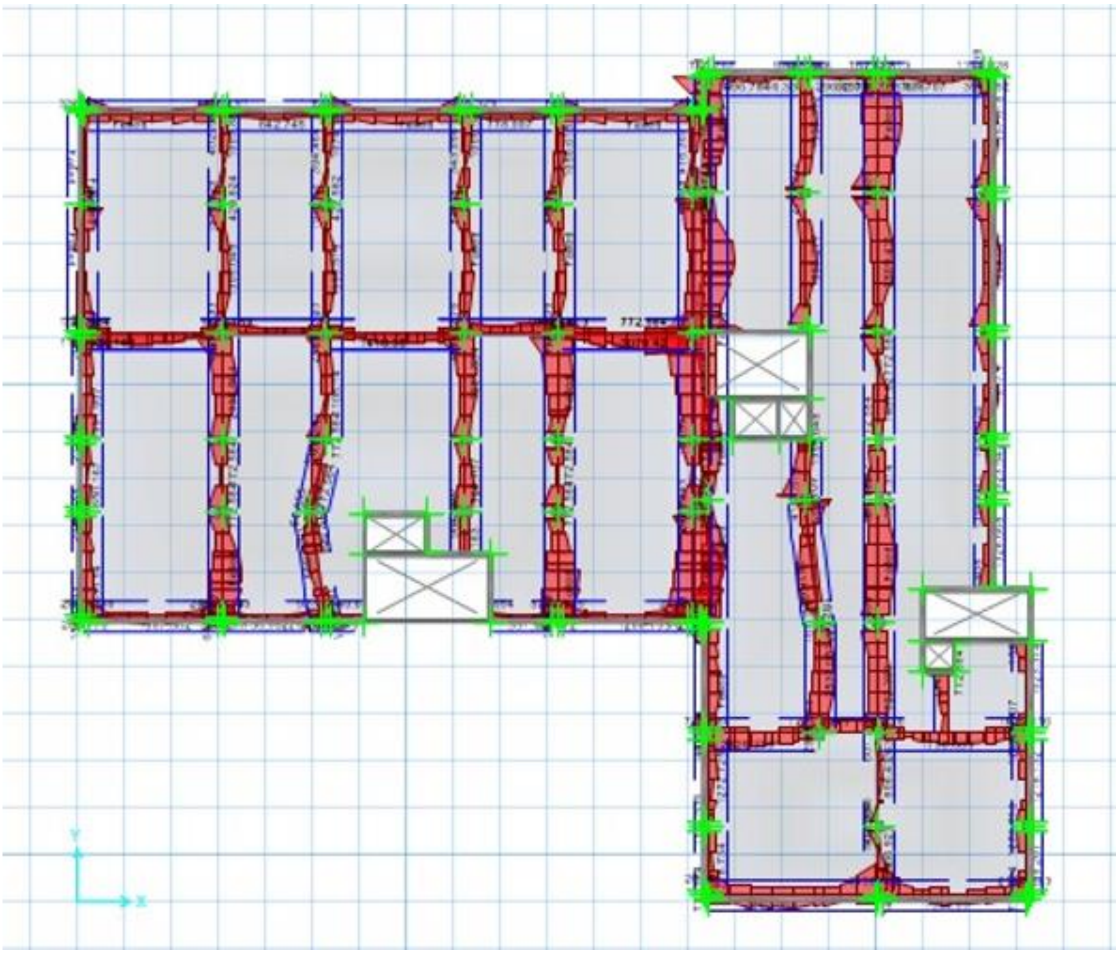


Additional top reinforcement (#ø14@15cm) which equal area / cm 154/0.15 = 1026.67 mm2/m PLUS (#ø18@15cm)) which equal area / cm 254.4/0.1 = 2544mm2/m
1026.67+2544 = 3570 mm2/m Which cover !



4.7.2 DESIGN BEAM

Safe 2016 Beam Design



4.7.3 Design of Column

ETABS 2017 Concrete Frame Design

Column Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
basement 1	C54	522	Column600*600	1.2D+1.6L	0	4000	0.424	Sway Intermediate

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
600	600	60	27.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
23025.2	24	1	420	420

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
4669.5313	-62.943	155.2152	155.2152	155.2152	4271	1.19

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Effective Length mm
Major Bend(M3)	0.314761	1	1	1	3500
Minor Bend(M2)	0.309991	1	1	1	3500

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear ΦV_c kN	Shear ΦV_s kN	Shear ΦV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	66.3331	383.6605	0	0	0
Minor, V_{u3}	31.0223	383.6605	0	41.5476	0

	Joint Shear Force kN	Shear $V_{u,Top}$ kN	Shear $V_{u,Tot}$ kN	Shear ΦV_c kN	Joint Area cm²	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(6/5) Beam/Column Capacity Ratio

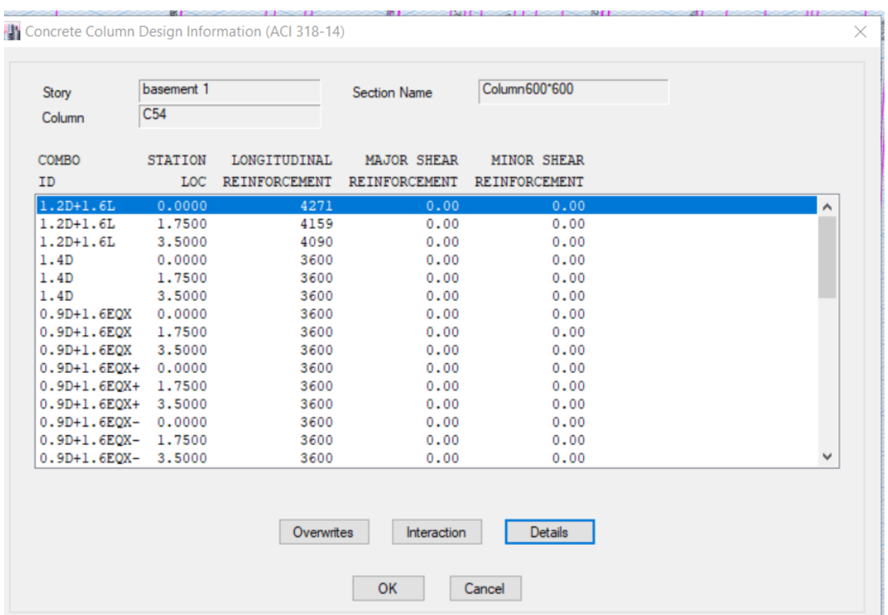
Major Ratio	Minor Ratio
N/A	N/A

Notes:

N/A: Not Applicable

N/C: Not Calculated

N/N: Not Needed


Concrete Column Design Information (ACI 318-14)

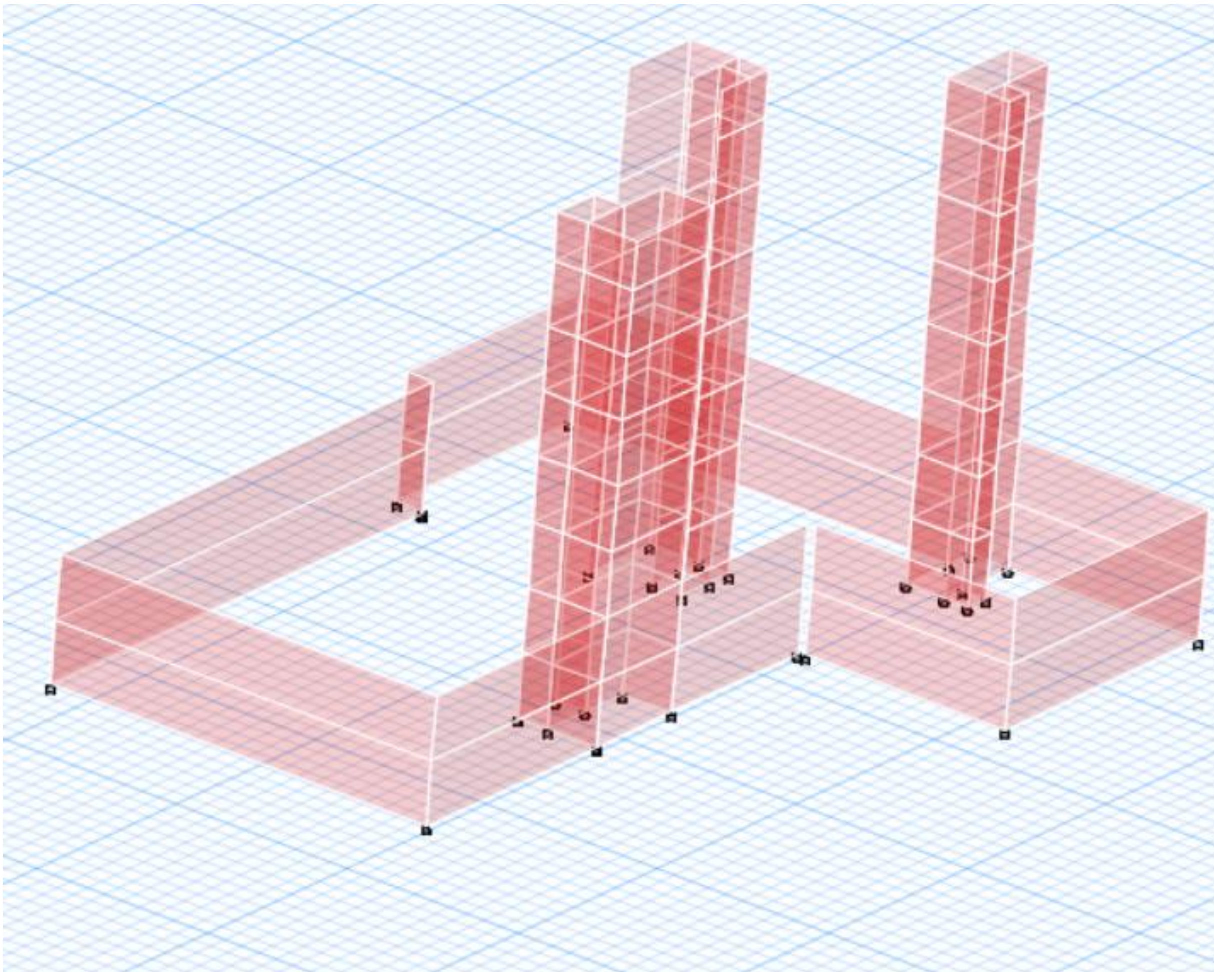
Story: basement 1
Column: C54
Section Name: Column600*600

COMBO ID	STATION LOC	LONGITUDINAL REINFORCEMENT	MAJOR SHEAR REINFORCEMENT	MINOR SHEAR REINFORCEMENT
1.2D+1.6L	0.0000	4271	0.00	0.00
1.2D+1.6L	1.7500	4159	0.00	0.00
1.2D+1.6L	3.5000	4090	0.00	0.00
1.4D	0.0000	3600	0.00	0.00
1.4D	1.7500	3600	0.00	0.00
1.4D	3.5000	3600	0.00	0.00
0.9D+1.6EQX	0.0000	3600	0.00	0.00
0.9D+1.6EQX	1.7500	3600	0.00	0.00
0.9D+1.6EQX	3.5000	3600	0.00	0.00
0.9D+1.6EQX+	0.0000	3600	0.00	0.00
0.9D+1.6EQX+	1.7500	3600	0.00	0.00
0.9D+1.6EQX+	3.5000	3600	0.00	0.00
0.9D+1.6EQX-	0.0000	3600	0.00	0.00
0.9D+1.6EQX-	1.7500	3600	0.00	0.00
0.9D+1.6EQX-	3.5000	3600	0.00	0.00

Overwrites Interaction Details

OK Cancel

4.7.4 Design of Shear Wall , according Earthquake loads



ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
basement 1	P8	4078202.3	1849948.4	4500	250	0.655

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
23025.2	24	1	420	420

Design Code Parameters

Φ_T	Φ_C	Φ_v	Φ_v (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	4078202.3	1847698.4	4078202.3	1852198.4	4500	250
Bottom	Leg 1	4078202.3	1847698.4	4078202.3	1852198.4	4500	250

