Palestine Polytechnic University
College of Engineering
Civil Engineering Department

Graduation Project

"STRUCTURAL DESIGN OF A RESIDENTIAL-COMMERCIAL BUILDING"

Project Team

Ayat Alwawi
Raghad Diab
Loren sous

Supervisor:

Dr. Belal Al-massri

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

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The undersigned hereby certify that they have read, examined, and recommended to the Department of Civil Engineering in the College of Engineering at Palestine Polytechnic University the approval of a project entitled: **Structural Design of a Residential Commercial Building**, submitted by Ayat Alwawi , Raghad Diab and Loren Sous for partial fulfillment of the requirements for the bachelor's degree.

Dr. Belal Almassri (Supervisor) :

Signature:……………..………………… Date:…..……………..…………………

Project Approved by :

ENG. Belal ALmassri

Head of Civil Engineering Department

Palestine Polytechnic University

Signature:……………..………………… Date:…..……………..…………………

Dr. Yousef Sweiti

Dean of College of Engineering

Palestine Polytechnic University

Signature:……………..………………… Date:…..……………..…………………

STRUCTURAL DESIGN OF A RESIDENTIAL COMMERCIAL BUILDING

Supervisor : Dr. Belal Al-massri

Project Team: Ayat Alwawi .

Raghad Diab.

Loren Sous.

ABSTRACT

This project aims to apply the theoretical knowledge that has been acquired during the years of study through making a complete analysis and design of a 9-story residential building with an estimated total area of 3222.1 m^2.

In this regards, the architectural plans of the building were studied. Then structural planning of the building was done, in which the location of columns and beams was determined to fit with the architectural plans. A detailed structural study also was carried out to estimate loads that act on each member using the Jordanian code for gravity loads estimation and (ASCE7-16) code for the definition of lateral seismic loads.

Analysis and design then were done in accordance with ACI 318-14 Building code based on the ultimate strength method for concrete design and working stress method for soil design. That was using structural programs such as Atir BeamD, Found, SP Column, Safe 2016, and Etabs2018. Finally, structural working drawings were prepared to present the reinforcement details of all members.

التصميم الانشائي لمبنى سكني تجاري .

فريق العمل: ايات الواوي , رغد دياب , لورين صوص .

الملخص

يهدف المشروع الى عمل تصميم انشائي لجميع العناصر الانشائية المكونة لمبنى سكني تجاري مكون من 10 طوابق تقدر مساحتها الاجمالية بـ 3222.1م^2. وذلك لما للتصميم االنشائي من اهمية فهو من اهم المراحل التي يمر بها المبنى والتي يتم فيها تحديد اماكن الاعمدة و الأنظمة الانشائية لمختلف عناصر المبنى وبذلك يتم تحويل المخططات المعمارية الأولية الى مخططات قابلة للتنفيذ .

وتحقيقا لهدف المشروع تم في البداية دراسة المخططات المعمارية و اختيار انسب الية لتوزيع العناصر االنشائية بما لا يتعارض مع التصميم المعماري للمبنى , ثم تم عمل دراسة انشائية مفصلة تم فيها تقدير الاحمال المتوقعة على جميع العناصر الانشائية بالاعتماد على الكود الأردني (في تحديد الأحمال الحية) والكود الأمريكي ASCE-16 لتقدير احمال الزلازل , بعد ذلك تم تحليل وتصميم جميع تلك العناصر.

بالاعتماد على الكود الامريكي ACI318-14 وباستخدام مجموعة من البرامج الهندسية . في النهاية تم إعداد المخططات التنفيذية لجميع العناصر االنشائية المكونة المبنى ليصبح المبنى قابلاً للتنفيذ .

DEDICATION

To those who have always believed in us ...

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

To those who gave us strength ...

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

To our families ...

Project Team

ACKNOWLEDGEMENT

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

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

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

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

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

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

Project Team

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LIST OF ABBREVIATIONS

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

CHAPTER 1

INTRODUCTION

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

1.1 General Overview

Any building is supported by a framed arrangement known as Structure which is a system formed from the interconnection between structural members. The structural design requires an intelligent manner in making decisions regarding the systems of different structural elements and that cannot be achieved by an understanding of basic concepts of structures only. Rather, that understanding must be applied through practice.

From this point of view, a residential building was chosen to be designed and the reason for choosing this building is because it is the most common and most requested in the engineering labor market, because of the recent tendency of people to live in such buildings.

The building was designed by applying the acquired knowledge in the design of different structural elements to provide a safe design that achieves the required engineering specifications and standards.

1.2 Project Problem

As a result of the variety of construction systems and the need of making a balance between costs and safety in the design, it was necessary to find the most appropriate structural system that satisfies the strength and serviceability requirements for the chosen residential building.

1.3 Project Objectives

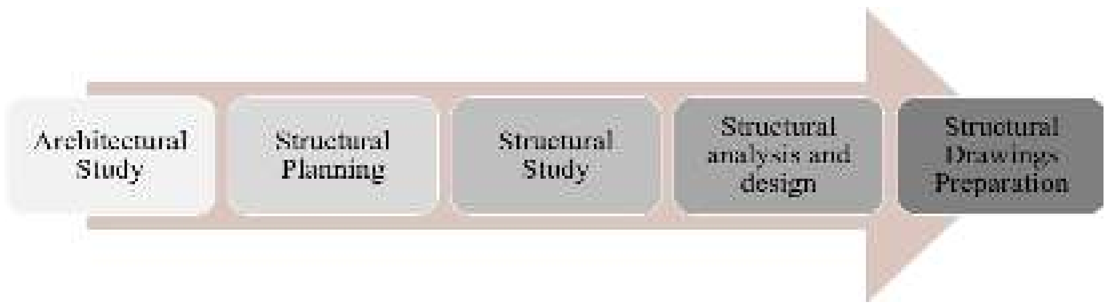
This Project was chosen to achieve the following galls:

- Correlate the theory that has been gained in the design courses with practical life.
- Increase the ability to choose a suitable structural system of elements that meets design requirements.
- Get experience in dealing with different problems encountered in the design process.
- Practice the structural analysis and design programs as well as theoretical knowledge.

1.4 Work Procedure

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

1. Architectural study in which the site, building plans, and elevations were been studied.
2. Structural planning of the building, in which the location of columns, beams, and shear walls was determined to fit with architectural design.
3. Structural study in which all structural members were identified and different loads were been estimated.
4. A complete analysis and design for all elements were done according to the ACI Code.
5. Preparation of Structural drawings of all existing elements in the building.
6. Project Writing in which all these stages were presented in detail.



Figure(1- 1) :Work Procedure

1.5 Project Scope

This Project contains the following Chapters :

CHAPTER 1: A general introduction.

CHAPTER 2: An architectural description of the project.

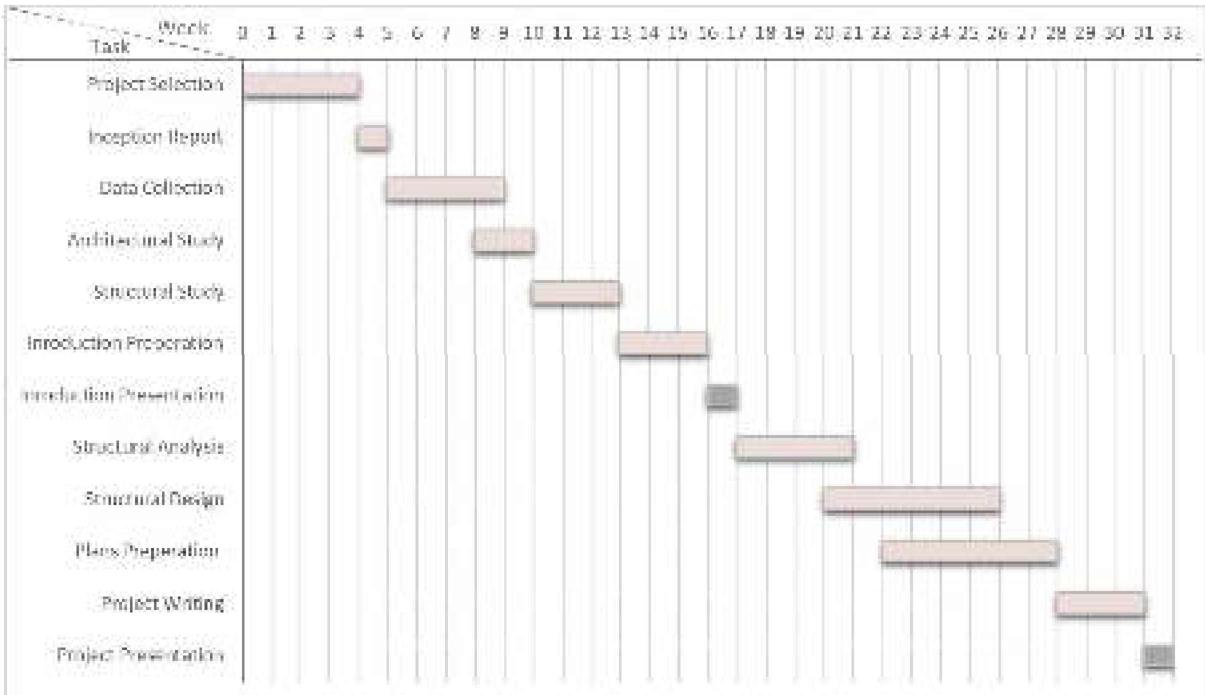
CHAPTER 3: A general description of the structural elements.

CHAPTER 4: Structural analysis and design of all structural elements.

CHAPTER 5: Results and Recommendations.

1.6 Project Timeline

The following chart shows the project plan and timeline :



Figure(1- 2):Project Timeline

1.7 Programs used in the project

There are several computer programs used in this project:

1. Microsoft Office: It was used in various parts of the project such as text writing, formatting, and project output.
2. AUTOCAD 2014: for detailed drawings of structural elements.
3. ATIR18: Structural design and analysis of structural elements.
4. SP Column: design of columns.
5. Etabs18: design and analysis of structural elements especially for walls.
6. Safe16: design of combined and matt foundation.

CHAPTER 2

ARCHITECTURAL DESCRIPTION

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

2.1 Introduction

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

Starting first with the architectural design stage, at this stage, the shape of the structure is determined and take into account the inquiry of the various functions and requirements for which you will create this building, here the initial distribution of the facilities is made, to achieve the required spaces and dimensions, and in this process, lighting, ventilation, movement, mobility, and other functional requirements are also studied.

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

2.2. General Identification of the project

The proposed project is a residential commercial building with 10 floors two of them is a commercial centers, and Basement floor, the rest is residential floors with two apartments in each (In addition to the roof)

The building is proposed to be built on 751 square meters land. Although this area is considered small for the construction of such a project, The architect showed her proficiency in design as she was able to use the space to design a building that meets the standards and provides comfort for its residents taking into account the architectural beauty in the overall design.

Figure (2- 1): 3D shot of the building

2.3 General site description

The proposed project is located in - Ras al-Jura in the city of Hebron, near to Traffic Department, in a residential area with a good infrastructure of roads. It is an easily accessible location with available needed services such as electricity and communications link. The location of the project is clearly shown in Figure (2-1).

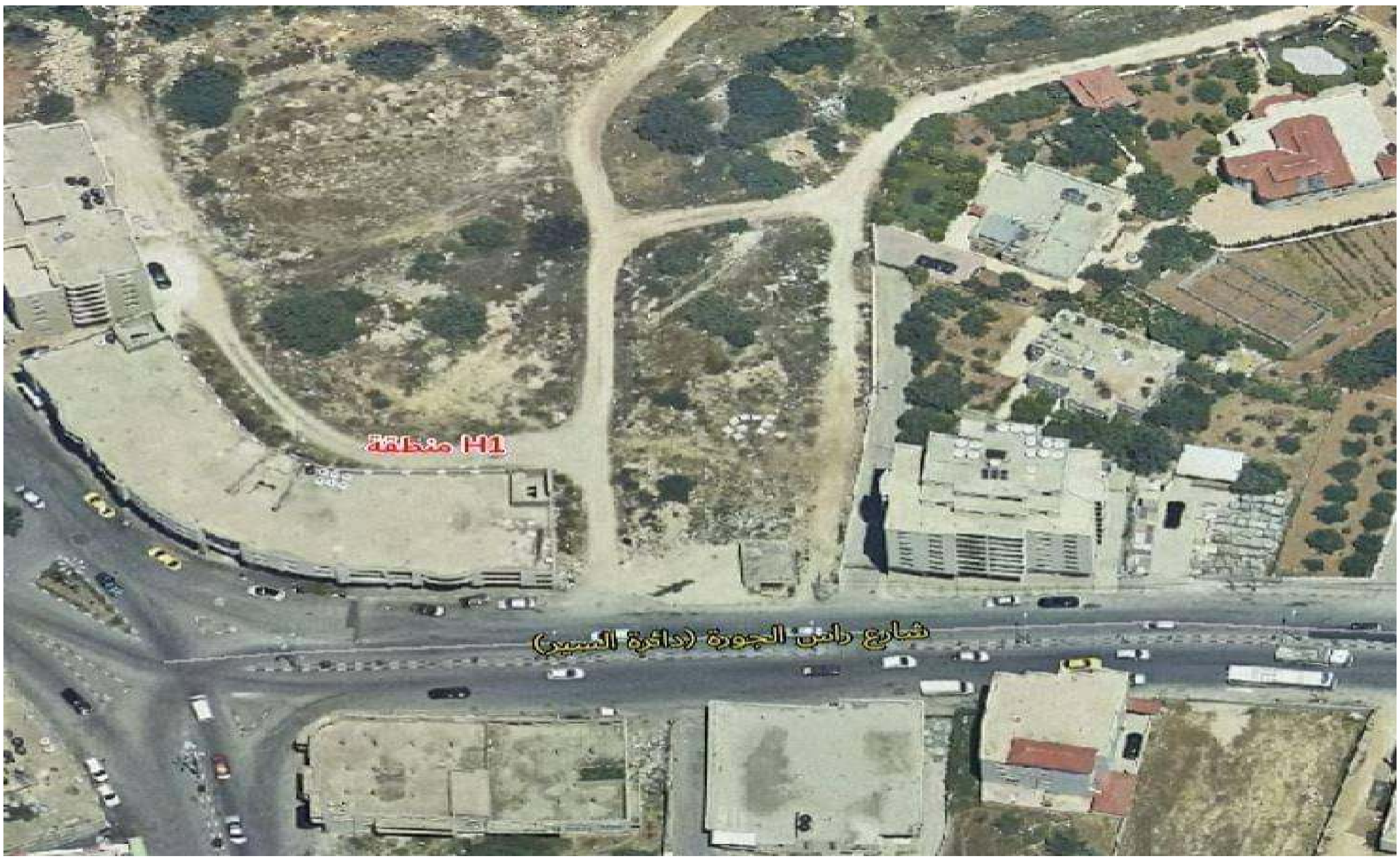


Figure (2- 2): Site Location

2.4 Floors Description

The Project contains five types of floors: Basement, Ground floor ,first floor , and 6 residential floors ,Roof floor ,with a total area of 3222.1 m^2. The following is a brief description of each floor.

1. BASEMENT FLOOR

The basement floor level is 3.12m below the level of Main Street with an area of 490.4 m^2. It can accommodate 15 parking spaces, The entrance and exit to the basement is from the North side of the building Which is clearly shown in figure (2-2).

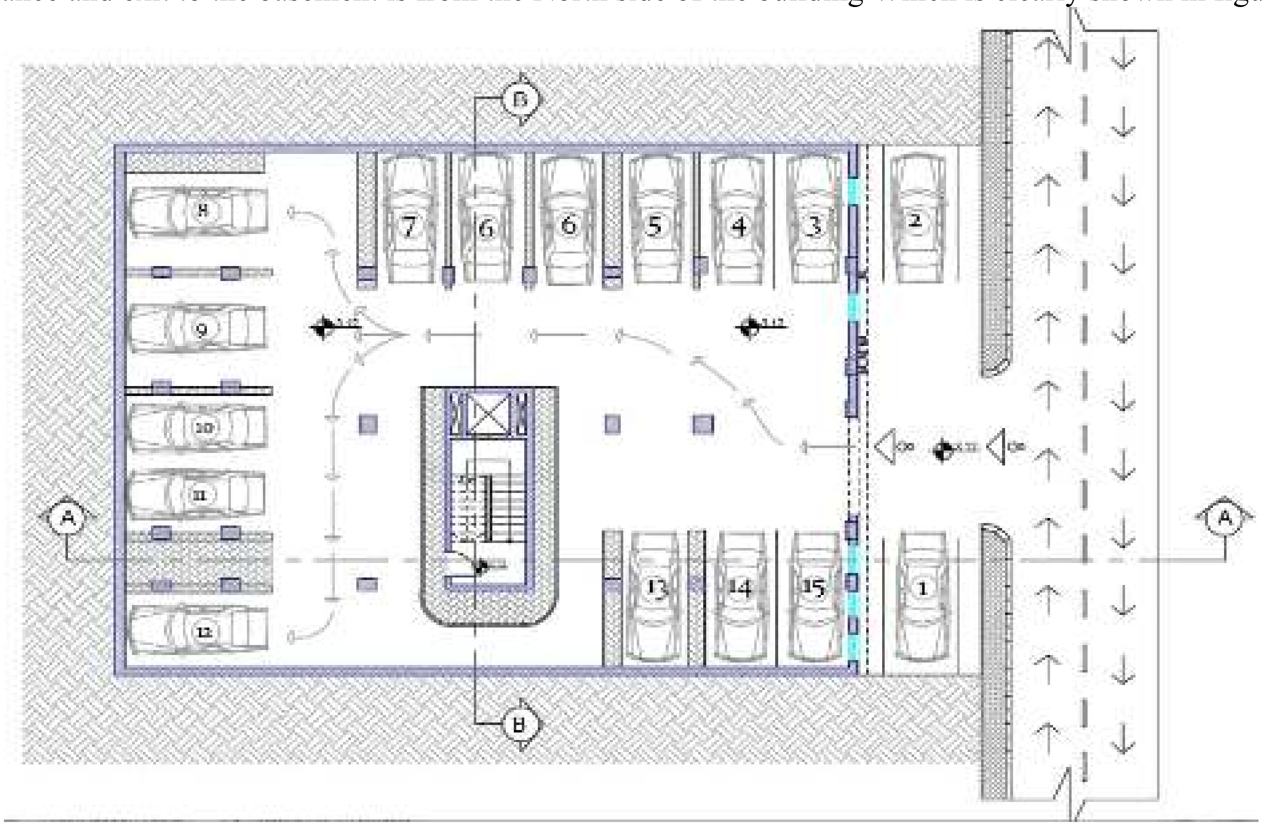


Figure (2- 2): Basement floor plan

2. GROUND FLOOR

It contains a first commercial center with an area of 292.4 m2. The entrance is on the south elevation of the building at main street level.

