Palestine Polytechnic University

College of Engineering

Department of Civil Engineering

Building Engineering

Hebron – Palestine

The Graduation Project
Structural Design For A Residential Commercial Building/Hebron

Team of Work

Ahmad Ghayyadah

Mohammed Al Mohtaseb

Supervisor

Dr. Abed Al Sameea Halahla

May, 2023

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"Graduation Project Submitted to the Civil Engineering Department in Partial Fulfillment of the Requirements for the Degree of B.Sc. in Civil Engineering depends on the instructions of the project supervisor and with the consent of all members."

Project Supervisor Signature

Dr. Abd Al Samee Halahla

Signature of the Head of Department

Dr. Bilal Al-Masri

Dedication

To those who have taken credit for us from our birth to this day, to those who taught us to read and write, to those who motivated us every time we felt we were going to fail.

To those who made great efforts to convey all the information to us, our dear doctors.

To those who gave us everything they could, and their prayers protected us from all harm, and always remained by our side, our dear mothers.

To those whose names we proudly bear, from whom we learned human values, and who accompanied us with their great hearts, our dear fathers.

To those who looked upon us with pride after every success, our brothers, sisters and friends.

Acknowledgement

Praise be to God in the beginning and in the end, it did not come with our hard work, not even with our efforts, but rather with the blessings of God upon us.

First of all, thanks to our esteemed university, Palestine Polytechnic University, which was the best choice for us.

Moreover, the greatest thanks go to our supervisor Eng. Abed Al Sameea Halahla, who did not spare us anyguidance and any information, may God reward her with all the best.

The final and all thanks are given to all the college Doctors, without whom we would not have been able to reach this final stage in our university life, and to all those who provided us with any great advice or assistance.

Abstract

The work of a residential building project was chosen because the city of Hebron and other cities of the West Bank suffers from overcrowding and the scarcity of land allowed for construction due to the occupation's limitation of the areas allowed for construction, and the division of land into areas (A, B, C).

This project aims to create a safe, feasible and economical structural design for a residential building.

As the process of distributing columns and studying the behavior of the building under the influence of main and horizontal loads, choosing the best type of knots, walls and foundations, and following the best methods and solutions from an economic and safety point of view, is the main objective of this project.

This building consists of 10 floors, with a total area equal to 4456 square meters, and it has 2 staircases in addition to 1 electric elevator. A dam for the warehouse with an area of 400 square meters, and it contains five residential floors with an area of 482 square meters, each floor contains two apartments, It also contains a roof floor, in addition to a floor containing a roof with an area of 236 square meters, and a repeating staircase with an area of 40 square meters..

After the completion of the project, we expect to be able to provide a structural design for all structural elements of the project in accordance with the requirements of the American and Jordanian code.

In this project, I will use several engineering programs, namely:

1- AutoCAD.

2- Ater.

3-Safe.

4-SP Column .

5-E Tabs

الملخص

تم اختيار عمل مشروع عمارة سكنية وذلك لما تعانيه مدينة الخليل وغيرها من مدن الضفة الغربية من اكتظاظ سكاني وقلة الاراضي المسموح البناء بها بسبب تحديد الاحتلال للمساحات المسموح البناء بها،وتقسيم الاراضي الى مناطق (A,b,c) .

يهدف هذا المشروع لعمل تصميم انشائي أمن قابل للتنفيذ واقتصادي في نفس الوقت لعمارة سكنية .

اذ أن عملية توزيع الاعمده ودراسه سلوك المبنى تحت تأثير الأحمال الرئسية والأفقية واختيار أفضل نوع للعقدات والجدران ولأساسات واتباع أفضل الطرق والحلول من الناحيه الاقتصادية و الامان هو الهدف الأساسي لهذا المشروع.

يتكون هذا المبنى من 10طوابق بمساحة كلية تساوي 4456 متر مربع ويوجد فيه 2 بيت درج بالاضافة الى1 مصعد كهربائي حيث يحتوي اول طابقين تسوية على مواقف سيارات يحتوي الطابق على 9مواقف ومساحة الطابق 444متر مربع ، والطابق الارضي بمنسوب 0:0 يستخدم كمستودع بمساحة 482متر مربع بالاضافة الى سدة للمستودع بمساحة 400متر مربع ، ويحتوي على خمس طوابق سكنيةبمساحة 482 متر مربع كل طابق يحتوي على شقتين ،ويحتوي ايضا على طابق روف ، بالاضافة الى طابق يحتوي على روف بمساحة 236مربع،ومكرر بيت الدرج بمساحة 40متر مربع .

وبعد إنجاز المشروع نتوقع أن نكون قادرين على توفير تصميم هيكلي لجميع العناصر الهيكلية للمشروع وفقاً لمتطلبات الكود الأمريكي والأردني.

في هذا المشروع سوف استخدم العديد من البرامج الهندسية وهي:

1- الاوتوكاد .
2- العتير.
3- السيف
4- اي تابس
5- اس بي كولوم

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List of abbreviation:

- 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 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.
- W = width of beam or rib.
- W_u = factored load per unit area.
- $\emptyset$ = strength reduction factor.
- ε_c = compression strain of concrete = 0.003.
- ε_s = strain of tension steel.
- $\acute{\varepsilon}_s$ = strain of compression steel.
- ρ = ratio of steel area.

CHAPTER

1 Introduction

1.1 INTRODUCTION

1.2PROJECT PROBLEM

1.3PROJECT OVERVIEW

1.4OBJECTIVE OF THE PROJECT

1.5PROJECT METHEDOLOGY

1.6REASONS FOR CHOSING THE PROJECT

1.7SCOPE OF THE PROJECT

1.8PROJECT SCHEDUAL

1.1 Introduction

All countries of the world, and especially third world countries, are experiencing profound transformations and tremendous changes, the source of which is the high population increase and the resulting overcrowding of the population on the one hand, and the increase, development and growth of cities in terms of number, size and shape (morphology). The two phenomena of the demographic explosion and the emergence of huge cities (the urban explosion) coincided with the beginning of the Renaissance and the industrialization movement.

Where these factories were the magnetic cores of attractive to the rapidly growing population and those looking for opportunities to work in factories and leave the countryside and villages.

As for the countries of the third factor, these phenomena were not known sharply until after the middle of the twentieth century, when most of the countries under European colonialism gained their independence, and by benefiting from the achievements of European civilization, as they had a strong impact on demographic shifts through the massive and unprecedented population increase in history, and from That began the problems of demographic growth on the one hand and the problems of urban growth on the other hand, due to the rural exodus coming to the cities and the transformation of large rural villages and agricultural and forest areas into areas of population overcrowding automatically or randomly.

Since the end of World War II and until 1980, the population of cities in the Third World quadrupled, and this increase (estimated at about 700 million people) corresponds to the total urban population in the world in 1950.

What is remarkable at the beginning of the new millennium is that more than half of the world's population currently lives in cities, and that the number of million cities has exceeded 100, while the giant cities whose population exceeds ten million people has become constantly increasing to the extent that there are cities with a population of more than twenty million people and most of them It is located in developing countries, and the number is expected to double in the coming years due to the high population growth that causes urban explosion and rural erosion, especially in third world countries.

In a report issued by the Population Department of the United Nations, its data disclosed that the world's urban population currently represents (mid-2018) 3.55%, with an expected increase to 4.60% and then to 4.68% by 2030 and 2050, respectively. . In the sense that the shift from living in rural areas to urban areas will continue in the future to expand the scope of urban life by sweeping more rural areas, transforming them into urban areas, which results in an increase in the number of cities and enlargement of some of them, especially in developing countries, as revealed by the statistical data that It expects urban areas to add an estimated growth of 5.2 billion people within the next three decades, the same number as the entire world population at the middle of the last century, and that nearly 90% of this increase will occur in the continents of Asia and Africa. (1)

In Palestine, the number of Palestinians residing in the State of Palestine at the end of 2021 is estimated at about 5.3 million, including 3.2 million individuals in the West Bank (59.6%), and 2.1 million individuals (40.4%) in the Gaza Strip. Concerning the distribution of the Palestinian population among the governorates, the data indicates that Hebron governorate recorded the highest percentage of the population, reaching 15.0% of the total population in the State of Palestine, then Gaza governorate, which recorded 13.7%, while the population percentage in Jerusalem governorate reached 9.0%. The data also indicates that the governorate of Jericho and the Jordan Valley recorded the lowest percentage of the population at the end of 2021, at 1.0% of the total population in the State of Palestine. (2)

Hebron is a Palestinian city, and the center of Hebron Governorate. It is located in the West Bank, about 35 km south of Jerusalem. Today, it is the largest city in the West Bank in terms of population and area, with a population of 222,136 in 2021. It is also an important commercial center in Palestine. (3)

As a result of this large population growth in this city, it was necessary to find an appropriate solution to contain the large number of residents while exploiting the space at the same time, although most people prefer independent residential homes separated from buildings, but it is necessary to adopt residential buildings to mitigate inflation Horizontal urban.

Recently, citizens in the city of Hebron have been complaining about the high prices of lands, and the reason for this rise is due to the scarcity of land areas offered for sale in the city after many citizens from Hebron and owners of capital purchased lands in various areas of the governorate. For example, in the city of Dura in the Hebron governorate, the price of one dunam in the city is not less than 100,000 dinars, while the price of a dunam in villages ranges from 15,000 to 50,000 dinars. The increase in some areas has reached 300%, so that some areas increase their prices monthly, such as the Sinjar area, which connects between Dura and Hebron. The last five years witnessed a remarkable rise in land prices in the city of Dora, to reach in certain areas to 220 thousand dinars per dunam, after it was only 35 thousand dinars, and this increase in land prices led to the difficulty of horizontal expansion of residential houses. (4)

Among the factors that led to the difficulty of the horizontal expansion of buildings in the West Bank regions in particular, are the areas classified (C) and (H2), as these areas, according to the Oslo Accords between the Palestine Liberation Organization and Israel, are the most prominent targeting square in the demolition of Palestinian buildings approved by the occupation authorities. , which constitutes 61% of the total area of the West Bank, under the pretext that it was established in violation of the provisions of the Israeli master plans; Where the Palestinians residing there must obtain the necessary permits from the Israeli Civil Administration for construction and land reclamation for any purpose whatsoever; On the other hand, successive Israeli governments are taking decisions to confiscate the lands of Palestinian citizens to build more settlement housing units.

1.2 Project problem

The problem is centered in analyzing the residential building from the structural point of view, and thenmaking the architectural and structural design of all the residential building's elements.

An analysis and design of beams, columns and ribs will be done.

Dimensions for the different sections and Reinforcement for all selected structural elements will be specified.

1.3 Project overview

The residential building consists of parking lots distributed on two floors below the street level (second settlement and first settlement), and the ground floor is a commercial warehouse, and the remaining five floors are apartments. Each floor has two apartments in addition to a roof floor for one apartment.
The total area of the building is about 4456 square meters, as the area of the first and second settlement is 444 square meters, and the area of the ground floor and the repeated floors from the first to the fifth is about 478 square meters, and the area of the roof floor is 236 square meters.
The building also contains an ascending staircase with an elevator in the middle to facilitate access and movement, and public safety measures have been taken into account by providing an additional stairway to escape in emergency cases.
In our project, we seek to apply what we learned during our university years in the field of civil engineering at Palestine Polytechnic University, where we will do the structural analysis and design of the facility, taking into account the architectural elements in it.

1.4 Objective of the project

1. Calculation of the different loads to which the structural structure is subjected.
2. Selection of the proportional structural system for the design based on quality and cost.
3. Distribution of structural elements on the basis of architectural plans.
4. Analysis and design of structural elements based on ground and horizontal loads, especially Seismic load.
5. Using manual methods in designing, analyzing and using the experiences and knowledge gained in previous courses.
6. Use of structural design software

1.5Project Methodology

1. Studying the architectural plans and making the necessary modifications.

2. Distribution of columns and then making beams and ribs in a way that achieves the economic aspect.

3. Analysis of the loads affecting the structural elements.

4. Design of all structural elements.

5. Writing the project text depending on scientific research methods.

1.6Reasons for choosing a project

This project was chosen for several reasons:

Practical application of what has been learned regarding the structural design of the various elements in a series of reinforced concrete courses.

The use of various programs in the design of columns, slabs, beams and foundations.

Identify the mechanism through which projects are designed in the market.

Choosing the best structural solutions.

This project is submitted to the Department of Civil Engineering in order to fulfill the conditions for graduation and obtaining a Bachelor's degree in Building Engineering.

1.7 Scope of the project

This project contains five chapters:

1. Chapter One: It includes the general introduction to the project.

2. Chapter Two: It includes the architectural description of the project.

3. Chapter Three: It includes the structural study of the project, including its structural elements and loads, and the functional description of these elements.

4. Chapter Four: Analysis and Structural Design of Structural Elements.
5.Chapter Five: Findings and Recommendations

1.8Project Schedule

Table 1-1: The Stages of the project (graduation project @ second semester 2022-2023)

(Graduation Project @ Second semester 2022-2023)

Week NO. / Task	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
Select project	■	■	■	■																												
Inception report					■	■	■																									
Collect information about the project							■	■																								
Architectural study of the building									■	■	■																					
Structural study of the building												■	■	■																		
Prepare the introduction															■	■																
Display the introduction																	■															
Structural analysis																		■	■	■	■	■	■									
Structural design																						■	■	■	■							
Prepare the project plans																									■	■	■	■				
Write the project																												■	■	■	■	
Project presentation																																■

Figure 1:Table 1-1: The Stages of the project (Introduction of graduation project @ first semester 2022-2023)

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CHAPTER

2

Architecture Description

2.1 INTRODUCTION

2.2 BASIC ELEMENT OF Residential Commercial Building

1.2.2 Interior spaces
2.2.2 External spaces

2.3 Residential Commercial Building PLANS

2.3.1 Second basement floor plan
2.3.2 First basement floor plan
2.3.3 Ground floor plan
2.3.4 Mezzanine floor plan
2.3.5 First floor plan
2.3.6 Second floor plan
2.3.7 Third floor plan
2.3.8 Fourth floor plan
2.3.9 Five floor plan
2.3.10 Roofing plan

2.4 Residential Commercial Building ELEVATIONS

2.4.1 Northwest
2.4.2 Southwest
2.4.3 Southeast
2.4.4 Northeast

2.5 Residential Commercial Building SECTIONS

2.5.1 Section A-A
2.5.2 Section f-f

2.1 Introduction

The site analysis process is the first step before architectural design; In order to determine the exact mechanism of the building orientation process and then design it architecturally considering the basic functions, the best view and many other things.

The architectural design process begins with the idea of the project, then the final form of the project, taking into account all the details such as ventilation, lighting, circulation, parking lots, and all services and other details such as spaces, openings, defining columns, locations, and other things...

Then comes the structural design of the building, through which all structural elements such as beams, columns, slabs and foundations are determined with certain dimensions and characteristics based on the nature of the load on each of them.

Since commercial residential buildings provide many social services, unlike other buildings, the project was chosen to be the design project for a commercial residential building in Hebron, which is intended to be multi-use and takes into account the requirements of the people of Hebron.

2.2 Basic Elements of Building.

The project areas are divided into internal and external spaces tied together to reach the goals that

we need it:

2.2.1 Interior spaces

The interior area of the project is 4456 m^2.

Interior spaces divided to:

1-Car parking: Car parking is located in the two underground floors, where each floor can accommodate 9 cars, with an area of 444 square meters per floor.

2- Stores (Warehouse): The building contains a floor and a block used as a warehouse for goods, with an area of the store floor of 482 square meters and a block of 400 square meters.

3- Residential apartments: The building contains five floors as residential apartments. The floor contains two apartments with a balcony for each apartment, three bedrooms, 2 bathrooms, and two living rooms, with an area of 482 square meters for each floor.
4- Roof: The building contains a roof layer with an area of 236 square meters divided into 2 roofing

2.2.2 External spaces

It contains small green spaces and parking for two vehicles.
Corridors for cars to enter and exit from the garages

2.3 Building Plans

Building consists of 10 floors as follows:

2.3.1Second basement floor plan

The plan for the first car park is at -5.25m, with an area of 444 square meters, containing 9 cars.

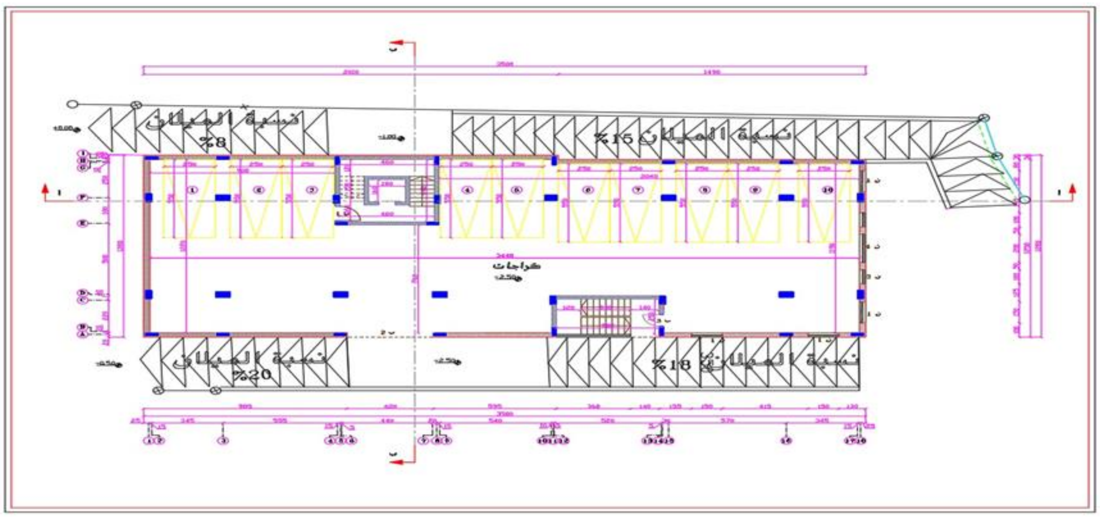

Figure 2-1: Basement two Floor

Figure 1

2.3.2First basement Floor Plan:

The plan for the first car park is at -5.25m, with an area of 444 square meters, containing 9 cars.