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	9615	0.0085	0.0027	0.9D+1.6EQX+e	-2263.6249	-8.5163	-2215.73	1125000
Bottom	8109	0.0072	0.0027	0.9D+1.6EQX-e	-2631.975	0.6579	104.5682	1125000

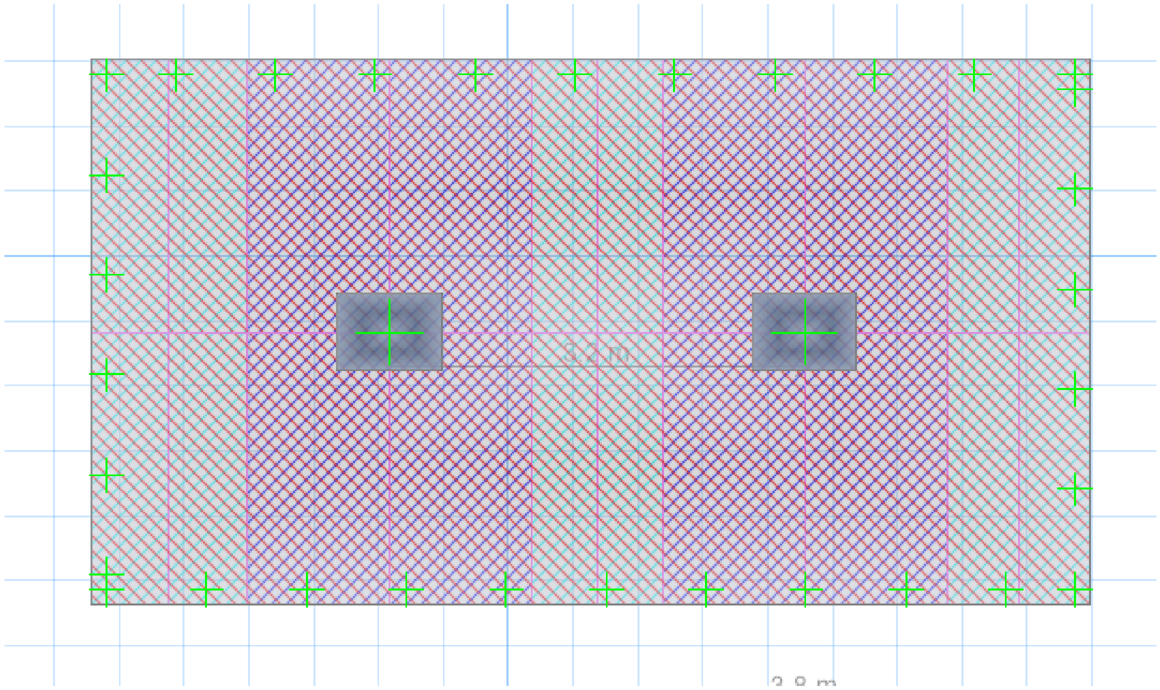
Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ΦV_c kN	ΦV_n kN
Top	Leg 1	1023.85	0.9D+1.6EQY+e	602.38	7217.385	1967.3307	823.74	1967.3307
Bottom	Leg 1	1023.85	0.9D+1.6EQY+e	703.63	-651.9377	1967.3307	823.74	1967.3307

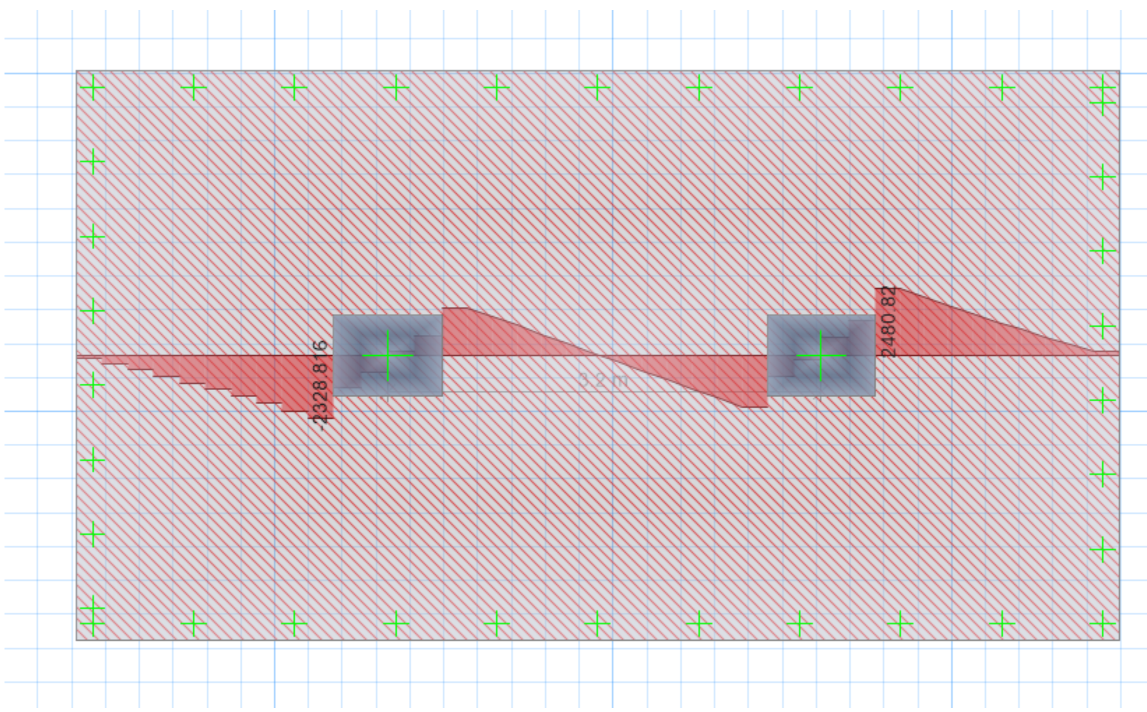
Boundary Element Check (ACI 21.9.6.3, 21.9.6.4)

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	Not Stressed	1.2D+1.6L	0	0	0	0	0	0
Top–Right	Leg 1	326.9	1.2D+1.6L	1828.3804	3749.3356	6.07	4.8	653.7	1000
Bottom–Left	Leg 1	Not Required	1.2D+0.5L+EQY-e	1963.3804	-374.2394	2.19	4.8	684.9	1000
Botttom–Right	Leg 1	Not Stressed	1.2D+0.5L+EQY-e	0	0	0	0	0	0

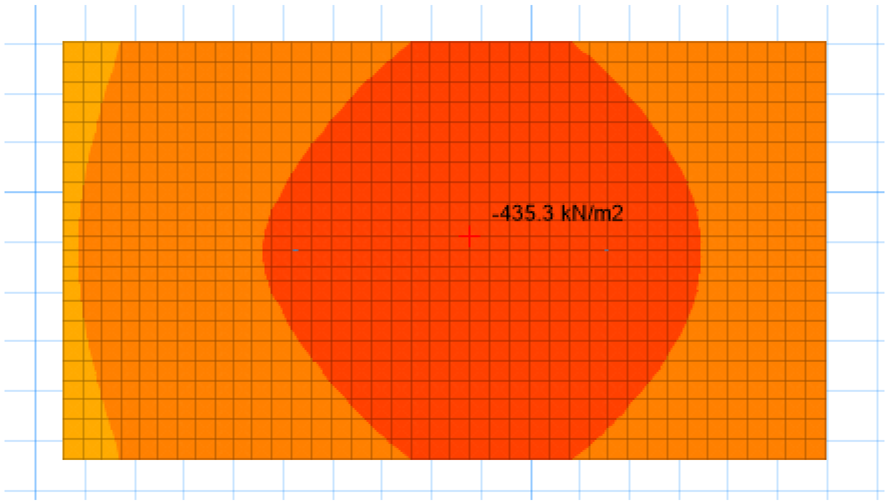
4.7.5 Design of Combined Footing



- Check one way shear :-

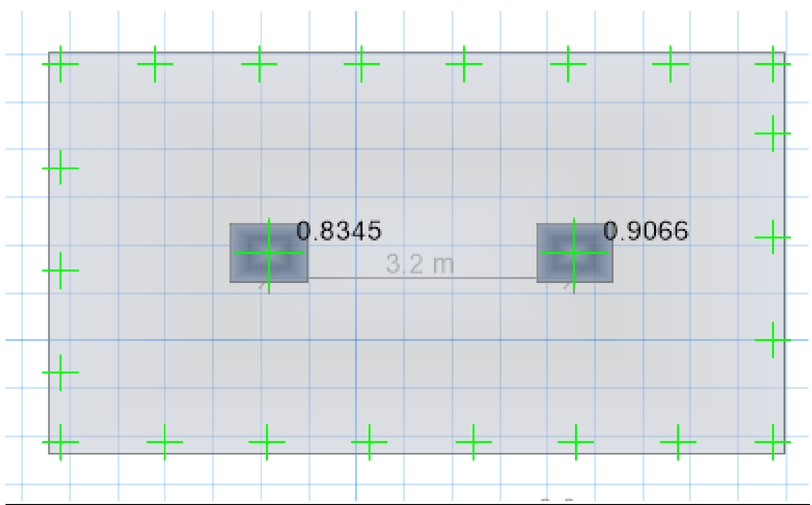


- Check soil pressure :



less than 450KN/m2 >> ok .

- Check punching shear :-



End report .

CHAPTER 5

STRUCTURAL ANALYSIS AND DESIGN (By Calculations)

5.1 Introduction

5.2 Factored load

5.3 Determination of slab thickness

5.4 Design of topping

5.5 Design of one-way ribbed slab

5.6 Design of Beam B1

5.7 Design of Two-way ribbed slab

5.1 Introduction :

Concrete is the only major building material that can be delivered to the job site in a plastic state. This unique quality makes concrete desirable as a building material because it can be molded to virtually any form or shape.

A bond forms between the steel and the concrete, and stresses can be transferred between both components. The design strength provided by a member flexure, and load, and shear is taken as the nominal strength calculated in accordance with the requirements and assumptions of ACI-code.

NOTE:

*****Concrete B300, {fc'= 24 MPa for rectangular and L section}.

*The specified yield strength of the reinforcement {fy = 420MPa}.

5.2 Factored load

The structure may be exposed to different loads such as dead and live loads. The value of the load depends on the structure type and the intended use. The factored loads on which the structural analysis and design is based for our project members, is determined as follows:

$$q_u = 1.2DL + 1.6L$$

ACI – 318 - 14 (9.2.1)

5.3 Determination of slab thickness

Determination of Thickness for One Way Ribbed Slab:

According to ACI-Code-318-08, the minimum thickness of no prestressed beams or one-way slabs unless deflections are computed as follow:

The maximum span length for one end continuous (for ribs):

h_{min} for one-end continuous = L/18.5

=522.3 /18.5 = **28.232 cm**

The maximum span length for both end continuous (for ribs):

h_{min} for both-end continuous = L/21

= 530/21 = **25.238 cm**

Select Slab thickness **h= 35cm** with **block 27 cm** & **Topping 8cm.**

Load calculations:

One-way ribbed slab:

For the one-way ribbed slabs, the total dead load to be used in the analysis and design is calculated as in the following table:

Table (5 – 1) Calculation of the total dead load for one-way rib slab.