Figure (2-3) shows the plan of the ground floor on which the entrance to the commercial center appears, in addition to the connection between spaces.

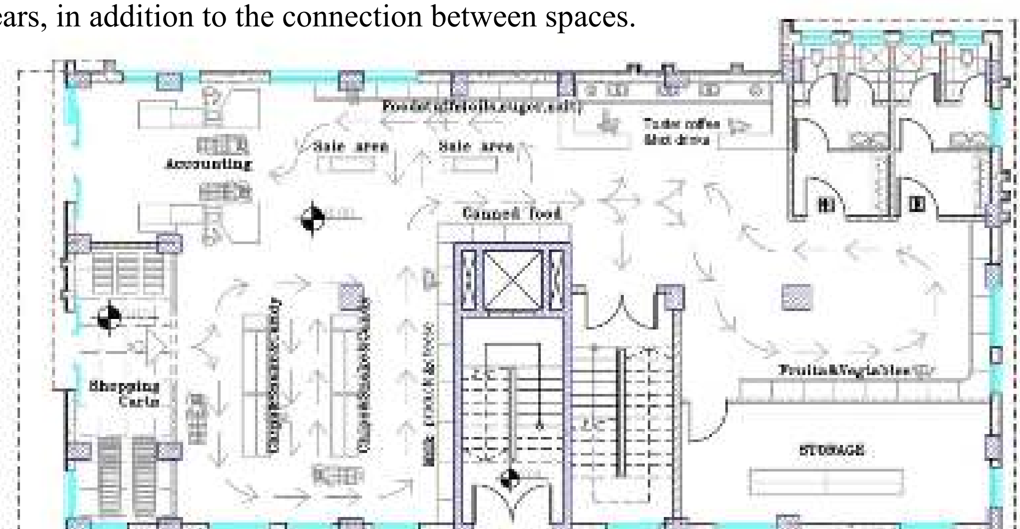
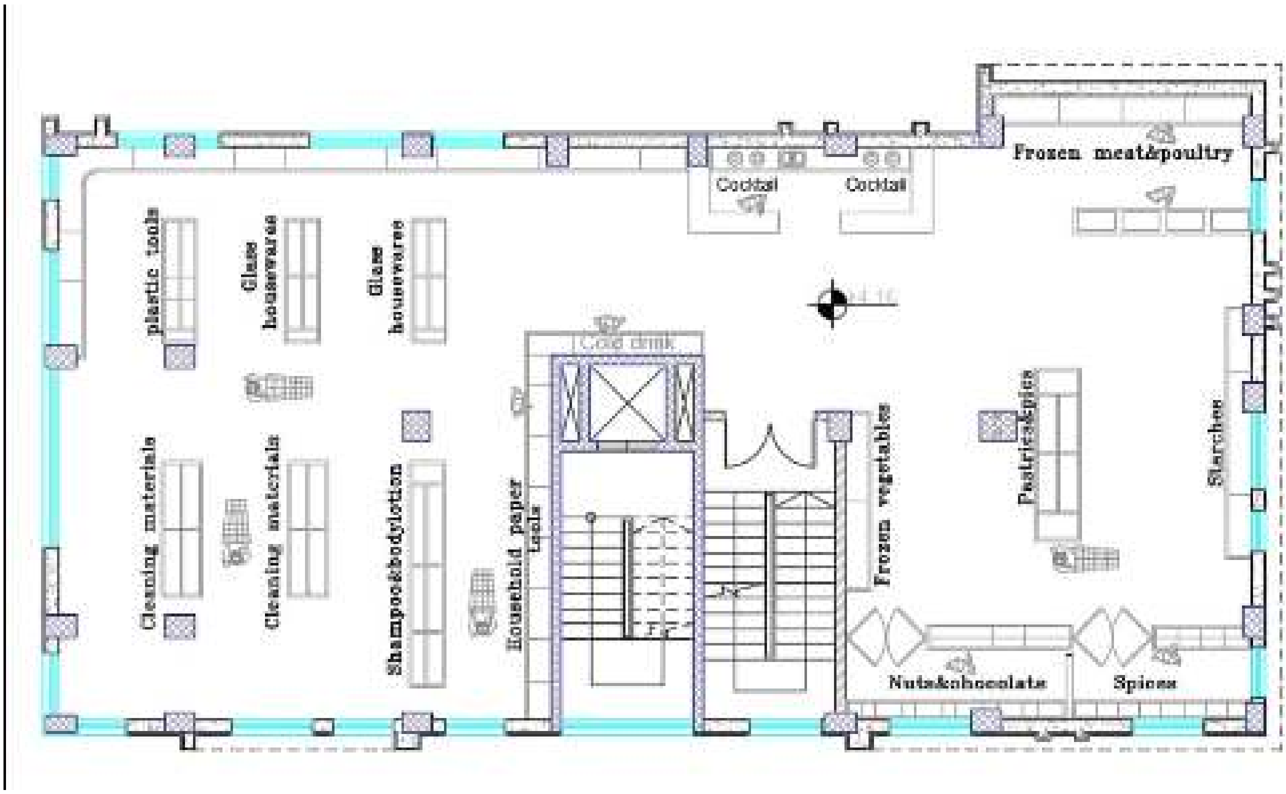


Figure (2- 4):Ground floor Plan

3. First FLOOR

It contains a second commercial center with an area of 292.4 m2.



4. TYPICAL FLOORS

There are 6 typical floors, each floor consists of two apartments and has a total area of 306.7m^2, Each apartment consists of:

- Guest room.
- Living room.
- Master bedroom.
- Kitchen.
- Three bathrooms.
- Two bedrooms.
- Dining room.

Figure (2-4) shows the plan of the typical residential floor on which the entrances appear, in addition to the connection between parts of each apartment.

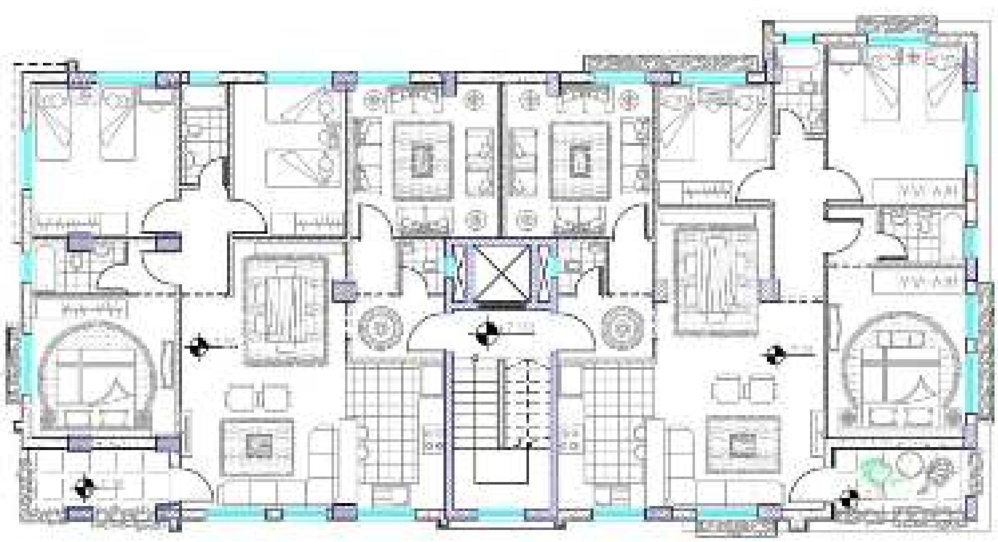


Figure (2- 5): Typical floor Plan

5. ROOF FLOOR

It's a two apartment with an area equivalent to 188.9 m^2.

Roof floor consist of the following :

- Bedroom.
- Living room.
- Dining room.
- Kitchen.
- Bath.

Figure (2-5) shows the plan of the Roof floor on which the entrances appear, in addition to the parts of the apartment.

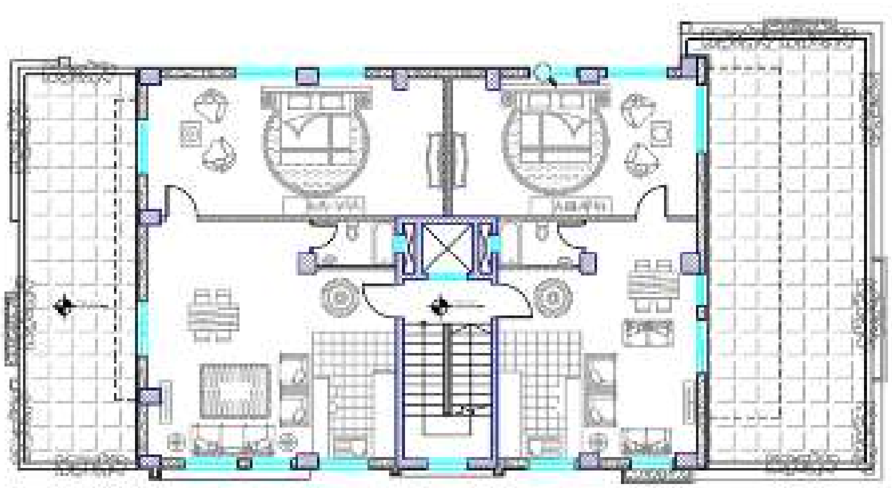


Figure (2- 6):Roof floor Plan

6. MOVEMENT AREAS

This building contains one stairs and one elevator,.The main stairs and elevator are located on the main elevation, they are also located in the middle of the building, which makes a connecting point for the two apartments.

2.5 Elevations Description

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

2.3.1 North elevation

This elevation contains the entrance to the basement. And This elevation is characterized by its glass that is integrated with stones, as it appears in figure (2-6).

Figure (2- 7):North elevation

2.3.2 South elevation

As shown in figure (2-7) this elevation overlooks the main street and has the main entrance and exit to the commercial centers (Hebron Mall).

Figure (2- 8):South elevation

2.3.3 East elevation:

Figure (2- 9): East elevation

2.3.4 West elevation:

Figure (2- 10): West elevation

2.4 Sections of the building

These sections explain the movement inside the building through the stairs and elevator. It also shows more details for the heights and levels for slabs, windows, and doors. Figures (2-11) and (2-12) shows two sections of the building.

Figure (2- 11):Section A-A

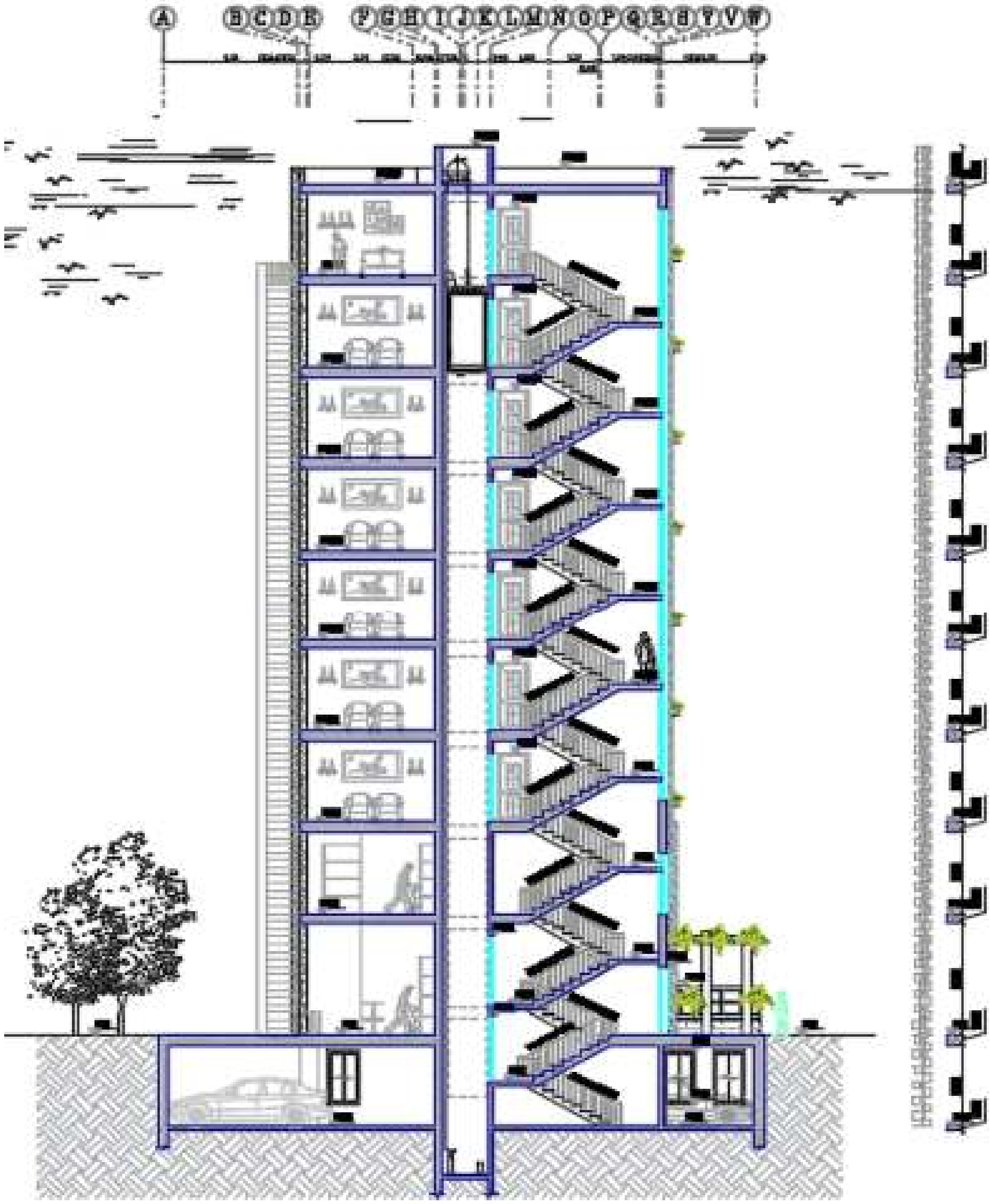


Figure (2- 12):Section B-B

CHAPTER 3

STRUCTURAL DESCRIPTION

3.1 Introduction

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

The knowledge of structural elements of any project is essential in the design of reinforced concrete structures. In this chapter, a study of the different structural elements such as columns, bridges, foundations, and other elements was conducted. Also, different loads were estimated in accordance with the requirements, standards, and standard specifications that will be mentioned later.

3.2 The Aim of the Structural Design

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

1- **Safety**: The structure should be able to carry all expected loads safely, without failure, that is, without breaking or collapsing under the loads.

2- **Durability:** The structure should last for a reasonable period of time.

3- **Stability:** to prevent overturning, sliding, or buckling of the structure, or parts of it, under the the action of loads.

4- **Strength:** to resist safely the stresses induced by the loads in the various structural members.

5- **Serviceability**: To ensure satisfactory performance under service load conditions - which implies providing adequate stiffness and reinforcements to contain deflections, crack-widths, and vibrations within acceptable limits, and also providing impermeability and durability (including corrosion-resistance), etc.

There are two other considerations that a sensible designer must bear in mind, economy and aesthetics. As any engineer can always design a massive structure, which has more than adequate stability, strength, and serviceability, but the ensuing cost of the structure may be exorbitant, and the end product, far from aesthetic.

3.3 Scientific Tests

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

3.4 Loads Acting on the Building

Loads that acting on the building must be calculated and selected carefully because any error in identifying and calculating loads reflects negatively on the structural design of various structural elements. The building is exposed to loads of live and dead loads, wind loads, snow loads, and loads of earthquakes.

3.4.1 dead loads

a constant load in a structure (such as a bridge, building, or machine) that is due to the weight of the members, the supported structure, and permanent attachments or accessories.

Figure(3- 1): Dead Load

3.4.2 live load

Consider any dynamic force that could move on or in a building over the course of its lifetime: books in a library, people passing through an office building, the weight of furniture in a house. Each of these scenarios is considered a live load.

Figure(3- 2): Live Load

3.3.3 Environmental loads

Environmental forces that might affect a structure based on its geography may technically be considered live loads (because they're not inherently part of the structure, and because they change over time). These could include seismic activity, wind, rain, and snow. They are considered separate from live or dead loads because they may act laterally on a structure (whereas other loads act vertical.)It is the third type of load that must be taken into account in the design, and these loads are:

1. Wind Loads

They are horizontal forces that affect the building and their effect appears in tall buildings. They are the forces that the wind affects buildings, installations, or parts of, and they are positive if they are caused by pressure and negative if they are caused by tension, and are measured in kilotons per square meter (KN / m2). Wind loads are determined depending on the height of the building above the ground, and the location in terms of surrounding buildings, whether high or low.

Figure(3- 3):Wind Loads

2. Snow

Snow loads can be evaluated based on the following principles:

• Height of the facility above sea level.

• Slope of the roof exposed to snow.

The following table shows the value of snow loads according to the height above sea level, according to the Jordanian code

Figure(3- 4): Snow Loads

Table 3 - 1 The value of snow loads by height above sea level

SNOW LOADS (KN/m^2)	HEIGHT OF THE FACILITY ABOVE .SEA LEVEL (m)
0	h < 250
(h-250) /1000	500 > h > 250
(h-400) / 400	1500 > h > 500
(h – 812.5)/ 250	2500 > h > 1500

3. SeismicLoads

is one of the basic concepts of earthquake engineering which means application of a seismic oscillation to a structure. It happens at contact surfaces of a structure either with the ground or with adjacent structures, which are horizontal and vertical forces that generate torque, and can be resisted by using shear walls designed with thicknesses and sufficient reinforcement to ensure the safety of the building when it is exposed to such loads that must be observed in the design process to reduce Risks and maintenance of the building's performance of its function during earthquakes.

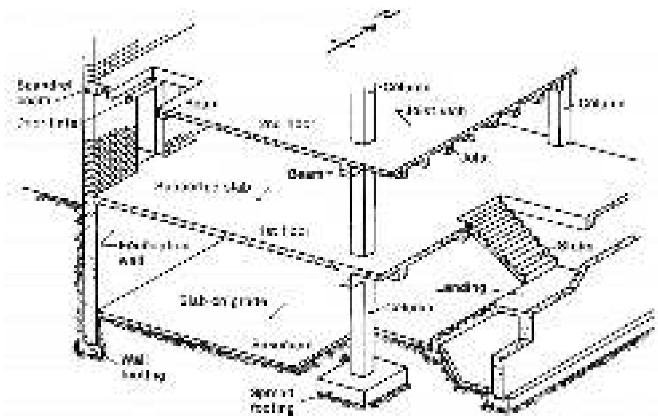
Figure(3- 5):Seismic Loads

4. Shrinkage and expansion loads

As a result of the contraction and expansion of the concrete elements of the building due to the variation in temperature during the seasons of the year, stresses have generated that lead to cracks in the building, where they are avoided and prevented from appearing using the phi 8 reinforcement mesh and also using expansion joints.