Figure 2

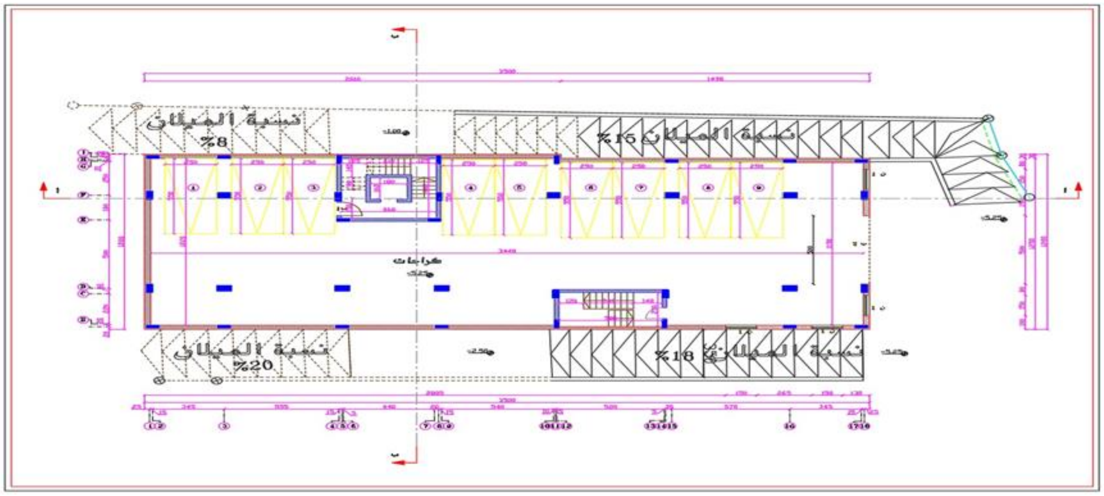

Figure 2-2: Basement One Floor

2.3.3Ground Floor Plan:

Warehouse of 482 square meters on a level of 0.00 m

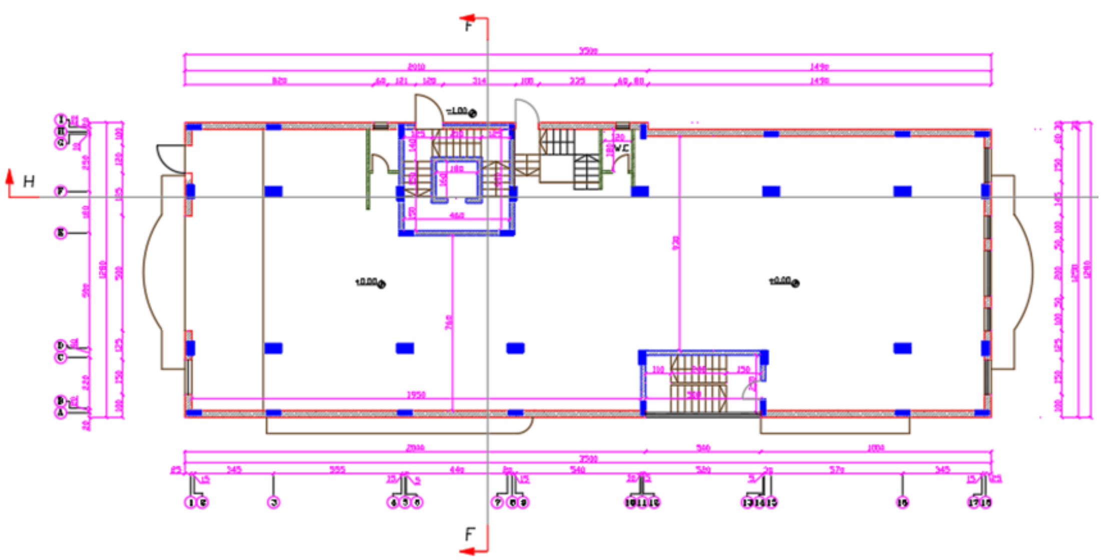

Figure 3

Figure 2-3: Ground Floor

2.3.4 commercial block plan:

A dam attached to a warehouse of 400 square meters at +3m level.

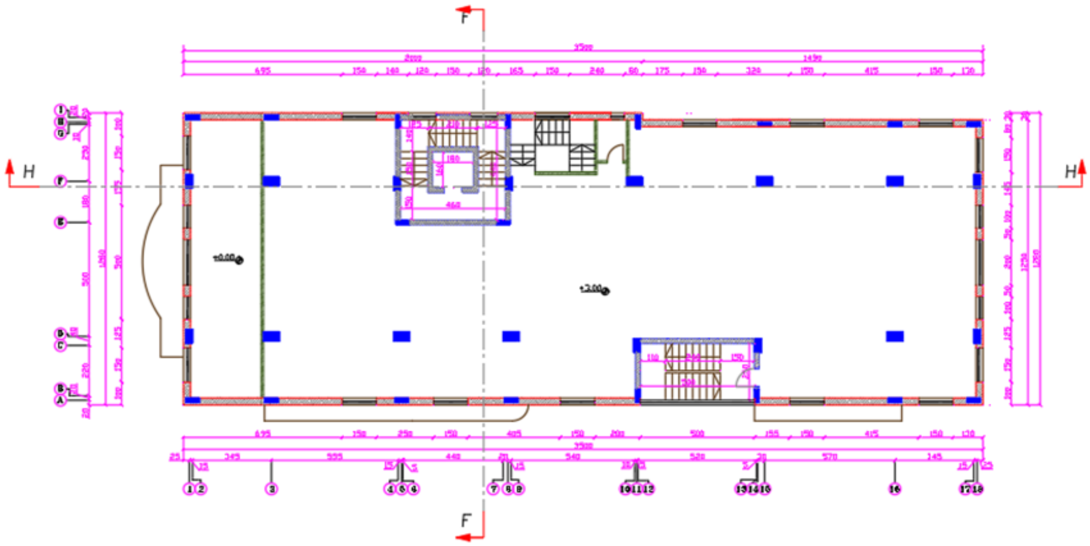

Figure 4

Figure 2.4 commercial block Floor

2.3.5 First Floor Plan:

2.3.6 Second Floor Plan:

2.3.7 Third Floor Plan:

2.3.8 FourFloor Plan:

2.3.9 Five Floor Plan:

The five floor contains two apartments, each apartment contains three bedrooms, a living room, in addition to a salon and a balcony with an area of 482 square meters , it located at level +6.00

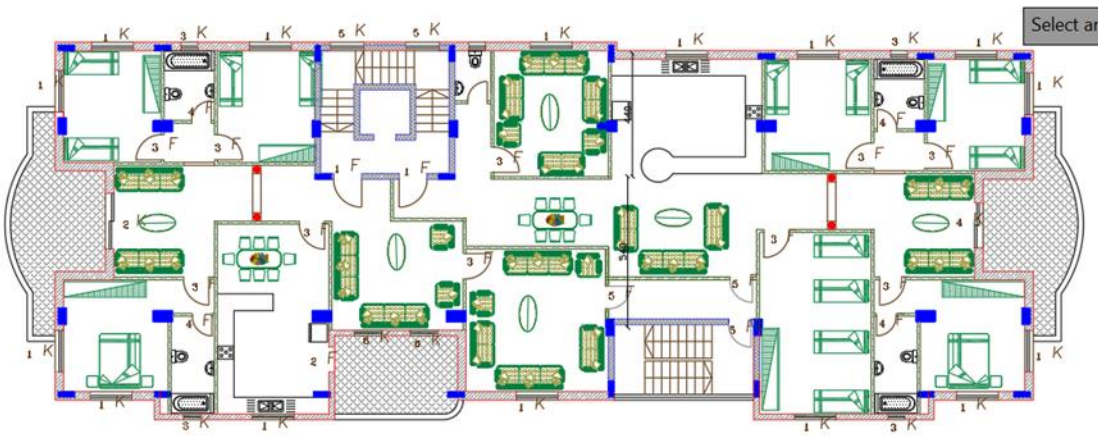

Figure 5

Figure 2.5 : Apartments Floors

2.3.10 floor plan roof.

Roof floor with an area of 236 square meters at +21m level

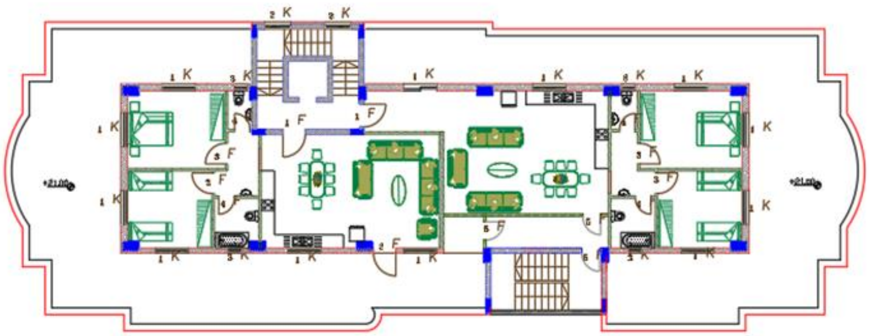

Figure 6

Figure 2.6: plan roof Floor

2.4 Building Elevatns

The design of building facades is considered one of the most important works of building design. The first factor for judging a building with success or failure from an architectural point of view is the facade of the building because it is the visible part of the architectural work.

In architecture, the building's front façade is the most important part from a design point of view, as it determines the style of the remaining parts of the building. Many of the front façades have historical value and local zoning regulations or other laws greatly limit or even prohibit alteration of these façades

2.4.1Northwest elevation:

This interface shows the difference in levels and the distribution of windows in the interface.

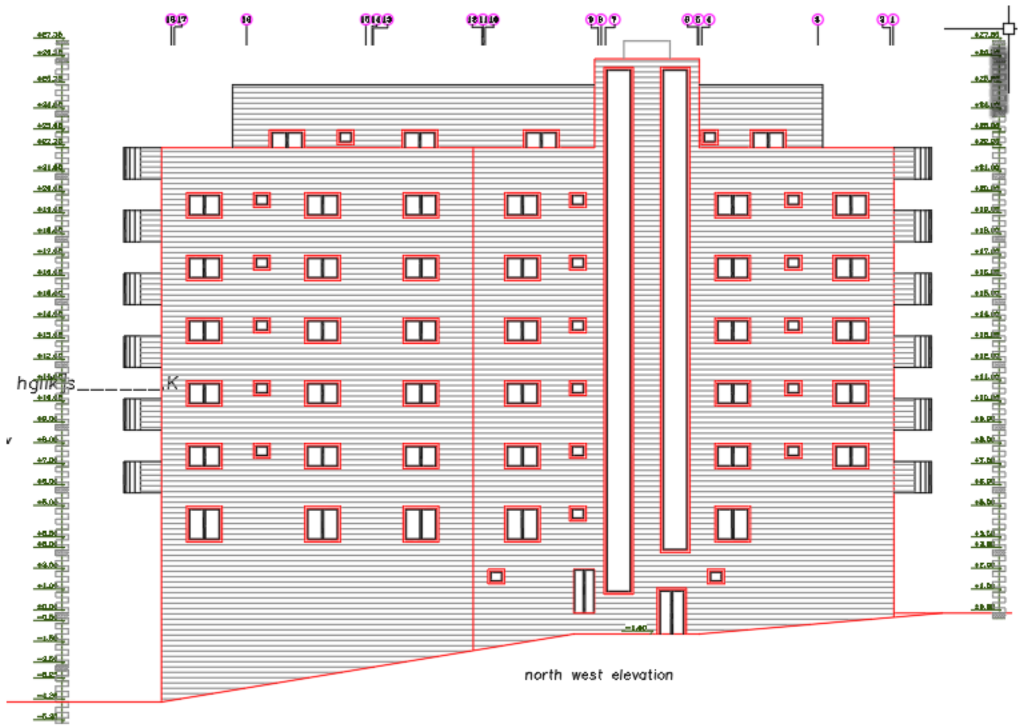

Figure7

Figure 2-7: Northwest elevation:

2.4.2 South east:

The southern elevation shows the difference in the heights of the building and the different facades. It shows the symmetry of the southeastern façade with the northwestern one on the upper floors, and shows the presence of an emergency staircase.

Figure 8

Figure 2-8: South east Elevation

2.4.north east

On this elevation, the entrance to the first parking lot appears, 5.25 F.

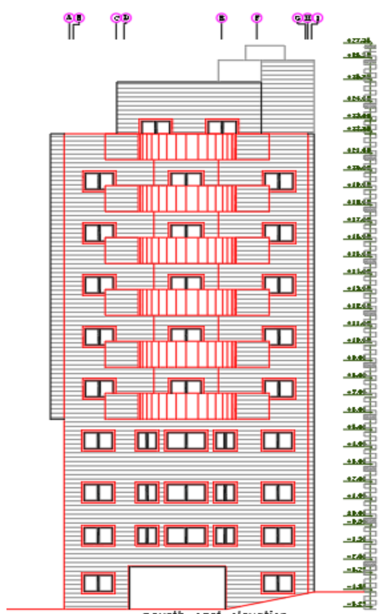

Figure 9

2.4.4 South west:

The southwestern facade shows the main entrance to the warehouse and the apartments at street level, and the level difference between the floors is visible from the northeastern facade.

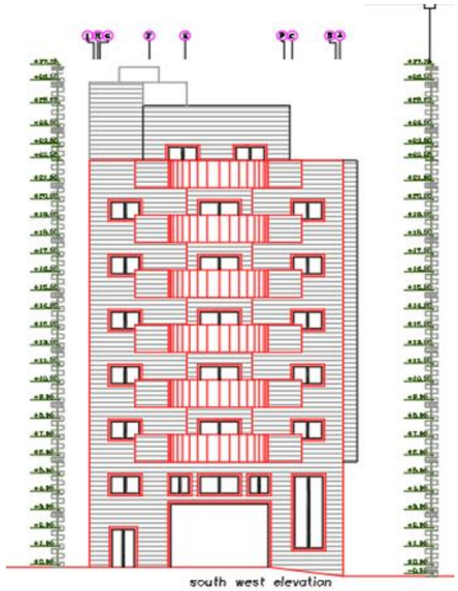

Figure 10

Figure 2-10: Northwest Elevation

2.5 Building Sections

The sections are important to clarify some of the details inside the building.

2.5.1 Section A-A

This section was taken in the staircase, and shows some interior details such as doors, seats, etc.

Figure 11

2.5.2 SectionF-F

This section shows us many interior details such as the doors and their heights, in addition to the height difference in the entrance to the F-1 floor and the ground floor.

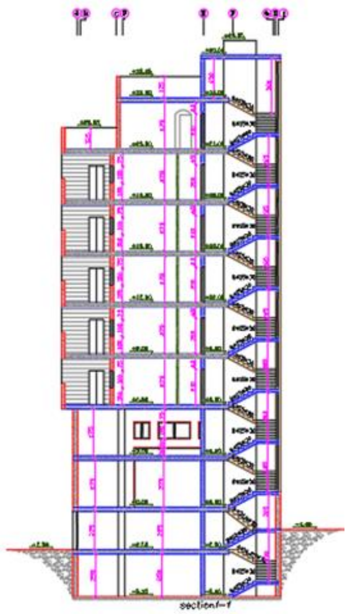

Figure 12

CHAPTER

3

Structural Description

3.1 INTRODUCTION.

3.2 THE GOAL OF THE STRUCTURAL DESIGN.

3.3 SCIENTIFIC TESTS.

3.4 STAGES OF STRUCTURAL DESIGN.

3.5 LOADS ACTING ON THE BUILDING.

3.5.1 Vertical Loads

3.5.1.1 Dead Loads

3.5.1.2 Live Loads

3.5.2 Horizontal Loads

3.5.2.1 Earthquake Loads

3.5.2.2 Wind Loads

3.5.2.3 Snow Loads

3.5.3 Secondary Loads

3.6 STRUCTURAL ELEMENTS OF THE BUILDING.

3.6.1 Slabs

3.6.1.1 Solid Slabs

3.6.1.1.1 One Way Solid Slab

3.6.1.1.2 Two Way Solid Slab

3.6.1.2 Ribbed Slabs

3.6.1.2.1 One Way Ribbed Slab

3.6.1.2.2 Two Way Ribbed Slab

3.6.2 Beams

3.6.3 Columns

3.6.4 Shear Walls

3.6.5 Footing

3.6.6 Stairs

3.6.7 Basement Wall

3.1 Introduction:

The main objective of the process design is to ensure the existence of necessary operating advantages with structural elements on the most suitable dimensions in terms of security and economic terms.

The knowledge of structural elements of any project is essential in the design of reinforced concrete structures to make comparisons between different types of these elements for the construction of safer system. So, the structural elements that go into the design of this project will described.

3.2 The Goal of the Structural Design:

The structural design is an integrated and balanced structural system capable of carrying it meet the established requirements and desires of users, and thus determines the structural elements from the following:

- Factor of Safety: Is achieving by selecting sections for structural elements capable of withstanding the forces and resulting stresses.
- Economy: Checked by choosing the appropriate building materials and by selecting the perfect low-cost section.
- Serviceability: To avoid excessive landing (deflection), fissures (cracks).
- Preservation of architectural design.
- Preserving the environment.

3.3 Scientific Tests:

Before the design of any construction project some test must done. For example, tests of the soil to know bearing capacity of the soil, specifications, type, the underground water level and depth of the foundation layer.

3.4 Steps for Structural Design:

We will divide the structural design of the project in two phases:

The first step:

In this step, the appropriate structural system of project construction and analysis for this system will be determined.

The second step:

The structural design of each element detailed and examined according to the chosen construction system and executive structural plans.

3.5 Loads Acting on the Building:

Is a group of forces that is designed to bear, and that any building is subjected to several types of loads must be calculated and selected carefully because any error in identifying and calculating loads reflect negatively on structural design of various structural elements.

The permanent forces and resulting from gravity and location and do not change during the age of the building, and the loads on the weight of structural elements and the weights of the items based upon permanently as cutters and walls, as well as the weight of the body adjacent to the building. Beside the calculation and estimate of the loads by knowing the dimensions of the structural elements and specific gravity of the material used in the manufacture of structural elements. These elements include concrete, steel reinforcement, plaster, bricks, tiles, finishes, and the stone used in building coverage abroad.

3.5.1 Vertical Loads:

It's dividing to:

3.5.1.1 Dead Loads:

These loads result from the self-weights of the structural elements, the self-weight of the building, in addition to the weight of the soil in the retaining walls. These loads are considered to have a permanent effect on the building.

The following table **(Table 3-1)** shows the Specific weights of the materials for which the self-weights are calculated according to the ACI code for loads.

Table 3-1: Specific weights for materials which use at the structural elements

Table 1:Specific weights for materials which use at the structural elements

Material	Specific weight (KN/m^3)
Tile	23
Mortar	22
Sand	17
Hollow Block	10
Reinforced concrete	25
Plaster	22
Partitions	2.3

3.5.1.2 Live Loads:

Buildings of all kinds are exposed to several uses, and for these uses loads may be concentrated or distributed. Live loads are classified into:

1. Static loads: such as furniture and electrical appliances.

2. People Loads: varies according to the use of the building

3. Executing Loads: Like Cranes

Table (3-2) shows the values of live loads according to the Jordanian code of loads and forces.

Table 2:Live loads in different buildings

Type of area	Live load (KN/m^2)
parking & Commercial warehouse	5
mezzanine floor	3
Stairs	4
residential apartments.	4
Roof floor	5

Table 3-2: Live loads in different buildings

3.5.2 Horizontal Loads:

They include snow, earthquake, and wind loads, they vary by amount, location, and direction also it's depend primarily on the unit area they faced.

There are several factors to determine these loads which are:

1. The speed.
2. Building height.
3. The importance of the building.

3.5.2.1Earthquake Loads:

Produce earthquakes of horizontal and vertical vibrations due to the relative motion of the earth rock layers, resulting in strong cut affect the origin, and these loads must take into account during the design to ensure resistance of earthquakes. This will be resisted by shear walls in the building.

3.5.2.2Wind Loads:

It is determined based on the maximum wind speed that varies with the height of building above sea level.

Figure 3-1 shows positive and negative pressures that are formed from wind load.

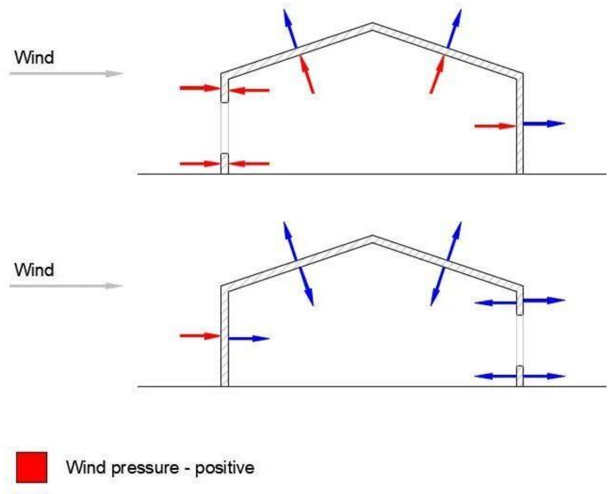

Figure13:wind pressures

Figure 3-1: wind pressures

3.5.2.3Snow Loads:

It is determined based on the height from the sea level.

Table (3.3) Shown the snow loads based on the height from the sea level according to the Jordanian code of loads and forces 2006.