Parts of Rib	Density	Calculation
RC. Rib	25	0.27*0.14*25= 0.945 KN/m
Top Slab	25	0.08*0.54*25 = 1.08 KN/m.
Plaster	22	0.03*0.54*22 = 0.356 KN/m.
Block	12	0.4*0.27*12= 1.296 KN/m
Sand Fill	17	0.07*0.54*17= 0.643 KN/m
Tile	23	0.03*0.54*23 = 0.373KN/m
Mortar	22	0.03*0.54*22 =0.356 KN/m.
partition	-	2.3*0.54 =1.242 KN/m

Nominal Total Dead load = **6.966 KN/m** of rib

Nominal Total live load =5*0.54=**2.7 KN/m** of rib

5.4 Design of topping

The calculation of the total dead load for the topping is shown below:

Table (5 – 2) Calculation of the total dead load on topping

No.	Material	Calculation
1	**Tile**	**0.03*23*1= 0.69 KN/m**
2	**mortar**	**0.03*22*1= 0.66 KN/m**
3	**Coarse sand**	**0.07*17*1= 1.19 KN/m**
4	**topping**	**0.08*25*1 = 2.0 KN/m**
5	**Interior partitions**	**2.3 *1 =2.3 KN/m**
Sum		**6.84 KN/m**

W_u = 1.2 DL + 1.6 LL

= 1.2 * 6.84+ 1.6 * 5 = 16.208 KN/m^2. (Total Factored Load)

$$M_u = \frac{W_u * l^2}{12} =$$

5.5 Design of one way Ribbed slab

Material: -

concrete B300 $Fc' = 24$ N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Section: -

b =12cm b_f=52 cm

h =30cm T_f=8 cm

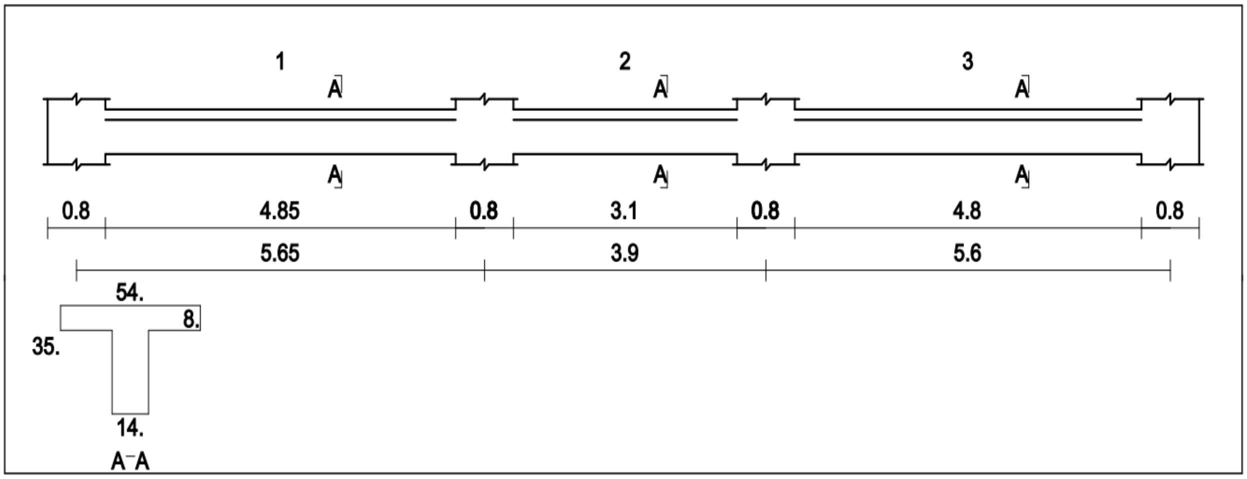


Figure (5-1): Rib geometry.

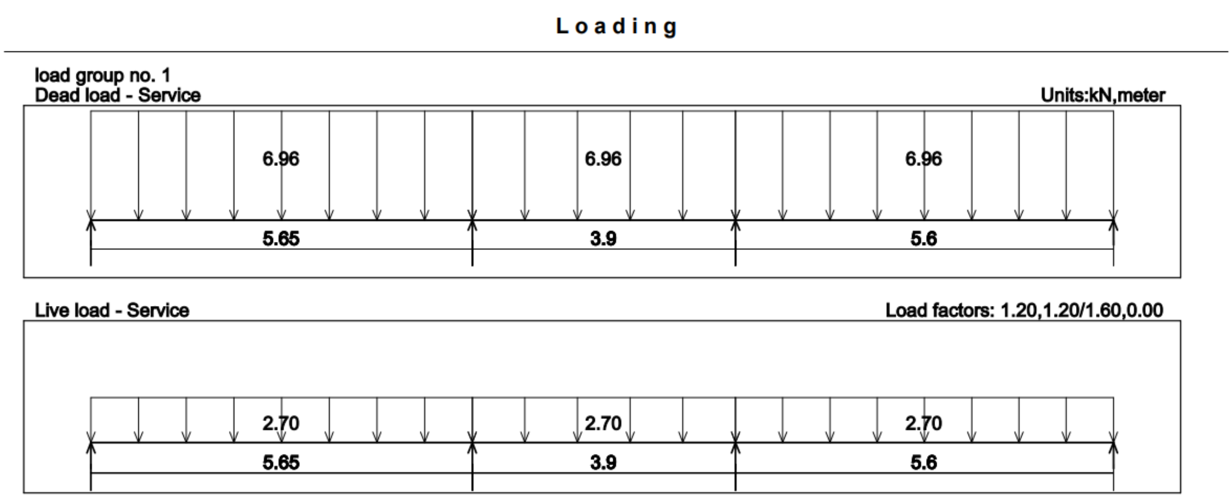


Figure (5-2): loading of rib (1)

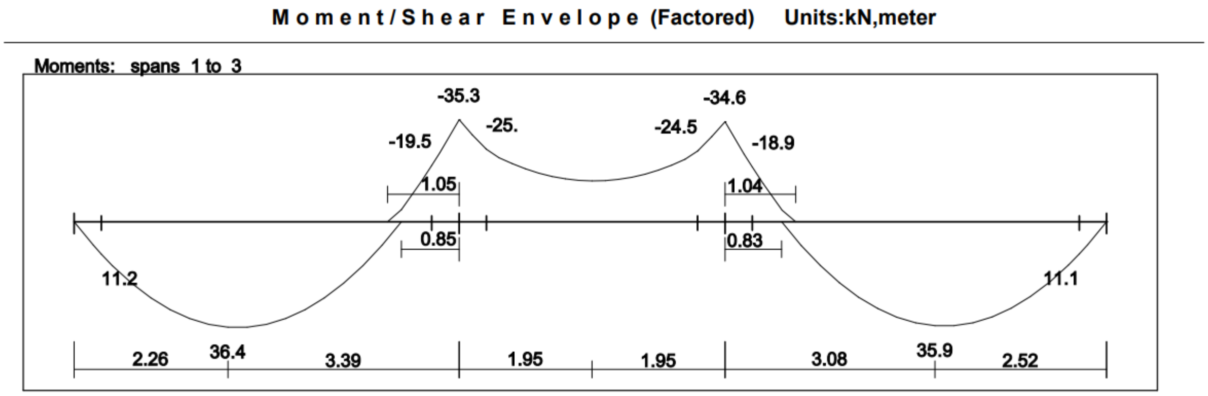


Figure (5-3): Moment Envelop of rib (1)

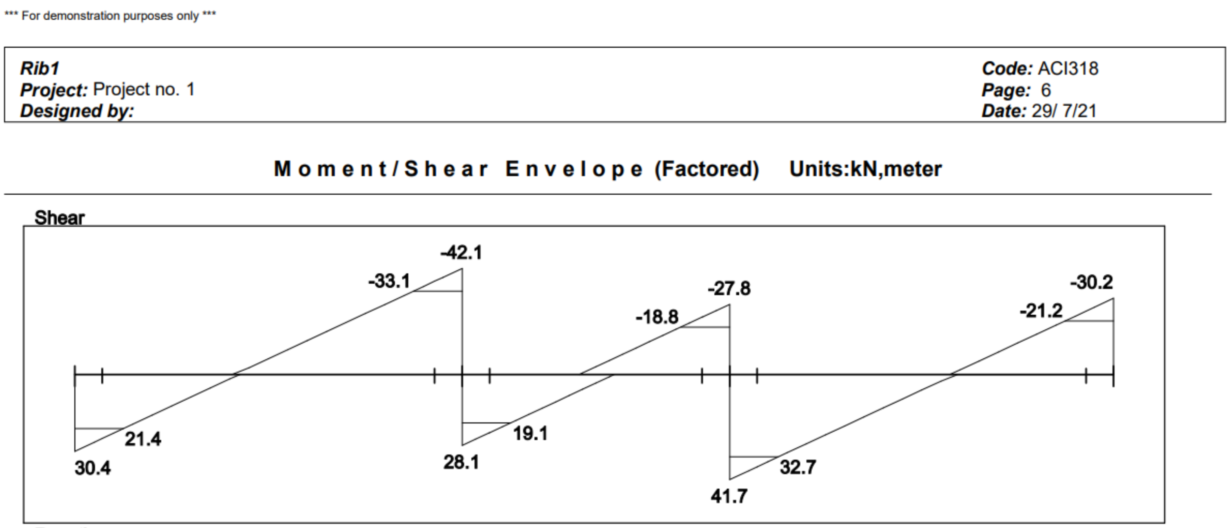


Figure (5-4): Shear Envelop of rib (1)

5.5.1 Design of flexure: -

5.5.1.1 Design of Positive moment of rib (RIB 1):

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

$= 350 - 20 - 10 - \frac{12}{2} = 314$ mm.

→$M_{u\ max}$ = 36.4 KN.m

be ≤ Distance center to center between ribs = 540 mm………… Controlled.

≤ Span/4 = 5650/4 = 1412.5 mm.

≤ (16* t_f) + b_w =(16* 80) +140 =1420 mm.

→b_E= 540 mm.

$$\rightarrow M_{nf} = 0.85\, f_c' * b_E * t_f * \left(d - \frac{t_f}{2}\right)$$

$$= 0.85 * 24 * 0.54 * 0.08 * \left(0.314 - \frac{0.08}{2}\right) * 10^3 = 241.47\ KN.m$$

ϕM_{nf} = 0.9 * 241.47 = 217.32 KN.m

→ϕM_{nf} =217.32> $M_{u\ max}$= 36.4 KN.m.

∴ **DESIGN AS RECTANGULAR SECTION**.

1) Maximum positive moment $Mu^{(+)}$ = 36.4 KN.m

M_n = Mu /ϕ =36.4/ 0.9 = 40.44 KN.m

$$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.58$$

$$R_n = \frac{M_n}{b*d^2} = \frac{36.4*10^6}{540*(314)^2} = 0.683 \text{ MPa}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2*0.683*20.58}{420}}\right) = 0.00165$$

→A_s = ρ * b *d = 0.00165 * 540 * 314= 280.51 mm^2.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d \quad \text{…………(ACI-10.5.1)}$$

$$= \frac{\sqrt{24}}{4*420} * 140 * 314 \ \geq \frac{1.4}{420} * 140 * 314$$

= 128.2 mm^2<146.53 mm^2 ………… Larger value is control.

→As_{min} = 146.53 mm^2<As_{req} = 280.51 mm^2.

∴ As = 280.51 mm^2.

2 Φ14 =307.88 mm^2>As_{req} = 280.51 mm^2. OK.

∴ Use 2 Φ14

→ Check for strain:-($\varepsilon_s \geq 0.005$)

Tension = Compression

A_s * fy = 0.85 * f_c' * b * a

307.88* 420 = 0.85 * 24 * 140 * a

a = 45.28 mm.

f_c' = 24 MPa< 28 MPa→ β_1 = 0.85

$c = \frac{a}{\beta_1} = \frac{45.28}{0.85}$ = 53.27 mm.

$\varepsilon_s = \frac{d-c}{c}$ * 0.003

$= \frac{313-53.27}{53.27}$ * 0.003 = 0.0146 > 0.005 ∴ ϕ =0.9 … OK!

Maximum negative moment $Mu^{(-)}$ = 25 KN.m

M_n = Mu /ϕ =25/ 0.9 = 27.77 KN.m

$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24}$ = 20.58

$R_n = \frac{M_n}{b*d^2} = \frac{27.77*10^6}{540*(314)^2}$ = 0.522 MPa

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2*0.522*20.58}{420}}\right) = 0.00126$$

→A_s = ρ * b *d = 0.*00126** 140 * 314= 55.35 mm^2.

$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\; \geq \;\; \frac{1.4}{f_y} * b_w * d$ …………(ACI-10.5.1)

$= \frac{\sqrt{24}}{4*420} * 140 * 314 \geq \frac{1.4}{420} * 140 * 314$

= 128.2 mm^2<146.53 mm^2 ………… Larger value is control.