3.5 Structural Elements of the Building

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



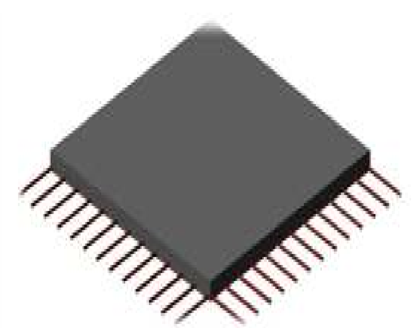
Figure(3- 6):Structural elements of a typical RC structure

3.5.1 Slabs

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

3.5.1.1 Solid slab (one or two way)

A reinforced concrete slab is a crucial structural element and is used to provide flat surfaces(floors and ceilings) in buildings. On the basis of reinforcement provided, beam support, and the ratio of the spans, slabs are generally classified into one-way slab and two-way slab. It is known that solid slabs should be supported by drop beams.

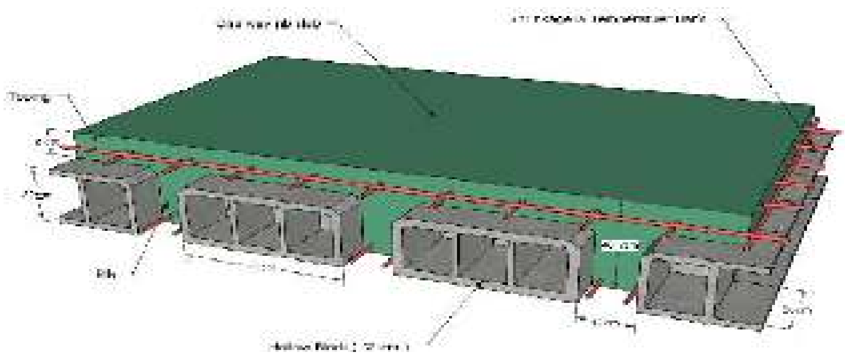


Figure(3- 7): Solid slab

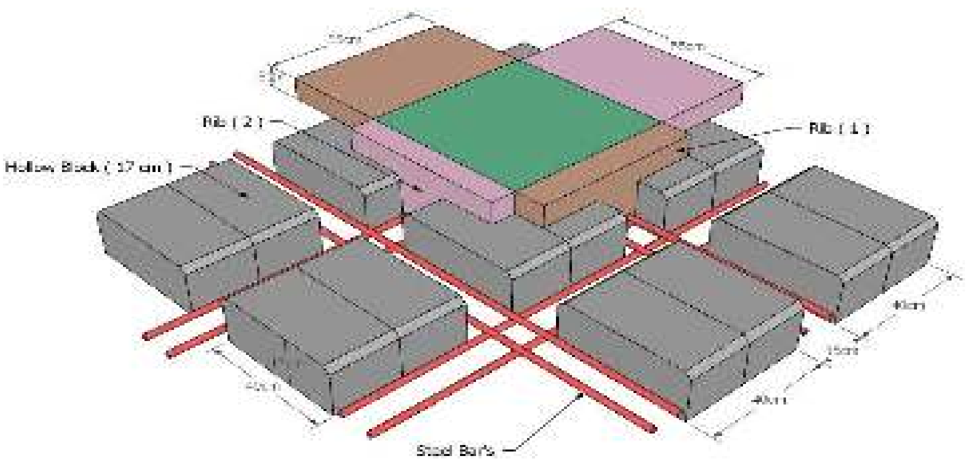
3.5.1.2 Ribbed slab (one or two way)

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

Figures (3-8),(3-9) describe one-way and two-way ribbed slabs respectively.



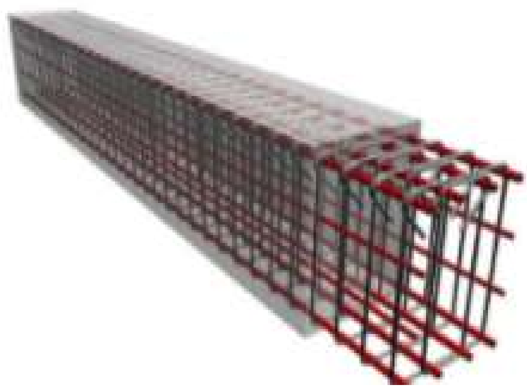
Figure(3- 8):One way ribbed slab.



Figure(3- 9):Two way ribbed slab

3.5.2 Beams

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



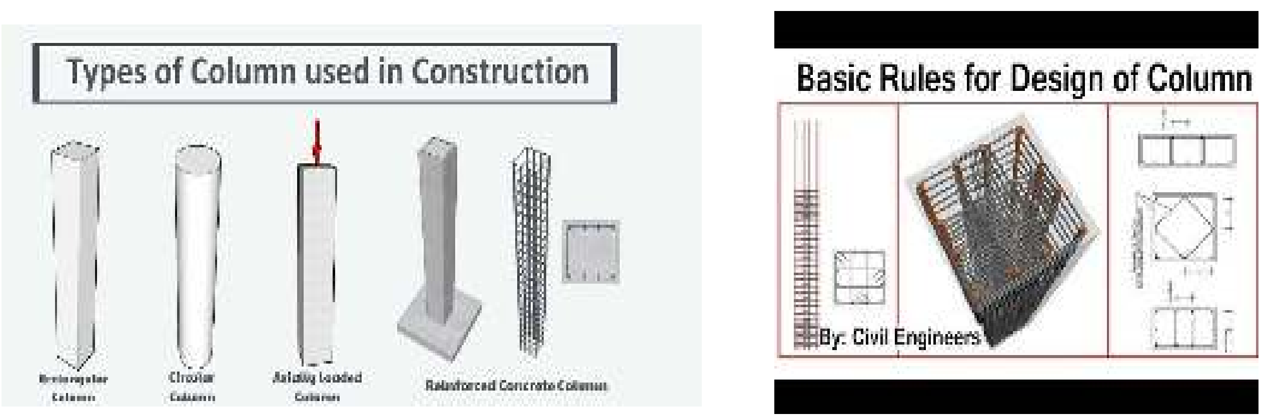
Figure(3- 10):Beams

3.5.3 Columns

vertical structures transmit the compressive loads.

The floor and the columns on the floors above are supported by columns; the columns of the bottom floor must be large enough to bear the accumulative weight of each floor above it. They can move loads to the foundations and soil below from the slab and beams.

Columns should be positioned uniformly on all floors for the most efficient support, if possible. The stability of the lowest set of columns would be improved by this.

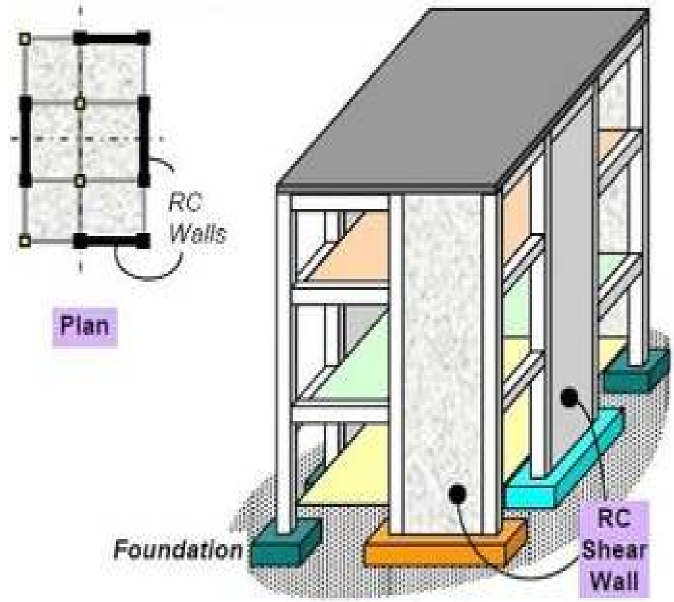


Figure(3- 11):Different types of Columns

3.5.4 Shear walls

Shear wall is a structural member in a reinforced concrete framed structure to resist lateral forces such as wind forces.

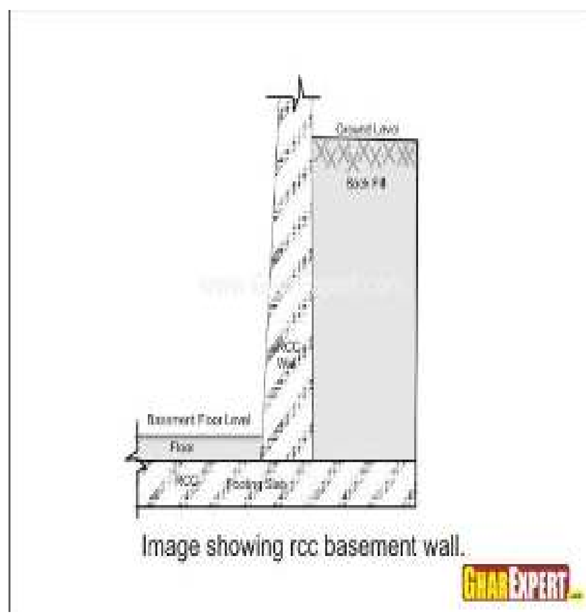
Shear walls are generally used in high-rise buildings subject to lateral wind and seismic forces. In reinforced concrete framed structures the effects of wind forces increase in significance as the structure increases in height. Codes of practice impose limits on horizontal movement or sway.



Figure(3-12)Shear wall

3.5.7 Basement walls

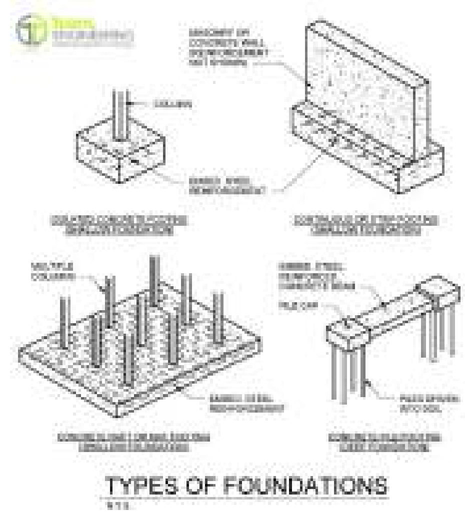
A basement wall is a wall that is used on the floor and ceiling to provide support to the side walls as well as to the structure. It handles the pressure of the sidewalls and provides space for living inside the walls. Basement walls bear the load of the whole structure.



Figure(3-13)Basement wall

3.5.5 Foundations

is the part of a building that fixes it into the soil. These structures provide support for the main structures that appear above the soil level, much like the roots of a tree support the stem , One of its functions is to transfer loads from the structure to the ground. For example, slabs transfer their weight to girders, which in turn transfer that load as well as loads applied to them to the beams. Beams transfer that load and any additional loads applied to them to the columns, and finally, columns transfer that load to the foundations.

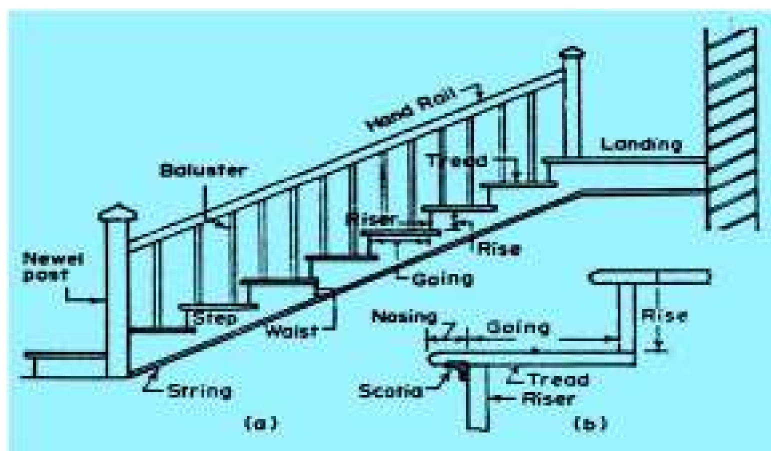


Figure(3- 14): Types of Footing

3.5.6 Stairs

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

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



Figure(3- 15): General sections of stairs

CHAPTER 4

STRUCTURAL ANALYSIS AND DESIGN

4.1. Introduction
4.2. Determination of slab thickness
4.3. Design of one-way ribbed slab
4.4. Design of Beam B40
4.5. Design of Column C18
4.6. Design of Shear Wall
4.7. Design of Basement Wall
4.8. Design of Basement Footing
4.9. Design of Combined Footing F5
4.10. Design of Stairs

4.1. Introduction

After finishing the structural planning of the building, in which the location of columns and beams was determined. A complete design for all elements was done for flexure, shear, and deflection.

In this chapter, the analysis and design procedure for a sample of each structural element in the building are explained in detail.

The following General considerations are taken throughout the analysis and design processes of this project:

1. All members were designed according to ACI 318-14 Building code.
2. Gravity loads were estimated using the Jordanian code.
3. (ASCE7-16) is used for the definition of lateral seismic loads.
4. The ultimate strength design method is used during the analysis and design of this project.
5. Working Stress Method is used for soil design.
6. The compressive strength of concrete for all elements is B300 which equals to $Fc' = 24$ MPa.
7. Yield strength of reinforcing rebars $Fy = 420$ MPa .

4.2. Determination of slab thickness

The thickness of the one-way ribbed slab is obtained according to the ACI code to achieve deflection requirements. The following table summarizes the determination of thickness for ribs that gives maximum values:

Table(4- 1): Determination of thickness for ribs from maximum values of cases

Supporting type	min. h equation	Rib	Span	min. h (cm)
Simply Supported	L/16	10	1	$= \frac{488}{16} = 30.50$
One end continues	L/18.5	11	3	$= \frac{488}{18.5} = 26.38$
Both ends continuous	L/21	12	3	$= \frac{466}{21} = 22.20$

Since the previous are approximate equations for determination the thickness of a slab ,it will be selected (32cm) and deflection will be checked later.

$\therefore$ Select slab thickness = 32cm with 24cm block & 8cm topping

The following figure shows a typical section in a 32cm thick one-way ribbed slab.

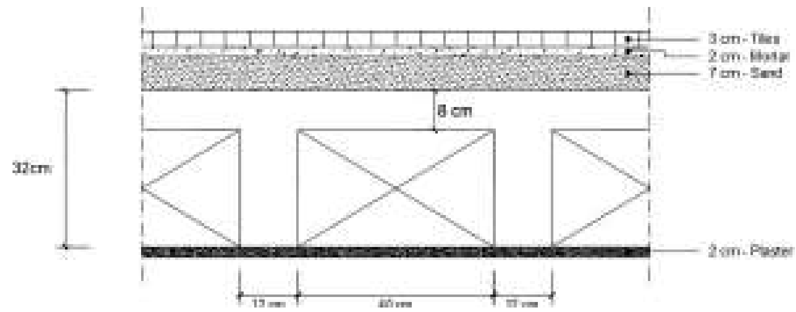


Figure (4- 1) :Typical section of one-way ribbed slab

4.3. Design of one-way ribbed slab

One way ribbed slab Design procedure is explained in the following steps :

4.3.1. Design of topping

Topping in One way ribbed slab can be considered as a strip of 1-meter width and span of hollow block length with both ends fixed in the ribs.

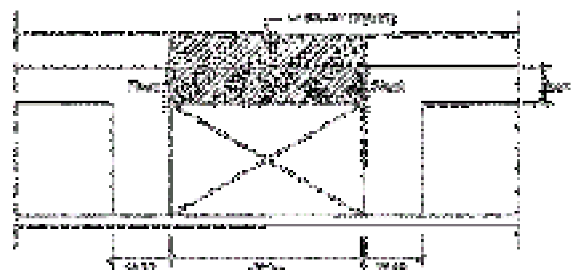


Figure (4- 2):System of topping

4.3.1.1. Calculation of Loads on Topping

Dead loads that act on Topping can be calculated as shown in the following table :

→ Dead Load For 1m strip:

Table(4- 2): Dead Load Calculation for topping

Material	Quality Density (kN/m^3)	Calculation	Dead Load (kN/m)
Tiles	23	= 0.03×23×1	0.69
Mortar	22	= 0.02×22×1	0.44
Sand	16	= 0.07×16×1	1.12
Topping	25	= 0.08×25×1	2
Partitions		= 1.85×1	1.85
∴ Dead Load for 1m strip of topping = 6.1 kN/m			

→ Live Load For 1m strip = **3.0 × 1 = 3.0 kN/m**

→ Factored load (W_u) = 1.2 ×D.L + 1.6 ×L.L = 1.2 * 6.1 + 1.6 * 3.0 = **12.12 kN/m.**

4.3.1.2. Analysis of topping

-Vu = (Wu*L)/2 = (12.12*0.4)/2
=2.4KN

- $Mu = \frac{Wu \times (L)2}{12} = \frac{10.52 \times (0.4)2}{12} = 0.16$ kN.m

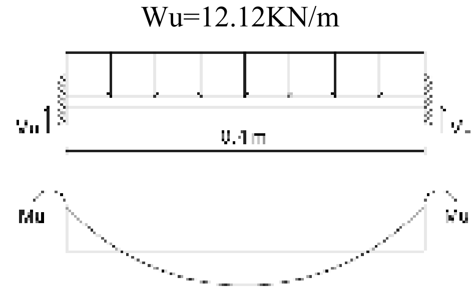


Figure (4- 3): System and analysis of topping

4.3.1.3. Design Strength of topping

→ **Shear Design Strength :**

For Plain concrete section one way shear is calculated using the following equation:

$\Phi \, . \, Vc = \Phi \times 0.11 \times \lambda \times \sqrt{Fc'} \times bw \times h$

$\Phi \, . \, Vc = 0.6 \times 0.11 \times 1 \times \sqrt{24'} \times 1000 \times 80 = 25.87$ kN > Vu=2.4KN → **SAFE**

→ **Moment Design Strength :**

For Plain concrete section with "b = 1 m & h = 8 cm"

$\Phi \, . \, Mn = 0.6 \times 0.42 \times \sqrt{Fc'} \times \frac{b\,h^2}{6}$

$\Phi \, . \, Mn = 0.6 \times 0.42 \times \sqrt{24'} \times \frac{1000 \times 80^2}{6} = 1.32$ kN.m > Mu=0.16 KN.m → **SAFE**

∴ **Plain Concrete Section is SAFE #**

But According to ACI , As_{min} shall be provided for slabs as shrinkage and temperature reinforcement.