Table 3-3: snow loads based on the height from the sea level

Table 3:snow loads based on the height from the sea level

Height of the building above sea level (h) m	S_0 **(KN/m^2)**
h>250	0
500>h>250	(h – 250) / 800
1500>h>500	(h – 400) / 320

$$S_d = \mu_i * S_0$$

S_0: $Snow\ load\ (KN/m^2)$

$\mu_i = Shape\ Factor\ (0.8)$

Dura is located at 800 m above sea level.

$$S_0 = \frac{1000-400}{320} = 1.875\ KN/m^2$$

$$S_d = 0.8 * 1.875 = 1.5\ KN/m^2$$

3.5.3 Secondary Loads:

Such as shrinkage and expansion loads resulting from the drying of concrete and Settlement of the foundation soil. This problem is solved by placing expansion joints inside the building. We don't need expansion joint at our project.

3.6 Structural Elements of the Building:

All buildings are usually consisting of a set of structural elements that work together to satisfy the continuity of the building and its suitability for human use. The most important of these slabs, beams, columns and load-bearing walls, it must define.

Figure 3-2 shows all structural elements.

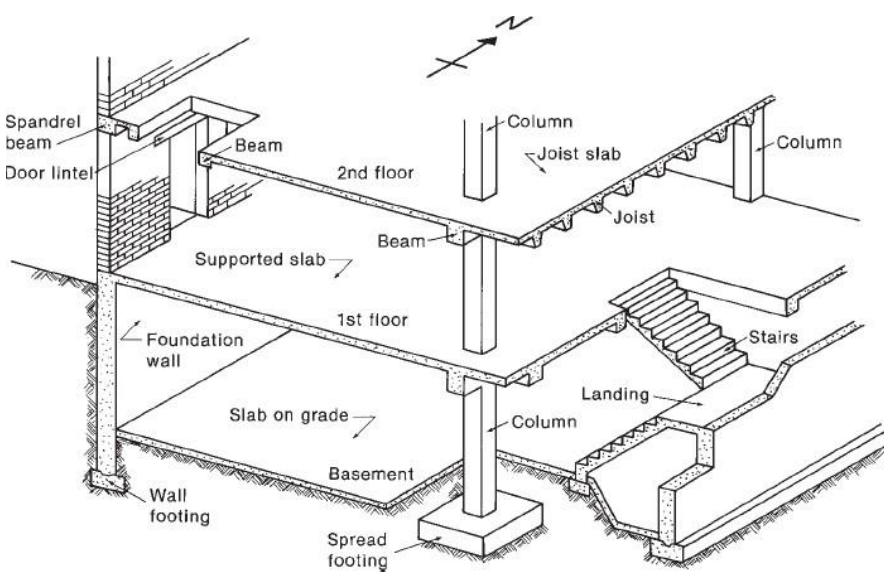

Figure 14

3.6.1 Slabs:

Structural elements such as beams, columns and walls are capable of delivering vertical forces due to the loads affecting on the building.

Slabs divided into two types:

Solid slabs.

Ribbed slabs.

In this project, three types of Slabs will use, and will clarify the structural design in the subsequent chapter, and these types are:

One Way Ribbed Slab.

One Way Solid Slab.

Two Way Ribbed Slab.

The following is a description of the different types of slabs:

3.6.1.1 Solid Slabs:

It's divided into:

3.6.1.1.1 One way Solid Slab:

The solid slab called one way if it has those categories:

L/b is equal or more than 2.

Beams located at two directions.

The concrete slab is cast in one uniform thickness without any voids.

The loads transferring in the short direction and the slab may be treated as a beam.

The design done by using strip of 1m.

Figure 3-3 shows One Way Solid Slab.

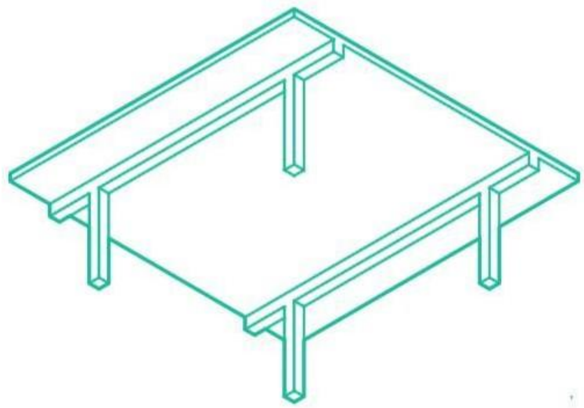

Figure 15
Figure 3-3: One way Solid Slab

3.6.1.1.2 Two way Solid Slab:

The solid slab called one way if it has those categories:

If L/b is less than 2.

Beams are located at four directions to support them.

The loads transferring to all four supporting beams

Figure 3-4 Shows two way Solid Slab.

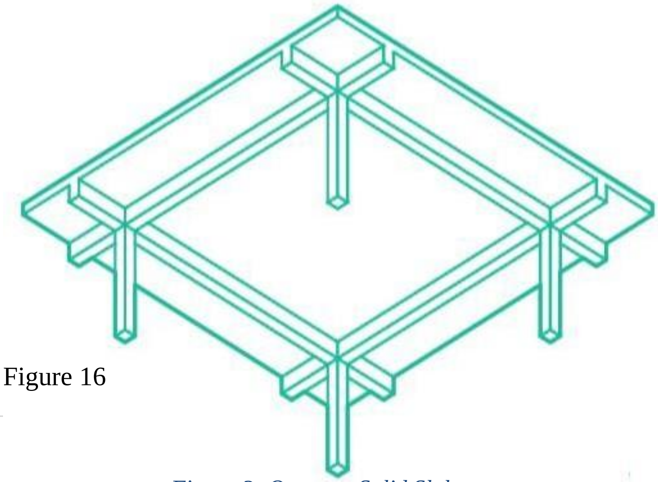

Figure 16

Figure 3- One way Solid Slab

3.6.1.2Ribbed Slabs:

It’s divided into:

3.6.1.2.1 One way Ribbed Slab:

Consists of hollow slabs with a total depth greater than of solid slab.

This system more economical for buildings when the loads are small and the spans large such as schools, residential buildings…

The concrete in the tension zone is ineffective so the area is left open between ribs or filled with lightweight material to reduce the self-weight of the slab.

Figure 3-5 Shows One way Ribbed slab.

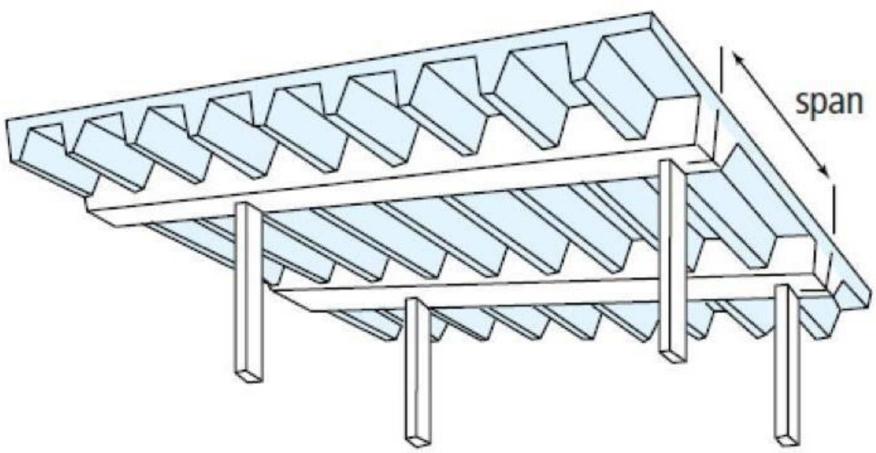

Figure 17

Figure 3-5 One way Ribbed Slab

3.6.1.2.2 Two way Ribbed Slab:

In Two way Ribbed Slab Both ends of the slab become supported by walls or columns at different levels with one of the edges having a waffle design pattern. The Ribs are vertical to one edge, and inclined to the other edge.

It is used to cover large areas, especially when the distances between spans are close and the distances are greater than 6 m.

Figure 3-6 Shows two way Ribbed slab.

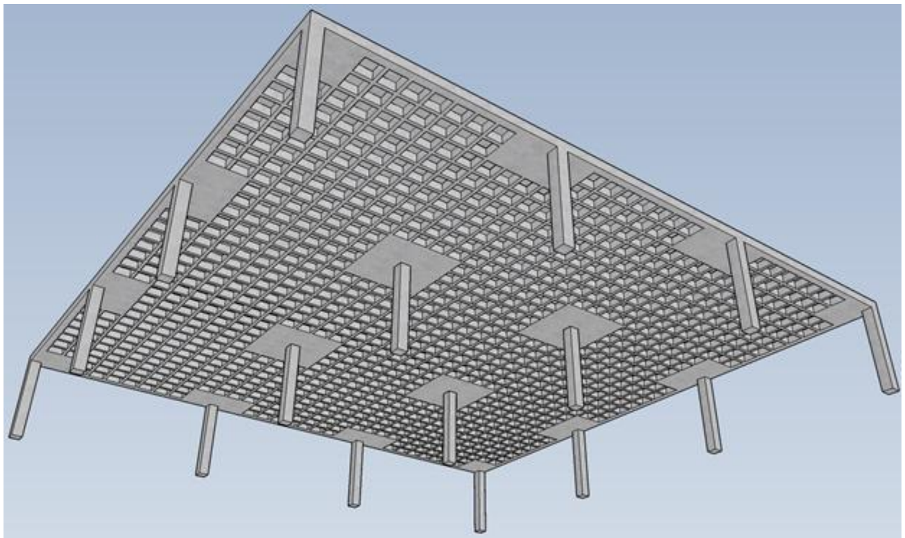

Figure 18
Figure 3-6 two way Ribbed Slab

3.6.2 Beams:

The basic structural elements in moving load of tiles into columns, and are of two types:

Hidden Beam: Hidden inside Slabs, its height equally to the height of slab.

Dropped Beam: its height is larger than the height of slab, also called T-section or L-section.

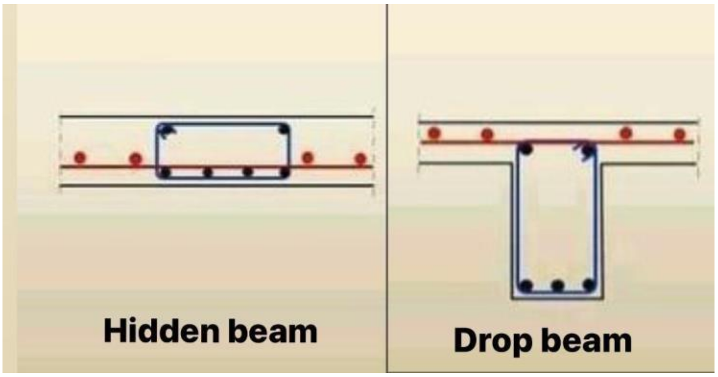

Figure 19

Figure 3-7 Hidden and Drop beams

Beams are used for:

Make frames by attaching columns to each other.

They are placed under the walls to load the walls on them.

Reduce the buckling for columns.

3.6.3 Columns:

Columns carry loads from slabs and beams to the foundations, and there are two types of columns: long columns and short columns.

There are several types for columns, including circular, square, rectangular and composite.

The columns used in this residential building vary between short and long concrete columns. The whole columnsare rectangular.

Figure 3-7 shows the types of columns.

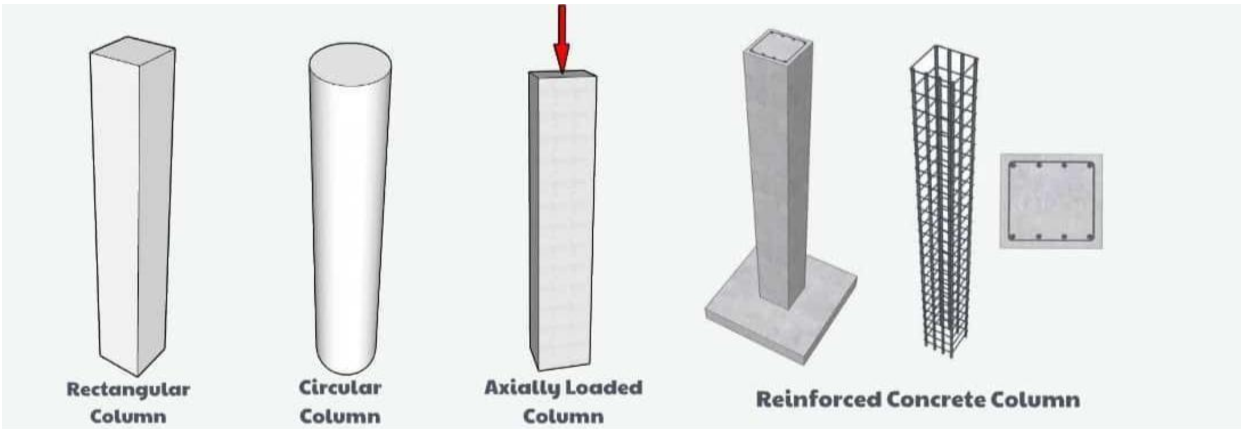

Figure 20

Figure 3-8 Types of columns

3.6.4 Shear Walls:

They are the elements that resist lateral loads such as wind and earthquakes.

It must be taken into account that these walls are available in both directions, and their number is at least 3, two in the same direction and the third at the other direction.

Shear walls shall be sufficient to minimize torque.

Shear walls in this project in stair walls and elevator walls as well as others starting from the foundations.

Figure 3-9 shows shear walls.

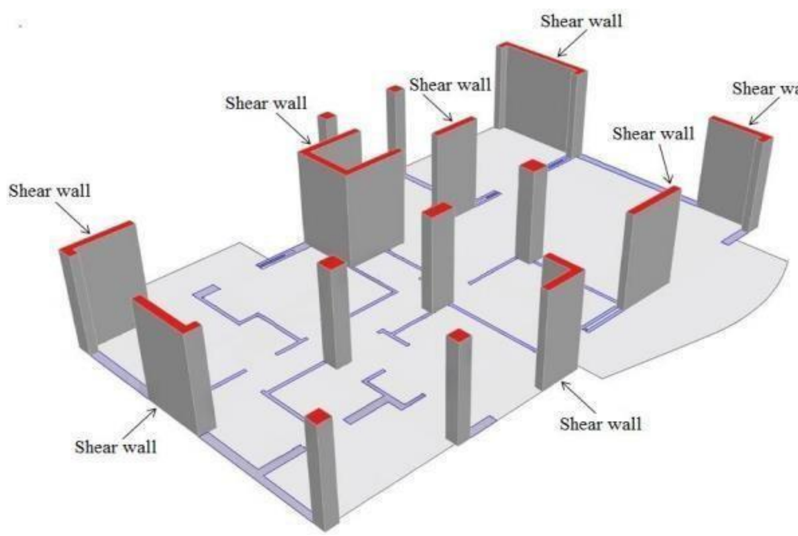

Figure 20 Shear walls

3.6.5 Footing:

It is a structural member used to support columns and walls and to transmit and distribute their loads to the soil.

The last element to be designed and the first to be executed.

Responsible for bearing loads of various kinds: dead loads, dynamic loads and live loads.

The foundation that is close to the surface of the earth is called the **Surface foundation**.

The deep foundation that transfers the loads of the structure to the deep soil layers is called the **Deep foundation**.

There are several types of foundations, including Isolates, wall footing, Mat footing, Strap footing, Pile footing and combined footing.

Several types of foundation are expected to be used in this project.

Figure 3-10 shows Reinforcing details for isolated footing.

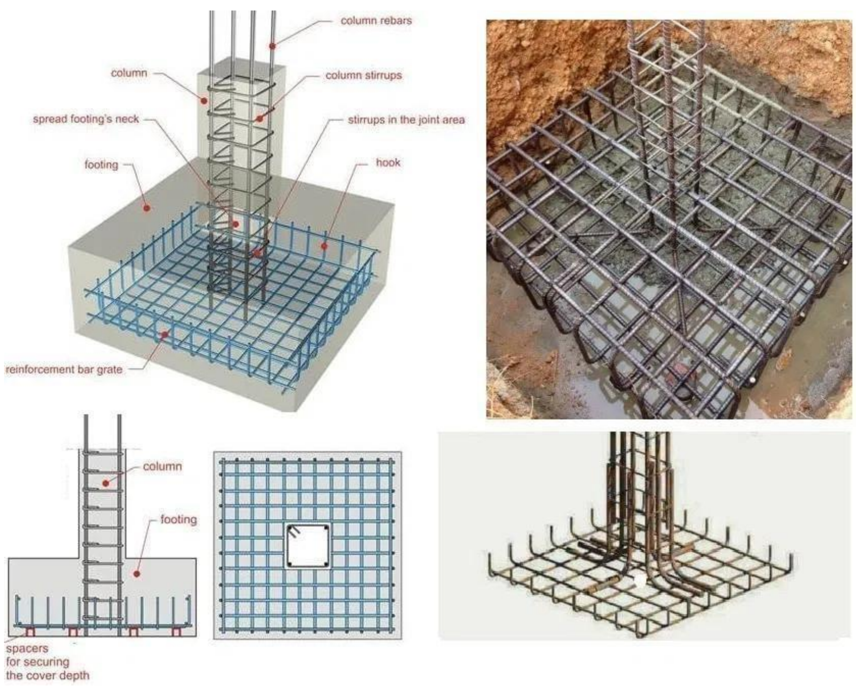

Figure 21 Reinforcement detailing of isolated footing

3.6.6Stairs:

Consist of rises (vertical distance between two steps), runs (The depth of the step) and landings (the horizontal part of the staircase without rises).

There are different types of stairs: Single flight stairs, double flight stairs, three or more flights of stairs and cantilever stairs…

Figure 3-11 shows single flight stair.

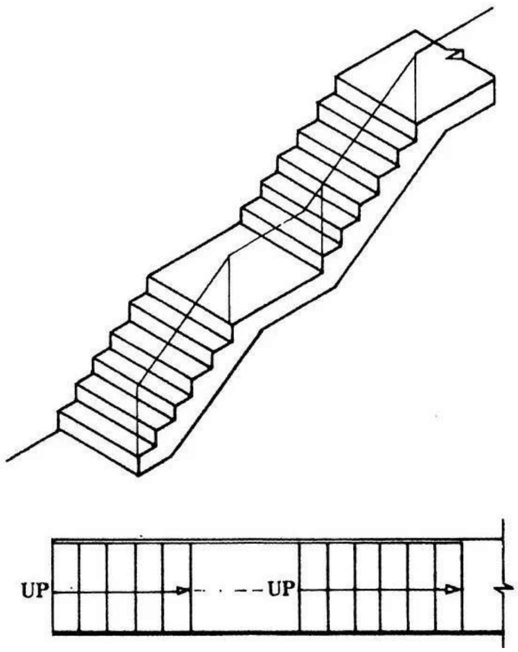

Figure 22 Single flight stairs

Figure 22

3.6.7 Basement walls:

The basement walls are either load bearing to bear the load of super structure or as non load bearing walls.

The basement outer walls are act as retaining wall and it is some time called as cantilever retaining wall because it is free standing structure without lateral supports at top.

The lateral pressure at the top of basement wall is minimum and increased with the depth and maximum at the bottom.

The forces created by the earth may push the basement wall forward or over turn when it is not designed well properly.

Designing the basement wall, all the forces should be taken in to consideration such as earth pressure, rain water effect and vibratory pressure due to side traffic.

The width of foundation is increase from top level of earth up to the bottom of the foundation and the width should be maximum at bottom.

The basement wall should be safe against excessive foundation pressure, sliding and over turning.

Figure 3-12 shows Basement wall.