→As_{min} = 146.53 mm^2 ≥ As_{req} = 55.35 mm^2.

∴ As = 146.53 mm^2.

2 Φ10 =157 mm^2>As_{req} = 146.53 mm^2 . OK.

∴ Use 2 Φ10

4.6.2 Design of shear of rib (RIB 1):

1) Vu = 33.1 KN.

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

$= 1.1 * \frac{\sqrt{24}}{6} * 0.14 * 0.313 * 10^3$ =39.36 KN.

ϕV_c = 0.75 * 39.36 =29.52 KN.

→Check for Cases: -

1- <u>Case 1:</u> $V_u \leq \frac{\phi V_c}{2}$.

$33.1 \leq \frac{29.52}{2}$=14.76

∴ Case (1) is NOT satisfied

2- <u>Case 2:</u> $\frac{\phi V_c}{2} < V_u \leq \phi V_c$

14.76 ≤ 33.1 ≤ 29.52

∴ Case (2) is NOT satisfied → shear reinforcement is required.

Vs = $\frac{V_u}{\phi} - Vc = 4.77$

Vs max = $\frac{2}{3} * \sqrt{f_c'} * d * b_w = \frac{2}{3} * \sqrt{24} * 140 * 313 * 10^{-3}$ **= 143.11**

$Vs' = \frac{Vs\ max}{2}$ =71.56

Vs min=$\frac{1}{16} * \sqrt{f_c'}$ * b_w * **d** =13.42

Vs min=$\frac{1}{3}$* b_w* **d** = 14.61 ....**Control.**

<u>Try 2Φ8: -</u>

$\frac{100.5*420*313}{S}$ = $14.61*10^3$ → S = 904.3 mm.

S ≤$\frac{d}{2}$ = $\frac{313}{2}$ = 156.5 mm. … **Control**

≤ 600 mm.

∴ **Use 2Φ8 @ 15 Cm**

5.6 Design of Beam 35

Material: -

concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Section: -

B = 80 cm

h =50 cm "choose h=50 , for deflection requirments L/240"

According to ACI-Code-318-08, the minimum thickness of no prestressed beams or one way slabs unless deflections are computed as follow:

h_{min} for one end cont. = L/18.5

=747/18.5 = 40.37 cm.

→Select Total depth of beam **h=50cm. (35cm slab and 15 cm drop)**

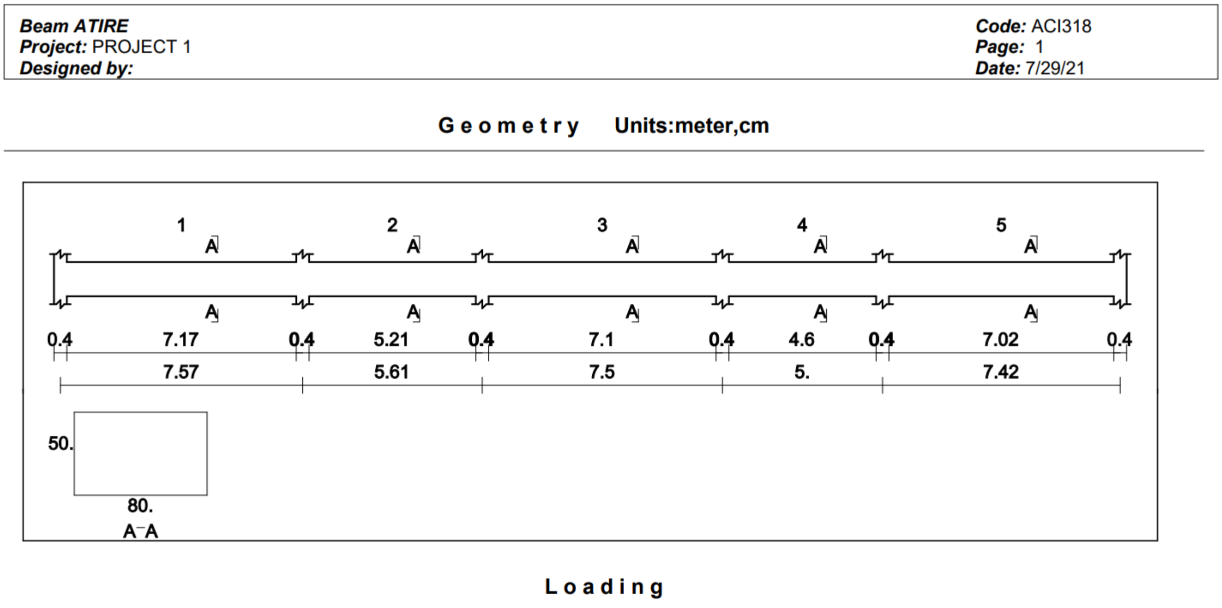


Figure (5-5): Beam Geometry.

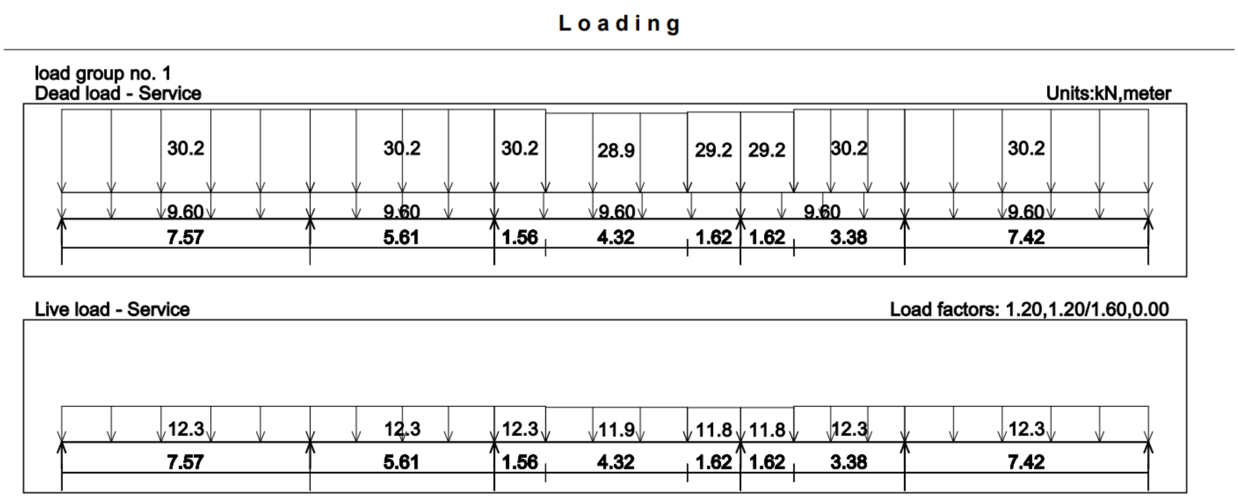


Figure (5-6): Load of Beam (B.35)

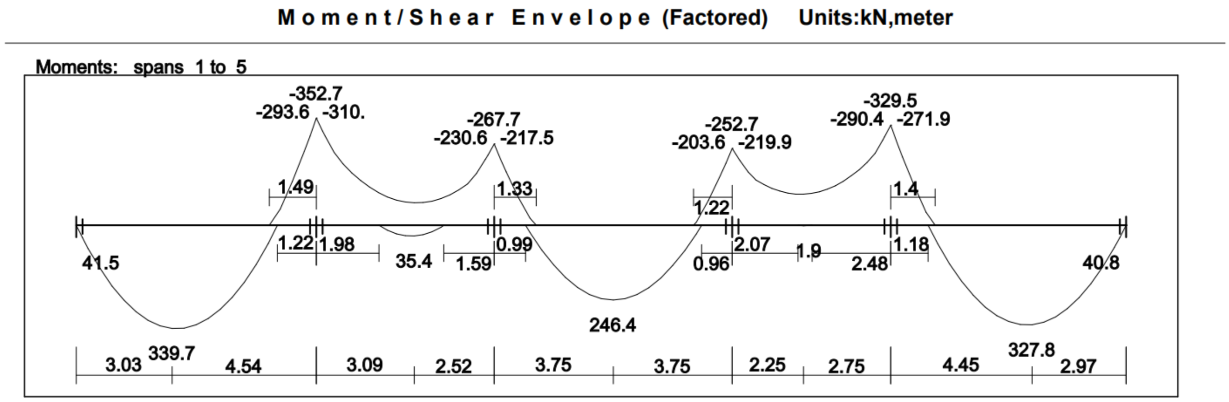


Figure (5-7): Moment Envelop for Beam (B.35)

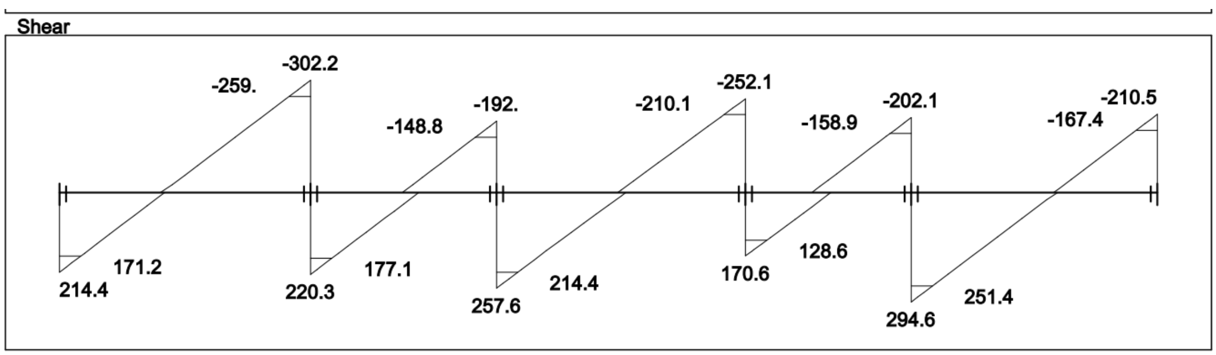


Figure (5-8): Shear Envelop for Beam (B.35)

5.7.1 Design of flexure: -

5.7.1.1 Design of Positive moment: -

$\rightarrow Mu_{max}$ =339.7 KN.m

b_w = 80 Cm.   h =50 Cm.

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

$=500 - 40 - 10 - \frac{18}{2} = 441mm$

$C_{max} = \frac{3}{7} * d = \frac{3}{7} * 441 = 189$ mm.

$f_c' = 24$ MPa < 28 MPa$\rightarrow \beta_1 = 0.85$

a_{max} = β_1* C_{max} = 0.85 * 189= 160.65mm.

*Note:

$M_{n\,max}$ = 0.85 * f_c' * b * a * (d - $\frac{a}{2}$)

= 0.85 * 24 * 0.8 * 0.161 * (0.441 – 0.161/2) * 10^3

=947.22 KN.m

ϵ_s = 0.004

ɸ=0.65+ $\frac{250}{3}$ *(0.004-0.002) = 0.82

→ɸMn_{max} = 0.82 * 947.22=776.72 KN.m

→ Mu = 339.7 KN.m< ɸMn_{max} 776.72 KN.m

∴Singly reinforced concrete section.

1) Maximum positive moment $Mu^{(+)}$ = 339.7 KN.m

Mn = Mu /ɸ= 339.7/ 0.9 =377.44 KN.m .

→m=20.58

$R_n = \frac{M_n}{b*d^2} = \frac{339.7*10^6}{800*\,(441)^2}$ =2.18 MPa

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$\frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*2.18*20.58}{420}}\right)$=0.00552

A_s = ρ * b *d =0.00552*800*441=1947.46 mm^2

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b * d \geq \frac{1.4}{f_y} * b * d$$

$$\frac{\sqrt{24}}{4 * 420} * 800 * 441 \geq \frac{1.4}{420} * 800 * 441$$

$=1028.7mm^2 < 1176mm^2$ …. Larger value is CONTROL

As= 1947.46 mm^2

Use Φ18…. As=254.46mm^2

#of bars= (1947.46/254.46) =8

∴ **Use 8 Φ18…As=2035**.75 > 1947.46mm^2

→ Check for strain:-($\varepsilon_s \geq 0.005$)

Tension = Compression

A_s * fy = 0.85 * f_c' * b * a

2035.75* 420 = 0.85 * 24 * 800 * a

a = 52.4 mm.

f_c' = 24 MPa< 28 MPa→ β_1 = 0.85

$c = \frac{a}{\beta_1} = \frac{52.4}{0.85}$ = 61.64 mm.