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

Minimum (As) = $\rho_{shrinkage} \times Ag$

= 0.0018 × b × h
= 0.0018 × 100 * 8
= 1.44 cm²/m

Step (s) is the smallest of :

1. 3h = 3×80 =240 mm « **controlled**
2. 450 mm.
3. $S = 380(\frac{280}{fs}) - 2.5Cc = 380\,(\frac{280}{\frac{2}{3}*420}) - 2.5*20 = 330mm$

But $S \leq 300(\frac{280}{fs}) = 300\,(\frac{280}{\frac{2}{3}*420}) = 300mm$

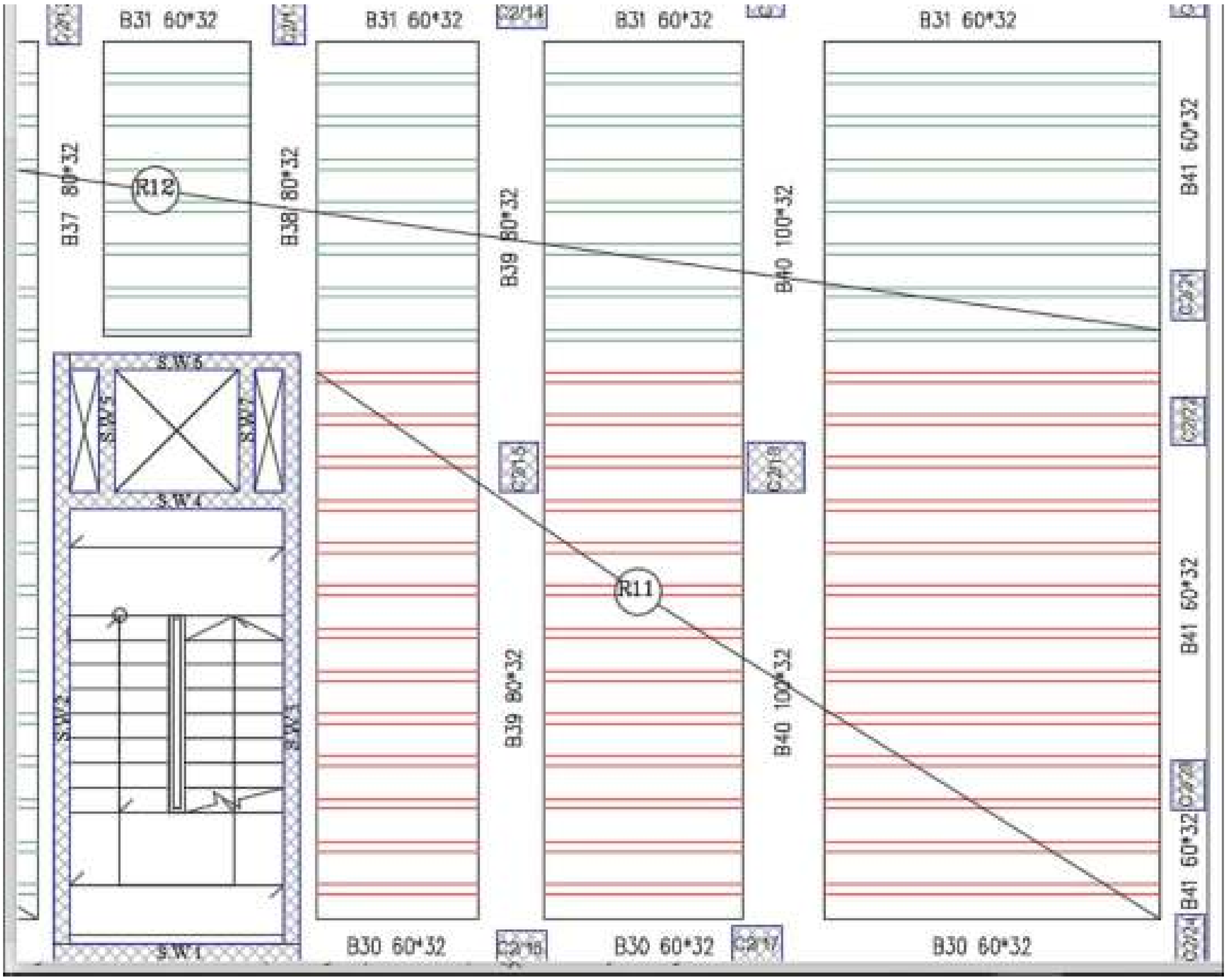
Take S=200mm < Smax=240mm

∴ **Select Mesh Ø8/20cm in both directions.**

Provided As = (π×8²/4)*(100/20) = 2.5 cm²/m > min As = 1.44 cm²/m

4.3.2. Design of Rib (R11)

Rib (R11) is selected to be designed , the following figure shows its location in Typical floor slab:



4.3.2.1. Rib geometry

Requirements for Ribbed Slab (T-Beam Consideration According to ACI) are as follows :

- $bw \geq 10cm \rightarrow$ **select** $bw = 12$ **cm**
- $h \leq 3.5\,bw = 3.5 \times 12 = 42cm \rightarrow$ **select** $h = 32$ **cm**
- $tf \geq \frac{Ln}{12} \geq 50$ mm $\rightarrow$ **select** $tf = 8cm$

4.3.2.2. Loads Calculation for Rib (R11)

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

→ **Dead loads :**

Table(4- 3): Dead Load Calculation for rib (R11)

Material	Quality Density (kN/m^3)	Calculation	Dead Load (kN/m/Rib)
Tiles	23	= 0.03×23×0.52	0.359
Mortar	22	= 0.02×22×0.52	0.229
Sand	16	= 0.07×16×0.52	0.582
Topping	25	= 0.08×25×0.52	1.040
Block	12	= 0.24×12×0.40	1.152
Rib	25	= 0.24 ×25×0.12	0.720
Plaster	22	= 0.02×22×0.52	0.229
Partitions		= 1.85×0.52	0.962
∴ Dead Load =5.27 kN/m/Rib			

→ **Live loads** = 3.0 × 0.52 = **1.56 kN/m/rib**

→ **Factored Load (W_u)** = 1.2×D.L + 1.6×L.L

WuD = 1.2× 5.27 = **6.32 kN/m/rib**

WuL = 1.6*1.56 = **2.49 kN/m/rib**

4.3.2.3. Analysis

Figure (4-5)& (4-6) shows the shear and Moment envelope of the rib (R11) obtained from Atir 2018 software .

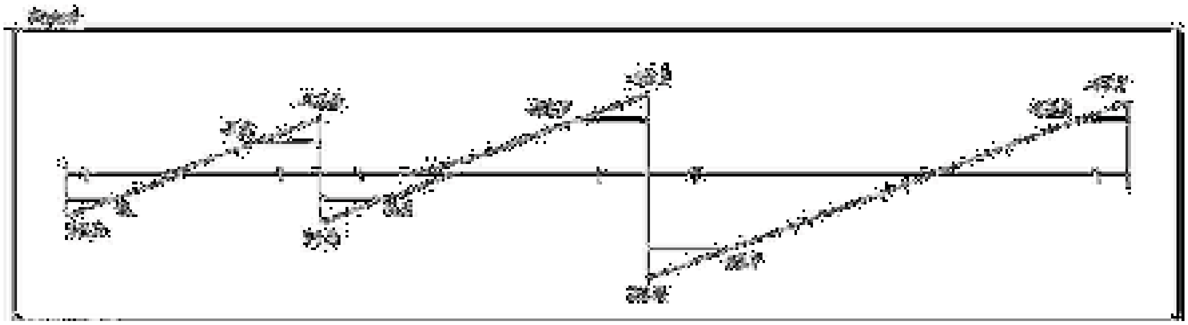


Figure (4- 5):Shear envelope of rib (R11) – [kN]

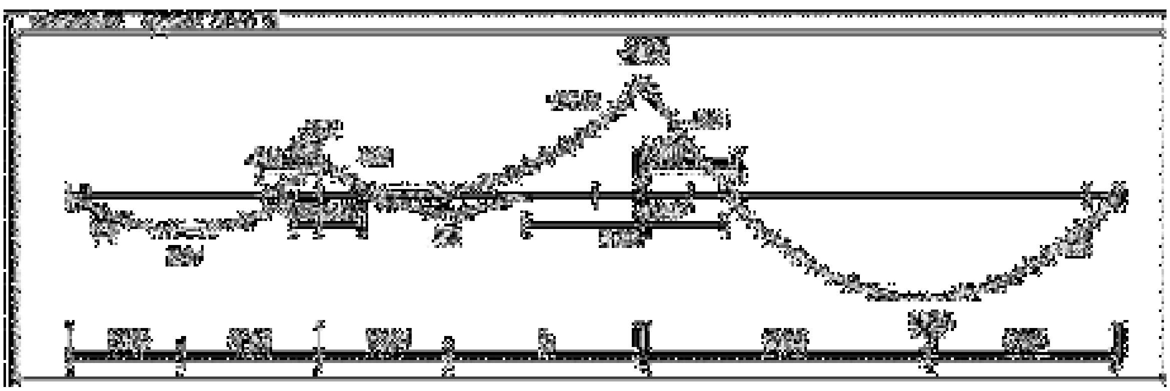


Figure (4- 6): Moment envelope of rib (R11) – [kN.m]

4.3.2.4. Design of Rib for Shear

Shear strength Vc , provided by concrete for the ribs may be taken greater than that for beams. This is mainly due to the interaction between the slab and the closely spaced ribs.

Max. Vu at the critical section at distance d from the face of support is obtained from figure (4-5) , where Vu = 22.8 kN

If $\frac{1}{2}$ Ø.Vc < Vu ≤ Ø.Vc …. No shear Reinforcement is required for slabs .

$\rightarrow$ Ø .Vc = Ø * 1.1 * $\frac{1}{6}$ * $\sqrt{Fc'}$ * bw * d

= 1.1 * 0.75 * $\frac{1}{6}$ * $\sqrt{24}$ * 120 * 283 *10^{-3}

= **22.88 kN**

Ø.Vc=**22.88 kN** < Vu max = 18.70 kN … No shear Reinforcement is required .

∴ Select Ø8/30cm as montage for construction requirements .

4.3.2.5. Design Rib for Flexure

4.3.2.5.1. Design of Positive Moment – Bottom Reinforcement

Check for chosen effective flange width (***be***) :

According to (ACI 318-14) (be) is the smallest of:

- $be \leq Span/4 \leq (608/4) = 152$ cm
- $be \leq 16*hf + bw = 16*8 + 12 = 140$ cm
- $be \leq bw + ½ Lc = 12 + ½ * 40 + ½ * 40 =$ **52 cm** « **Cont.**

⇒ **Design of span 1 - Max $Mu^{+} = 5.90$ kN.m**

1. **Check if (a ≤ t) or (a > t)**

   Assume a=t=8cm

   Ø * Mn = Ø * C or T * (d – ½*t)

   C = (0.85 * Fc′ * t * bE)

   Ø * Mn = Ø * C or T * (d – ½*t)

   $= 0.9 * 0.85 * 24 * 80 * 520 * (283 - \frac{80}{2}) *10^{-6}$

   = **185.6 kN.m** > $Mu^{+} = 5.90$ kN.m

   ∴ **a < t → Compression zone is in the flange**

2. **Design as Rectangular Section with b=be**

   $\rightarrow m = \frac{Fy}{0.85*Fc'} = \frac{420}{0.85*24} = 20.59$

   $\rightarrow kn = \frac{Mu/\emptyset}{b*d^2} = \frac{5.9*10^6/0.9}{520*283^2} = 0.158$ MPa

   $\rightarrow \rho = \frac{1}{m} * (1- \sqrt{1 - \frac{2*KN*m}{Fy}}) = \frac{1}{20.59} * (1- \sqrt{1 - \frac{2*0.158*20.59}{420}}) = 0.00038$

   $\rightarrow Asreq = \rho * b * d = 0.00038 * 520 * 283 = 55.92\ mm^2$

   ∴ **Select 2Ø12 with As = 226 mm^2>Asmin = 113.2mm2**

3. **Check Strain :**

   C=T

   0.85*Fc'*a*b=As *Fy

   0.85 * 24 * a * 520 = 226* 420

   a=8.95 mm ⇒ X = a / β = 8.95 / 0.85 = 10.53 mm

   $\varepsilon_s = \frac{0.003d}{x} - 0.003 = \frac{0.003*283}{10.53} - 0.003 = 0.077 > 0.005 \Rightarrow \emptyset = 0.9$ … (OK)

⇒ Design of span 3 - Max Mu^+ = 17.80 kN.m

1. **Check if (a ≤ t) or (a > t)**

Assume a=t=8cm

Ø * Mn = Ø * C or T * (d – ½*t)

C = (0.85 * Fc′ * t * bE)

Ø * Mn = Ø * C or T * (d – ½*t)

$= 0.9 * 0.85 * 24 * 80 * 520 * (283 - \frac{80}{2}) * 10^{-6}$

= **185.6 kN.m** > Mu^+ = 17.80 kN.m

∴ **a < t → Compression zone is in the flange**

2. **Design as Rectangular Section with b=be**

$\rightarrow m = \frac{Fy}{0.85 * Fc'} = \frac{420}{0.85 * 24} = 20.59$

$\rightarrow kn = \frac{Mu/Ø}{b * d^2} = \frac{17.8 * 10^6/0.9}{520 * 283^2} = 0.475$ MPa

$\rightarrow \rho = \frac{1}{m} * (1 - \sqrt{1 - \frac{2 * KN * m}{Fy}}) = \frac{1}{20.59} * (1 - \sqrt{1 - \frac{2 * 0.475 * 20.59}{420}}) = 0.001144$

$\rightarrow$ Asreq $= \rho * b * d = 0.001144 * 520 * 283 = 168.35$ mm^2

∴ **Select 2Ø12 with As = 226 mm^2>Asreq = 168.35mm2**

3. Check As min :

$$As\ (min) = 0.25 * \frac{\sqrt{Fc'}}{Fy} * bw * d = 0.25 * \frac{\sqrt{24}}{420} * 120 * 283 = 99.03\ mm^2$$

Or

$$As\ (min) = \frac{1.4}{Fy} * bw * d = \frac{1.4}{420} * 120 * 283 = \mathbf{113.2\ mm^2}$$ « **Controlled**

∴ Use 2Ø12 with As = 226 mm² > Asmin = 113.2 mm²

4. Check Strain :

C=T

0.85*Fc'*a*b=As *Fy

0.85 * 24 * a * 520 = 226* 420

a=8.95 mm ⇒ X = a / β = 8.95 / 0.85 = 10.53 mm

ε = (0.003d)/10.53 – 0.003 = (0.003∗283)/10.53 – 0.003 = 0.077 > 0.005 ⇒ Ø = 0.9 … (OK)

4.3.2.5.2. Design of Negative Moment – Top Reinforcement (at support C)

Max Mu^- = 11.80 kN.m
(Compression zone in web ⇒ design as rectangular RC section)

→ $kn = \frac{11.8*10^6/0.9}{120*283^2} = 1.36$ MPa

→ $\rho = \frac{1}{20.59} * (1-\sqrt{1-\frac{2*1.36*20.59}{420}}) = 0.00335$

→ Asreq $= \rho * b * d = 0.00335 * 120 * 283 = 113.77$ mm^2

∴ **Select 2Ø12 with As = 226 mm2 > As req = 113.77 mm^2**

→ **Check Strain :**
C=T
0.85 * 24 * a * 120 = 226* 420
a=38.77 mm ⇒ X = a / β = 38.77 / 0.85 = 45.62 mm
$\varepsilon_s = \frac{0.003*283}{45.62} - 0.003 = 0.0156 > 0.005$ ∴ Ø = 0.9 (Ok)

4.3.2.6. Check Deflection

The value of Deflection should not exceed Δlimit, Which according to ACI Code = $\frac{L}{240}$. The following Table shows values of Δlimit compared with deflection calculated by Atir software .

Table(4- 4):Deflection Check for rib (R11)

Span No.	Span Length (mm)	Δlimit (mm)	ΔCalculated (mm)	Check
Span 1	2580	2580/240 =10.75	2580/8469 =0.305	ΔCalculated < Δlimit (OK)
Span 2	3330	3330/240 =13.9	3330/8798 =0.378	
Span3	4880	4880/240=20.3	4880/600=8.13	

4.4. Design of Beam B40

Beam (B30) is selected to be designed , the following figure shows its location in Typical floor slab:

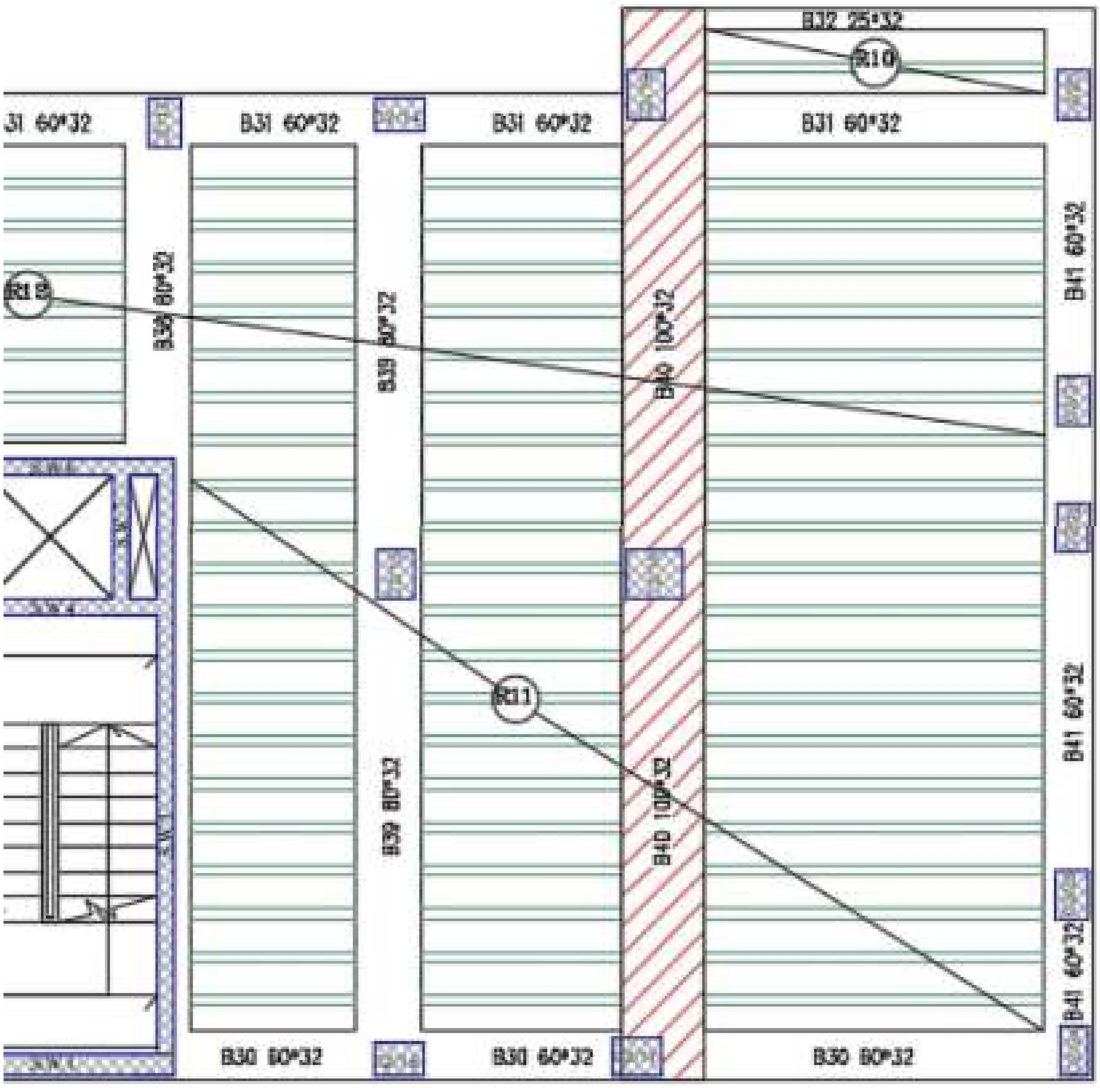


Figure (4- 7):Beam (B40) Location In Typical Floor Slab

4.4.1. Load Calculation for beam

The following figure shows the geometry of beam and loads that act on it :