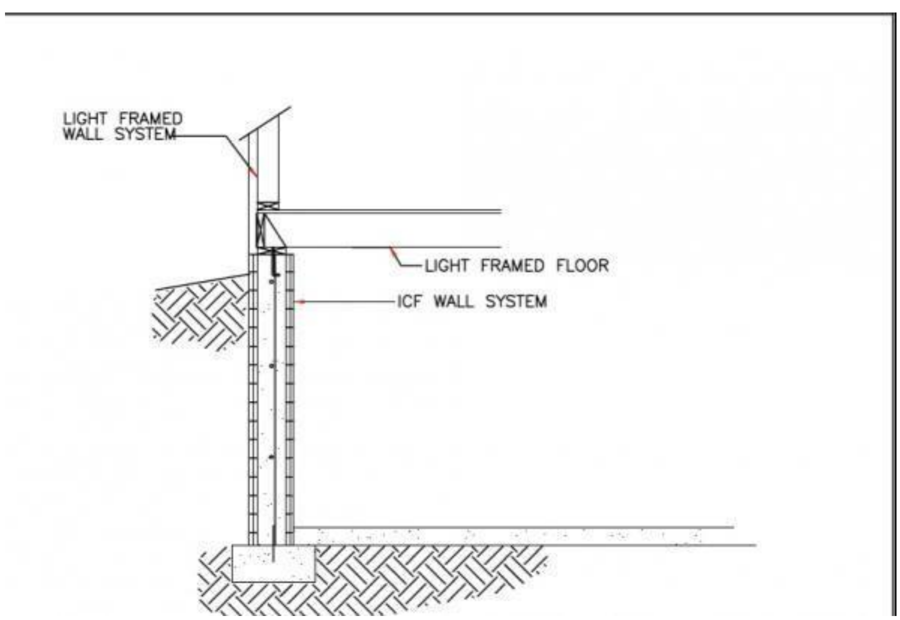

Figure 23-Basement wall

CHAPTER

4

Design Of Structural Members

4.1 Introduction

4.2 Factored Load

4.3 Determination of thickness for one way ribbed slab

4.4 One way ribbed slab design

4.5 Two way solid slab design

4.6 Beam design

4.7 Stair Design

4.8 Isolated footing (F1) Design

4.9 Shear wall Design

4.10 Basement wall Design

4.11 Long column (C1) from group A Design

4.1 Introduction

Concrete is a one of construction materials which is composed of cement, aggregate, water and chemical admixtures.

After mixing the previous components, the concrete will take a shape and it will reach to its hardens, the water will reacts with the cement which bonds the other components together. Concrete is used to make pavements, structures, roads…

In this project there are three types of slabs: one way solid slab, one way ribbed slab and two way ribbed slabs.

"ATER-software" will use to find the shear and moment, then hand calculations will make.

$f'_c = 24\ MPa$

$Fy = 420\ MPa$

4.2 Factored Load

The factored loads for the structural analysis and design for our project determined as follows:

$U = 1.2D + 1.6\ L \ldots\ldots\ldots\ldots.\ ACI - 318 - 08(9.2.1)$

D: Dead Load (KN)

L: Live Load (KN)

4.3 Determination of thickness for one way ribbed slab

Any structure may expose to different types of loads such as dead and live loads (its value depends on the type of structure).

The minimum thickness must satisfy by ACI table (9.5.a) shown below: Minimum thickness for one way ribbed slab:

$h\ min = \frac{l}{21} = \frac{6800}{21} = 320mm$ .... (Both end continuous)

$h_{min} = \frac{L}{16} = \frac{5.55}{16} = 0.346\ m$ $(Simply\ supported\ beam)$

For Rib 3 in the mezzanine floors the thickness is 35 cm including (27cm block and 8 cm topping).

4.4 Design of one way ribbed slab

For Rib 1in the first floor as shown in figure 4-1:

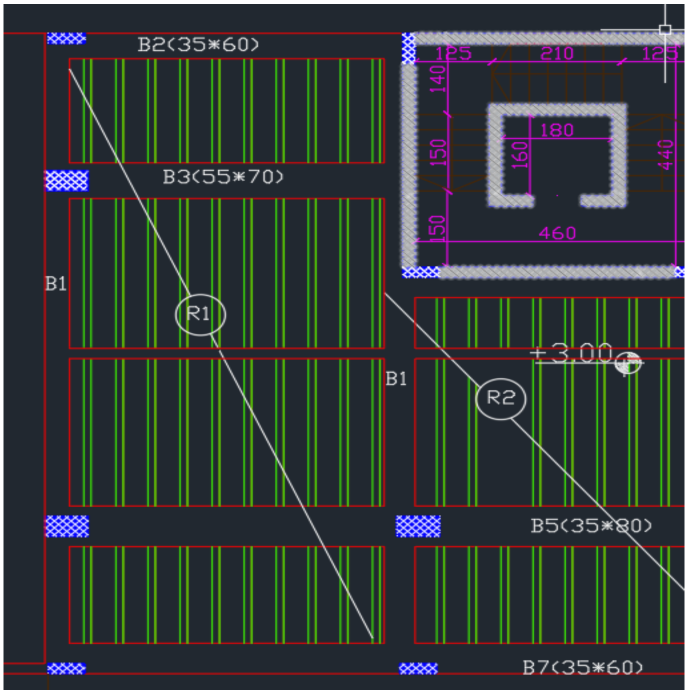

Figure2 4-1 Rib 1

4.4.1 Determination of b_e

Effective Flange width b_e
b_e For T-section is the smallest of: **(ACI 8.12.2)**

$$be \le \frac{l}{4} = \frac{6.8}{4} = 1.7m$$

$$b_e \le b_{\mathrm{w}} + 16\, h_{\mathrm{f}} = 14 + 16 * 8 = 142\, cm$$

. $be \le$ Center to Center spacing between adjacent beam $= \frac{40}{2} + \frac{40}{2} + 14 = 54$……..control

4.4.2 Load Calculations

For one way ribbed slab, the total dead load to be used in the analysis and design as follows:

Table (4 – 1) Calculation of the total dead load for Rib1

Table 4:Calculation of the total dead load for Rib1

#	Dead Load From	Density (KN/m^3)	Calculations	KN/m/Rib
1	Rib	25	0.27* 25 * 0.14	**0.945**
2	Topping	25	0.08 * 25 * 0.54	**1.08**
3	Plaster	22	0.03 * 22 * 0.54	**0.3564**
4	Block	10	0.27 * 10 * 0.4	**1.08**
5	Sand	17	0.07 * 17 * 0.54	**0.6426**
6	Tile	23	0.03 * 23 * 0.54	**0.3726**
7	Mortar	22	0.03 * 22 * 0.54	**0.3564**
8	Partition	2.3	2.3 * 0.54	**1.242**
s				**6.075**

Total Dead Load = $6.075/0.54 = 11.25KN/m^2$
Live Load = 3 * 0.54 = 1.62KN/m/Rib Factored

dead load = 1.2 * 6.075= 7.3 KN/m/Rib

Factored live load = 1.6 * 1.62= 2.59KN/m/Rib

$w_u = 7.3 + 2.59 = 9.9\ KN/m/Rib$

4.4.3 Topping design

We will design the topping as one way solid slab.

Table 5:Calculation of the total dead load for topping

Dead load from	Specific density	calculations	KN/m
Tiles	23	0.03 * 23 * 1	**0.69**
Mortar	22	0.03 * 22 * 1	**0.66**
Coarse Sand	17	0.07 * 17 * 1	**1.19**
Topping	25	0.08 * 25 * 1	**2**
Interior Partition	2.3	2.3	**2.3**
s			**6.84**

Live Load calculations3 * 1 = 3 KN/m Total

Factored Load: $w_u = 1.2D + 1.6\,L$

$w_u = 1.2D$ .L+

1.6L.L=1.2*6.84+1.6*3=13KN/m

For slabs less than 3m or equal the

moment :

Mu =$\frac{WuL^2}{12}$=$\frac{13*0.4*0.4}{12}$=0.173KN .M/M

strip of width

$\emptyset M_n \geq M_u$ – Strength condition, where $\emptyset = 0.55 - for\ plain\ concrete$.

Mn=0.42*$\lambda\sqrt{fc} * Sm$

Where S_m for rectangular section of the slab:

$Sm = \frac{bh^2}{6}$=$\frac{100*80^2}{6} = 1066666.67\ KN.m$

Mn=0.42*1*$\sqrt{24} * 1066666.67$=2.19KN.m

$\emptyset Mn = 0.55 * 2.19 = 1.2045 KN.m$ ≫0.173KN .M……..ok

No reinforcement is required by analysis, so minimum reinforcement will use (Shrinkage and temperature reinforcement):

□□$_{□□□□□□g}$ $= 0.0018$ $(ACI$ 7.12.2.1)

As=$\rho * b * h = 0.0018 * 1000 * 80 = 144 mm^2$ /m

Try bar Ø8 $with\ A_s = 50.27\ mm^2$

$n = \frac{As}{As\emptyset 8} = \frac{144}{50.27} = 2.87 \approx 3bars$/m

Check of $Step\ (s)$is $the\ smallest$:

1- 3h=3*80=240mm…….control

2- 450mm

3- $s = 380 * \frac{280}{fs} - 2.5Cc = 380 * \frac{280}{\frac{2*420}{3}} - 2.2 * 20 = 330mm$

4- S=300*$\frac{280}{fs} = 300 * \frac{280}{\frac{2*420}{3}} = 300mm$

Then take Ø8 @20mm in both direction

4.4.4 Rib1 Design

By using ATIR program we get the envelope shear and moment diagrams as the follows:

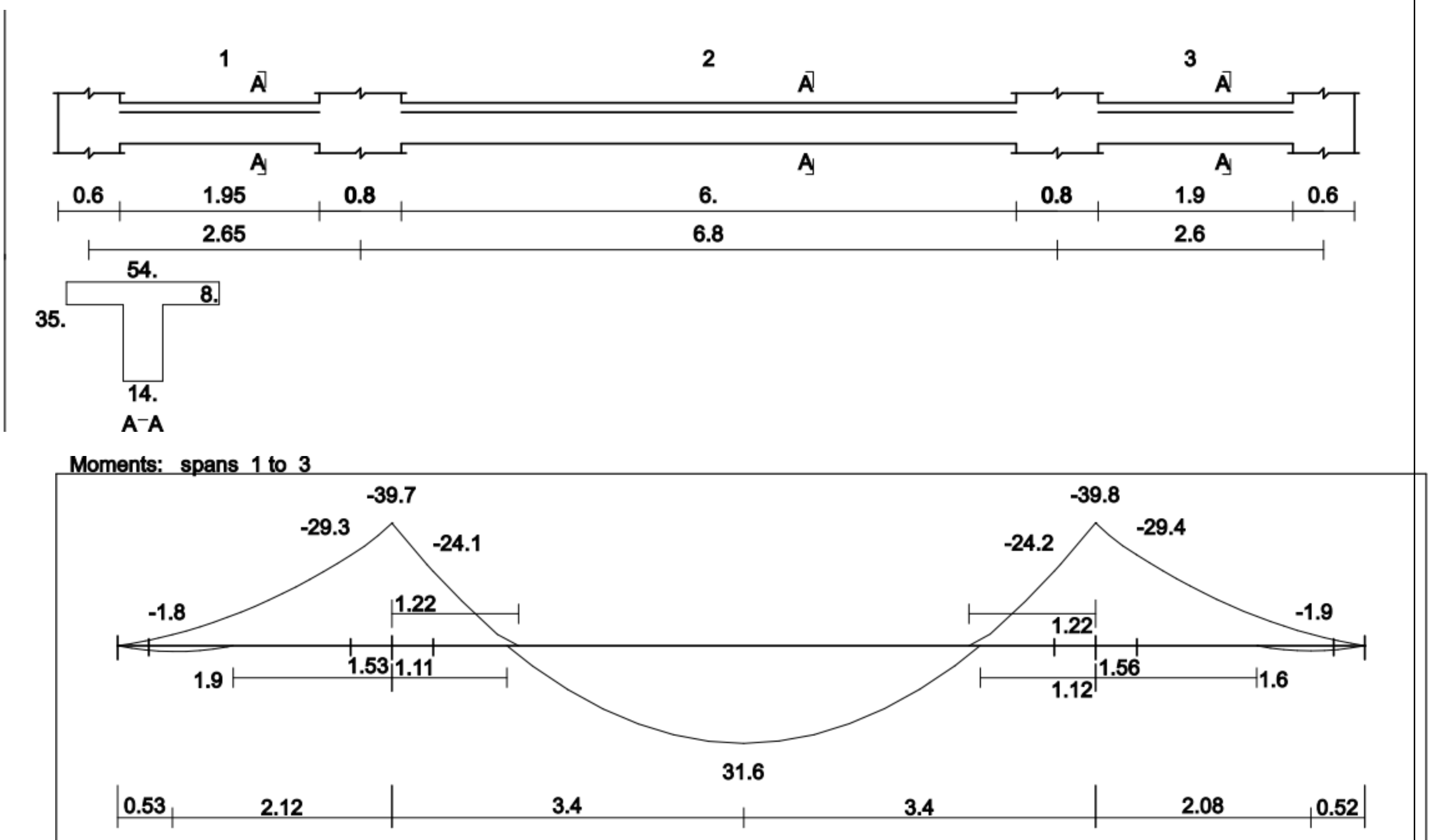

Figure 25:Moment Envelop for Rib3

Figure25-3 Moment Envelop for Rib3(KN.m)

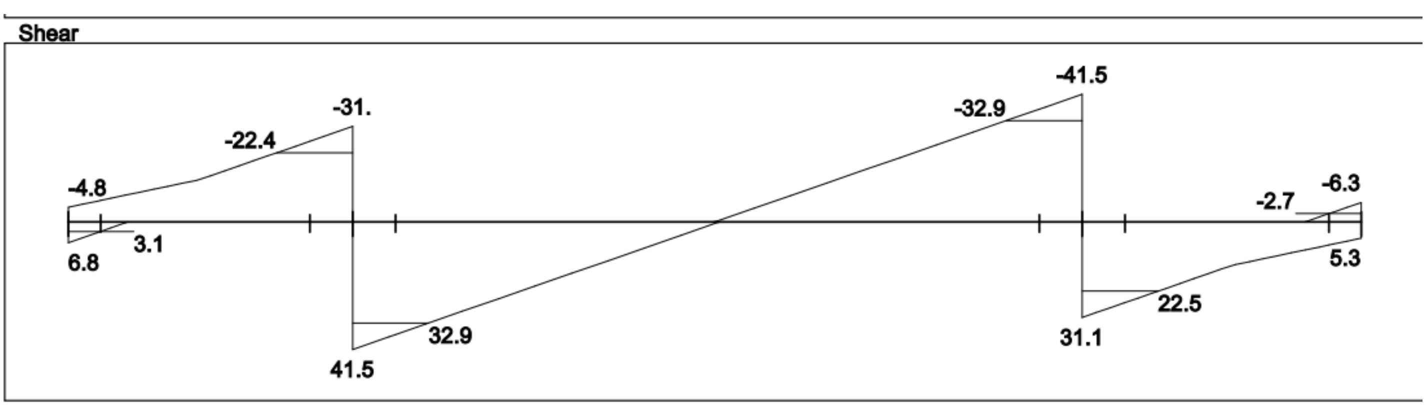

Figure 4-4 Shear Envelop for Rib8 (KN)

Figure 26- Shear Envelop for Rib1

4.4.4.1 Design of Negative Moments of Rib1:

➢ **The Maximum negative moment at the face of support Mu = −29.4KN.m**

Assume bar diameter Ø 16 for main negative reinforcement andØ10 for stirrups.

$$d = h - cover - \text{Ø}stirrups - \text{Ø}\frac{14}{2} = 350 - 20 - 10 - \frac{14}{2} = 313mm$$

$$Rn = \frac{Mu}{\text{Ø}bd^2} = \frac{29.4 * 10^6}{0.9 * 140 * 313^2} = 2.38MPA$$

$$m = \frac{fy}{0.85fc} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{(2 * Rn * m)}{fy}}) = \frac{1}{20.59}(1 - \sqrt{1 - \frac{(2 * 2.38 * 20.59)}{420}}) = 0.00604$$

As=$\rho * b * d = 0.00604 * 140 * 313 = 264.67mm^2$

Check for As, min:

$A_{s,\ min}$ is the maximum of: -

$$As\ min \geq 0.25 * \frac{\sqrt{fc}}{fy} * bw * d \geq \frac{1.4}{fy} * bw * d$$

$$As\ min \geq 0.25 * \frac{\sqrt{24}}{420} * 140 * 313 = 127.8mm^2$$

$$As\ min = \frac{1.4}{420} * 140 * 313 = 146.06mm^2 \text{ ... .control}$$

$$\text{As}=264.67mm^2 > As\ min = 146.06mm^2$$

Try bar Ø14 $with\ A_s = 153.9\ mm^2$

$$n = \frac{As}{As\text{Ø}14} = \frac{264.67}{153.9} = 1.72 \approx 2bars$$

Then take2 Ø14 $As.provided = 307.9mm^2$

Check of strain :

$$\epsilon s > 0.005 \text{ ....} tension - control\ section$$

$$a = \frac{As * fy}{0.85 * fc * bw} = \frac{307.9 * 420}{0.85 * 24 * 140} = 45.3mm$$

$c = \frac{a}{\beta} = \frac{45.3}{0.85} = 53.3mm \ldots .* \ Note\ f' = 24\ MPa < 28\ Mpa \rightarrow B = 0.85$

$$\epsilon s = \frac{0.003 * (d - c)}{c} = \frac{0.003 * (313 - 53.3)}{53.3} = 0.0146 > 0.005 \ldots ok$$

$$\emptyset Mn > Mu$$

$$\emptyset Mn = 0.9 * As * Fy * \left(d - \frac{a}{2}\right) = 0.9 * 307.9 * 420 * \left(313 - \frac{45.3}{2}\right) * 10^{-6} = 33.8KN.m$$

4.4.4.2 Design of positive Moments of Rib1:

- **The Maximum positive moment at the face of support Mu =+ 31.6 KN.m**

Assume bar diameter Ø 14 for main negative reinforcement andØ10 for stirrups.

$$d = h - cover - \emptyset stirrups - \emptyset\frac{14}{2} = 350 - 20 - 10 - \frac{14}{2} = 313mm$$

$$Rn = \frac{Mu}{\emptyset bd^2} = \frac{31.6 * 10^6}{0.9 * 540 * 313^2} = 0.66MPA$$

$$m = \frac{fy}{0.85fc} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{(2 * Rn * m)}{fy}}) = \frac{1}{20.59}(1 - \sqrt{1 - \frac{(2 * 0.66 * 20.59)}{420}}) = 0.0016$$

As=$\rho * b * d = 0.0016 * 540 * 313 = 270.432mm^2$

Check for As, min:

$A_{s,\,min}$ is the maximum of: -

$$As\ min \geq 0.25 * \frac{\sqrt{fc}}{fy} * bw * d \geq \frac{1.4}{fy} * bw * d$$

$$As\ min \geq 0.25 * \frac{\sqrt{24}}{420} * 140 * 313 = 127.8mm^2$$

$$As\ min = \frac{1.4}{420} * 140 * 313 = 146.06mm^2 \ldots \ldots ..\text{control}$$

$$\text{As}=270.432mm^2 > As\ min = 146.06mm^2$$

Try bar Ø14 $with\ A_s = 153.9\ mm^2$

$$n = \frac{As}{As\emptyset 14} = \frac{270.432}{153.9} = 1.75 \approx 2bars$$

Then take2 Ø14 $As.provided = 307.9mm^2$

Check of strain :

$$\epsilon s > 0.005 \ldots .tension - control\ section$$

$$a = \frac{As * fy}{0.85 * fc * bw} = \frac{307.9 * 420}{0.85 * 24 * 540} = 11.74mm$$

$c = \frac{a}{\beta} = \frac{11.74}{0.85} = 13.8mm \ldots ..*\ Note\ f' = 24\ MPa < 28\ Mpa \rightarrow B = 0.85$

$$\epsilon s = \frac{0.003 * (d - c)}{c} = \frac{0.003 * (313 - 13.8)}{13.8} = 0.065 > 0.005 \ldots ok$$

$$\emptyset Mn > Mu$$

$$\emptyset Mn = 0.9 * As * Fy * \left(d - \frac{a}{2}\right) = 0.9 * 307.9 * 420 * \left(313 - \frac{13.8}{2}\right) * 10^{-6} = 35.62KN.m$$