$\varepsilon_s = \frac{d-c}{c}$ * 0.003

$= \frac{441-61.64}{61.64}$ * 0.003 = 0.0185 > 0.005 ∴ ϕ =0.9 … OK!

5.7.2 Design of shear: -

1) Vu = 259 KN .

ϕVc = ϕ * $\frac{\sqrt{f_c'}}{6}$ * b * d

= 0.75 * $\frac{\sqrt{24}}{6}$ * 800* 441 * 10^{-3} = 216.044KN.

\→ Check For Cases:-

1- <u>Case1 :</u>

$V_u \leq \frac{\phi V_c}{2}$.

$259 \leq \frac{216.04}{2} = 108.02$

∴ Case (1) is NOT satisfied

2- <u>Case 2 :</u>

$\frac{\phi V_c}{2} < V_u \leq \phi V_c$

$108.08 < 259 \leq 216.04$

∴ Case (2) is NOT satisfied

3- <u>Case 3 :</u> $\phi V_c < V_u \leq \phi V_c + \phi Vs_{min}$

$\phi\, Vs_{min} \geq \frac{\phi}{16}\sqrt{f_c'} * b_w * d = \frac{0.75}{16}\sqrt{24} * 0.8 * 0.441 * 10^3 = 81.01KN.$

$\geq \frac{\phi}{3} * b_w * d = \frac{0.75}{3} * 0.8 * 0.441 * 10^3 = 88.2KN$ …. CONTROL.

$\therefore \phi Vs_{min} = 88.2$ KN.

$\phi V_c + \phi Vs_{min} = 216.04 + 88.2 = 304.24KN.$

$\phi V_c < V_u \leq \phi V_c + \phi Vs_{min}$

$216.04 < 233.2 \leq 304.24$ OK

∴ Case (3) is satisfied → $(\frac{Av}{S}) = \frac{Vs}{(fy_t * d)}$.

$Vs = (\frac{Vu}{\phi} - Vc)$

$Vc = \frac{216.04}{0.75} = 288.05KN$

Vs= ($\frac{259}{0.75}$ – 288.05) = 57.283 KN.

Try 2Φ10 =2 * 78.5 = 157 mm^2.

$\frac{2*78.5*}{S} = \frac{11.68 * 10^{-3}}{(420*441)} \longrightarrow$ s = 1270.96mm

s ≤ $\frac{d}{2} = \frac{441}{2}$ = 220.5 mm …. CONTROL

≤ 600 mm.

∴ **Use Φ10 @ 20Cm 2L.**

5.7 Design of two way ribbed slab

1. Approximate method:

Approximate value of minimum(h) according to ACI

Minimum (h) ≥ (Maximum clear perimeter/180)

Minimum (h) ≥ (2*7.93+2*8.4)/180=18.14 cm

Select (h=35 cm) > minimum (h); 8cm Topping+27cm Block

2. Accurate method:

All exterior and interior beams have a rectangular section of 80 width and 50 cm depth:

$$\text{I for beam} = \frac{b * h^3}{12}$$

$$\text{I for beam} = \frac{80 * 50^3}{12}$$

=833333.33 cm^4

The moment of inertia for the ribbed slab:

Be =5$4$ cm was defined in one way ribbed slab

$$\text{yc} = \frac{40 * 8 * 4 + 35 * 14 * 17.5}{40 * 8 + 35 * 14}$$

=12.17 cm

$$\text{I for rib} = \frac{54 * 12.17^3}{3} - \frac{40 * 4.17^3}{3} + \frac{14 * 22.83^3}{3}$$

=87007.51 cm^4

Slab section for exterior beam

Short direction: L=7.93 m =793cm

$$\text{Is} = \frac{\text{Irib} * (\frac{l}{2}\text{L} + \text{bw})}{\text{bf}}$$

$$\text{Is} = \frac{87007.51 * (\frac{793}{2} + 80)}{54}$$

=767760.7cm^4

Long direction L=8.4m=840 cm

$$\text{Is} = \frac{87007.51 * (\frac{840}{2} + 80)}{54}$$

805625.09 cm^4=

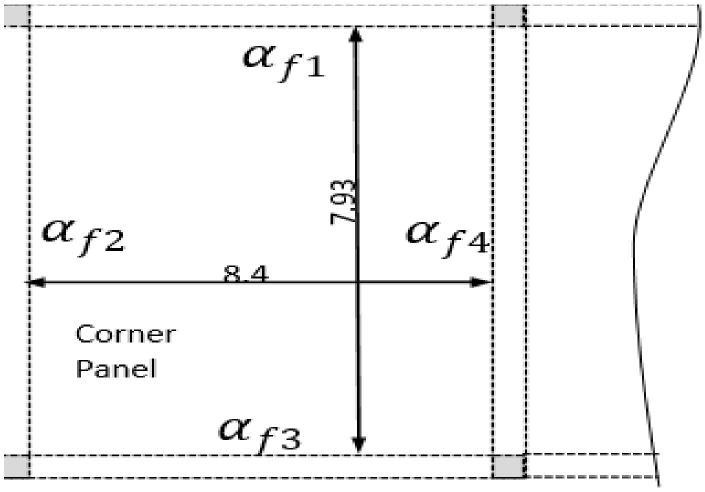
Short direction l_{right}=l_{left} =793 m=793 cm

Slab section for interior beam

Long direction l_{left}=l_{right} =6.33m=633cm

$$\text{Is} = \frac{87007.51 * (\frac{840}{2} + \frac{720}{2} + 80)}{54}$$

=1385675.16cm^4



$$\alpha = \frac{Ib}{Is}$$

α1=833333.33/767760.7=1.085

α2=833333.33/805625.09 =1.034

α3=833333.33/767760.7=1.085

α4=833333.33/1385675.16=0.6014

αfm=(1.085+1.034+1.085+0.06014)/4

= 0.9514 <2.0 the minimum slab thickness will be :

$$h = \frac{\ln\,(0.8 + \frac{fy}{1400})}{36 + 5\beta(\alpha fm - 0.2)}$$

$$h = \frac{8400\;(0.8 + \frac{420}{1400})}{36 + 5 * 1.06(0.9514 - 0.2)}$$

=231.1 mm>125mm

β=8.4/7.93=1.06

First trial thickness h=350mm >231.1 mm_ok

Take slab thickness h_{slab} 350mm ,80mm topping ,270mm concrete block

Load calculation:

Material	Quality Density $KN/\ m^3$	$W = \gamma \cdot V$ KN
Tiles	23	$23 \times 0.03 \times 0.54 \times 0.54 = 0.201$
mortar	22	$22 \times 0.03 \times 0.54 \times 0.54 = 0.1925$
Sand	17	$17\times 0.07 \times 0.54 \times 0.54 = 0.347$
Reinforced Concrete Topping	25	$25 \times 0.08 \times 0.54 \times 0.54 = 0.583$
Reinforced Concrete Rib	25	$25 \times 0.27 \times 0.14 \times (0.54 + 0.4) = 0.888$
Concrete Block	12	$12 \times 0.27 \times 0.4 \times 0.4 = 0.518$
Plaster	22	$22 \times 0.03 \times 0.54 \times 0.54 = 0.1925$
For ceiling	1.25	$1.25 \times 0.54\times0.54 = 0.3645$
Partitions 2.3 $KN/\ m^2$		$2.3 \times 0.54 \times 0.54 = 0.671$
Total Dead Load, KN		3.96

$$DL = \frac{3.96}{0.54 \times 0.54} = 13.58\ \mathrm{KN/m2}$$

$$w_D = 1.2 \cdot 13.58 = 16.3\ KN/m^2$$

Live Load of slab:

$LL = 5\ KN/m^2$

$w_L = 1.6 \cdot 5 = 8\ KN/m^2$

$w = 16.3 + 8 = 24.3\ KN/m^2$

Moments calculations:

$M_a = C_a w l_a^2$ and $M_b = C_b w l_b^2$

La/Lb = 0.95

Design of bending moment:

Ma,pos ,DL=0.031*16.3*7.93^2

= 31.78 KN.m/m

Mb,pos ,DL=0.031*16.3*8.4^2

= 35.65 KN.m/m

Ma,pos,LL=0.036*8*7.93^2

=16.1 KN.m/m

Mb,pos,LL=0.032*8*8.4^2

=18.06 KN.m/m

Ma,pos =31.78+16.1=47.88 KN.m/m

Mb,pos =35.65+18.06=53.71 KN.m/m

Mb,neg =0.067*24.3*8.4^2=114.88 KN.m/m

Design of positive moments:-

Mu=53.71 KN.m

ØMn =Mu

Mn=53.71/0.9=59.68 KN.m

Assume bar diameter Ø16 for main reinforcement

d=350-20-8-16/2=314mm

Rn=Mn/(b*d^2)

=59.68/(540*314^2)

=1.12 Mpa

m=fy/(0.85 fc)

=(420/.85*24)

=20.58

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

$$=\frac{1}{20.58}(1-\sqrt{1-\frac{2*1.12*20.58}{420}})$$

=0.00274

As=0.00274*540*314=465.3 mm2

Check of As min:

As,min= 0.25*(√fc'/ fy)*bw*d ≥ 1.4/fy *bw*d

128.19<146.53 mm^2

As min = 146.53 mm2 < As req = 465.3 mm2

∴As = 465.3 mm2 …. control

2Φ18 = 508.9 mm2 > Asreq = 465.3mm2 OK

∴ Use 2 Φ18

Check for strain:

($\varepsilon_s \geq 0.005$)

Tension = Compression

As * fy = 0.85 * fc' * bw * a

508.9 * 420 = 0.85 * 24 * 540 * a

a = 19.4 mm

fc' = 24 MPa< 28 MPa→ β = 0.85

c=a/β = 19.4/0.85 = 22.82 mm

ε_s=(d-c)/c * 0.003

∴ ϕ =0.9 0.0381 > 0.005 Ok

Design of negative moments:-

Mu=114.88 KN.m

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

=114.88 /0.9=127.64 KN.m

Assume bar diameter Ø16 for main reinforcement

d=350-20-8-16/2=314 mm.

Rn=$\frac{M_n}{b*d^2}$

=127.64*10^6/(540*$314^{2)}$

=2.4 Mpa

`

m=420/(0.85*24)=20.58

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2 * R_n * m}{f_y}})$$

=$\frac{1}{20.58}(1 - \sqrt{1 - \frac{2*2.4*20.58}{420}})$

=0.0061

As=0.0061*540*314=1033.77 mm^2

As,min= 0.25*(√fc'/ fy)*bw*d ≥ 1.4/fy *bw*d

128.19<146.53 mm^2

As min = 146.53 mm2 < As req = 1033.77mm2

∴ As = 1033.77 mm2 control

3Φ22 = 1140.4 mm2 > Asreq = 1033.77mm2 OK

∴ Use 3 Φ22

Check for strain: (ε_s≥0.005)

Tension = Compression

As * fy = 0.85 * fc' * bw * a

1140.4 * 420 = 0.85 * 24 * 540 * a

a = 43.4 m

fc'= 24 MPa< 28 MPa→ β = 0.85

c=a/β = 43.4/0.85 = 51.15 mm

εs =(d-c)/c * 0.003

Ok 0.0152 > 0.005

∴ ϕ =0.9

Design of shear:

W_b= 0.67

The total load on the panel being =(7.93*8.4*24.3=1618.67 KN

The load per rib at the face of short beam is (0.67*1618.67*0.54)/(2*7.93)=36.93KN

V_{ud} =Vu_{face}-(Wu*bf*d)=36.93-(24.3*0.54*0.313)=32.82 KN

The maximum shear force at the distance d from the face of support, Vu = 346 KN

$Vc = 1.1(\sqrt{fc'}/6 * bw * d)$

$= 1.1(\sqrt{24}/6 * 140 * 313) = 39.35$ KN

$\phi Vc = 0.75 * 39.35 = 29.5$ KN

Case1: $Vu \leq (\phi Vc)/2$

32.82 < 29.5/2 =14.75 ∴ Case (1) is not satisfied

Case2: $Vu \leq (\phi Vc)$

32.82 < 29.5 ∴ Case (2) is not satisfied

∴ shear reinforcement is required

Case3:

$Vs = (\frac{Vu}{\phi} - Vc)$

$Vs = (\frac{32.82}{0.75} - 39.35) = 4.41$ KN.