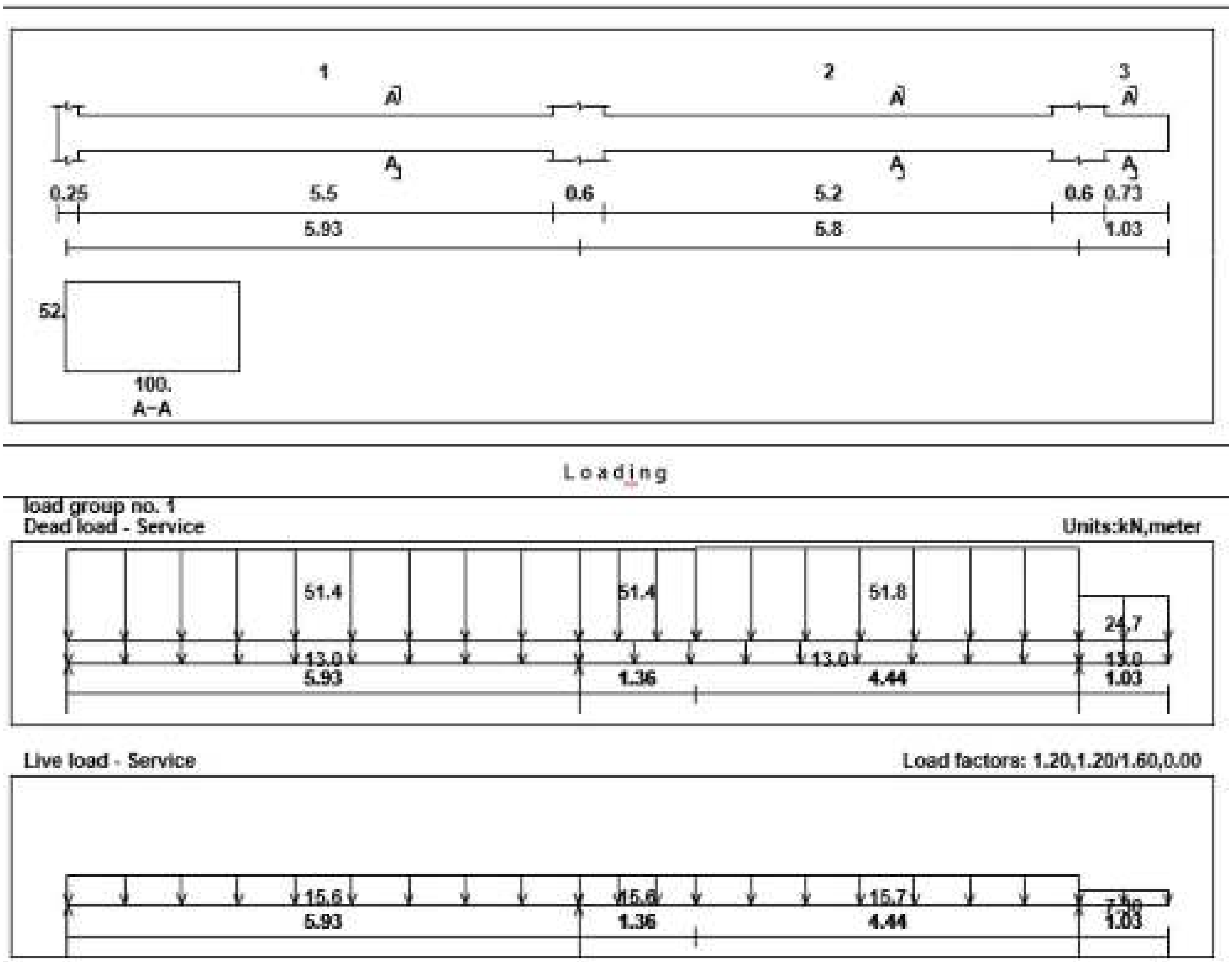


Figure (4- 8) : Beam B30 Geometry and loads

Calculation of Loads that acts on beam B30 :

1. Own weight of the beam :

   Own wt. = 25*0.52*1.00 = **13 kN/m**

2. Reactions of ribs that acting on it .

The following table shows calculation of loads that act on B40 from ribs .

Table(4- 5):Loads on B40 from ribs

	Rib(R10)	Rib(R11)	Rib (R12)
quD(kN/m)	12.86/0.52=24.7	26.74/0.52 = 51.4	26.95/0.52 = 51.8
quL (kN/m)	3.81/0.52 = 7.3	8.16/0.52=15.7	8.19/0.52 = 15.8

4.4.2. Design of beam B40 for Flexure

The following figure shows moment envelope resulted from analysis of beam (B30) using Atir 2018 Software :

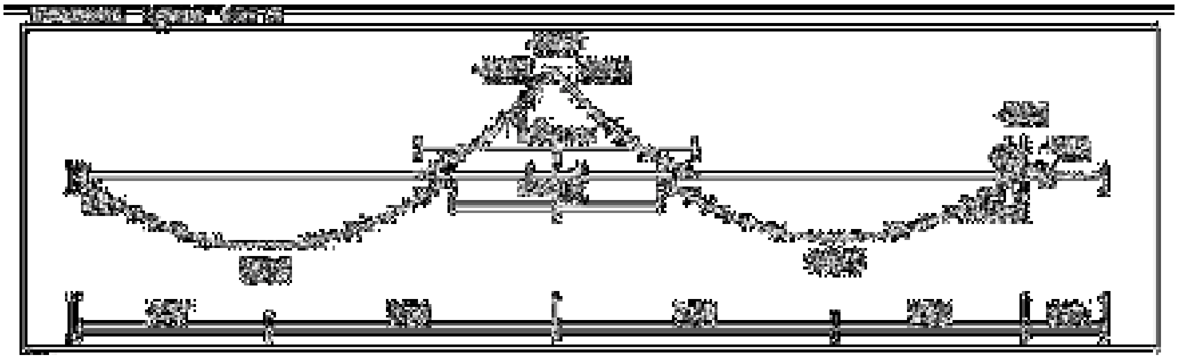


Figure (4- 9):Moment Envelope of beam (B30) – [kN.m]

4.4.3.1 Design of Negative Moment – Top Reinforcement

⇒ **Design of negative moment Mu- = 329.1 kN.m @ support (2)**

1. **Check whether the section will be act as singly or doubly reinforced section :**

Maximum nominal moment strength from strain condition $\varepsilon_s = 0.004$.

$\boldsymbol{d}$ = 520 – 40 -10 - 20/2 = 460 $\boldsymbol{mm}$

→ Mn req = $\frac{Mu}{\emptyset}$, Take Ø = 0.9 for flexure as tension-controlled section.

→ Mn req = $\frac{329.1}{0.9}$ = 366 kN.m

→ m = $\frac{Fy}{0.85*Fc'} = \frac{420}{0.85*24}$ =20.59

→ kn $= \frac{\text{Mn req}}{b*d^2} = \frac{366*10^6}{1000*460^2}$ = 1.73 Mpa

→ $\rho req = \frac{1}{m} * (1- \sqrt{1-\frac{2*KN*m}{Fy}}) = \frac{1}{20.59} * (1- \sqrt{1-\frac{2*1.73*20.59}{420}}) = 0.0043$

But $\rho max = 0.85 * \frac{Fc'}{Fy} * \beta 1 * \frac{3}{7} = 0.85 * \frac{24}{420} * 0.85 * \frac{3}{7} = 0.01769$

∴ $\rho req < \rho max$ **…** Design the section as singly reinforced concrete section.

1. **Design the section as singly reinforced concrete section :**

Assume rectangular & tension control section.

→ Asreq = $0.0043 * 1000 * 460 = 1978$ mm^2

∴ Select 8Ø18 with As = 2032 mm².

2. Check As min :

$$As\ (min) = 0.25 * \frac{\sqrt{Fc'}}{Fy} * bw * d = 0.25 * \frac{\sqrt{24}}{420} * 1000 * 460 = 1341\ mm^2$$

Not less than :

$$As\ (min) = \frac{1.4}{Fy} * bw * d = \frac{1.4}{420} * 1000 * 460 = \mathbf{1533\ mm^2}$$ « **Controlled**

As = 2032 mm² > Asmin = 1533mm² … (OK)

3. Check Strain for Ø and Asmax

C=T

0.85*Fc'*a*b=As *Fy

0.85 * 24 * a *1000 = 2032* 420

a=41.8 mm

X = a / β = 41.8 / 0.85 = 49.2 mm

$$\varepsilon_s = \frac{0.003d}{x} - 0.003 = \frac{0.003*460}{49.2} - 0.003 = 0.025$$

$\therefore\ \varepsilon_s = 0.025 > 0.005$ then Ø = 0.9 … (OK)

also, $\varepsilon_s = 0.025 > 0.004$ then As < Asmax … (OK)

4. Check for spacing

$$S = \frac{1000-2(40)-2(10)-8(18)}{7} = 108\ mm > 25\ mm\ …\ (OK)$$

> db=18mm … (OK)

⇒ Design of negative moment Mu- = 15.20 kN.m @ support (3)

Since Mu =15.20 kN.m < max Mu @ support 3 ,which was designed as singly reinforced section , then also this section must be designed as singly reinforced concrete section.

→ Mn req=15.2/0.9 =16.9 kN.m

→ m = 20.59

→ $kn = \frac{16.9*10^6}{1000*460^2} = 0.08\ MPa$

→ $\rho = \frac{1}{20.59} * (1 - \sqrt{1 - \frac{2*0.08*20.59}{420}}) = 0.00019$

As = ρ*b*d = .00019 * 1000*460=87.4mm^2

Select 7 **Ø** 18 …As = 1778 mm^2> Asmin =1533mm^2

→ **Check Strain for Ø and Asmax**

C=T

0.85 * 24 * a *1000 = 1778* 420

a=36.6 mm , X = 36.6 / 0.85 = 43.1 mm

$\varepsilon_s = \frac{0.003*460}{43.1} - 0.003 = 0.029$

$\therefore\ \varepsilon_s = 0.029 > 0.005\ then\ Ø = 0.9$ … (OK)

also, $\varepsilon_s = 0.029 > 0.004$ then As < Asmax … (OK)

→ **Check for spacing:**

S= $\frac{1000-2(40)-2(10)-7(18)}{6}$ = 129 mm >25 mm … (OK)

> db=18mm … (OK)

4.4.3.2 Design of Positive Moment – Bottom Reinforcement

⇒**Design of span 1 - Max Mu+ = 278.9 kN.m**

Since max Mu in this span < max Mu @ support 2 ,which was designed as singly reinforced section , then also this section must be designed as singly reinforced concrete section.

→ Mn req=278.9 /0.9 =309.9 kN.m

→ m = 20.59

→ kn = $\frac{309.9*10^6}{1000*460^2}$ = 1.46 MPa

→ $\rho req = \frac{1}{20.59} * \left(1- \sqrt{1 - \frac{2*1.46*20.59}{420}}\right) = 0.00361$

→ Asreq = 0.00361 ∗ 1000 ∗ 460 = 1660 mm^2

∴ **Select 7Ø18 with As = 1778 mm^2**

→ As =1778 mm^2> Asmin =1533 mm^2 … (OK)

→ **Check Strain for Ø and Asmax :**

C=T

0.85 * 24 * a *1000 = 1778* 420

a=36.6 mm , X = 36.6 / 0.85 =43.1 mm

$\varepsilon_s = \frac{0.003*460}{43.1} - 0.003 = 0.029$

$\therefore\ \varepsilon_s = 0.029 > 0.005$ *then* Ø = 0.9 … (OK)

also, $\varepsilon_s = 0.029 > 0.004$ then As < Asmax … (OK)

→ **Check for spacing:**

$S = \frac{1000-2(40)-2(10)-7(18)}{6} = 129mm > 25\ mm$ … (OK)

> db=18mm … (OK)

⇒ Design of span 2 - Max Mu+ = 249.3 kN.m

Since max Mu in this span < max Mu @ support 2 ,which was designed as singly reinforced section , then also this section must be designed as singly reinforced concrete section.

→ Mn req=249.3 /0.9 =277 kN.m

→ m = 20.59

→ $kn = \frac{277*10^6}{1000*460^2} = 1.31MPa$

→ $\rho req = \frac{1}{20.59} * (1- \sqrt{1 - \frac{2*1.31*20.59}{420}}) = 0.00323$

→ Asreq = $0.00323 * 1000 * 460 = 1485.8\ mm^2$

select 7Ø18 with As=1778 mm²

→ As =1778 mm² > Asmin =1533 mm² … (OK)

→ **Check Strain for Ø and Asmax :**

C=T

0.85 * 24 * a *1000 = 1778* 420

a=36.6 mm , X = 36.6 / 0.85 =43.1 mm

$\varepsilon_s = \frac{0.003*460}{43.1} - 0.003 = 0.029$

$\therefore\ \varepsilon_s = 0.029 > 0.005$ *then* Ø = 0.9 … (OK)

also, $\varepsilon_s = 0.029 > 0.004$ then As < Asmax … (OK)

→ **Check for spacing:**

$S = \frac{1000-2(40)-2(10)-7(18)}{6} = 129\ mm > 25\ mm$ … (OK)

> db=18mm … (OK)

4.4.4 Design Beam B40 for Shear

The following figure shows shear force envelope resulted from analysis of beam (B40) using Atir 2018 Software :

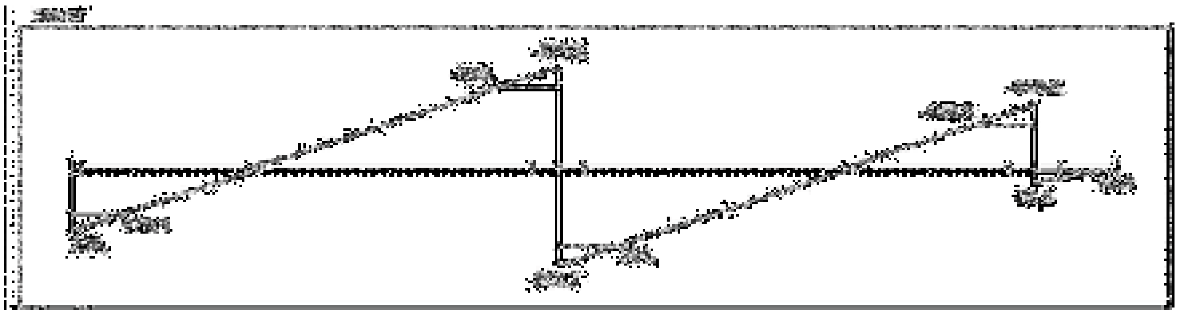


Figure (4- 10):Shear envelope of beam B40 – [kN]

The following are steps of shear force design :

1. **Check for dimensions:**

If Vu max $\leq$ Ø.Vc + Ø $\frac{2}{3}\sqrt{Fc'}$ * bw * d, then section dimensions are adequate. If not, section must be increased.

Overall maximum shear value = 299 kN as shown in figure (4-10).

$$Ø.Vc = Ø * \frac{1}{6} * \sqrt{Fc'} * bw * d$$
$$= 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 460 * 10^{-3}$$
$$= \mathbf{281.69\ kN}$$

$$Ø \frac{2}{3}\sqrt{Fc'} * bw * d = 0.75 * \frac{2}{3}\sqrt{24} * 1000 * 460 * 10^{-3} = \mathbf{1126.7\ kN}$$

$$\mathbf{Ø.Vc + Ø \frac{2}{3}\sqrt{Fc'} * bw * d = 1408.4\ \ kN > Vu\ max = 299} \quad ...(OK)$$

∴ **Section is adequate .**

2. **Category (III) :**

$$Ø.Vc < Vu \leq Ø.Vc + Ø.Vs\ min$$

Ø .Vs min is the maximum between :

$\rightarrow$ Ø .Vs min = $0.75 \times \frac{1}{16} \times \sqrt{fc'} \times bw \times d = 0.75 \times \frac{\sqrt{24}}{16} \times 1000 \times 460 \times 10^{-3}$ = **105.6 kN**

OR

$\rightarrow$ Ø .Vs min=$0.75 \times \frac{1}{3} \times bw \times d = 0.75 \times \frac{1}{3} \times 1000 \times 460 \times 10^{-3}$= **115 kN** « **Cont.**

$$Ø.Vc + Ø.Vs\ min = \mathbf{105.6 + 115 = 220.6\ kN}$$

∴ For all shear values that is ≤ 220.6 kN , minimum shear reinforcement is required .