4.4.5 Shear design for Rib1

The maximum shear force at the distance(d) from the face of support **V_u= 32.9 KN**

Shear strengthV_c, provided by concrete for the rib may be taken 10% greater than for beams. This is mainly due to the interaction between the slab and closely spaced ribs. **(ACI, section 8.13.8).**

$$\boldsymbol{vc} = 1.1 * \frac{1}{6} * \sqrt{fc} * bw * d$$

$$\text{vc} = 1.1 * \frac{1}{6} * \sqrt{24} * 140 * 313 * 10^{-3} = 39.35KN$$

$$\emptyset \boldsymbol{vc} = 0.75 * 1.1 * \frac{1}{6} * \sqrt{fc} * bw * d$$

$$\emptyset \boldsymbol{vc} = 0.75 * 1.1 * \frac{1}{6} * \sqrt{24} * 140 * 313 * 10^{-3} = 29.5KN$$

$$\emptyset \frac{\text{VC}}{\mathbf{2}} = \frac{29.5}{2} = 14.75KN$$

$$\emptyset \boldsymbol{vc} < \text{vu}$$

$$29.5\text{KN} < 32.9\text{KN}$$

then we need to shear reinforcement

shear force of stirups equal:

$$vs = \frac{vu}{0.75} - vc$$

$$vs = \frac{32.9}{0.75} - 39.35 = 4.5KN$$

CaseIII:

$V_{s,\min}$ is *the Maximum of*:

1- $V_{s,\min} = \frac{1}{16}\sqrt{fc}$*bw*d=$\frac{1}{16}\sqrt{24}$*140*313*$10^{-3}$=13.4 KN
2- $V_{s,\min} = \frac{1}{3} bw * d = \frac{1}{3} * 140 * 313 * 10^{-3}$=14.6 KN….CONTROL

$$\boldsymbol{\emptyset vc < vu \le \emptyset(vc + vsmin)}$$
$$\boldsymbol{0.75 * 39.35 < 32.9 \le 0.75(39.35 + 14.6)}$$
$$\boldsymbol{29.5KN < 32.9KN \le 40.46}\text{KN}$$

Then we have minimum shear reinforcement is provided(Av,min):

$$Smax \le \frac{d}{2} = \frac{313}{2} = 156.5mm\text{…control}$$
$$Smax \le 600mm$$

$$Av, min = \frac{1}{16} * \sqrt{fc} * \frac{bw * s}{fyt}$$
$$Av, min = \frac{1}{16} * \sqrt{24} * \frac{140 * 156.5}{420} = 16\ mm^2$$
$$Av, min = \frac{1}{3} * \frac{bw * s}{fyt}$$

$$Av, min = \frac{1}{3} * \frac{140 * 156.5}{420} = 17.4\ mm^2 \ldots\ldots control$$

Select $\emptyset 8\ (2leg) with\ As = 100.5mm^2 > \ Av, min = 17.4\ mm^2$….ok

$$s = \frac{Av * fy * d}{vs}$$
$$s = \frac{100.5 * 420 * 313}{4.5 * 10^3} = 2936mm$$
$$S > Smax$$
$$2936mm > 141mm$$

Then Select $\emptyset 8\ (2leg)$@100mm(10cm).....ok

4.5 Design of two way solid slab

1. Approximate method:

Approximate value of minimum(h) according to ACI

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

Minimum (h) ≥ (2*6+2*4.9)/180=12.11 cm

Select (h=20 cm) > minimum (12.11cm)

2. accurate method:

interior beams have a rectangular section of 80 width and 40cm depth:

$bw + 2hw = 80 + 2 * 20 = 120cm$

$bw + 8h = 80 + 8 * 20 = 240cm$

$bw + 2hw < \ bw + 8h.........120 < 240$

(20*(80+20*2)*(20+$\frac{20}{2}$)))+80*20*$\frac{20}{2}$)/(

$$\text{yc} = \frac{(20 * (80 + 20 * 2) * (20 + 20/2))) + 80 * 20 * 20/2)}{20 * (80 + 20 * 2) + 80 * 20}$$

=21.78cm

$$\text{Ib} = \frac{(80 + 20 * 2) * (20 - 1.78)^3}{3} + \frac{80 * (21.78)^3}{3} = 517451.7cm^4$$

The moment of inertia for the internal solid slab:

-short direction $Lright$=4.9m …… short direction $Lleft$=4.79m

$$\text{Is1} = \frac{h^3*(\frac{1}{2}\text{Lleft}+\frac{1}{2}\text{Lleft}+\text{bw})}{12} = \frac{20^3*(\frac{1}{2}490+\frac{1}{2}479+80)}{12} = 376333.3cm^4$$

-short direction $Lright$=2.65m …… short direction $Lleft$=4.90m

$$\text{Is2} = \frac{h^3*(\frac{1}{2}\text{Lleft}+\frac{1}{2}\text{Lleft}+\text{bw})}{12} = \frac{20^3*(\frac{1}{2}4.90+\frac{1}{2}265+80)}{12} = 305000cm^4$$

-long direction $Lright$=1.80m …… short direction $Lleft$=6.00m

$$Is3 = \frac{h^3 * (\frac{1}{2}\text{Lleft}+\frac{1}{2}\text{Lleft}+\text{bw})}{12} = \frac{20^3 * (\frac{1}{2}180+\frac{1}{2}600+80)}{12} = 313333.34 cm^4$$

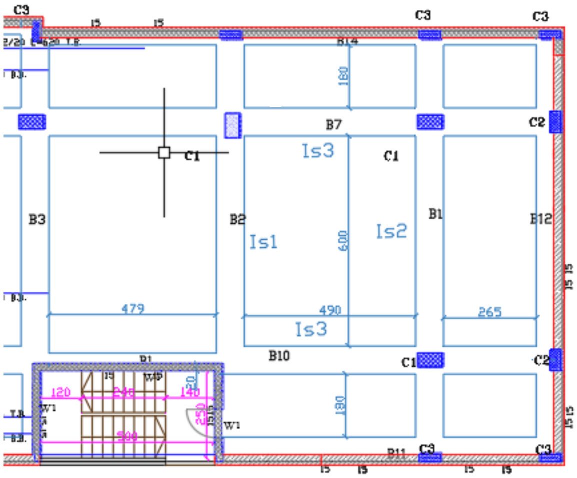

Figure 4- 1 Two way solid case

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

α1=517451.7/376333.3=1.374

α2=517451.7/305000=1.696

α3=517451.7/313333.34 =1.651

α4=1.651

αfm=(1.374+1.696+1.651+1.651)/4

= 1.593 <2.0 the minimum slab thickness will be :

$$h = \frac{\ln(0.8+\frac{fy}{1400})}{36+5\beta(\alpha fm-0.2)} \text{.....................} \ \beta=7.03/6.84=1.03$$

$$h = \frac{6000(0.8+\frac{420}{1400})}{36+5*1.22(1.593-0.2)}$$

=148.32 mm>125mm…..ok

First trial thickness h=200mm >148.32 mm_ok

Take slab thickness h_{slab} 200mm .

Load calculation:

Table 6:Calculation of the total dead load for solid slab two way

Material	Quality Density $KN/\ m^3$	$W = \gamma \cdot V$ KN
Tiles	23	$23\times\ 0.03\ = 0.69$
mortar	22	$22 \times 0.02\ = 0.44$
Sand	17	$17\times\ 0.07 = 1.19$
Reinforced Concrete solid slab	25	$25\times\ 0.2 = 5$
Plaster	22	$22 \times 0.02 = 0.44$
Partitions 2 $KN/\ m^2$		2
Total Dead Load, KN		9.76

DL=9.76 $KN/m2$
$W_D = 1.2 \cdot *9.76 = 11.7\ KN/m^2$

Live Load of slab:
$LL = 5\ KN/m^2$
$w_L = 1.6 \cdot 5 = 8\ KN/m^2$

$w = 11.7 + 8 = 19.7\ KN/m^2$

Moments calculations:

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

La/Lb = 4.9/6=0.816

Case 2

C_a&C_b from tables by interpolation

$C_{a,dl}=0.02536$ $C_{b,dl}=0.01132$

$C_{a,ll}=0.03972$ $C_{b,ll}=0.01764$

$C_{a,neg}=0.0634$ $C_{b,neg}=0.02828$

Design of bending moment:

Ma,pos ,DL= $C_{a,dl}$ * DL* La^2

Ma,pos ,DL=0.02536 *11.7*4.9^2

= 7.12KN.m/m

Mb,pos ,DL=0.01132*11.7*6^2

= 4.767 KN.m/m

Ma,pos ,LL= $C_{a,\text{ll}}$ * LL* La^2

Ma,pos,LL=0.03972*8*4.9^2

=7.63 KN.m/m

Mb,pos,LL=0.01764*8*6^2

=5.08 KN.m/m

Ma,pos =7.12+7.63=14.75 KN.m/m

Mb,pos =4.767+5.08=9.847 KN.m/m

Ma.neg= $C_{a,\text{neg}}$ * wu* La^2

Ma.neg=0.0634*19.7*4.9^2=30 KN.m/m

Mb.neg=0.02828*19.7*6^2=20.05KN.m/m

Design of positive moments in short direction :-

Ma,pos=14.75 KN.m/m

Assume bar diameter Ø14 for main reinforcement

d=200-20-8-14/2=165mm

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{14.75 * 10^6}{0.9 * 1000 * 165^2} = 0.602\ Mpa.$$

$$\text{m} = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.602}{420}}\right) = 0.001455$$

$As = \rho bd = \mathbf{0}.001455 * 1000 * 165 = 240.1mm^2$

$A_{smin} = 0.0018bh = 0.0018 * 1000 * 200 = 360\ mm^2$

$\boldsymbol{A_{s,min}} = 360mm^2$>$\boldsymbol{A_{s,req}} = 240.1\ mm^2$

Take $\boldsymbol{A_{smin}} = 360\ mm^2$

Use Ø12 , $n = \frac{\boldsymbol{A_{s,req}}}{\boldsymbol{A_{s,\emptyset 12}}}$ = $\frac{360}{113.1}$ = 3.18=4bar

take Ø12,@25cm

Step (s) is the smallest of:

1. $3h = 3 * 200 = 750\ mm$
2. $450\ mm\ \ -Control$

$S = 250mm < Smax = 450\ mm \dots\dots\dots OK$

Mb,pos =9.847 KN.m/m< Ma,pos ==14.75 KN.m/m

Then will be reinforcement as area of steel minimum like short direction

take Ø12,@25cm

<u>Design of ngative momentsin short direction :-</u>

Ma,neg=30 KN.m/m

Assume bar diameter Ø14 for main reinforcement

d=200-20-8-14/2=165mm

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{30 * 10^6}{0.9 * 1000 * 165^2} = 1.22\ Mpa.$$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.59$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2*20.59*1.22}{420}}\right) = 0.00299$$

$$As = \rho bd = \mathbf{0}.00299 * 1000 * 165 = 493.35mm^2$$

$$A_{smin} = 0.0018bh = 0.0018 * 1000 * 200 = 360\ mm^2$$

$\boldsymbol{A_{s,min}} = 360mm^2 < \boldsymbol{A_{s,req}} = 493.35\ mm^2$

Take $\boldsymbol{A_{sreq}} = 493.35\ mm^2$

Use Ø12 , $n = \frac{A_{s,req}}{A_{s,\emptyset12}} = \frac{493.35}{113.1}$ = 4.36=5bar/m

take Ø12,@20cm

Step (s) is the smallest of:

1. $3h = 3 * 200 = 750\ mm$
2. $450\ mm\ \ - Control$

$S = 200mm < Smax = 450\ mm \ldots\ldots\ldots OK$

Check for strain:

According to AC -318-11 (10.3.5)، ($\varepsilon_s \geq 0.005$)

$a = \frac{A_s f_y}{0.85b\ f_c'}$

$= \frac{493.35*420}{0.85*1000*24} = 10.157mm$

$c = \frac{a}{\mathcal{B}_1}$

$= \frac{9.314}{0.85} = 11.95mm$ * Note $f_c' = 24\ MPa < 28$ Mpa → $\mathcal{B}_1 = 0.85$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right)$$

$= 0.003\left(\frac{165-11.95}{11.95}\right) = 0.0384 > 0.005\ \ldots\ldots\ldots.\ \ \boldsymbol{0k}$

∴ Ø = **0.9**

Design of ngative moments in long direction :-

Mb,neg=20.05KN.m/m

Assume bar diameter Ø14 for main reinforcement

d=200-20-8-14/2=165mm

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{20.05 * 10^6}{0.9 * 1000 * 165^2} = 0.82\ Mpa.$$

$$\text{m} = \frac{f_y}{0.85f_c'} = \frac{420}{0.85*24} = 20.59$$

$$\rho\ = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.82}{420}}\right) = \ 0.00199$$

$$As = \ \rho bd = \mathbf{0}.00199 * 1000 * 165 = 328.35mm^2$$

$$A_{smin} = 0.0018bh = 0.0018 * 1000 * 200 = 360\ mm^2$$

$\boldsymbol{A_{s,min}} = 360mm^2$>$\boldsymbol{A_{s,req}} = \ 328.35\ mm^2$

Take $\boldsymbol{A_{sreq}} = 360\ mm^2$

Use Ø12 , $n = \frac{\boldsymbol{A_{s,req}}}{\boldsymbol{A_{s,\emptyset 12}}}$ = $\frac{360}{113.1}$ = 3.2=4bar/m

take Ø12,@25cm

Step (s) is the smallest of:

1. $3h = 3 * 200 = 750\ mm$
2. $450\ mm\ \ - Control$

$S = 250mm < Smax = 450\ mm \ldots \ldots \ldots OK$

Check for strain:

According to AC -318-11 (10.3.5)، ($\varepsilon_s \geq 0.005$)

$a = \frac{A_s f_y}{0.85 b\, f_c'}$

$= \frac{360*420}{0.85*1000*24} = 7.4mm$

$c = \frac{a}{\mathcal{B}_1}$

$= \frac{7.4}{0.85} = 8.72mm$ * Note $f_c' = 24\ MPa <$ 28 Mpa → $\mathcal{B}_1$ = 0.85

$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right)$

$= 0.003\left(\frac{165-8.72}{8.72}\right) = 0.053 > 0.005\ \ldots\ldots\ldots.\ \boldsymbol{0k}$

∴ Ø = **0.9**

Design of shear:

W_a= 0.694

W_b= 0.306

The total load on the panel being =(4.9*6*19.7=579.18 KN

The load per meter at the face of short beam is (0.3*579.18*1)/(2*4.9)=17.73KN

The load per meter at the face of long beam is (0.7*579.18*1)/(2*6)=33.78 KN

Vc = (√fc' /6 * bw * d)

= (√24/6 * 1000 * 165) 10^{-3}= 134.7 KN

ϕVc = 0.75 * 134.7= 101 KN

Vu max =33.78KN< 0.5ϕVc=50.5

The thickness of the slab is adequate enough

4.6 Design Of Beam10

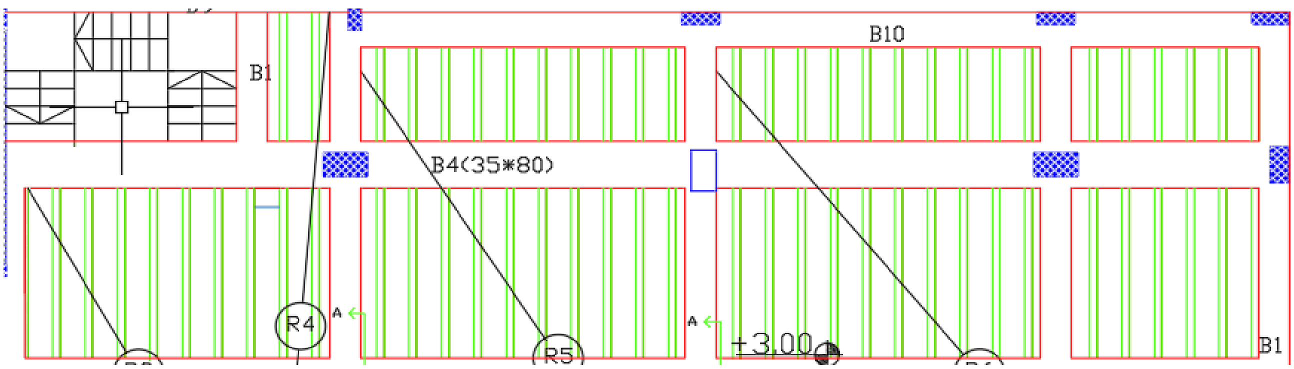

Figure27: Beam4

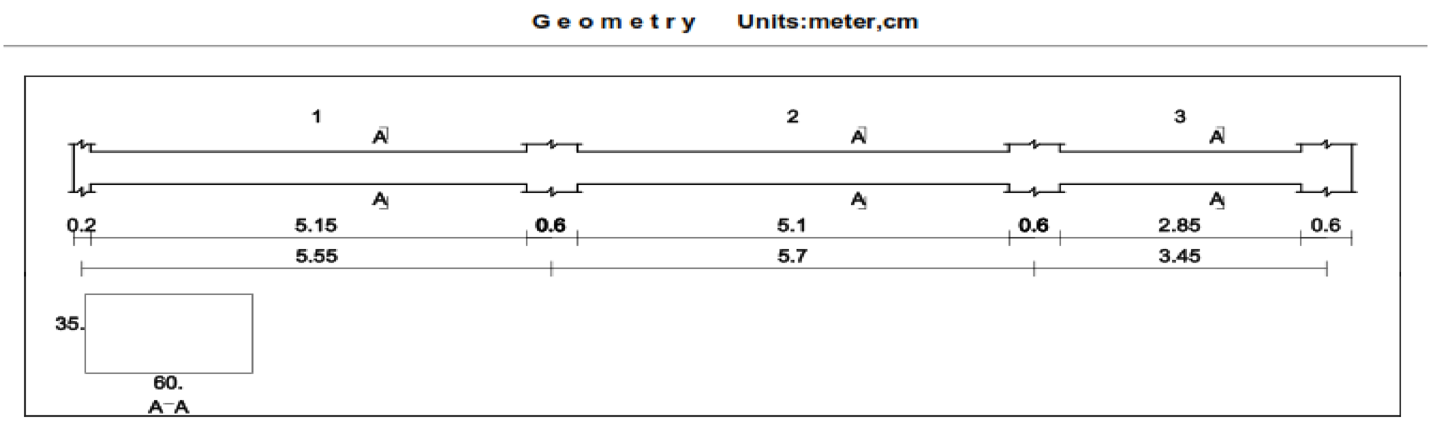

Figure 28-Span diagram for Beam10

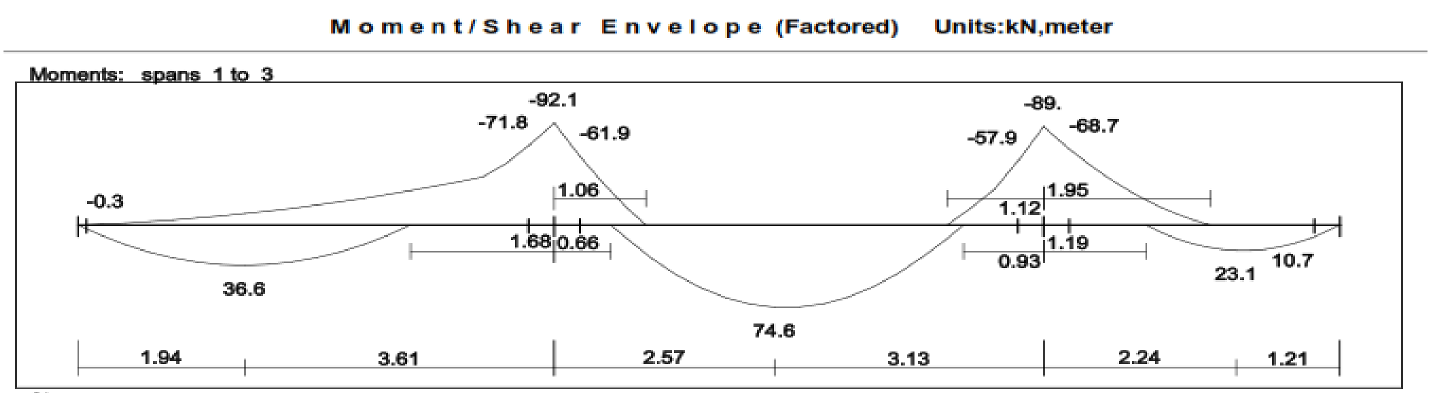

Figure 29:Moment Envelop for Beam10

Figure 4-6Moment Envelop for beam4(KN.m)