$Vs_{min} \geq \frac{1}{16}\sqrt{f_c'} * b_w * d = \frac{1}{16}\sqrt{24} * 140 * 0.313 * 10^3 = 13.42$ KN.

$\geq \frac{1}{3} * b_w * d = \frac{1}{3} * 140 * 0.313 * 10^3 = 14.61$ KN …. CONTROL.

∴Vs_{min} = 14.61 KN.

<u>**Try 2Φ8: -**</u>

$\frac{100.5*420*313}{S} = 14.61*10^3 \rightarrow S = 904.3$ mm.

$S \leq \frac{d}{2} = \frac{313}{2} = 156.5$ mm. … **Control**

≤ 600 mm. ∴ **Use 2Φ8 @ 15 Cm**

5.8 Design of Column (C8)

Calculation of Loads act on Column (C8)

Loads acting on columns are obtained from support reaction when analyzing the system on etabs

Dead Load = (Service Dead reaction from etabs)

=1628 KN

Live Load = (Service Live reaction from etabs)

=434.7 KN

Loads acting on column (C8) are as follows:

Factored loads (Pu) = 2649

Calculation of Required Dimension of Column (C8)

Total load Pu =2649 KN

Pn = 2649 /(0.65) = 4075.6 KN

$\rho g = 2.0$ %

$\mathrm{Pn} = 0.8 * \mathrm{Ag}\{0.85 * \mathrm{fc}' + \rho\mathrm{g}(\mathrm{fy} - 0.85\mathrm{fc}')\}$

$4075.6 * 10^{-3} = 0.8 * \mathrm{Ag}[0.85 * 24 + 0.02 * (420 - 0.85 * 24)]$

$\mathrm{Ag} = 1794\ cm^2$

∴**Select 40*60cm with Ag = 2400 cm².**

- **Check Slenderness Effect :**

For braced system if $\lambda \leq 34 - 12\frac{M1}{M2} \leq 40$, then column is classified as short column and slenderness effect shall not be considered.

$$\lambda = \frac{Klu}{r}$$

Where :

Lu: Actual unsupported (unbraced) length = 3.65 m

K: effective length factor (K= 1 for braced frame).

R: radius of gyration → for rectangular section = $\sqrt{\frac{I}{A}}$ 0.3 h

<u>System about X</u>

$$\rightarrow \lambda = \frac{1 * 3.65}{0.3 * 0.6} = 20.28$$

$\lambda \leq 34 - 12(1) = \mathbf{22} \leq 40$

$\lambda = 20.28 < 22$ ∴ Short about X .

<u>System about Y</u>

$$\rightarrow \lambda = \frac{1 * 3.65}{0.3 * 0.6} = 20.28$$

$\lambda \leq 34 - 12(1) = \mathbf{22} \leq 40$

$\lambda = 20.28 < 22$ ∴ Short about Y.

∴ Column is Short , So Slenderness effect will not be considered.

Calculation of Required Reinforcement Ratio

Since Column is short and slenderness effect will not be considered, then Design Strength of column can be calculated using the following equation :

$\phi\text{Pn} = 0.65 * 0.8 * \text{Ag}\{0.85 * \text{fc}' + \rho\text{g}(\text{fy} - 0.85\text{fc}')\}$

Where , Pu =2649 KN

$2649 * 10^3 = 0.65 * 0.8 * 400 * 600\{0.85 * 24 + \rho\text{g}(420 - 0.85 * 24)\}$

$\Rightarrow \rho_g = 0.0134 > \rho_{min} = 0.01$ & $< \rho_{max} = 0.08$

As req = 0.0134 *400 *600 =3216 mm^2

Use Φ 18 $>> \# of\ bar = \frac{3216}{254} = 12.64$

∴ **<u>Use 14 Ø 18 with As = 3562 mm² >As $_{req}$ = 3216 mm²</u>**

- **Check spacing between the bars :**

S = $\frac{400-2*40-2*10-4*18}{3} = 76\ mm$

S = 76 mm $\geq 40mm$

$\geq$ 1.5db = 37.5 mm

Determination of Stirrups Spacing

<u>According to ACI :</u>

$\text{Spacing} \leq 16 \times d_b(\text{Longitudinal. bar. diameter}) = 16 \times 1.8 = 28.8\ \text{cm.}$

$\text{Spacing} \leq 48 \times d_t(\text{tie. bar. diameter}) = 48 \times 1.0 = 48\text{cm.}$

$\text{Spacing} \leq \text{Least. dim e nsion} = 40\ \text{cm}$

∴ **<u>Select Ø 10/20cm</u>**

Column (C8) Section is shown in figure(4-11) where bars arrangmenet and stirrups detailing appear :

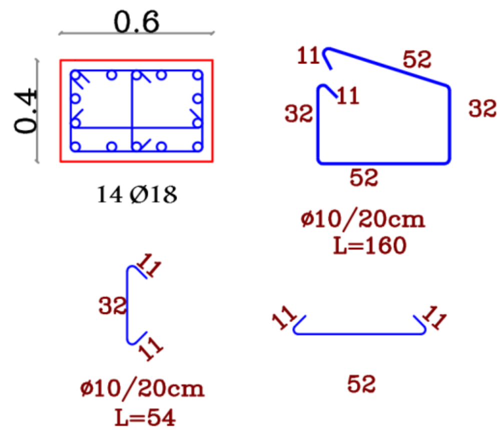


Figure (5- 9): C14 Reinforcement Details

5.9 Design of Isolated Footing (F5)

Loads that act on footing F5 are :

- PD =3389.5 kN , PL = 664.5 kN → Pu = 1.2 * 3389.5 + 1.6*664.5 = 5130.6 kN

The following parameters are used in design :

- γ_{concrete} = 25 kN/m^3
- γ_{soil} = 18 kN/m^3
- σ_{allow} = 450 kN/m^2
- clear cover = 7.5 cm

Determination of footing dimension (a)

Footing dimension can be determined by designing the soil against bearing pressure .

→ Assume h = 80 cm

→ $\sigma_{b(allow)\text{net}}$ = 450 – 25*0.80 = 430 kN/m²

→ $\sigma_{\text{bu(allow . net)}}$ = 1.4* 430 = 602 kN/m²

→ $\sigma_{\text{bu}} = {Pu}/{Areq} \leq \sigma_{\text{bu(allow . net)}}$

∴ $5130.6/\text{a}^2$ = 602 → **a=2.92m** → **Select a=3 m**

→ $Bearing\ Pressure\ \sigma bu = \text{Pu}/\text{A} = \frac{5130.6}{3*3}$ = **570.1 kN/m^2** ≤ 602 kN/m² **…. (SAFE)**

Determination of footing depth (h)

To determine depth of footing both of one and two way shear must be designed.

Design of one way shear

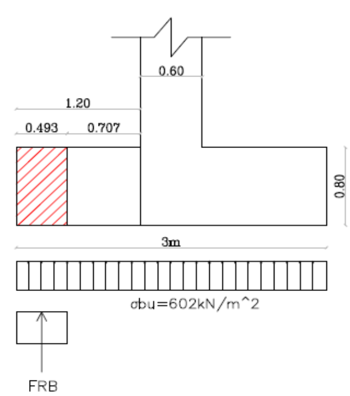


Figure (5- 10): Critical Section of Shear Force

→ $d = \text{h} - cover - \emptyset = 800 - 75 - 18 = 707\ mm$

→ Vu at distance d from the face of column :

Vu = $FRB = \sigma bu \times 0.493 \times b$

= 570.1 × 0.493 × 3= 843.18 kN

→ $\emptyset * \text{Vc} = 0.75 * \frac{1}{6} * \sqrt{\text{fc}'} * \text{b} * \text{d}$

= $0.75 * \frac{1}{6} * \sqrt{24} * 3000 * 707$ =1298.84 kN >Vu

∴ **h = 80 cm is correct** ✓

Design of Punching (two way shear)

→ $d = 707\ mm$

→ $b_\circ$ = 2× 1307+2×1107= 4828 mm

→ Bc = 1

→ αs = 40 (interior column)

Vu = 5130.6 – (570.1 *1.305*1.1.107) = **4305.75 kN**

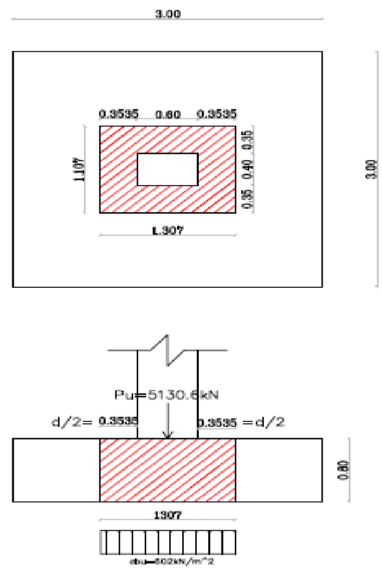


Figure (5- 1):Punching Shear Critical Section

<u>Ø×Vc is the smallest of :</u>

1. $\mathrm{Vc} = \left(2 + \frac{4}{\mathrm{Bc}}\right) \times \frac{\sqrt{\mathrm{fc}'}}{12} \times \mathrm{b_o} \times \mathrm{d}$

$= \left(2 + \frac{4}{1}\right) \times \frac{\sqrt{24}}{12} \times 4828 \times 707$

$= 8361.08\ \mathrm{kN}$

2. $\mathrm{Vc} = \left(\frac{\propto_s \times d}{\mathrm{b_o}} + 2\right) \times \frac{\sqrt{\mathrm{fc}'}}{12} \times \mathrm{b_o} \times \mathrm{d}$

$= \left(\frac{40 \times 707}{4828} + 2\right) \times \frac{\sqrt{24}}{12} \times 4828 \times 707$

$= 10949.53\ \mathrm{kN}$

3. $\mathrm{Vc} = 4 \times \frac{\sqrt{\mathrm{fc}'}}{12} \times \mathrm{b_o} \times \mathrm{d}$

$= 4 \times \frac{\sqrt{24}}{12} \times 4828 \times 707 =$ **5574.05 kN .. ≺ cont.**

→ Ø×Vc = 0.75 × 6009.6 = **4180.53** kN > Vu = **4159.7** kN

∴ <u>**h = 80 cm is correct ✓**</u>

Design of Reinforcement

Mu = 570.1 *1.2*3*(1.2/2) = 1231.4 kN.m

→ $m = \frac{Fy}{0.85*Fc'} = \frac{420}{0.85*24} = 20.6$

→ Mn = 1231.4/0.9= 1368.22 kN.m

→ $Rn = \frac{Mn}{b*d^2} = \frac{1368.22*10^6}{3000*707^2} = 0.912$ MPa

→ $\rho = \frac{1}{m} * (1-\sqrt{1-\frac{2*Rn*m}{Fy}})$

$= \frac{1}{20.6} * (1-\sqrt{1-\frac{2*0.912*20.6}{420}}) = 0.002222$

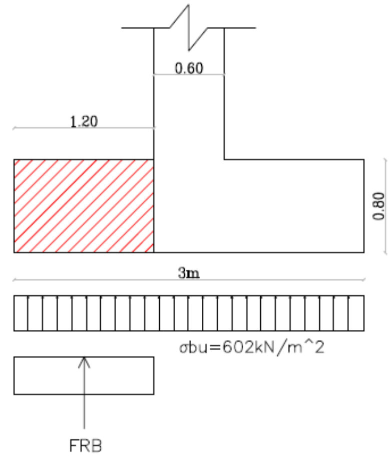


Figure (5- 2):Critical Section of Bending Moment

→ Asreq = $\rho * b * d = 0.002222 * 3000 * 707 =$ 4712.862 mm^2

→ As (min) = 0.0018*b*h = 0.0018*3000 * 800 = 4320 mm^2

→ Asreq > As (min)