→ Minimum Shear Reinforcement :

$$Sreq = \frac{0.75*Av*Fyt*d}{Ø.Vs\ min}$$

$$\rightarrow Sreq = \frac{0.75*158*420*460}{115*10^3} = \mathbf{200\ mm}$$

But , **Smax** ≤ d/2 → 460/2 = **230 mm« Cont**

Or , **Smax ≤ 600 mm**

Note :
Assume Ø10 stirrups with 10 legs are used ,
then $Av = 2 * \frac{\pi * 10^2}{4} = 158\ mm^2$

∴ **Select Ø10/15cm ,2legs**

3. Category (IV) :

$$Ø.Vc + Ø.Vs\ min < Vu \leq Ø.Vc + Ø \times \frac{1}{3} \times \sqrt{fc'} \times bw \times d$$

$$\rightarrow Ø \times \frac{1}{3} \times \sqrt{fc'} \times bw \times d = 0.75 \times \frac{1}{3} \times \sqrt{24} \times 1000 \times 460 \times 10^{-3} = \mathbf{563.4\ kN}$$

$$\rightarrow Ø.Vc + Ø \times \frac{1}{3} \times \sqrt{fc'} \times bw \times d = 845.1\ \mathbf{kN} > Vu\ max = 299\ \mathbf{kN}$$

$$Sreq = \frac{Av*Fyt*d}{Vs}, \text{ where } Vs = \frac{Vu - Ø.Vc}{Ø} = \frac{299 - 281.69}{0.75} = 23.08\ kN$$

$$\rightarrow Sreq = \frac{158*420*460}{23.08*10^3} = \mathbf{1322\ mm\ «\ Cont}$$

But , **Smax** ≤ d/2 → 460/2 = **230 mm**

Or **Smax ≤ 600 mm**

∴ **Select Ø10/15cm ,2legs**

4.5. Design of Column (C18)

4.5.1. Calculation of Loads act on Column (C18)

Loads acting on columns are obtained from support reaction when analyzing the supported beams.

Loads acting on column (C18) are as follows:

Dead Load = (Service Dead reaction from B14) + (Service Dead reaction from B27) + (Service Dead reaction from B40 x 7) +(Service Dead reaction from B50) + (Self weight of the column x 10)

= (235.14) + (267.92) + (168.18 x 7) +(252.87) + (0.6 x 0.45 x 3 x 25 x 10) = 3828.03 kN

Live Load =(Service Live reaction from B14) + (Service Live reaction from B27) + (Service Live reaction from B50) +(Service Live reaction from B40 x 7)

=(118.26) + (95.92) + (16.27) + (71.68 x 7) =1139.9 kN

Factored loads (Pu) = 1.4 DL = 1.4 x 3828.03 = 6084.82 kN.

OR Pu = 1.2 DL + 1.6 LL = 1.2 x 3828.03+ 1.6 x 1139.9 = **6417.48 kN** << Cont.

4.5.2. Calculation of Required Dimension of Column (C18)

Total load Pu =7039.4 KN

Pn =6417.48 /(0.65) = 9873.04 KN

Assume $\rho g = 2.0$ %

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

$8508 * 10^{-3} = 0.8 * \mathrm{Ag}[0.85 * 24 + 0.02 * (420 - 0.85 * 24)]$

$\mathrm{Ag} = 0.37\ \mathrm{m}^2$

∴**Select 60*70cm with Ag =4200 cm².**

- **Check Slenderness Effect :**

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

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

Where :

Lu: Actual unsupported (unbraced) length = 3.84 m

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

R: radius of gyration → for rectangular section =√(1/A) *0.3h

System about X

$\rightarrow \lambda = \frac{1 * 3.84}{0.3 * 0.7} = 18.29$

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

$\lambda = 18.29 < 22$ ∴ Short about X .

System about Y

$\rightarrow \lambda = \frac{1 * 3.84}{0.3 * 0.6} = 21.33$

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

$\lambda = 21.33 < 22$ ∴ Short about Y.

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

4.5.3. Calculation of Required Reinforcement Ratio

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

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

Where , Pu =6417.48 KN

$6417.48 * 10^3 = 0.65 * 0.8 * 600 * 700\{0.85 * 24 + \rho g(420 - 0.85 * 24)\}$

$\Rightarrow \rho_g = 0.0225 > \rho_{min} = 0.01 \ \& < \rho_{max} = 0.08$

As req = 0.0225 *600 *700 =9450 mm^2

Use Φ 25 >> $\# of\ bar = \frac{9450}{491} = 19.25$

∴ **Use 20 Ø 25 with As = 9820 mm^2 > As $_{req}$ =9450 mm^2**

- **Check spacing between the bars :**

S x= $\frac{600-2*40-2*10-6*25}{5} = 70\ mm$ Sy $= \frac{700-2*40-2*10-6*25}{5} = 90\ mm$

S = 70 mm or 90 mm $\geq \frac{4}{3}$M.A.S

$\geq 40mm$

$\geq$ 1.5db = 37.5 mm

4.5.4. Determination of Stirrups Spacing

<u>According to ACI :</u>

$S \leq$ 16 db (longitudonal bar diameter)

S $\leq$ 48dt (tie bar diameter).

S $\leq$ Least dimension.

Spacing $\leq 16 \times d_b$(Longitudinal. bar. diameter) $= 16 \times 2.5 = 40$cm.

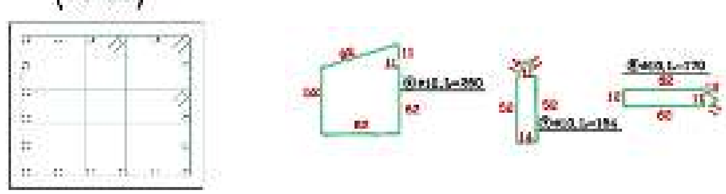
Spacing $\leq 48 \times d_t$(tie. bar. diameter) $= 48 \times 1.0 = 48$cm.

Spacing $\leq$ Least. dimension $= 60$ cm

∴ **Select Ø 10/20cm**

Column (C18) Section is shown in figure(4-11) where bars arrangmenet and stirrups detailing appear :

(70*60)



20Ø25

Figure (4- 11): C18 Reinforcement Details

4.6. Design of Shear Wall

Analysis and design were done using ETABS program in which the seismic loads were taken into account. The following is a sample calculation for one of the walls, S.W6.[For detailed information see appendix C]

The following data that used in design :

Vu = 824.87 KN , Mu =1798.38KN.m

- Shear Wall thickness = h = 20 cm
- Shear Wall length Lw = 3.0 m
- Building height Hw=30.42 m
- Critical section shear : Lw<hw → d =0.8*3 =2.40 m

4.6.1. Design of Horizontal Reinforcement

Calculation of Shear Strength Provided by concrete Vc:

- Shear Strength of Concrete is the smallest of :

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

$= \frac{1}{6}\sqrt{24} \times 200 \times 2400 = 392\ \mathbf{kN} \ll \text{Controlled}$

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

$= \frac{\sqrt{24} \times 200 \times 2400}{4} + 0 = 587.8\ kN$

3- $Vc = \left[\frac{\sqrt{fc'}}{2} + \frac{Lw\left(\sqrt{fc'} + \frac{2Nu}{Lw.h}\right)}{\frac{Mu1}{Vu} - \frac{Lw}{2}}\right] \times \frac{h \times d}{10}$

Vc=1154KN

Where :

- Mu1=1798.38 kN.m

$\therefore Vc = 392\ \mathrm{kN} \rightarrow \emptyset \mathrm{Vc} < \mathrm{Vumax} = 824.87\ \mathrm{kN}$ →Horizontal Reinforcement is Required.

$\rightarrow \mathrm{Vs} = \frac{\mathrm{Vu}}{\emptyset} - \mathrm{Vc} = \frac{824.87}{0.75} - 392 = 707.8\ \mathrm{kN}$

$\rightarrow \frac{\mathrm{Avh}}{\mathrm{s}} = \frac{\mathrm{Vs}}{\mathrm{fy} * \mathrm{d}} = \frac{707.8 * 10^3}{420 * 2400} = 0.702 \ll \text{Controlled}$

but $\left(\frac{\mathrm{Av}}{\mathrm{s}}\right)\mathrm{min} = 0.0025 * \mathrm{h} = 0.0025 * 200 = \mathbf{0.5}.$

→ Avh : For 2 layers of Horizontal Reinforcement

Select Ø10 :

Avh = 2 *79 = 158 mm^2

$\frac{Avh}{s}$ =.702

→$Sreq$ =225mm

Smax= Lw/5 = 3000/5 = 600 mm

= 3h = 3*200 =600mm

= $45cm$ ≪ Controlled.

Sreq = 225mm < Smax=450mm

∴Select Ø10 @ 200 mm at each side.

4.6.2. Design of Vertical Reinforcement

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

→ $\frac{hw}{lw} = \frac{30.42}{3.0} = 10.14 > 2.50$

→ $\frac{Avv}{Sver}$ = [0.0025 +0.5 (0)($\frac{2*79}{200*200}$ − 0.0025)] ∗ 200

∴ $\frac{Avv}{Sver} = 0.5$

Smax = Lw/3 = 3000/3 = 1000 mm
= 3h = 3*200 =600mm
= 450 mm ≪ Controlled

Select Ø12 :
Avv = 2 *113 = 226 mm^2
$\frac{Avv}{s} = 0.5 \rightarrow Sreq = \frac{226}{0.5} = 452$ mm

∴Select Ø12 @ 200 mm at each side

Design of Bending Moment

Moment diagram were obtained from ETABS [See Appendix – Page]

→ Max Mu =1798.38 kN.m

→ Part of Moment that resisted through Avv :

Muv =0.9 [0.5*Asv*fy*Lw (1-$\frac{Z}{2\,Lw}$)]

Where :

- Asv=2*113 *$\frac{3000}{200}$ $=$ 3390 mm^2
- $\frac{Z}{Lw} = \frac{1}{2+\frac{0.85*\beta1*fc'*Lw*h}{Asv*fy}} = \frac{1}{2+\frac{0.85*0.85*24*3000*200}{3390*420}} = 0.1074$

∴ Muv $=$ 0.9 [0.5*3390*420*3000 (1-$\frac{0.1074}{2}$)] $=$ 1819 kN.m

Muv = 1819 kN.m > Mu =1798.38 kN.m

<u>So, Boundary Element is not required. #</u>

4.7. Design of Basement Wall

4.7.1. System and Loads

The wall spans vertically and it is considered to be pinned at both ends as shown in figure (4-12) which also illustrate loads that act on the wall.

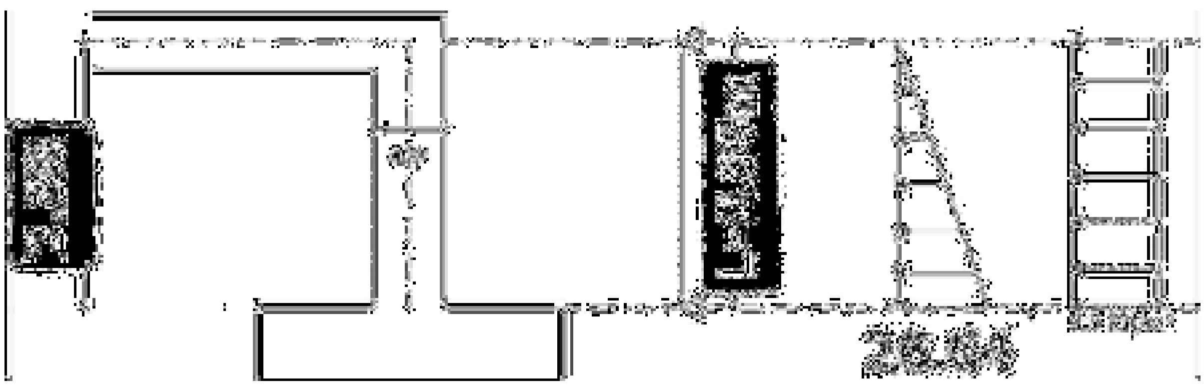


Figure (4- 12):Basement Wall system and loads

The different lateral pressures on a 1m length of the wall are calculated as follows:
$k_o = 1 - \sin 30 = 0.5$
Due to soil pressure at rest : $qu1 = k_o.\gamma.h = 0.5*18*2.96=26.64\ kN/m^2$
Due to surcharge : $qu2 = 5*0.5 = 2.5\ kN/m^2$

The following are shear and moment diagrams that obtained from Atir Software.

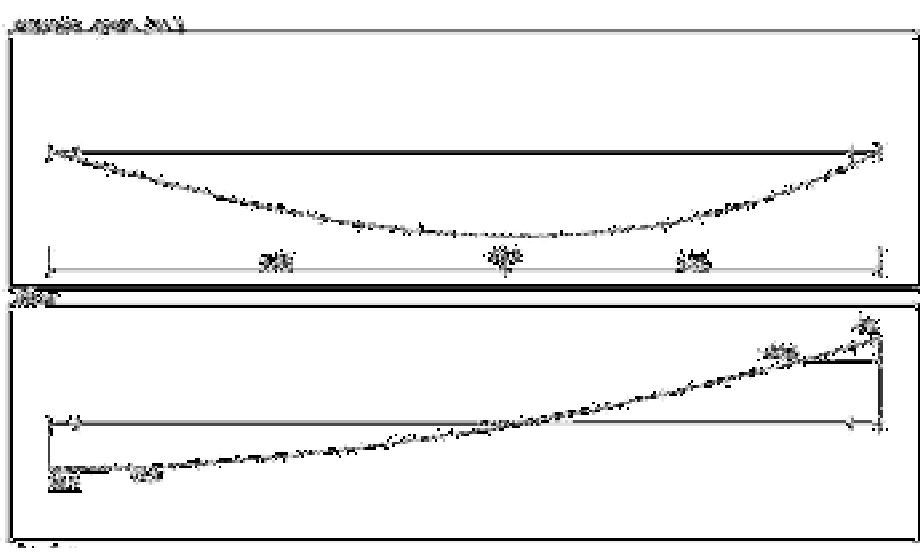


Figure (4- 13): Moment and Shear Envelope of Basement wall

4.7.2. Design of Shear Force

Max value shear force is obtained from figure(4-13) , Vu= 26.3 kN

d=30-2-2 = 26 cm

$Ø * Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 260 = 159$ kN > Vu

∴ **h=30cm is correct.**

4.7.3. Design of Wall Reinforcement

1. Design of Vertical Reinforcement at Tension Side :

Max value Moment is obtained from figure(4-13) , Mu = 21.2 kN.m

→ $m = \frac{420}{0.85*24} = 20.6$

→ Mn = 21.2 /0.9= 23.5 kN.m

→ $kn = \frac{Mn}{b*d^2} = \frac{23.5*10^6}{1000*260^2}$ = 0.347 MPa

→ $\rho = \frac{1}{20.6} * (1- \sqrt{1 - \frac{2*0.347*20.6}{420}}) = 0.00083$

→ Asreq = $\rho * b * d = 0.00083 * 1000 * 260 = 215.8$ mm^2 /1m

→ As (min) = 0.0012*b*h = 0.0012*1000* 300 = 360 mm^2 /1m > Asreq

∴ Select Ø12/20cm with As = 452 mm^2 /m > As min

2. Design of Vertical Reinforcement Compression Side:

→ As = As (min) = 360 mm^2

∴ Select **Ø10/20cm with As = 632 mm^2 /m**

3. Design of Horizontal Reinforcement:

→ As = As (min) = 0.001 *1000*300 = 300 mm^2 /m for one layer

∴ Select **Ø10/25cm**

4.8. Design of Basement Footing

Loads that act on Wall footing is obtained from ETABS where :

qD=48.42 kN/m & qL=9.53 kN.m

Total Service Loads : qtot =48.42+9.53 = 57.95 kN/m

Total Factored Loads : qu =1.4 *65.56 = 67.79 kN/m

4.8.1. Check if footing width is correct

Assume h = 30cm.

$\therefore\ \mathbf{57.95}/_{1.0\,*\,1.0} = 57.95 < \sigma_{\text{b(allow . net)}} = 400$ kN/m²

$\therefore$ **a=1.0m is correct#**

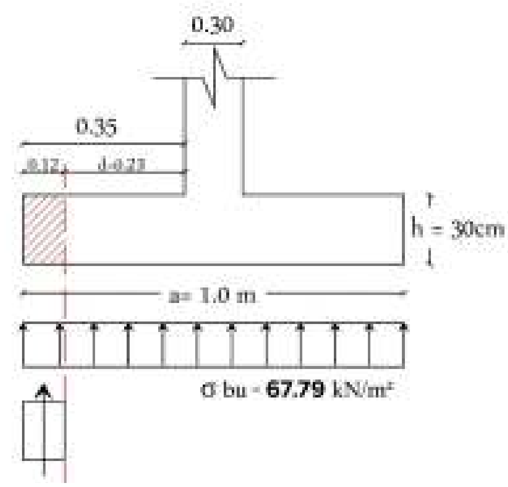


Figure (4- 1): Critical Section of Shear force

4.8.2. Design of One way shear

→ Assume h =30cm

→ d=300-50-20 = 230 mm

→ Vu=67.79*0.12*1m = 8.13 KN

→ $Ø * Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 230 = 140.8$ kN > Vu

$\therefore$ **h =30 cm (SAFE) .**

4.8.3. Design of Bending Moment

Main Steel:

Mu =67.79*0.35*1*(0.35/2) = 4.15 kN.m

→ Mn = 4.15 /0.9= 4.61 kN.m

→ $kn = \frac{Mn}{b*d^2} = \frac{4.61*10^6}{1000*230^2} = 0.087$ MPa

→ $\rho = \frac{1}{20.6} * (1 - \sqrt{1 - \frac{2*0.087*20.6}{420}}) = 0.000208$

→ $Asreq = 0.000208 * 1000 * 230 = 47.84$ mm²/m

→ As (min) = 0.0018*1000 * 300 = 540 mm²/m

→ $\therefore$ **Select Ø12/20cm with As = 565 mm² > Asmin**

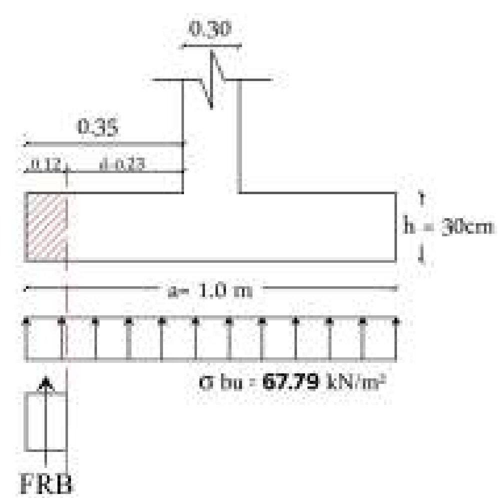


Figure (4- 2):Critical Section of Bending Moment

➢ **Secondary Steel:**

→ As (min) = 0.0018*b*h = 0.0018*1000 * 300 = 540 mm^2

∴ Select **Ø12/20cm 5Ø12/1m with As = 565 mm^2 > Asmin**

The Following figure shows details of a section taken in a basement wall and its footing.