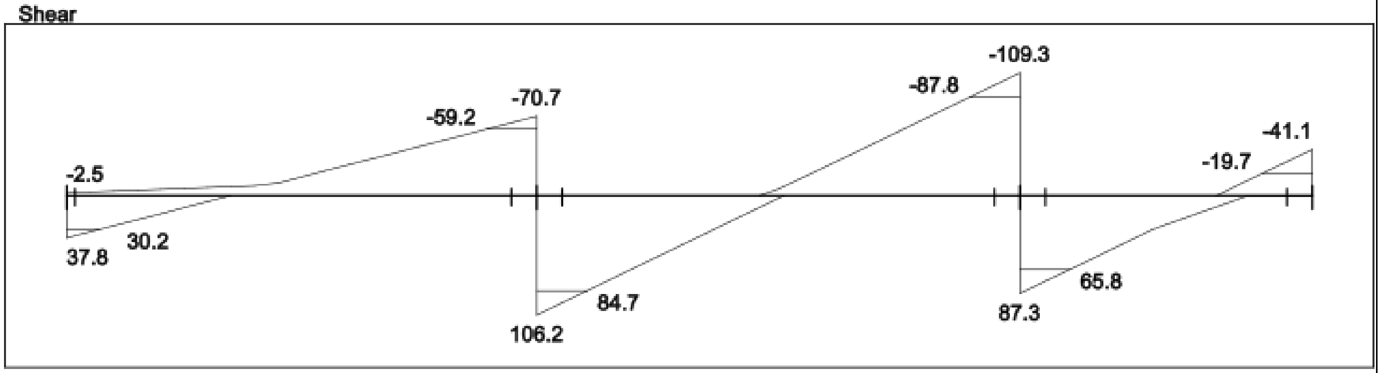

Figure 4-7Shear Envelop for Beam10(KN)

Figure 30:7Shear Envelop for Beam 10

4.6.1 Determination of thickness

According to ACI 9.5.2.1 Table 9.5(a)

H=L/21 = 5.7/21= 27.14 m................ (Both Ends Continous)

Take h = 35cm
B= 60cm

4.6.1 Design beam10for flexure

Assume bar diameter Φ14 for main positive reinforcement, stirrups Φ10

$b = 80cm$ $h = 35cm$
D=350-40-10-(14/2)=302mm

Design for positive $_u = 74.6KN.m$

Check whether the section will be act as single or doubly reinforced section:

$c = \frac{3}{7}d = \frac{3}{7}302 = 129.4mm$

$a = \beta * c = 0.85 * 129.4 = 110mm$

$M_{n,} = 0.85 f_c ab \left(d - \frac{a}{2}\right)$

$= 0.85 * 24 * 110 * 600 * \left(302 - \frac{110}{2}\right) * 10^{-6} = 332.5\ KN.m$

Ø $M_{n,}$ = 0.82* 332.5 = 272.7 KN.m * Note: $\epsilon s = 0.004 \rightarrow$ Ø = 0.82

$$(74.6KN.m) < \emptyset\ M_{n,}(272.7\ \text{KN.m})$$

Design section as singly reinforced concrete section.

$$Rn = \frac{Mu}{\emptyset bd^2} = \frac{74.6 * 10^6}{0.9 * 600 * 302^2} = 1.51MPA$$
$$m = \frac{fy}{0.85fc} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{(2 * Rn * m)}{fy}}) = \frac{1}{20.59}(1 - \sqrt{1 - \frac{(2 * 1.51 * 20.59)}{420}}) = 0.00374$$

As=$\rho * b * d = 0.00374 * 600 * 302 = 677.54mm^2$

Check for As, min:

$A_{s,\ min}$ is the maximum of: -

$$As\ min \geq 0.25 * \frac{\sqrt{fc}}{fy} * bw * d \geq \frac{1.4}{fy} * bw * d$$
$$As\ min \geq 0.25 * \frac{\sqrt{24}}{420} * 600 * 302 = 524mm^2$$
$$As\ min = \tfrac{1.4}{420} * 302 * 600 = 604mm^2 \ldots\ldots\text{..control}$$

$$\text{As}=677.54mm^2 > As\ min = 604mm^2$$

Try bar Ø14 $with\ A_s = 153.9\ mm^2$
$$n = \frac{As}{As\emptyset 14} = \frac{677.54}{153.9} = 4.4 \approx 5bars$$

Then take5 Ø14 $As.provided = 769.5mm^2$

Check of strain :

$$\epsilon s > 0.005 \ldots.tension - control\ section$$
$$a = \frac{As * fy}{0.85 * fc * bw} = \frac{769.5 * 420}{0.85 * 24 * 600} = 26.4mm$$

$c = \frac{a}{\beta} = \frac{26.4}{0.85} = 31.06mm.....*$ $Note\ f' = 24\ MPa < 28\ Mpa \rightarrow B = 0.85$

$$\epsilon s = \frac{0.003*(d-c)}{c} = \frac{0.003*(302-31.06)}{31.06} = 0.026 > 0.005 \ldots ok$$

Check for bar placement :

Sb= $\frac{600-2*40-2*10-5*14}{4}$=107.5mm>25mm……..ok

Then take 5Ø14

Design Of Negative Moment In The Part 1 Second Support Of Beam 10

Design for negative moment $_u = -71.8KN.m$

Ø $M_{n,Max}$ =271.37 KN.m

$(71.8KN.m)$ < Ø $M_{n,}$(272.7KN.m)

Design section as singly reinforced concrete section

$$Rn = \frac{Mu}{\emptyset bd^2} = \frac{71.8*10^6}{0.9*600*302^2} = 1.46MPA$$

$$m = \frac{fy}{0.85fc} = \frac{420}{0.85*24} = 20.59$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{(2*Rn*m)}{fy}}) = \frac{1}{20.59}(1 - \sqrt{1 - \frac{(2*1.46*20.59)}{420}}) = 0.0036$$

As=$\rho*b*d = 0.0036*600*302 = 652.32mm^2$

Check for As, min:

$A_{s,\ min}$ is the maximum of: -

$$As\ min \geq 0.25*\frac{\sqrt{fc}}{fy}*bw*d \geq \frac{1.4}{fy}*bw*d$$

$$As\ min \geq 0.25*\frac{\sqrt{24}}{420}*600*302 = 524mm^2$$

$$As\ min = \frac{1.4}{420}*302*600 = 604mm^2 \ldots\ldots..\text{control}$$

$$As=652.32 > As\ min = 604mm^2$$

Try bar Ø14 $with\ A_s = 153.9\ mm^2$

$$n = \frac{As}{AsØ14} = \frac{652.32}{153.9} = 4.24 \approx 5bars$$

Then take5 Ø14 $As.provided = 769.5mm^2$

Check of strain :

$$\epsilon s > 0.005 \ldots .tension - control\ section$$

$$a = \frac{As * fy}{0.85 * fc * bw} = \frac{769.5 * 420}{0.85 * 24 * 600} = 26.4mm$$

$c = \frac{a}{\beta} = \frac{26.4}{0.85} = 31.06mm \ldots ..*$ $Note\ f' = 24\ MPa < 28\ Mpa \rightarrow B = 0.85$

$$\epsilon s = \frac{0.003 * (d - c)}{c} = \frac{0.003 * (302 - 31.06)}{31.06} = 0.0115 > 0.005 \ldots ok$$

Check for bar placement :

Sb $=\frac{600-2*40-2*10-5*14}{4}$=107.5mm>25mm……..ok

Then take5Ø14

4.6.2 Design beam10 for shear:

Critical section at distance d=302 mm from the face of support.

$V_{u,=}$ 87.8 KN

Vc=$\frac{1}{6} * \sqrt{fc}$*bw*d

$=\frac{1}{6}\sqrt{24} * 600 * 302 * 10^{-3} = 148$ KN

$Ø\ V_c = 0.75 * 148 = 110.9\ KN$

$\frac{1}{2} ØV_c = \frac{1}{2} * 110.9 = 55.5KN$

$\frac{1}{2} Ø\ V_c (55.5KN) < V_{u,max}\ (87.7KN) < Ø\ V_c\ (148KN)$

Minimum shear reinforcement is required ($A_{v,min}$)

$\boldsymbol{S_{max}}$ *is the smallest of*:

1. $\frac{d}{2} = \frac{302}{2} = 151\ mm$………. Control
2. $600mm$

$$A_{v,min} = \frac{1}{16}\sqrt{24}\frac{b_w S}{f_{yt}} \geq \frac{1}{3}\frac{b_w S}{f_{yt}}$$
$$A_{v,min} = \frac{1}{16}\sqrt{24}\frac{b_w S}{f_{yt}} = \frac{1}{16}\sqrt{24}\frac{600 * 151}{420} = 66mm^2$$
$$A_{v,min} = \frac{1}{3}\frac{b_w S}{f_{yt}} = \frac{1}{3}\frac{600 * 151}{420} = 71.9mm^2 \qquad \ldots.Control$$

Select Ø8 (2legs) with $A_v = 2 * \frac{\pi^2 * 8^2}{4} = 100.6\ mm^2 > A_{v,min} = 71.9\ mm^2$

$S = \frac{A_v, f_{yt}\ 3}{bw}$=$\frac{100.6*420*3}{600}$=211.26mm

Select Smax = 150 mm

We need to minimum shear reinforcement is provided

Take 2 leg Ø8 ……….Av = $100.6mm^2 > Av, min = 71.9mm^2$

Take 2 leg Ø8 @150mm

4.7stairs

4.7.1 determination of thickness

$h_{min} = \frac{L}{20} = \frac{520}{20} = 26\ cm$ ---- simply supported

$\theta = tan^{-1}\left(\frac{Rise}{Run}\right) = tan^{-1}\left(\frac{16.6667}{30}\right) = 29.055°$

4.7.2load calculations

4.7.2.1 Flight Dead Load Computation:

Table (7) Calculation of the total dead load for Flight

Material	Quality Density KN/m^3	calculations	$\boldsymbol{KN/m}$
Tiles	27	$0.03 * 27 *(\frac{0.166667+0.35}{0.3})$	**1.395**
Mortar	22	$0.02 * 22 *(\frac{0.166667+0.3}{0.3})$	**0.685**
Stair Steps	25	$(\frac{0.166667*0.3}{2})*\frac{25}{0.3} * 1$	**2.0834**
Reinforced Concrete Solid Slab	25	$(\frac{25*0.26*1}{cos29.055})$	**7.436**
Plaster	22	$(\frac{22*0.03*1}{cos29.055})$	**0.755**
$\boldsymbol{Total\ Dead\ Load\ (KN/m)}$			**12.35**

4.7.2.2 Landing Dead Load Computation:

Table (8) Calculation of the total dead load for Landing

Material	Quality Density KN/m³	Calculations	KN/m
Tiles	22	0.03 * 22 *1	**0.66**
Mortar	22	0.02 * 22 *1	**0.44**
Reinforced Concrete Solid Slab	25	25*0.26	**6.5**
Plaster	22	22*0.03*1	**0.66**
$\boldsymbol{Total\ Dead\ Load\ (KN/m)}$			**8.26**

$LL = 4KN/m^2$

Total Factored Load for flight: $w = 1.2D_L + 1.6L_L$

$1.2 * 12.35 + 1.6 * 4 = 21.22KN/m$

Total Factored Load for Landing=

$1.2 * 8.26 + 1.6 * 4 = 16.312KN/m$

Figure for load on the stairs on the flight

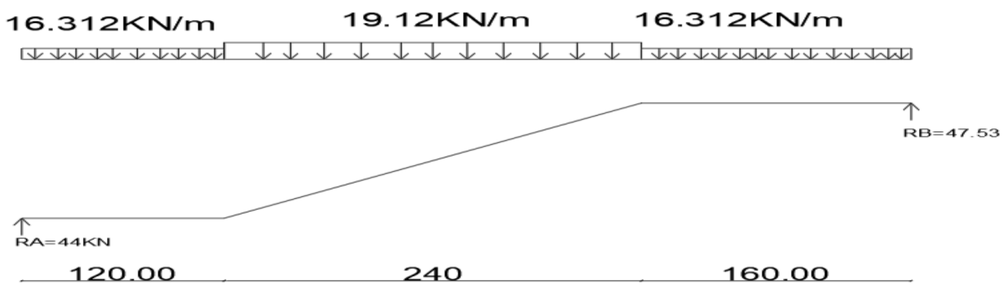

Figure 31(distribution of load on the stairs)

Moment for stairs

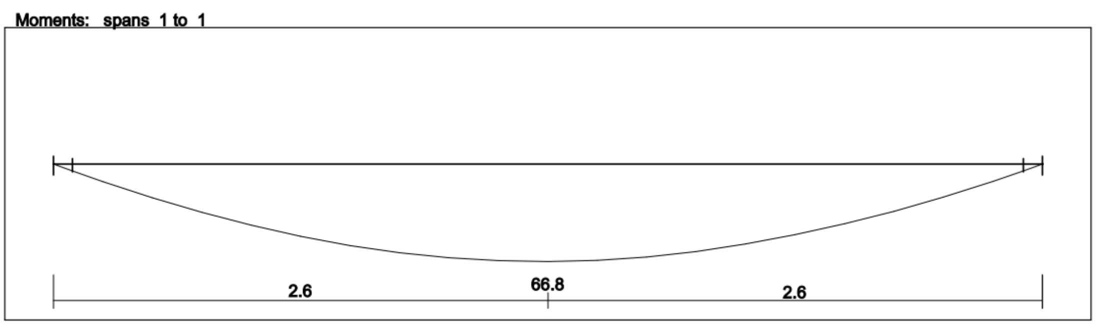

Figure 32(moment of stairs)

4.7.3.1 Shear Design for flight:

Take the maximum shear as the support reaction from beam d $\boldsymbol{V_u = 47.53\ KN}$

Assume bar diameter Ø 14 for main reinforcement.

$d = \boldsymbol{h - 20 - \frac{db}{2}}$

$$= 260 - 20 - \frac{12}{2} = 234\ mm$$

$Vc = \frac{1}{6}\sqrt{fc'}b_w d = \frac{1}{6} * \sqrt{24} * 1000 * 234 * 10^{-3} = 191.06\ KN/1\ m\ strip$.

$\emptyset = 0.75 - for\ shear$

$\emptyset Vc = 0.75 * 191.06 = 143.295 KN\ /\ 1m\ strip$

$$\frac{1}{2}\emptyset Vc = \frac{1}{2} * 143.295 = 71.647 KN\ /\ 1m\ strip$$

$Vu = 47.53\ KN < \frac{1}{2}\ \emptyset\ Vc = 71.647\ KN$

No shear Reinforcement required. The thickness is adequate enough.

4.7.3.2 Bending Moment Design:Design the stairs as solid slab one way

$from\ beam\ d\ Mu = 66.8\ KN.m$

$$Mn = \frac{Mu}{\emptyset} = \frac{66.8}{0.9} = 74.2\ KN.m$$

$$Rn = \frac{M_n}{bd^2} = \frac{74.2 * 10^6}{1000 * 234^2} = 1.355\ Mpa.$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 * m * R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 1.355}{420}}\right) = 3.341 * 10^{-3}$$

$A_s = \rho bd = 3.341 * 10^{-3} * 1000 * 234 = 781.82\ mm^2$

$A_{s,min} = 0.0018bh = 0.0018 * 1000 * 260 = 468\ mm^2$

$A_{s,req} = 781.82\ mm^2 > A_{s,min} = 468\ mm^2$

$A_{s,req} = 781.82\ mm^2 \quad Control$

Use Ø14then

$n = \frac{A_{s,req}}{A_{s\emptyset14}} = \frac{781.82}{153.9}$ = 5.1 = 6 bars

$s = \frac{1}{n} = \frac{1}{5.1} = 19.6c\ m$

Then TakeØ14@18cm with As=923.6mm^2

Step (s) is the smallest of:

1. $3h = 3 * 260 = 780mm$
2. $450mm$
3. $s = 380\left(\frac{280}{fs}\right) - 2.5Cc = 380\left(\frac{280}{\frac{2}{3} * 420}\right) - 2.5 * 20 = 330mm, but$

$$s \le 300\left(\frac{280}{\frac{2}{3} * 420}\right) = 300mm. \quad - \boldsymbol{control}$$

$s = 180mm < Smax = 300mm \quad - OK$

Check for strain:

According to AC -318-11 (10.3.5)، ($\varepsilon_s \geq 0.005$)

$a = \frac{A_s f_y}{0.85 b\, f'_c}$

$= \frac{923.6*420}{0.85*1000*24} = 19mm$

$c = \frac{a}{\mathcal{B}_1}$

$c = \frac{19}{0.85} = 22.37mm$ * Note $f'_c = 24\ MPa < 28$ Mpa $\rightarrow \mathcal{B}_1 = 0.85$

$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right)$

$= 0.003\left(\frac{234-22.37}{22.34}\right) = 0.0284 > 0.005\ \dots\dots\dots.\ \boldsymbol{0k}$

$\therefore \emptyset = \mathbf{0.9}$

4.7.5 Landing design:

I will reinforcement of landing on the temperature and shrinkage because the landing based on the flight as described previously

Take $\emptyset 12\ @\ 250mm$

4.7.3.3 Temperature and shrinkage reinforcement:

$\boldsymbol{As, Temperature\ \&\ shrinkage = 0.0018bh} = 0.0018*1000*260 = 468\ mm^2$

Use Ø12 then, $n = \frac{\boldsymbol{A_{s,req}}}{\boldsymbol{A_{s\emptyset 12}}} = \frac{468}{113.1} = 4.13$

$s = \frac{1}{n} = \frac{1}{4.13} = 0.2416\ m$

Take 5Ø12 with $As = 565.49mm^2 /\ m\ strip, or\ \emptyset 12\ @\ 250mm$

Step (s for $temperature\ \&\ shrinkage$) is the smallest of:

1. $5h = 5*260 = 1300mm$
2. $450mm\ - contorl$

$s = 250\ mm\ < Smax = 450\ mm\ \ - OK$

4.8 Isolated Footing Design (F1):

4.8.1 Determination of footing dimensions:

The following parameters are used in design:

$\gamma_{\ concrete} = 25\ KN/m^3$

$D_L = 2301\ KN$

$L_L = 802\ KN$

$\gamma_{\ soil} = 18\ KN/m^3$

$\sigma_{allow} = 400\ KN/m^2$

$column\ dimention = 40 * 70$

4.8.2Design of footing Area:

$Allowable\ soil\ pressure = 400KN/m^2$

Calculating the weight of footing and soil loads:

$assume\ h = 65cm$

$W_{footing} = 0.65 * 25 = 16.25KN/m^2$

$W_{soil} = 0.65 * 18 = 11.7KN/m^2$

$q_{a,net} = \sigma_{allow} - W_{footing} - W_{soil}$

$= 400 - 16.25 - 11.7 = 372.05\ KN/m^2$

$$Area(A) = \frac{Total\ Weight(P_n)}{Soil\ pressure(q_{a,net})}$$

$$= \frac{2301 + 802}{372.05} = 8.34m^2$$

Assume square footing:

$A = L^2$

$L = \sqrt{A} = \sqrt{8.34} = 2.888m$

$Take\ L = 3m$

$A = L^2$

$= 3 * 3 = 9m^2 \ldots \ldots OK$

$P_n = 2301 + 802 = 2833KN$

$q_u = \frac{P_n}{A}$

$= \frac{2833}{3 * 3} = 314.77\ KN/m^2$

4.8.3Design of footing:

4.8.3.1 Design for One way shear strength:

Assume $h = 65cm$ and $\emptyset 14$ for main reinforcement &7.5 cm cover.