∴ **Select for both directions: 21Ø18@15cm with As = 5343.87 mm^2 > Asreq … (ok)**

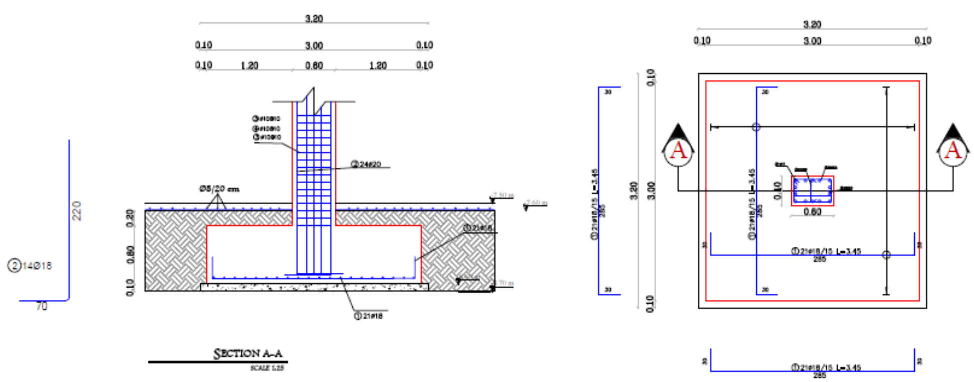


Figure (5- 13):F5 Reinforcement Details

5.10 Design of Stairs

The following figure shows a top view of the stairs :

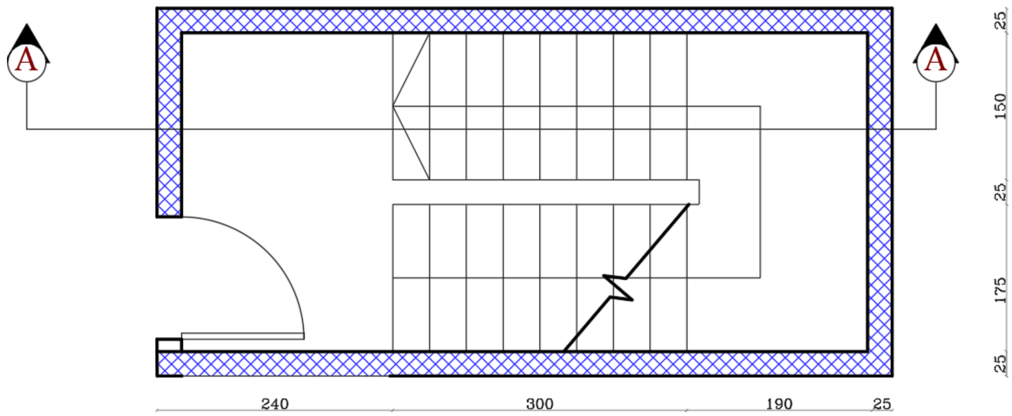


Figure (5- 14): Stairs Top View

Design of flight

The structural system of the flight is shown in figure (4-22) and the following steps explain the design procedure of the flight :

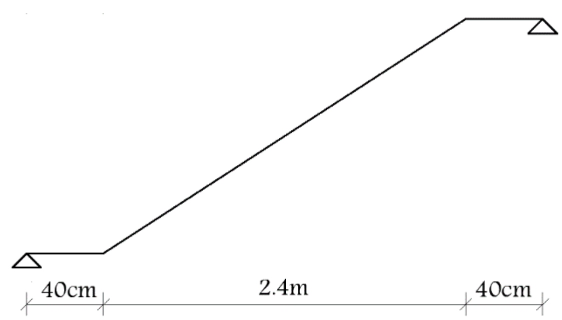


Figure (5- 15): Structural system of flight

1. **Determination of flight thickness :**

   Limitation of deflection: h ≥ minimum h

   h (min) = L/20 = 320/20 = 16cm

   ∴ Select h = 15 cm, but shear and deflection must be checked

   Angle (α): tan(α) = 16.7/30 → α = 29°

2. Loads calculation :

Figure (4-23) shows a section in the flight in which the layers carried by the flight appear.

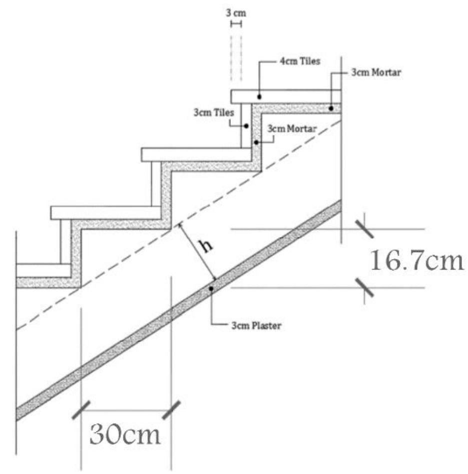


Figure (5- 16): Section of The Flight

Table(4- 1): Calculation of Dead Loads that act on Flight

Flight Dead Loads
Flight = (0.15 * 25* 1)/ cos(29) =4.28 kN/m
Plaster = (0.03 * 22* 1)/ cos(29) =0.75 kN/m
Hor.Mortar =0.03*22*1 =0.66 kN/m
Ver.Mortar = 0.03*22*($\frac{0.167}{0.3}$) = 0.36 kN/m
Hor.Tiles = 0.04*23*($\frac{33}{30}$) = 1 kN/m
Ver.Tiles = 0.03*23*($\frac{0.167}{0.3}$) = 0.38 kN/m
Triangle =0.5*0.167*25 =2.08 kN/m
Sum=9.51 kN/m

<u>Factored Loads :</u>

qu= 1.2 *9.51 + 1.6*2 = 14.6 kN/m

Vu= 14.6*2.4/2 = 17.52 kN

3. Analysis :

The following figures show shear and moment Diagrams resulted from analysis of the flight :

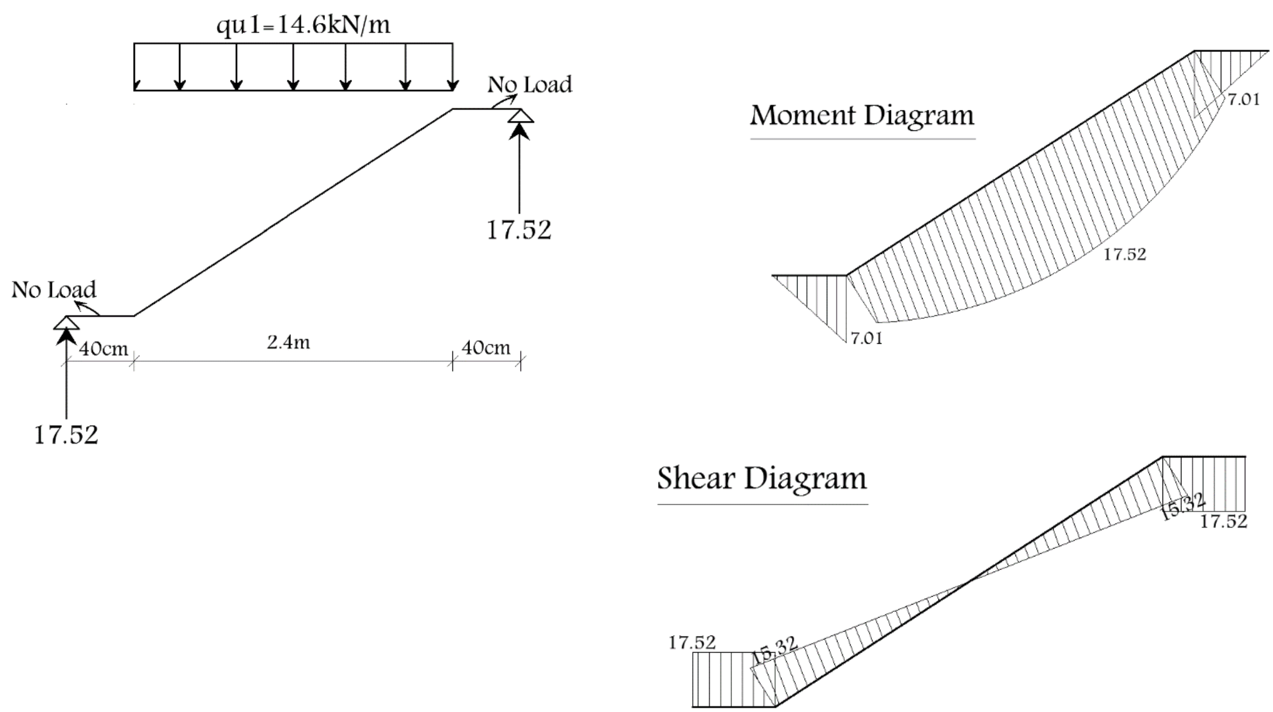


Figure (5- 17) :Analysis of the flight

4. Design :

- **Design of Shear Force :**

d=150-20-(12/2) = 124 mm

$Ø \times Vc = 0.75 * \frac{1}{6} * \sqrt{Fc'} * bw * d$

$= 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 124$

= 75.9 kN > Vu max = 15.32 kN

∴ No Shear Reinforcement is Required#

- **Design of Bending Moment :**

→ $m = \frac{Fy}{0.85 * Fc'} = \frac{420}{0.85 * 24} = 20.6$

→ $Rn = \frac{Mu/Ø}{b * d^2} = \frac{\mathbf{17.52} * 10^6/0.9}{1000 * 124^2} = 1.26$ MPa

→ $\rho = \frac{1}{m} * (1- \sqrt{1 - \frac{2 * Rn * m}{Fy}}) = \frac{1}{19.6} * (1- \sqrt{1 - \frac{2 * 1.26 * 20.6}{400}}) = 0.0031$

→ $Asreq = \rho * b * d = 0.0031 * 1000 * 124 = 384.4\ mm^2$

→ As min = 0.0018 *1000*16.7 = 300.6 mm^2

∴ Select **Ø12/20 with As = 565 mm^2** > As req …. For Main Reinforcement

For secondary Reinforcement select **Ø10 /20 with As=395 mm^2** = As min

→ Check Spacing :

20cm < **S max** = 3 * 15 = **45 cm … ok**
< **45 cm**

→ Check Strain:

C = T

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

0.85*24*a*1000 = 300.6 *420

a = 5.89 mm → c = a/β = 5.89/0.85 = 6.18 mm

$\varepsilon_s = \frac{0.003 * d}{c} - 0.003 = \frac{0.003 * 124}{6.18} - 0.003$

∴ **ε_s = 0.057 > 0.005 …. Ø = 0.9 (OK)**

Design of Landing

The structural system of the landing is shown in figure (4-25) and the following steps explain the design procedure of it :

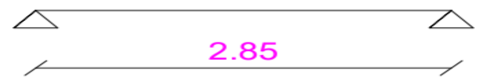


Figure (5- 18):Structural system of landing

- ### Determination of Landing thickness :

Limitation of deflection:

h ≥ minimum h

h (min) = L/20 = 285/20 = 14.25 cm

∴ Select h = 15 cm , but shear and deflection must be checked

- ### Loads calculation :

Figure (4-26) shows a section in the landing in which the layers carried by the landing appear.