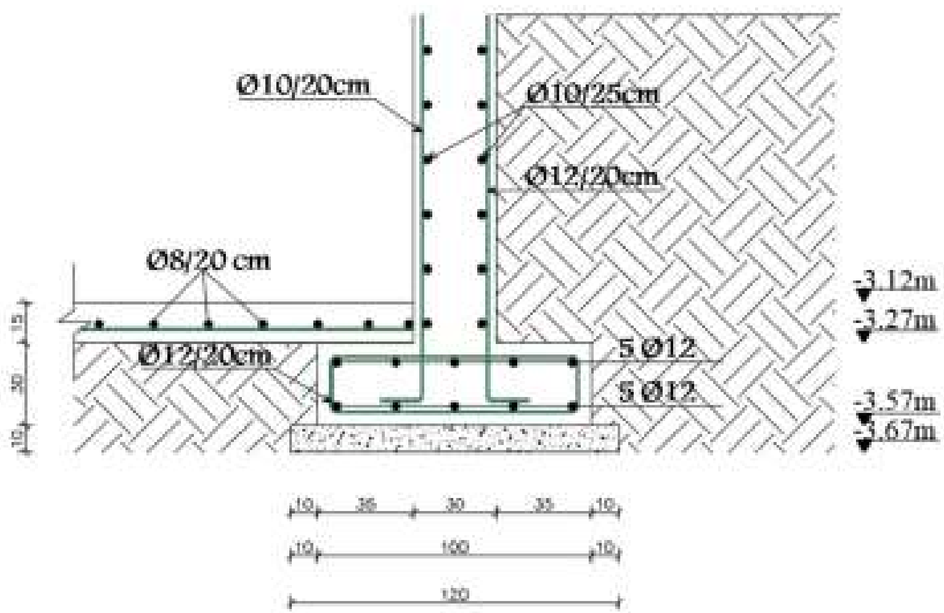


Figure (4- 3): Basement wall Reinforcement Details

4.9. Design of Combined Footing (C.F5)

PD1 =2687.32 kN PL1 = 548.02 kN

PD2 =3828.03 kN PL2 = 1139.9 kN

1. Design of Bearing Pressure:

Neglect the self weight of footing

Assume concentrically loaded footing :

$$\sigma b = \frac{Ptot}{Areq} = \frac{(2687.32+548.02+3828.03+1139.9)}{6.2*b} \leq 400KN/m^2$$

breq=3.31m

select b = 3.5m

Design as eccentrically loaded footing:

Ptot = 8203.27KN .

MRo = 4967.93*1.56 – 3235.34*1.56 = 2703 KN.m

e=2703/8203.27 = 0.329m.

a/6 = 6.2/6 = 1.03

e<a/6

$$\sigma bmax = \frac{8203.27}{6.2*3.5} \left(1 + \frac{6*0.329}{6.2}\right)$$

$\sigma bmax = 498KN/m^2 < 1.3*400 = 520\ KN/m^2$ ……………….SAFE

2-Design of one way shear :

Pu1 = 1.2*2687.32 + 1.6*548.02 = 4101.61 KN.

Pu2 = 1.2*3828.03 + 1.6*1139.9 = 6417.47 KN.

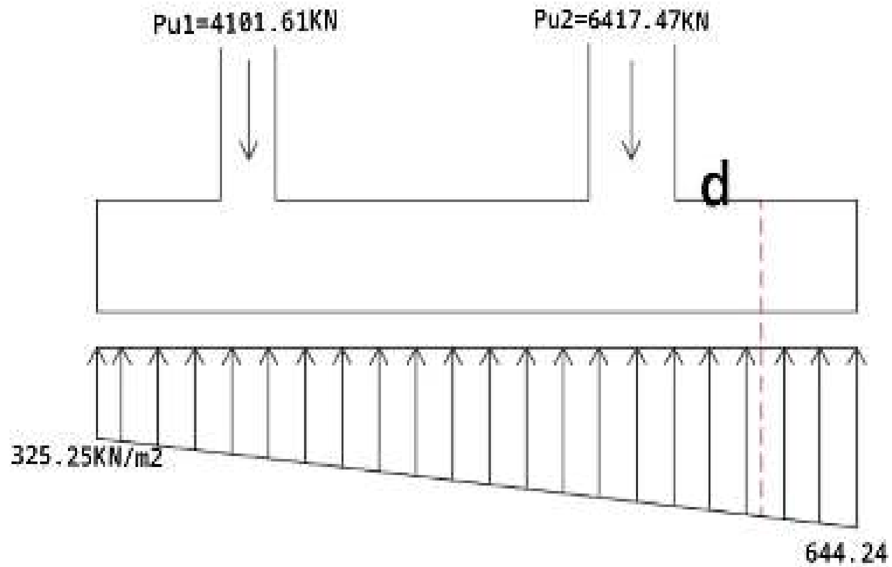
∑Pu =4101.61+6417.47 = 10519 KN .

MRo = (1.56*6417.47 – 1.56*4101.61) = 3612KN.m

e= MRo/∑Pu = 3612/10519 =0.34 < a/6.

$$\sigma bmax = \frac{10519}{6.2*3.5} \left(1 + \frac{6*0.34}{6.2}\right) = 644.24KN/m^2$$

$$\sigma bmin = \frac{10519}{6.2*3.5} \left(1 - \frac{6*0.34}{6.2}\right) = 325.25KN/m^2$$



Vu *at the critical section at a distance d from the face of column* :
Assume H=80cm
d= 800-75-18 = 707mm

$\sigma bu1 = \frac{644.24-325.25}{6.2} * 5.42 + 325.25 = 604.11 KN/m^2$
$Vu = \frac{604.11+644.24}{2} * 0.778 * 3.5 = 1699KN$.
$\Phi Vc = 0.75/6 * \sqrt{24} * 3500 * 707 = 1720KN > 1699KN$ ……..OK

3-Design of two way shear " punching shear " :

bo=4*1407 =5628mm

$\sigma bu2 = \frac{644.24-325.25}{6.2} * 5.0685 + 325.25$
$= 586.02\ KN/m^2$

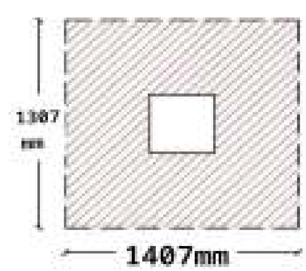
$\sigma bu3 = \frac{644.24-325.25}{6.2} * 3.6615 + 325.25$
$= 513.63\ KN/m^2$

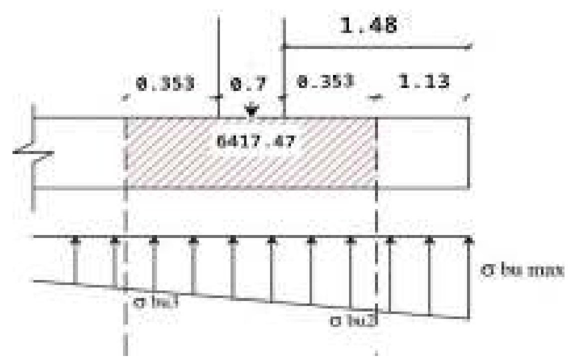
Vu = Pu – FRB
$= 6417.47 - (\frac{586.02+513.63}{2}) * 1.407^2$
= 4729 KN.

ΦVc:

1- $0.75(2 + \frac{4}{1})\frac{\sqrt{24}}{12} * 5628 * 707 = 7310KN.$
2- $0.75\ (\frac{40*707}{5628} + 2) * \frac{\sqrt{24}}{12} * 5628 * 707 = 8559KN.$
3- $0.75 * 4 * \frac{\sqrt{24}}{12} * 5628 * 707 = 4873KN$ … control

$\Phi Vc > Vu$ …. h= 80cm ok





3. Design of Bending Moment:

3.1 Bottom reinforcement (In x- direction):

Max Mu + :

$\sigma bu3 = \frac{644.24-325.25}{6.2} * 4.715 + 325.25 = 567.8 KN/m^2$

$Mu = 567.8 * \frac{1.485*1.485}{2} + (644.24-567.8)*1.485*0.5*\frac{2}{3}* 1.485$

$Mu = 2729 KN.m$

- Design (b=3500mm , d=707mm) :

- m=19.6.
- Mn = 2729 / 0.9 = 3032KN.m
- Kn= 1.73MPa.
- $\rho = 0.00043$ → Asreq= 0.00043*3500*707 = 1064mm²
  Asmin = 0.0018*3500*1000 = 6300mm²
  Select 25 Ø 18 with As =6350mm2 >Asmin.

TOP reinforcement (In x- direction):

Max Mu : at Vu=0.0

$-325.25 * x * 3.5 - \frac{644.24-325.25}{6.2} * x^2 * 3.5 * 0.5 + 4101.61$

X=2.9

$\sigma bu4 = \frac{644.24-325.25}{6.2} * 2.9 + 325.25 = 474.45 KN/m^2$

$Mu = 325.25 * \frac{2.9*2.9}{2} * 3.5 + ((644.24-567.8)/2)*2.9*2.9*\frac{1}{3}* 3.5 - 4101.61(2.9-1.9)$

$Mu = 1453.61 KN.m$

Design (b=3500mm , d=707mm)

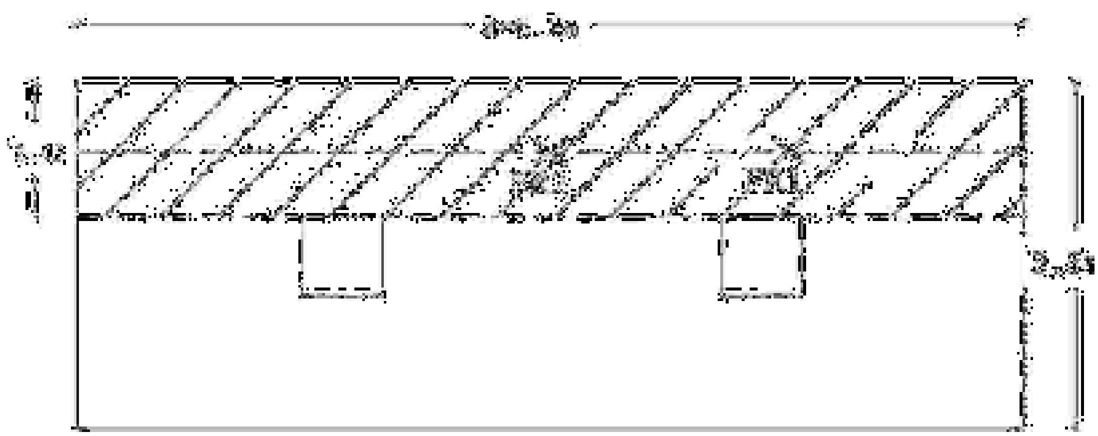
- m=19.6.
- Mn = 1453.61 / 0.9 = 1615KN.m
- Kn= 0.92MPa.
- $\rho = 0.00224$ → Asreq= 0.00224*3500*707 = 5542mm²
  Asmin = 0.0018*3500*1000 = 6300mm²
  Select 25 Ø 18 with As =6350mm2 >Asmin.

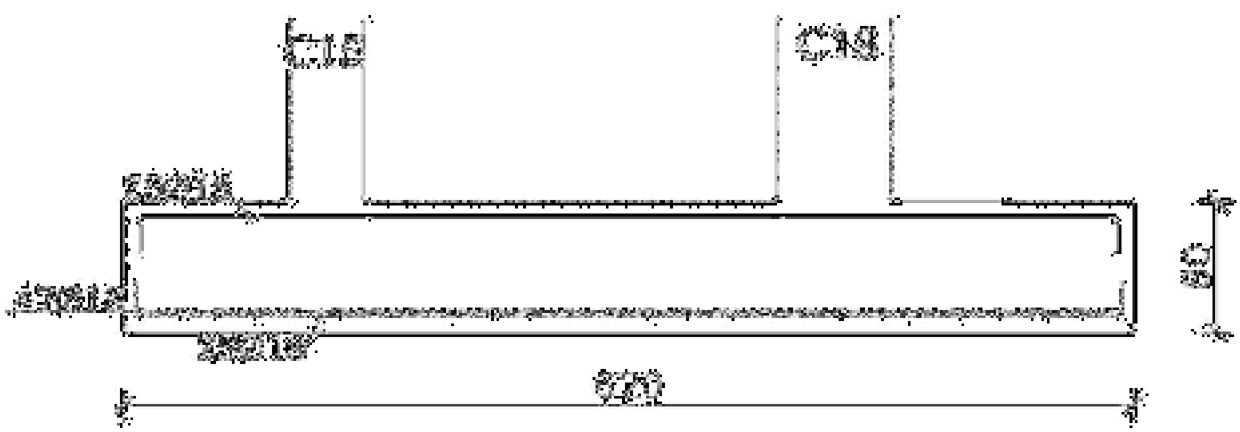
Bottom reinforcement in Y- direction:

$Mu^{+} = 325.25 * 6.2 * 1.45^2 * 0.5 + (644.24\text{-}325.25) * 6.\ 2*0.5 * 1.45^2 *0.5$
$Mu^{+} = 3159.4$ KN.m

Design (b=6200mm , d=707mm)

- m=19.6.
- Mn = 3159.40 / 0.9 = 3510.40KN.m
- Kn= 1.10MPa.
- $\rho = 0.0027$ → Asreq= 0.0027*6500*707 = 11835mm^2
  Asmin = 0.0018*6200*1000 = 11160mm^2
  Select 47Ø 18 with As =11938mm^2 >Asmin=11835mm^2.





4.10. Design of Stairs

The following figure shows a top view of the stairs :

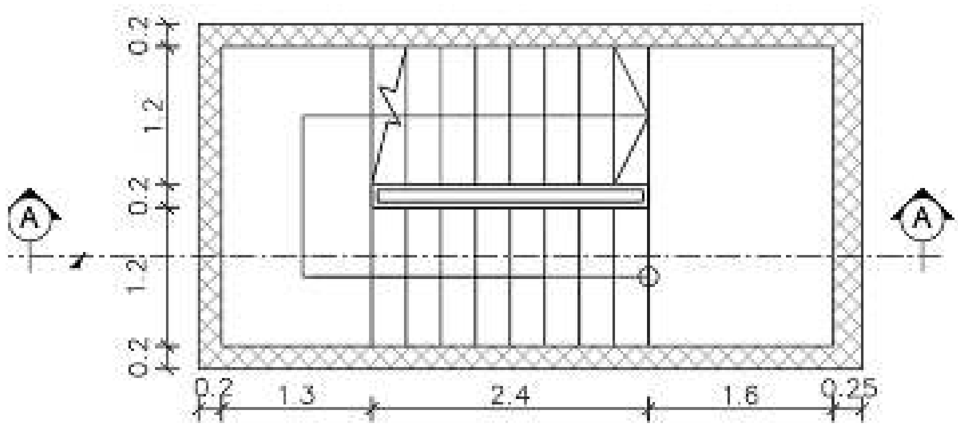


Figure (4- 21): Stairs Top View

4.10.1. Design of flight

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

1. **Determination of flight thickness :**

   Limitation of deflection: h ≥ minimum h

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

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

   Angle (α): tan(α) = 17.33/30 → α = 30°

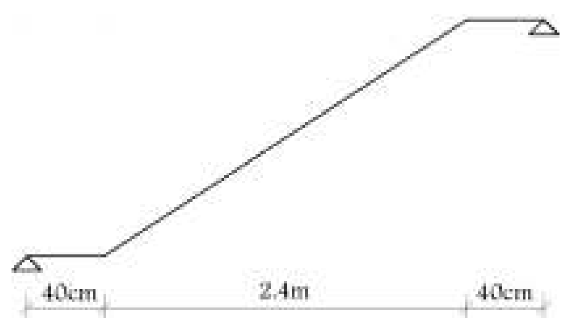


Figure (4- 22): Structural system of flight

2. **Loads calculation :**

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

Table(4- 6): Calculation of Dead Loads that act on Flight

Flight Dead Loads
Flight = (0.15 * 25* 1)/ cos(30) =4.30 kN/m
Plaster = (0.03 * 22* 1)/ cos(30) =0.80 kN/m
Hor.Mortar =0.03*22*1 =0.70 kN/m
Ver.Mortar = 0.03*22*($\frac{0.173}{0.3}$) = 0.40 kN/m
Hor.Tiles = 0.04*23*($\frac{33}{30}$) = 1 kN/m
Ver.Tiles = 0.03*23*($\frac{0.173}{0.3}$) = 0.40 kN/m
Triangle =0.5*0.173*25 =2.20 kN/m
Sum=9.80 kN/m

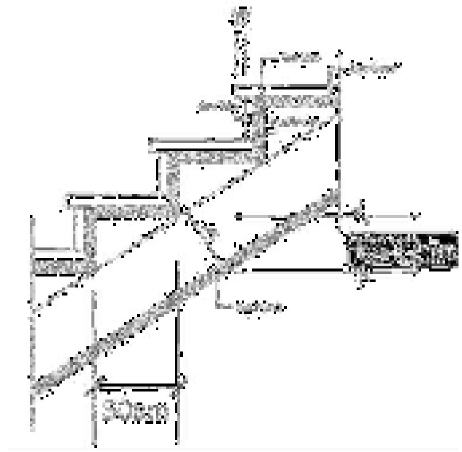


Figure (4- 23): Section of The Flight

Factored Loads :

qu= 1.2 *9.80 + 1.6*2 = 15 kN/m

3. **Analysis** : The following figures show shear and moment Diagrams resulted from analysis of the flight :