$d = 650 - 75 - 7 = 568mm$

V_u At distance d from the face of support:

$V_u = q_u\ b\left(\frac{l}{2} - \frac{a}{2} - d\right)$

$= 314.77 * 3\left(\frac{3}{2} - \frac{0.4}{2} - 0.568\right) = 691.2\ KN$

$Vc = \frac{1}{6}\sqrt{fc'}\,b_w\,d$

$= \frac{1}{6} * \sqrt{24} * 3000 * 568 * 10^{-3} = 1374.2KN$

$\emptyset Vc = 0.75 * 1374.2 = 1030.6KN$

$\emptyset Vc = 1030.6KN > V_u = 691.2KN$

- **Safe**

4.8.3.2 Design for Two way shear strength:

$$let\ V_u = \emptyset Vc, \emptyset = 0.75$$

$$V_u = 314.77\big(3 * 3 - (0.4 + 0.568) * (0.7 + 0.568)\big) = 2446.6 \text{ KN}$$

$$\beta = \frac{600}{300} = 2, \qquad \text{b}_\circ = 2(0.7 + 0.568) + 2(0.4 + 0.568) = 4.47\ m$$

$$\alpha_s = 40\ For\ interior\ column$$

$$Vc = \frac{1}{6}\left(1 + \frac{2}{\beta}\right)\sqrt{fc'}\text{b}_\circ d \qquad where \quad \frac{1}{6}\left(1 + \frac{2}{1.75}\right) = 0.357$$

$$Vc = \frac{1}{12}\left(\frac{\alpha_s d}{\text{b}_\circ} + 2\right)\sqrt{fc'}\text{b}_\circ d \qquad where \quad \frac{1}{12}\left(\frac{40 * 0.568}{4.47} + 2\right) = 0.59$$

$$Vc = \frac{1}{3}\sqrt{fc'}\text{b}_\circ d \qquad where \quad \frac{1}{3} = 0.333 - Control$$

$$Take\ Vc = 0.333\sqrt{fc'}\text{b}_\circ d = 0.333\sqrt{24} * 4470 * 568 * 10^{-3} = 4142\ KN$$

$$\emptyset Vc = 0.75 * 4072.61 = 3106.46\ KN < Vu = 2446.6 \text{ KN} \qquad - \quad OK$$

4.8.3.3 Design of flexure in short direction:

Take steel bars of Ø 16

$$d = 650 - 75 - \frac{16}{2} = 567\ mm$$

$$Mu = 314.77 * 3 * 1.3 * \frac{1.3}{2} = 797.94\ KN.m$$

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{797.94 * 10^6}{0.9 * 3000 * 567^2} = 0.919\ Mpa.$$

$$\text{m} = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.919}{420}}\right) = 0.00224$$

$$As = \rho b d = \mathbf{0}.00224 * 3000 * 567 = 3809.8 mm^2$$

$A_{smin} = 0.0018bh = 0.0018 * 3000 * 650 = 3510\,mm^2$

$\boldsymbol{A_{s,min}} = 3510mm^2 < \boldsymbol{A_{s,req}} = 3809.8\,mm^2$

Take $\boldsymbol{A_{s,req}} = 3809.8\,mm^2$

Use Ø16 , $n = \frac{\boldsymbol{A_{s,req}}}{\boldsymbol{A_{s,\emptyset 16}}} = \frac{3809.8}{201.1} = 18.94$

Take 19Ø16 $with\ \boldsymbol{A_{prov}} = 3820.9mm^2 > \boldsymbol{A_{s,req}} = 3809.8mm^2$

$$\boldsymbol{S} = \frac{3000 - 75 * 2 - 20 * 16}{19} = 141.44mm$$

take Ø16,@14cm

Step (s) is the smallest of:

1. $3h = 3 * 650 = 1950\ mm$
2. $450\ mm\ \ -Control$

$S = 141.44mm < Smax = 450\ mm \ldots\ldots\ldots OK$

Check for strain:

According to AC -318-11 (10.3.5)، ($\varepsilon_s \geq 0.005$)

$a = \frac{A_s f_y}{0.85 b\, f_c'}$

$= \frac{3820.9 * 420}{0.85 * 3000 * 24} = 26.22mm$

$c = \frac{a}{\mathcal{B}_1}$

$= \frac{26.22}{0.85} = 30.84mm$ * Note $f_c' = 24\ MPa < 28$ Mpa → $\mathcal{B}_1 = 0.85$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right)$$

$$= 0.003\left(\frac{567 - 30.84}{30.84}\right) = 0.0521 > 0.005\ \ldots\ldots\ldots.\ \ \boldsymbol{0k}$$

∴ Ø = **0.9**

4.8.3.4 Design of flexure in long direction:

Take steel bars of Ø 12

$$d = 650 - 75 - 16 - \frac{16}{2} = 551\ mm$$

$$Mu = 314.77 * 3 * 1.15 * \frac{1.15}{2} = 624.4\ KN.m$$

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{624.4 * 10^6}{0.9 * 3000 * 551^2} = 0.762\ Mpa.$$

$$\mathrm{m} = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.761}{420}}\right) = 1.8488 * 10^{-3}$$

$$As = \rho bd = 1.8488 * 10^{-3} * 3000 * 551 = 3056.06\ mm^2$$

$$A_{smin} = 0.0018bh = 0.0018 * 3000 * 650 = 3510\ mm^2$$

$$\boldsymbol{A_{s,min}} = 3510\ mm^2 > \boldsymbol{A_{s,req}} = 3056.06\ mm^2$$

Take $\boldsymbol{A_{s,min}} = 3510\ mm^2$

Use Ø16, $n = \frac{\boldsymbol{A_{s,req}}}{\boldsymbol{A_{s,\emptyset 16}}} = \frac{3510}{201.1} = 17.45$

Take $18\emptyset 16 with\ \boldsymbol{A_{prov}} = 3619.8\ mm^2 > \boldsymbol{A_{s,min}} = 3510\ mm^2$

$$\boldsymbol{S} = \frac{3000 - 75 * 2 - 18 * 16}{17} = 150.7\ mm$$

take Ø16,@15cm

Step (s) is the smallest of:

1. $3h = 3 * 500 = 1500\ mm$
2. $450\ mm\ - Control$

$S = 1150.7 mm < Smax = 450\ mm \ldots\ldots\ldots OK$

Check for strain:

According to AC -318-11 (10.3.5)، ($\varepsilon_s \geq 0.005$)

$a = \frac{A_s f_y}{0.85 b\, f_c'}$

$= \frac{3510*420}{0.85*3000*24} = 24.08mm$

$c = \frac{a}{\mathcal{B}_1}$

$= \frac{24.08}{0.85} = 28.34mm$ * Note $f_c' = 24\ MPa < 28$ Mpa $\rightarrow \mathcal{B}_1 = 0.85$

$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right)$

$= 0.003\left(\frac{551-28.34}{28.34}\right) = 0.0553 > 0.005$ ……… . $\boldsymbol{0k}$

$\therefore \emptyset = \mathbf{0.9}$

4.8.4Design of Dowels:

Load transfer in footing:

$\emptyset P_{n,b} = \emptyset(0.85 f_c'\ A_1 \sqrt{\frac{A_2}{A_1}}$

$A_1 = 0.7 * 0.4 = 0.28m^2$

$A_2 = 3 * 3 = 9m^2$

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{9}{0.28}} = 5.566 > 2$ ……………. $\sqrt{\frac{A_2}{A_1}} = 2$

$\emptyset P_{n,b} = 0.65(0.85 * 24 * 700 * 400 * 10^{-3} * 2) = 7425.6KN > P_u = 3103KN$

No need for dowels

Load transfer in column:

$\emptyset P_{n,b} = 0.65(0.85 * 24 * 700 * 400 * 10^{-3}) = 3712.8KN > P_u = 3103KN$

No need for dowels

$A_{s,min} = 0.005 * A_c = 0.005 * 400 * 700 = 1400mm^2$

Take $10\emptyset 14$ $with\ \boldsymbol{A_{prov}} = 1539\ mm^2 > \boldsymbol{A_{s,min}} = 1400\ mm^2$

4.8.5 Development Length:

4.8.5.1 Tension development length:

$$Ld_{t,req} = \frac{9}{10} * \frac{f_y}{\Upsilon\sqrt{f_c'}} * \frac{\Psi_e\Psi_s\Psi_t}{\frac{k_{tr} + c_b}{d_b}} * d_b > 300mm$$

$$= \frac{9}{10} * \frac{420}{1\sqrt{24}} * \frac{1 * 1 * 0.8}{\frac{0 + 56}{12}} * 14 = 15.435cm = 154.35\ mm < 300mm$$

$Take\ Ld_{t,req} = 30cm$

$$Ld_{t,available} = \frac{3000 - 400}{2} - 75 = 1225mm$$

$Ld_{t,available}(1225mm) > Ld_{t,req}(300\ mm)$

4.8.5.2 Compression development length:

$$Ld_{c,req} = \frac{0.24 * f_y}{\sqrt{f_c'}} * d_b > 0.0043 * f_y * d_b > 200\ mm$$

$$\frac{0.24 * 420}{\sqrt{24}} * 14 = 288.06mm > 0.0043 * 420 * 14\ (252.84mm) > 200\ mm \ldots\ldots\ldots OK$$

$Ld_{c,available} = 650 - 75 - 16 - 16 - 14 = 529mm > Ld_{c,req}(288.06246.91mm)$

4.8.5.3 Lab splice of dowels in column:

$L_{sc} = 0.071 * f_y * d_b$

$= 0.071 * 420 * 14 = 417.48mm > 300mm$

$L_{sc} = 450mm$

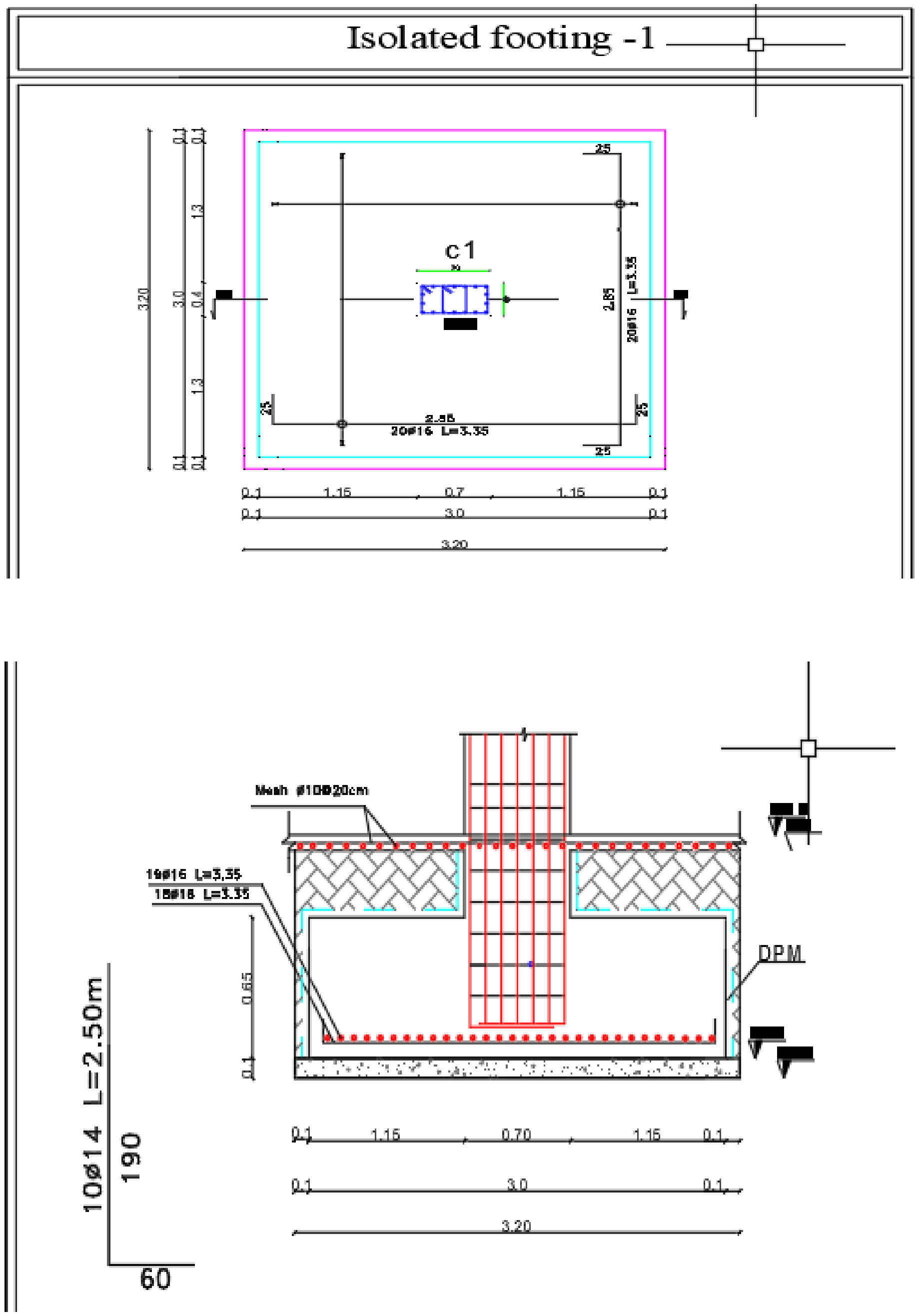

Figure 33(section on the isolated footing)

4.9 Shear Wall (S.W8) design:

From ETSAPS:

Pu=5500 KN Mu=4824 KN.m Vu=950 KN

Materials and sections:

- $f_c' = 24MPa$
- $f_y = 420MPa$
- $h = 20cm$
- $shear\ wall\ width\ (L_w = 5.40m)$
- $shear\ wall\ height\ (h_w = 3.00m)$

4.9.1 Horizontal reinforcement Design:

$\boldsymbol{\varepsilon f_x = V_u = 950\ KN}$

The critical section is the smallest of:

1. $\frac{h_w}{2} = \frac{29}{2} = 14.5m$
2. $\frac{L_w}{2} = \frac{5.40}{2} = 2.70m \ldots\ldots\ldots\ldots\ldots\ldots Control$
3. $story\ height = 3.0m$

$d = 0.8 * L_w = 0.8 * 5.4 = 4.32m$

$$\emptyset V_{n,max} = \emptyset * \frac{5}{6} * \sqrt{fc'} * h * d$$

$$= 0.75 * \frac{5}{6} * \sqrt{24} * 200 * 4320 * 10^{-3} = 2645.45\ KN$$

$\emptyset V_{n,max}\ (2645.45KN) > V_u\ (950\ KN) \ldots\ldots OK$

- Thickness is enough.

v_c is the smallest of:

1. $V_c = \frac{1}{6}\sqrt{fc'}hd$

$$V_c = \frac{\sqrt{24}}{6} * 200 * 4320 * 10^{-3} = 705.45\ KN \ldots\ldots\ldots control$$

2. $V_c = 0.27\sqrt{fc'} * h * d + \frac{Nu*d}{4Lw}$

$$= 0.27 * \sqrt{24} * 200 * 4320 * 10^{-3} + \frac{4000 * 4320}{4 * 5400} = 1942.8KN$$

3. $V_c = \left\{0.05\sqrt{fc'} + \frac{l_w\left(0.1\sqrt{fc'}+0.2\,\frac{Nu}{l_{wh}}\right)}{\frac{Mu}{Vu} - \frac{l_w}{2}}\right\} hd$

(Mu=4824-2.7*950 =2259 KN.m)

$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{2259}{950} - \frac{5.4}{2} = -0.322 < 0$ this equation is not avilable

Then

$V_c = 705.45KN$

$\emptyset V_c = 0.75 * 705.45 = 529.1KN$

$\frac{1}{2}\emptyset V_c = 264.54KN$

$$\boldsymbol{V_u = 950KN} > \frac{1}{2}\emptyset V_c = 264.54KN$$

shear reinforcement must be provide (needs reinforcement)

Ø* Vc + Ø*Vs = Vu

Vs = $\frac{vu}{\emptyset}$-vc =$\frac{950}{0.75}$- 705.45= 561.2 KN

$$\frac{A_{vh}}{s} = \frac{vs}{fyd} = \frac{561.2}{420 * 4320} = 0.00039\ m^2\ /m$$

$$\rho t = \frac{A_{vh}}{hs} = \frac{0.00039}{0.2} = 0.00195 < 0.0025$$

Take $\rho t = 0.0025$

$A_{vh}\ min = 0.0025 * s * h$

$\frac{A_{vh}\ min}{s} = 0.0025 * h$

$\frac{A_{vh}\ min}{s} = 0.0025 * 200$

Try 2 Ø10 with $A_{vh}\ min = 157.1mm^2$

$$\frac{157.1}{s} = 0.0025 * 200$$

$s = 314.2mm$

S_{max} is the smallest of:

1. $\frac{l_w}{5} = \frac{5400}{5} = 1080mm$
2. $450\ mm$
3. $3 * h = 3 * 200 = 600\ mm$

$Take\ s = 250\ mm < Smax = 450\ mm$

Select Ø10/25 cm

4.9.2 Vertical reinforcement Design:

ρl= 0.0025 + 0.5 (2.5 – hw/Lw) *(ρt) -0.0025)

=(0.0025 +0.5 (2.5 – 5.3) * (0.0025 – 0.0025) = 0.0025+0=0.0025

Select ρl =0.0025

Select Ø10/25 cm

4.9.3 Bending Moment Design(uniformly distributed flexural reinforcement):

$Mu = 4824\ KN.m$

Try Ø12/15cm

$$Ast = \frac{5400}{150} * 2 * 113.1 = 8143.2mm^2$$

$$w = \frac{Ast*fy}{lw*h*fc} = \frac{8143.2*420}{5400*200*24} = 0.13$$

$$\alpha = \frac{pu}{lw * h * fc} = \frac{5500}{5400 * 200 * 24} = 0.21$$

$$\frac{c}{lw} = \frac{(w + \alpha)}{(2w + 0.85\beta)} = \frac{(0.13 + 0.21)}{(2 * 0.13 + 0.85 * 0.85)} = 0.346$$

$$\emptyset Mn = \emptyset(0.5\, Ast * fy * lw * (1 + \frac{pu}{Ast * fy})(1 - \frac{c}{lw})$$

$$\emptyset Mn = 0.9(0.5 * 8143.2 * 420 * 5400 * \left(1 + \frac{5500}{8143.2 * 420}\right)(1 - 0.346) * 10^{-6}$$

$= 5444.1 KN.M > Mu$ =4824 KN.m ok (use $\emptyset 12@150mm$

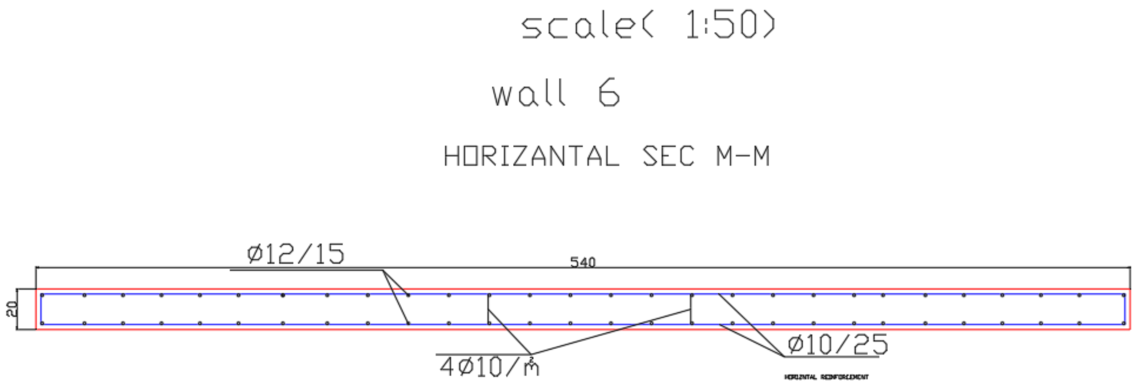

Figure 34 Detailing of shear wall

4.10 Basement Wall design:

Materials:

- $f_c' = 24MPa$
- $f_y = 240MPa$
- $h_{wall} = 25cm$
- $\emptyset = 30$
- $\gamma = 18KN/m^2$

4.10.1 Loads on basement wall:

Self-weight of earth (DL):

$k_0 = 1 - \sin 30 = 0.5$

$q_1 = \gamma * h * k_0$

$q_1 = 18 * 5.50 * 0.5 = 49.5\ KN/m^2$

Load from live load ($LL = 5\ KN/m^2$):

$q_2 = LL * C_0 = 5 * 0.5 = 2.5\ KN/m^2$

From Beam D

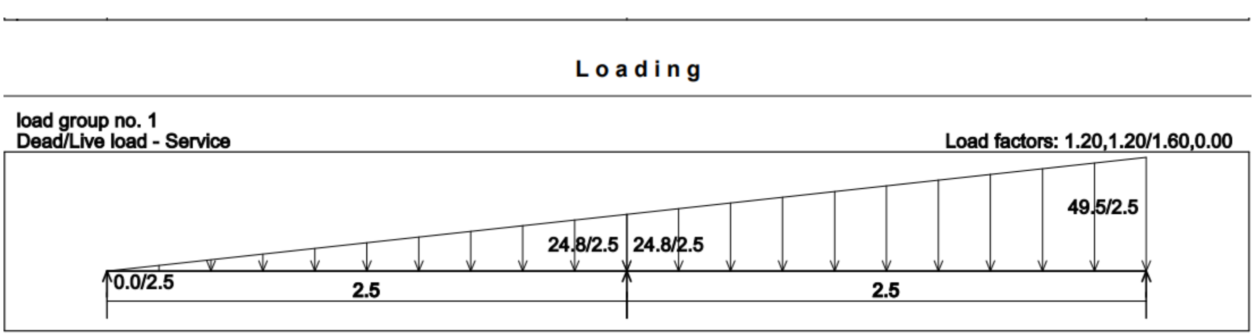

Figure 35(loading on basement wall)

4.10.2 Shear design:

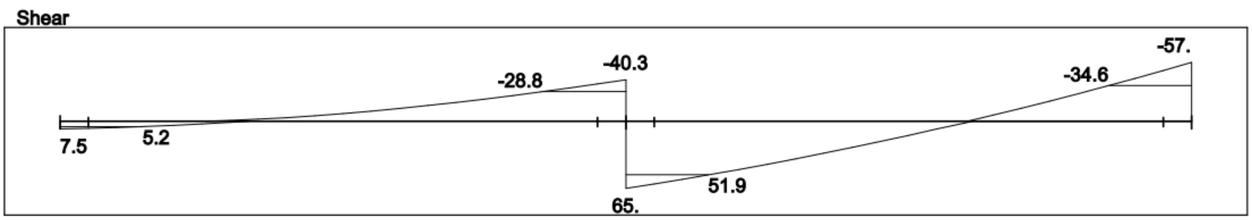

Figure 36(shear of basement)

From Beam D $V_u = 51.9KN$

Assume use bar $\emptyset 12$

$$d = 250 - 20 - \frac{12}{2} = 224\ mm$$

$$\emptyset\ Vc = 0.75\frac{\sqrt{fc'}}{6}bw * d = 0.75 * \frac{\sqrt{24}}{6} * 1000 * 224 = 137.17\ KN$$

$$\frac{1}{2}\emptyset\ Vc = 68.58\ KN$$

$$V_u = 51.9\ KN < \frac{1}{2}\emptyset\ Vc = 68.58\ KN$$

❖ Thickness is adequate enough.

4.10.3 Design of the Vertical reinforcement in tension side:

Moments of wall by beam d

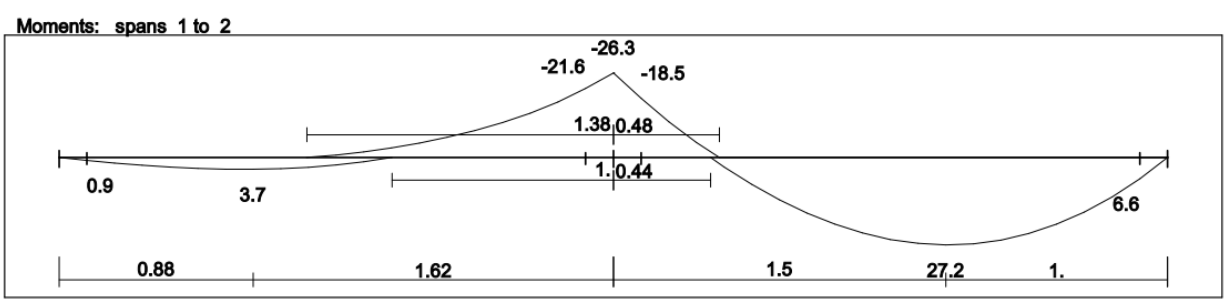

figure37(moment of basement wall)

4.10.3.1 Design of the Vertical reinforcement in positive tension side(span 1,internal side):

From Beam D $M_{u\,Max} = +27.2\ KN.M$

$$d = 250 - 20 - \frac{12}{2} = 224\ mm$$

$$Rn = \frac{M_n}{\emptyset bd^2}$$

$$= \frac{27.2 * 10^6}{0.9 * 1000 * 224^2} = 0.602 Mpa.$$

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

$$= \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right)$$

$$= \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.602}{420}}\right) = 0.001455$$

$$A_s = \rho bd$$

$$= 0.001455 * 1000 * 224 = 326 mm^2/m$$

$$A_{s,min} = 0.0018bh$$

$$= 0.0018 * 1000 * 250 = 450\ mm^2/m$$

$$A_{s,min} = 450 mm^2/m > A_{s,req} = 326\ mm^2/m$$

$$\#bars = \frac{450}{113.1} = 3.97 = 4 bars/1m$$

$\mathrm{Smax} = 3h = 3 * 250 = 750$

$\mathrm{Smax} = 450$...............*control*

Selected s=250mm $< 450mm$

Select $\emptyset 12$ */ 250 cm, with As provided =* $452.4\ mm^2/m$ *>* $A_{s,min} = 450\ mm^2/m$

4.10.3.2 Design of the Vertical reinforcement in negative tension side(external side ,middle side)

From Beam D $M_{u\,Max} = +21.6\ KN.m.$

$$d = 250 - 75 - \frac{12}{2} = 169\ mm$$

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

$$= \frac{21.6 * 10^6}{0.9 * 1000 * 169^2} = 0.84 Mpa.$$

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

$$= \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right)$$

$$= \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 20.59 * 0.84}{420}}\right) = 0.00204$$

$$A_s = \rho b d$$

$$= 0.00204 * 1000 * 169 = 345.3 mm^2/m$$

$$A_{s,min} = 0.0018 bh$$

$$= 0.0018 * 1000 * 250 = 450\ mm^2/m$$

$$A_{s,min} = 450 mm^2/m > A_{s,req} = 345.3\ mm^2/m$$

$$\#bars = \frac{450}{113.1} = 3..97 = 4 bars$$

$\text{Smax} = 3h = 3 * 250 = 750$

$\text{Smax} = 450$*..............control*

Select $\emptyset 12$ */ 25 cm, with As provided =* $452.4\ mm^2/m$ *>* $A_{s,min} = 450\ mm^2/m$

4.10.3.3Design of the vertical reinforcement in tension side(span 2, internal side) :

$A_{s,min} = 0.0018bh$

$= 0.0018 * 1000 * 250 = 450\ mm^2/m$

$A_{s,min} = 450mm^2/m > A_{s,req} = 345.3\ mm^2/m$

$\#bars = \frac{450}{113.1} = 3..97 = 4bars$

$\text{Smax} = 3h = 3 * 250 = 750$

$\text{Smax} = 450$..............*control*

Select $\emptyset 12$ */ 25 cm, with As provided =* $452.4\ mm^2/m$ *>* $A_{s,min} = 450\ mm^2/m$

4.10.3.4 Design of the vertical reinforcement in compression side :

$As, min = 0.0012bh$................ACI(14.3.2)
$= 0.0012 * 1000 * 250 = 300\ mm^2/m$................for internal and external side
As=$\frac{3000}{2}$=150 mm^2/m ....for one side

$\text{Smax} = 3h = 3 * 250 = 750mm$

$\text{Smax} = 450mm$..............*control*

Select $\emptyset 10$ */ 30 cm, with As provided = 253.6* mm^2/m *>* $A_{s,min} = 150\ mm^2/m$

4.10.4 Design of the horizontal reinforcement :

For Two layer:

$As, min = 0.002bh$................ACI(14.3.3)
$= 0.002 * 1000 * 250 = 500\ mm^2/m$................for internal and external side

As=$\frac{500}{2}$=250 mm^2/m ....for one side
Select $\emptyset 10$ */ 25 cm, with As provided = 314.2* mm^2/m *>* $A_{s,min} = 250\ mm^2/m$

4.11 Design of long column (C2) Group B:

$Concret\ B300$ $\quad f_c' = 24MPa$ $\quad f_y = 420MPa$

4.11.1 Load Calculation:

Service Load:

$Dead\ Load\ =\ 1183.13KN$

$Live\ Load\ =\ 288.21\ KN$

$\mathrm{P_u} = 1.2 * \mathrm{DL} + 1.6 * \mathrm{LL}$

$= 1.2 * 1183.13 + 1.6 * 288.21 = 1881KN$

$P_n = \frac{P_u}{\emptyset} = \frac{1881}{0.65} = 2893.8KN$ * Note for Tide Column → ∅= 0.65

4.11.2 Dimension of column:

$Assume\ \rho_g = 0.01 \quad \rightarrow \quad Ast = 0.01A_g$

$\emptyset Pn = \emptyset 0.8 * \{0.85 * f_c'(\mathrm{A}_g - \mathrm{A}_{st}) + \mathrm{A}_{st} * f_y\}$

$1881 * 10^3 = 0.65 * 0.8 * \{0.85 * 24(Ag - 0.01A_g) + 0.01A_g * 420\}$
$\rightarrow \quad A_g = 148274.622\ mm^2$

$A_g = b * \mathrm{h}$

$148274.622 = b * 350$

$b = 423.64\ mm$

Assume rectangular column.

Select b=600mm

$A_g = b * h = 600 * 300 = 180\ 000mm^2(0.18m^2) > A_{s,req} = 0.077m^2$

4.11.3 Check Slenderness Effect:

$\frac{Klu}{r} \leq 34 - 12\left(\frac{M1}{M2}\right) \leq 40 \rightarrow ACI - 10.12.2$

Where:

L_u: Actual unbraced length.

K: effective length factor k. According to ACI 318-2002 (10.10.6.3) the effective length factor k, shall be permitted to taken as 1.

R: radius of gyration = 0.3 h = $\sqrt{\frac{I}{A}}$ ………………….. For rectangular section

L_u= 3.00m

$\frac{M1}{M2} = 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. Braced\ fram$

$\boldsymbol{About\ x\ axis\ (h = 350mm)}$

$\frac{Klu}{r} \leq 34 - 12\left(\frac{M1}{M2}\right) \leq 40 \rightarrow ACI - 10.12.2$

$\frac{Klu}{rx} = \frac{1*3.00}{0.3*0.35} = 28.57 > 22$ $\therefore$ *Long column about x-axis direction.*

$\boldsymbol{About\ y\ axis\ (b = 600mm)}$

$\frac{Klu}{ry} = \frac{1*3.00}{0.3*0.6} = 16.6 < 22$ $\therefore$ *short column about y-axis direction.*

$$EI = \frac{0.4E_c I_g}{1 + \beta dns} \ldots\ldots\ldots\ldots\ldots\ldots. [ACI\ 318 - 2002(Eq. 10 - 15)]$$

$$Ec = 4700\sqrt{fc'} = 4700\sqrt{24} = 23025.20\ Mpa$$

$$\beta_{dns} = \frac{1.2DL}{1.2DL + 1.6\ LL} = \frac{1.2 * 1883.1}{2893.8} = 0.7808$$

$$Ig = \frac{bh^3}{12} = \frac{0.6 * 0.35^3}{12} = 2.143\ m^4$$

$$EI = \frac{0.4 * 23025.20 * 2.143}{1 + 0.7808} = 11083.33 KN.m^2$$

Determine the Euler buckling load P_c:

$$P_c = \frac{\pi^2 EI}{(KLu)^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots. ACI318 - 2002(Eq. 10 - 13)$$

$$P_{cr} = \frac{3.14^2 * 11083.3}{(1.0 * 3)^2} = 12141.87\ KN$$

Calculate the moment magnifier factor $\boldsymbol{\delta_{ns}}$:

$$Cm = 0.6 + 0.4\left(\frac{M1}{M2}\right) \ldots\ldots\ldots\ldots\ldots\ldots ACI318 - 2002(Eq. 10 - 16)$$

$$Cm = 0.6 + 0.4 * 1 = 1$$

$$\delta_{ns} = \frac{Cm}{1 - \left(\frac{Pu}{0.75Pc}\right)} \geq 1.0 \ \rightarrow ACI318 - 2002(Eq. 10 - 12)$$

$$\delta_{ns} = \frac{1}{1 - \left(\frac{1881}{0.75 * 12141.87}\right)} = 1.26 > 1$$

The magnified eccentricity and moment:

$$e_{min} = 15 + 0.03 * h$$
$$= 15 + 0.03 * 350 = 25.5mm = 0.0255m$$

$$e = e_{min} * \delta_{ns}$$

$$= 25.5 * 1.26 = 32.13mm$$

$$\frac{e}{h} = \frac{32.13 * 10^{-3}}{0.35} = 0.0918$$

Assume Ø14

$$\gamma = \frac{h - 2 * cover - 2 * d_s - d_b}{h}$$

$$= \frac{350 - 2 * 40 - 2 * 10 - 14}{350} = 0.67$$

γ: *the ratio of the distance between the centers of the outside layers of bars to the over all depthes of the column*

$$\frac{\emptyset P_n}{A_g} = \frac{1881 * 10^3}{350 * 600} * 0.145 = 1.3$$

From Interaction diagram:

$\Upsilon(0.6) < \Upsilon(0.67) < \Upsilon(0.75)$

$\rho g = 0.01$

Use linear interpolation to compute the value of $\Upsilon = 0.67$

Diagram A-9a for ($\Upsilon = \mathbf{0.6}$):

$$\frac{\emptyset P_n}{A_g} == 1.9$$

Diagram A-9b for ($\Upsilon = \mathbf{0.75}$):

$$\frac{\emptyset P_n}{A_g} = 2.1$$

$\frac{\emptyset P_n}{A_g}$ for $(\Upsilon = 0.67) = 1.9 + \frac{2.1-1.9}{0.75-0.6} * (0.67 - 0.6) = 1.99$

Diagram A-9a for ($\Upsilon = \mathbf{0.6}$): $\rho_g = 0.013$

Diagram A-9b for ($\Upsilon = \mathbf{0.75}$): $\rho_g = 0.012$

$$\rho_g for\ (\Upsilon = 0.67) = 0.012 - \frac{0.013 - 0.012}{0.75 - 0.6} * (0.67 - 0.6) = 0.0115$$

$\rho_g(0.0115) > \rho_{g,min}(0.01)$

$A_{st} = \rho_g * A_g = 0.0115 * 350 * 600 = 2415mm^2$

Select 16Ø14 with $A_s = 2462.4mm^2 > A_{st} = 2415mm^2$

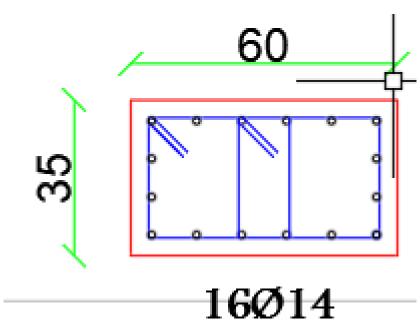

Figure 38 Column Section

4.11.4 Design of the Tie Reinforcement:

Use ties Ø10 with spacing of ties shall not exceed the smallest of:

$1.48\ times\ the\ tie\ \ diemeter, 48\ d_s = 48 * 10 = 480mm$

$2.16\ times\ the\ longitudinal\ bar\ diemeter, 16\ d_b = 16 * 14 = 224mm \ldots\ldots\ldots\ldots\ldots Control$

$3. The\ least\ dimension\ of\ the\ column = 350mm$

Use ties Ø10 /20

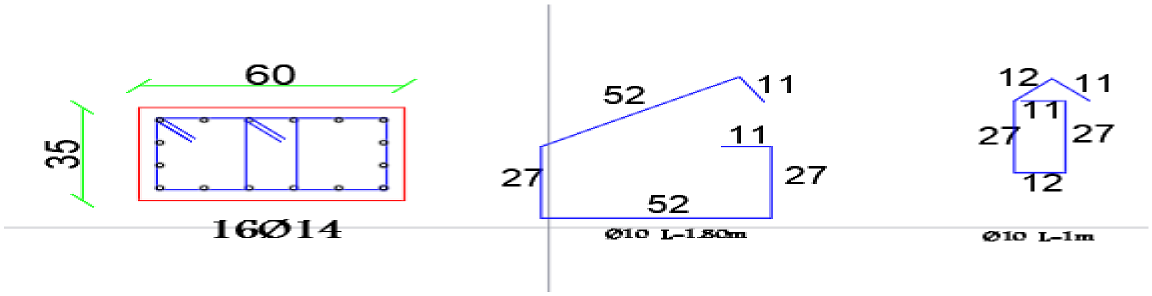

Figure (39): C2 Reinforcement Details

4.11.5 Check for code Requirements:

$$1. Clear\ space = \frac{650 - 40 * 2 - 10 * 2 - * 14}{5} = 107mm > 40mm \ldots\ldots..OK$$

$2. Gross\ Reinforcement\ ratio:$

$\rho_{g,min}(0.01) < \rho_g(0.0115) < 0.08 \quad \ldots\ldots..OK$

$3. Number\ of\ bars: 16 > 4 - For\ rectangular\ section \ldots\ldots\ldots\ldots OK$

$4. Minumum\ tie\ diameter: Ø10\ for\ Ø14\ \ \ldots\ldots\ldots\ldots OK$

$5. Spacing\ for\ ties: s = 20cm \ldots\ldots\ldots\ldots OK$

CHAPTER

5

Results and Recommendations

5.1 RESULTS

5.2 RECOMMENDATIONS

5.3 Reference

5.1 The Results

1. The most important step before you start designing is to carefully study architect ural plans for making column distribution.

2. Each student or structural designer should be able to design manually so he can get the experience and knowledge in using the computer software.

3. Experience in the use of construction programs cannot be reached without understanding the basic concepts of structural design.

4. One of the important steps of the structural design is how to connect the structural members to work together, then to divide these members and design them individually .

5. When choosing the construction system, it is best to distribute nerves in the long direction and bridges in the short direction to reduce the loads on bridges resulting i n less armament which means fewer costs.

1 We have used the live loads using the Jordanian code of loads.

5.2 The Recommendations

This project has an important role in widening and enhancing our understanding to the nature of the structural project 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 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 use after that in resisting the earthquake loads and other lateral loads.

5.3 Reference

[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] كود البناء الأردني، كود الأحمال والقوى، عمان، الأردن: مجلس البناء الوطني الأردني، 2006م