Table(4- 2):Calculation of Dead Loads that act on Landing

Landing Dead Loads
Tiles = 0.03*23*1=0.7 kN/m
Mortar = 0.03*22*1=0.4 kN/m
Sand = 0.07*16*1=1.1 kN/m
Slab = 0.15*25*1=3.75 kN/m
Plaster = 0.02*22*1=0.4 kN/m
Sum = 6.35 kN/m

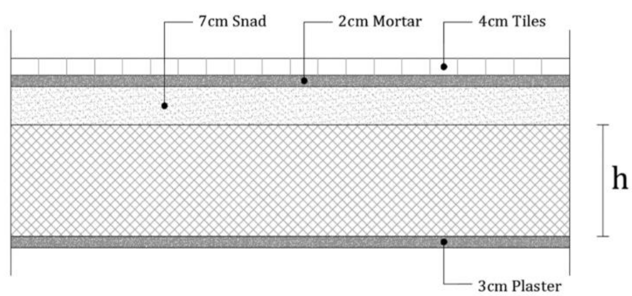


Figure (5- 19):Section of The Landing

<u>Factored Loads :</u>

qu = 1.2*6.35+1.6*2 = 10.82 kN/m

The landing carries (dead load & live load of landing + support reaction resulted from the flight)

qu = 10.82 + Support reaction of flight = 10.82 + 17.52 = **28.34 kN/m**

→ **Analysis :**

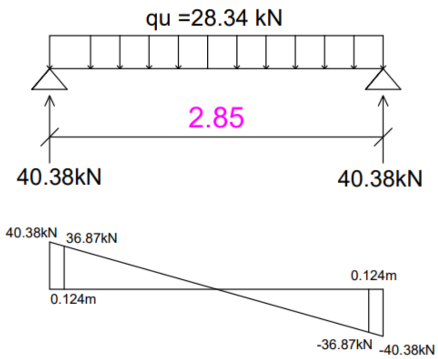


Figure (5- 3):Analysis of Landing

d= 150-20-(12/2) = 124mm

Vumax= 40.38 – (28.34*0.124) = **36.87 kN**

Mu max = $\frac{28.34*2.85^2}{8}$ = **28.77 kN.m**

Shear Force Design :

d=124mm & Vu max=36.87 kN

$Ø×Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 124 = 75.9kN > Vu\ max = 36.87\ kN$

∴No Shear Reinforcement is Required#

→ **Bending Moment Design : (Mu max = 28.77 kN.m)**

- m = 20.6
- Rn = $\frac{28.77 * 10^6/0.9}{1000*124^2}$ = 2.08 MPa
- $\rho = \frac{1}{20.6} * (1- \sqrt{1 - \frac{2*2.08*20.6}{420}}) = 0.00523$
- Asreq = 0.00523 ∗ 1000 ∗ 124 = 649.1 mm^2
- As min = 0.0018 *1000*150 = 270 mm^2

∴ Select **Ø12/15cm with As = $\frac{\pi*14^2}{4} * \frac{100}{15}$ = 753 mm^2** > As req …. For Main Reinforcement

- Check Spacing :

  15cm < **S max** = 3 * 15 = **45 cm … ok**

  < 45cm

- Check Strain:

$$C = T$$

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

$$0.85*24*a*1000 = 753*420$$

$$a = 15.5 \text{ mm} \rightarrow \quad c = a/\beta = 15.5/0.85 = 18.24 \text{ mm}$$

$$\varepsilon_s = \frac{0.003*(124-18.24)}{18.24}$$

$$\therefore \ \boldsymbol{\varepsilon_s = 0.0174 > 0.005 \ \ldots\ \text{Ø} = 0.9 \ \ (\text{OK})}$$

The following figure shows section A-A of the stairs in which reinforcement detailing appears .

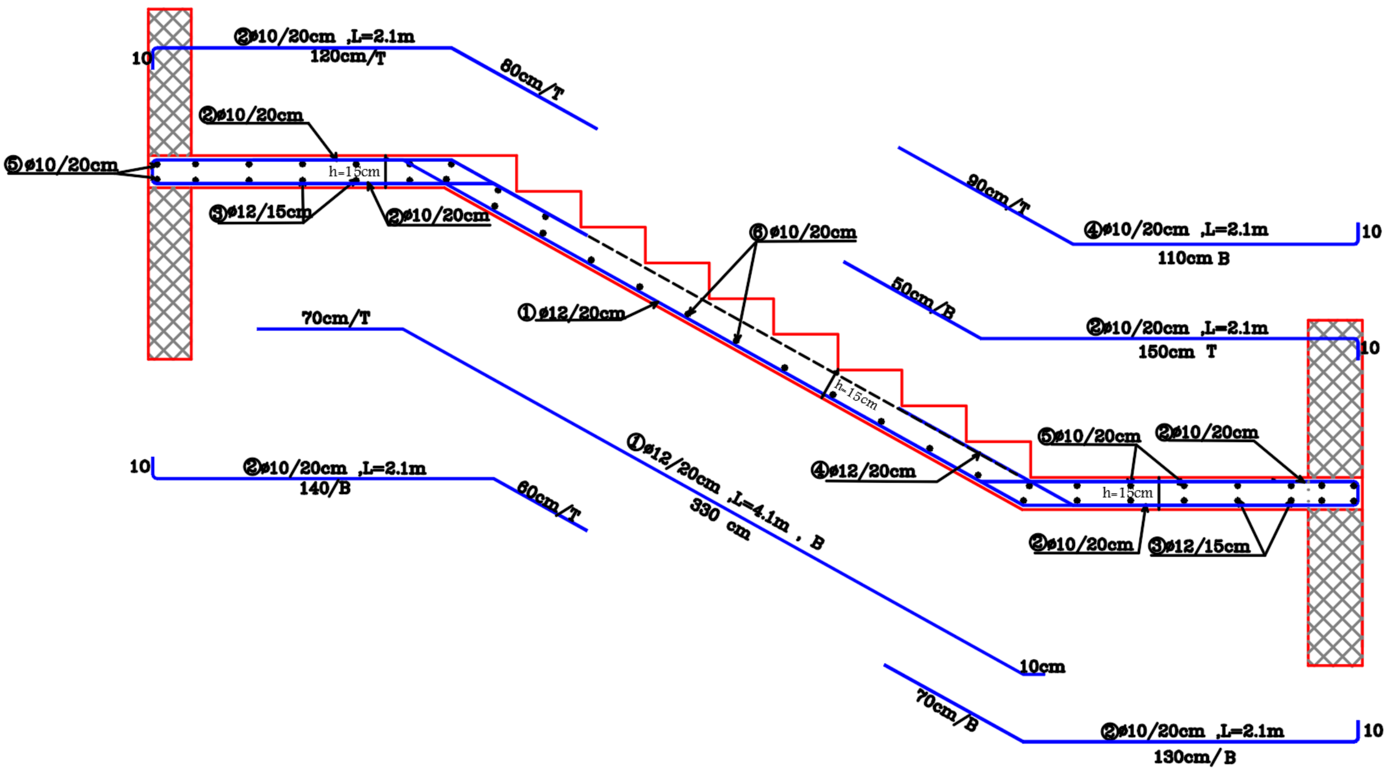


Figure (5- 21):Reinforcement Details of Stairs

5.10 Design of Shear wall

Analysis and design were done using ETABS program in which the seismic loads were taken into account. The following is a sample calculation for one of the walls, S.W9.

The following data that used in design**:**

- Shear Wall thickness = h = 25 cm
- Shear Wall length Lw = 6.0m
- Building height Hw=28 m
- Critical section shear :

  Lw/2=6/2=3 ….. control

  hw/2=28/2=14

  story height=3.65

→ d =0.8*Lw =0.8*6.0 = 4.8 m

4.7.1 Design of Horizontal Reinforcement

Calculation of Shear Strength Provided by concrete Vc:

- Shear Strength of Concrete is the smallest of :

1- Vc = $\frac{1}{6}\sqrt{fc'} \times b \times d$

$= \frac{1}{6}\sqrt{24} \times 250 \times 4800 = \mathbf{979.8\ kN} \ll$ Controlled

2- Vc = $0.27\sqrt{fc'} \times h \times d + \frac{Nu \times d}{4Lw}$

$= 0.27\sqrt{24} \times 250 \times 4800 + 0 = 1587.3\ KN$

3- $Vc = \left[0.05 * \sqrt{fc'} + \frac{Lw\left(0.1\sqrt{fc'}+0.2\frac{Nu}{Lw.h}\right)}{\frac{Mu1}{Vu} - \frac{Lw}{2}}\right] \times h \times d$

Where:

- Mu1= 920.5 kN.m

- $\frac{Mu1}{Vu} - \frac{Lw}{2} = \frac{920.5}{750.3} - \frac{6}{2} = -1.77 < 0$ → This equation is not applicable.

$\therefore Vc = 979.8$ kN → $\emptyset Vc = 734.85 < Vumax^1 = 750.3$ kN →Horizontal Reinforcement is Required.

$\rightarrow \mathrm{Vs} = \frac{\mathrm{Vu}}{\emptyset} - \mathrm{Vc} = \frac{750.3}{0.75} - 979.8 = 20.6$ kN

$\rightarrow \frac{\mathrm{Avh}}{\mathrm{s}} = \frac{\mathrm{Vs}}{\mathrm{fy}*\mathrm{d}} = \frac{20.6*10^3}{420*4800} = 0.0102$

but $\left(\frac{\mathrm{Avh}}{\mathrm{s}}\right)\mathrm{min} = 0.0025 * \mathrm{h} = 0.0025 * 250 = \mathbf{0.625} \ll$ Controlled.

→ Avh : For 2 layers of Horizontal Reinforcement

Select Ø10 :

Avh = 2 *79 = 158 mm^2

$\frac{Avh}{s} = 0.625 \rightarrow Sreq = \frac{158}{0.625} = 252.8\ mm$

Smax= Lw/3 = 6000/3 = 2000 mm

= 3h = 3*250 =750mm

$= 45cm \ll$ Controlled.

∴**Select Ø10 @ 200 mm at each side.**

4.7.2 Design of Vertical Reinforcement

→ Avv = $[0.0025 + 0.5\,(2.5 - \frac{hw}{lw})(\frac{Avh}{Shor*h} - 0.0025)] * h * Sver$

$\frac{hw}{lw} = \frac{28}{6} = 4.667 > 2.50$

→ $\frac{Avv}{Sver} = [0.0025 + 0.5\,(0)(\frac{2*79}{250*250} - 0.0025)] * 250$

∴ $\frac{Avv}{Sver} = 0.5$

Smax= Lw/3 = 6000/3 = 2000 mm

= 3h = 3*250 =750mm

$= 45cm \ll$ Controlled.

Select Ø12 :

Avv = 2 *113 = 226 mm^2

$\frac{Avv}{s} = 0.5 \rightarrow Sreq = \frac{226}{0.5} = 452$ mm

∴**Select Ø12 @ 150 mm at each side.**

CHAPTER 6

RESULTS AND RECOMMENDATIONS

6.1 Introduction

After finish the project and after dealing with problems that we had faced, it is necessary to summarize the results that were reached and to give some recommendations that will be helpful for students who will work on such projects.

The most prominent of these problems was deflection in beams and long term deflection in slabs that could have been solved by using drop beams . So that another solution had been found, and that was through changing the structural system by making two way ribbed slab instead of one way. After dealing with that problem a complete design for all slabs and beams were done and the results of the design is presented in a form of drawings.

6.2 Result

The following are results that had been reached during the work on this project :

1. The most important step before starting a design is to study the architectural plans carefully to distribute the columns correctly.
2. Gaining experience in using structural programs cannot be reached without an understanding of basic concepts of the structural design.
3. When choosing the structural system it is better to distribute ribs in the long direction and beams in the short one that will reduce loads that act on beams which leads to reducing of reinforcement which meant reducing costs.

6.3 Recommendations

After finish the project and after dealing with problems that we had faced, some recommendations should be mentioned that may help students who will work on such projects after us.

First of all, the architectural drawings had to be prepared and studied carefully to choose the most appropriate structural system. Collecting data about the project is an important step as the study of the site and the type of soil are important in choosing the construction materials to be used. Before starting the design of the building a good structural planning must be done to determine the location of columns, beams, and shear walls to fit with architectural plans.

References

[1] Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE7-16).

[2] Building code requirements for structural concrete (ACI-318-14), USA: American Concrete Institute, 2014.

[3] Uniform Building Code : UBC -97 code .

[4] كود البناء الأردني, كود الأحمال والقوى, عمان,الأردن: مجلس البناء الوطني الأردني, 2006م.