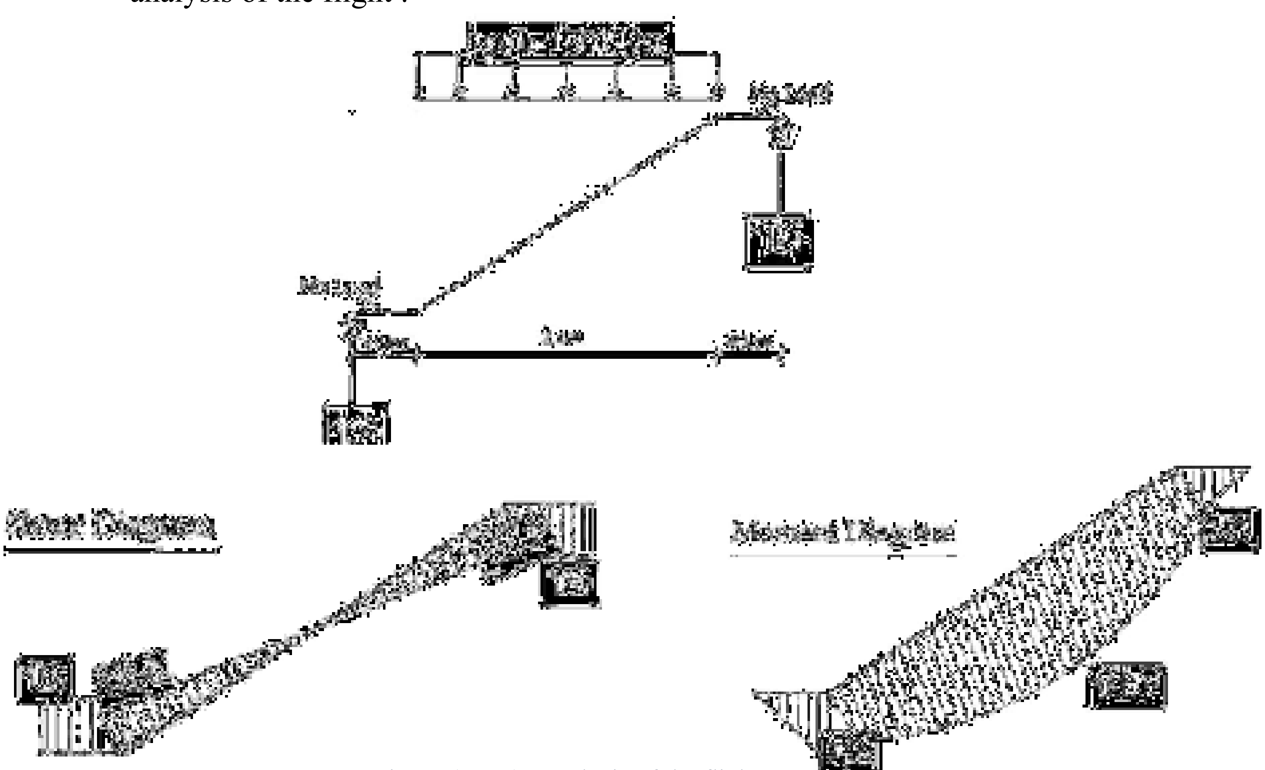


Figure (4- 24) :Analysis of the flight

4. Design :

- **Design of Shear Force :**

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

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

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

= 75.9 kN > Vu max = 15.32 kN

∴ No Shear Reinforcement is Required#

- **Design of Bending Moment :**

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

$\rightarrow kn = \frac{Mu/\emptyset}{b * d^2} = \frac{19.70 * 10^6 / 0.9}{1000 * 124^2} = 1.42$ MPa

$\rightarrow \rho = \frac{1}{m} * (1 - \sqrt{1 - \frac{2 * KN * m}{Fy}}) = \frac{1}{19.6} * (1 - \sqrt{1 - \frac{2 * 1.42 * 20.6}{400}}) = 0.0035$

$\rightarrow Asreq = \rho * b * d = 0.0035 * 1000 * 124 = 434 \text{ mm}^2$

$\rightarrow$ As min = 0.0018 *1000*17.33 = 311.9 mm^2

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

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

→ Check Spacing :

20cm > **S min** = 2.5 + 1.0 = **3.5 cm** or 2*(1.2)= **2.4 cm … ok**

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

→ Check Strain:

C = T

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

0.85*24*a*1000 = 565 *420

a = 11.6 mm → X = a/β = 11.6/0.85 = 13.70 mm

$\varepsilon_s = \frac{0.003 * d}{x} - 0.003 = \frac{0.003 * 124}{13.70} - 0.003$

ε_s = 0.024 > 0.005 …. Ø = 0.9 (OK)

4.10.2. Design of Landing

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

- **Determination of Landing thickness :**

Limitation of deflection:

Figure (4- 25):Structural system of landing

h ≥ minimum h

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

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

- **Loads calculation :**

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

Table(4- 7):Calculation of Dead Loads that act on Landing

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

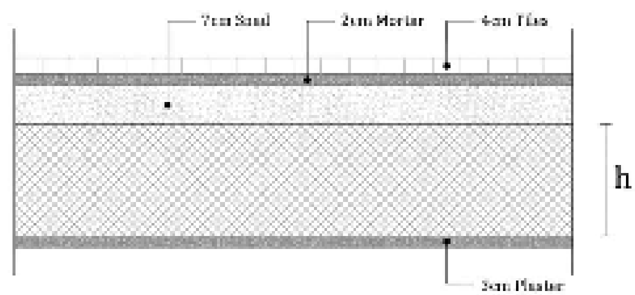


Figure (4- 26):Section of The Landing

<u>Factored Loads :</u>
qu = 1.2*6.35+1.6*2 = 10.82 kN/m

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

qu = 10.82 + Support reaction of flight = 10.82 + 18 = 28.82 **kN/m**

→ **Analysis :**

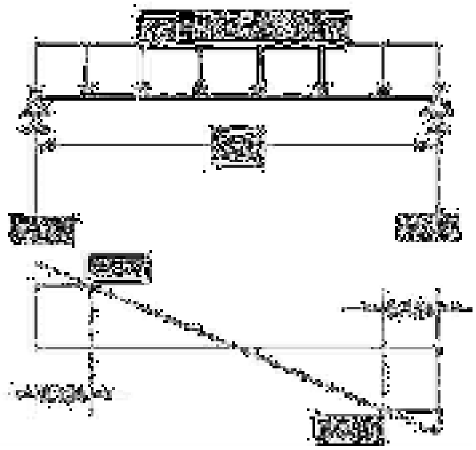


Figure (4- 27):Analysis of Landing

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

Vumax= 45.3 – (28.82*0.124) = 36.77 kN

Mumax = (28.82*2.82)/8 =28.24 KN/m^2

→ **Shear Force Design :**

d=124mm & Vu max=36.77 kN

$Ø×Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 124 = 75.9kN > Vu\ max = 36.77\ kN$

∴No Shear Reinforcement is Required#

→ **Bending Moment Design : (Mu max = 28.24 kN.m)**

- m = 20.6
- kn = $\frac{28.2 * 10^6/0.9}{1000*124^2}$ = 2.04 MPa
- $\rho = \frac{1}{20.6} * (1- \sqrt{1 - \frac{2*2.04*20.6}{420}}) = 0.0051$
- Asreq = $0.0051 * 1000 * 124 = 636.1\ mm^2$
- As min = 0.0018 *1000*150 = 270 mm^2

∴ Select **Ø12 /15cm with As = $\frac{\pi * 12^2}{4} * \frac{100}{15}$ = 753.3 mm^2** > As req …. For Main Reinforcement

- Check Spacing :
  15cm > **S min** = 2.5 + 1.2 = **3.7 cm** or 2*(1.2)= **2.4 cm … ok**
  15cm < **S max** = 3 * 15 = **45 cm … ok**

- Check Strain:

$$C = T$$

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

$$0.85*24*a*1000 = 753.3*420$$

$$a = 15.50 \text{ mm} \rightarrow X = a/\beta = 215.50/0.85 = 18.20 \text{ mm}$$

$$\varepsilon_s = \frac{0.003*124}{18.20} - 0.003$$

$\therefore$ **ε_s = 0.0172 > 0.005 …. Ø = 0.9 (OK)**

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

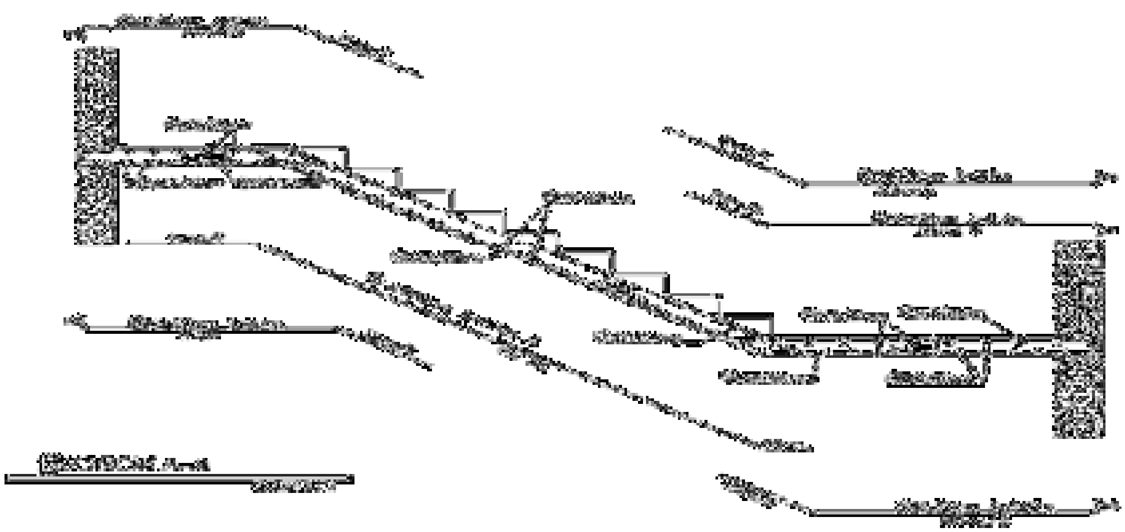


Figure (4- 28):Reinforcement Details of Stairs

CHAPTER 5

RESULTS AND RECOMMENDATIONS

5.1 Introduction

5.2 Results

5.3 Recommendations

5.1 INTRODUCTION

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

The most prominent of these problems was deflection in beams that could have been solved by using drop beams which are not preferred in a residential building. So that another solution had been found, and that was through changing the structural system by changing the bearing direction of ribs and beams. After dealing with that problem a complete design for all structural members were done and the results of the design is presented in a form of drawings in appendix B.

5.2 RESULTS

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

1. The most important step before starting a design is to study the architectural plans carefully to distribute the columns correctly.
2. The theoretical background is important but not enough, experience that reached by practicing the design is more important.it helps the engineer to be able to solve any problem that may appear in a project.
3. Gaining experience in using structural programs cannot be reached without an understanding of basic concepts of the structural design.
4. When choosing the structural system it is better to distribute ribs in the long direction and beams in the short one that will reduce loads that act on beams which leads to reducing of reinforcement which meant reducing costs.

5.3 RECOMMENDATIONS

This project has an important role in expanding the understanding of construction projects. So after completing this project, some recommendations should be mentioned that may help students who will work on such projects after us.

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

Before implementation, the electrical and mechanical plans of the project must be completed to introduce any possible modifications to the structural or architectural plans.It is recommended that a supervising engineer is present during the implementation of the project, and he admitted to the plans and conditions to complete the project in the best way .

References

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

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

[3] كود البناء األردني, كود األحمال والقوى, عمان,األردن: مجلس البناء الوطني األردني, 2006م.

Appendices

APPENDIX (A) :

Architectural Drawings

DRAWINGS ARE ATTACHED TO THE PROJECT

APPENDIX (B) :

Structural Drawings

DRAWINGS ARE ATTACHED TO THE PROJECT

APPENDIX (C) :

ETABS Results and Irrigulation checks

The following is a sample of analysis and design done using ETABS18

ASCE 7-16 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQX according to ASCE 7-16, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [ASCE Table 12.8-2]	$C_t = 0.02\text{ft}$
Coefficient, x [ASCE Table 12.8-2]	$x = 0.75$
Structure Height Above Base, h_n	$h_n = 79.46$ ft
Long-Period Transition Period, T_L [ASCE 11.4.5]	$T_L = 4$ sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	$R = 5$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	$\Omega_0 = 2.5$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	$C_d = 4.5$
Importance Factor, I [ASCE Table 1.5-2]	$I = 1$

Ss and S1 Source = 0.75

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.2]	$S_s = 0.56g$
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.2]	$S_1 = 0.28g$
Site Class [ASCE Table 20.3-1] = B - Rock	
Site Coefficient, F_a [ASCE Table 11.4-1]	$F_a = 0.9$
Site Coefficient, F_v [ASCE Table 11.4-2]	$F_v = 0.8$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.4, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.504g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.4, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.224g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.336g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.149333g$

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2]

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-3]

$$C_{S,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-5]

$$C_{S,min} = \max(0.044 S_{DS} I, 0.01) = 0.014784$$

[ASCE 12.8.1.1, Eq. 12.8-6]

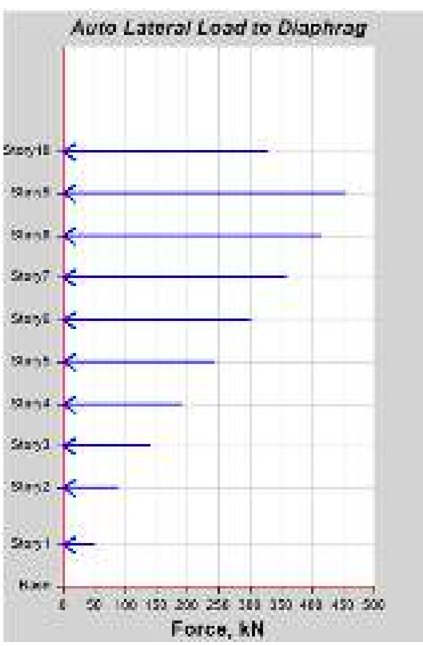
$$C_{S,min} = 0.5 \frac{S_1}{\left(\frac{R}{I}\right)} \text{ for } S_1 = 0.6g$$

$$C_{S,min} \leq C_s \leq C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	0.852	0.035039	16952.8432	594.016

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Stairs	26.92	0	0
ROOF	24.22	256.5838	0
6F	21.52	405.3770	0
5F	18.82	376.2204	0
4F	16.12	313.7557	0
3F	13.42	255.406	0
2F	10.72	198.4772	0
1F	8.02	143.2478	0
GF	5.32	97.842	0
BF	2.62	62.4266	0
Base	0	0	0

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Story5	P9	35650.5	23192.2	3000	200	0.781

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
23270	24	1	420	420

Design Code Parameters

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

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	34150.5	23192.2	37150.5	23192.2	3000	200
Bottom	Leg 1	34150.5	23192.2	37150.5	23192.2	3000	200

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

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	1972	0.0033	0.006	DWal24	-383.0261	6.5008	-335.3804	600000
Bottom	2355	0.0039	0.006	DWal24	-576.6086	-9.619	213.6154	600000

Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ΦV_c kN	ΦV_n kN
Top	Leg 1	500	DWal8	997.1412	591.0857	644.7198	632.832	1005.1489
Bottom	Leg 1	500	DWal8	819.0648	-1219.1933	645.6148	606.1205	978.4375

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	399.4	DWal13	1892.7243	-137.7626	3.61	4.8	699.4	666.7
Top–Right	Leg 1	387.3	DWal13	1850.6761	373.1452	4.33	4.8	687.3	666.7
Bottom–Left	Leg 1	441	DWal5	2037.0059	-119.5355	3.79	4.8	741	666.7
Botttom–Right	Leg 1	223.7	DWal5	1016.1776	1100.1693	5.36	4.8	447.4	666.7

ETABS 18.1.1

Elevation View - p1 Shear Force 2-2 Diagram (DWal7) [kN]

Elevation View - p1 Moment 3-3 Diagram (DWal19) [kN-m]

Irrigulation checks

a. Torsional Irrigulation Check

If $\frac{\Delta max}{\Delta av} > 1.2$, Eccentricities must be magnificated with an amplification factor

Table(1): Torsional irrigulating check for output case EQX+5

Story	Output Case	Ratio	Ok?	Eccentricity length
Story10	EQX+5	1.129	Ok	1.426889786
Story9	EQX+5	1.139	Ok	1.452278786
Story8	EQX+5	1.151	Ok	1.483041119
Story7	EQX+5	1.165	Ok	1.519337986
Story6	EQX+5	1.182	OK	1.5640027
Story5	EQX+5	1.203	N.Ok	1.620070075
Story4	EQX+5	1.232	N.Ok	1.699119644
Story3	EQX+5	1.269	N.Ok	1.802709675
Story2	EQX+5	1.143	Ok	1.462497075
Story1	EQX+5	1.012	Ok	1.146472311

Table(2): Torsional irrigulating check for output case EQX-5

Story	Output Case	Ratio	Ok?
Story10	EQX-5	1.042	Ok
Story9	EQX-5	1.045	Ok
Story8	EQX-5	1.051	Ok
Story7	EQX-5	1.056	Ok
Story6	EQX-5	1.062	Ok
Story5	EQX-5	1.068	Ok
Story4	EQX-5	1.073	Ok
Story3	EQX-5	1.062	Ok
Story2	EQX-5	1.087	Ok
Story1	EQX-5	1.055	Ok

Table(3): Torsional irrigulating check for output case EQY+5

Story	Output Case	Ratio	Ok?	Eccentricity length
Story10	EQY+5	1.197	Ok	1.603950075
Story9	EQY+5	1.314	N.Ok	1.9328283
Story8	EQY+5	1.351	N.Ok	2.043211119
Story7	EQY+5	1.403	N.Ok	2.203524519
Story6	EQY+5	1.469	N.Ok	2.415717453
Story5	EQY+5	1.564	N.Ok	2.738268578
Story4	EQY+5	1.728	N.Ok	3.3426432
Story3	EQY+5	2.188	N.Ok	5.359165644
Story2	EQY+5	2.164	N.Ok	5.242241911
Story1	EQY+5	1.062	Ok	1.2625587

Table(4): Torsional irrigulating check for output case EQY-5

Story	Output Case	Ratio	Ok?	Eccentricity length
Story10	EQY-5	1.175	Ok	1.545532986
Story9	EQY-5	1.291	N.Ok	1.865756786
Story8	EQY-5	1.321	N.Ok	1.953476453
Story7	EQY-5	1.352	N.Ok	2.046236978
Story6	EQY-5	1.393	N.Ok	2.172224853
Story5	EQY-5	1.453	N.Ok	2.363381186
Story4	EQY-5	1.565	N.Ok	2.741771319
Story3	EQY-5	1.879	N.Ok	3.952356453
Story2	EQY-5	1.808	N.Ok	3.659311644
Story1	EQY-5	1.037	Ok	1.203815853

b. Stiffness Check

If Ki/ki+1 > 0.7 → oK
If 3Ki/(Ki+1+Ki+2 ki+3) > 0.8 → oK

Table(5):Stiffness check for output case EQX+5

Story	**Output Case**	**Stiff X** **kN/m**	Ki/ki+1	Ki/(Ki+1+Ki+2 ki+3)/3
ROOF	EQX+5	113457.365	2.53	…
6F	EQX+5	286647.161	1.53	…
5F	EQX+5	437355.965	1.30	2.04
4F	EQX+5	570667.953	1.24	1.64
3F	EQX+5	706682.012	1.24	1.53
2F	EQX+5	875747.724	1.31	1.60
1F	EQX+5	1146407.01	…	…
GF	EQX+5	0	…	…
BF	EQX+5	0	…	…

c. Mass Check

If (mi/mi+1) or (mi/mi-1) < 1.5 , oK

Table(5):Mass Check

Story	**Mass X kg**	mi/mi+1	mi/mi-1
Story10	366808.14	1.556068385	…
Story9	570778.55	1.040453833	0.9611190507
Story8	593868.73	1.006428727	0.9936123374
Story7	597686.55	1	1
Story6	597686.55	1	1
Story5	597686.55	1	1
Story4	597686.55	1.002503453	0.9975027989
Story3	599182.83	0.9595612244	1.042142986
Story2	574952.61	1.585994557	0.6426452781
Story1	911871.71	…	…