Palestine Polytechnic University
College of Engineering
Civil Engineering and Architecture Department

Graduation Project

"STRUCTURAL DESIGN OF A RESIDENTIAL BUILDING"

Project Team

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

Hebron - Palestine

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The undersigned hereby certify that they have read, examined, and recommended to the Department of Civil Engineering and Architecture in the College of Engineering at Palestine Polytechnic University the approval of a project entitled: **Structural Design of a Residential Building**, submitted by Hala Mohammad Al-Qadi and Mai M. Sameeh Hashlamoun for partial fulfillment of the requirements for the bachelor's degree.

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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

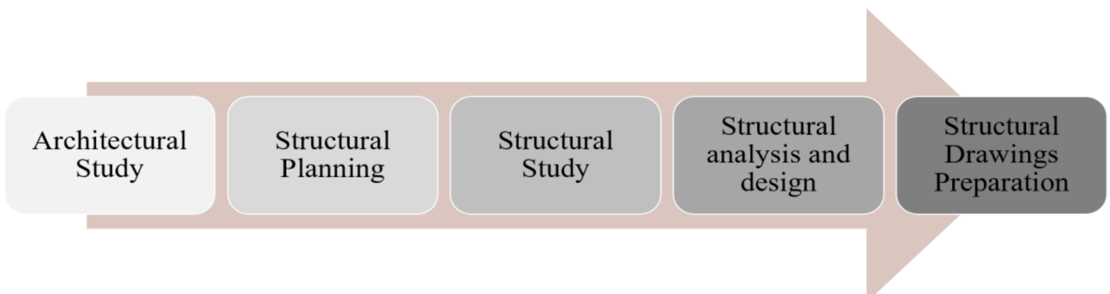
This Project was chosen to achieve the following galls:

- 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.

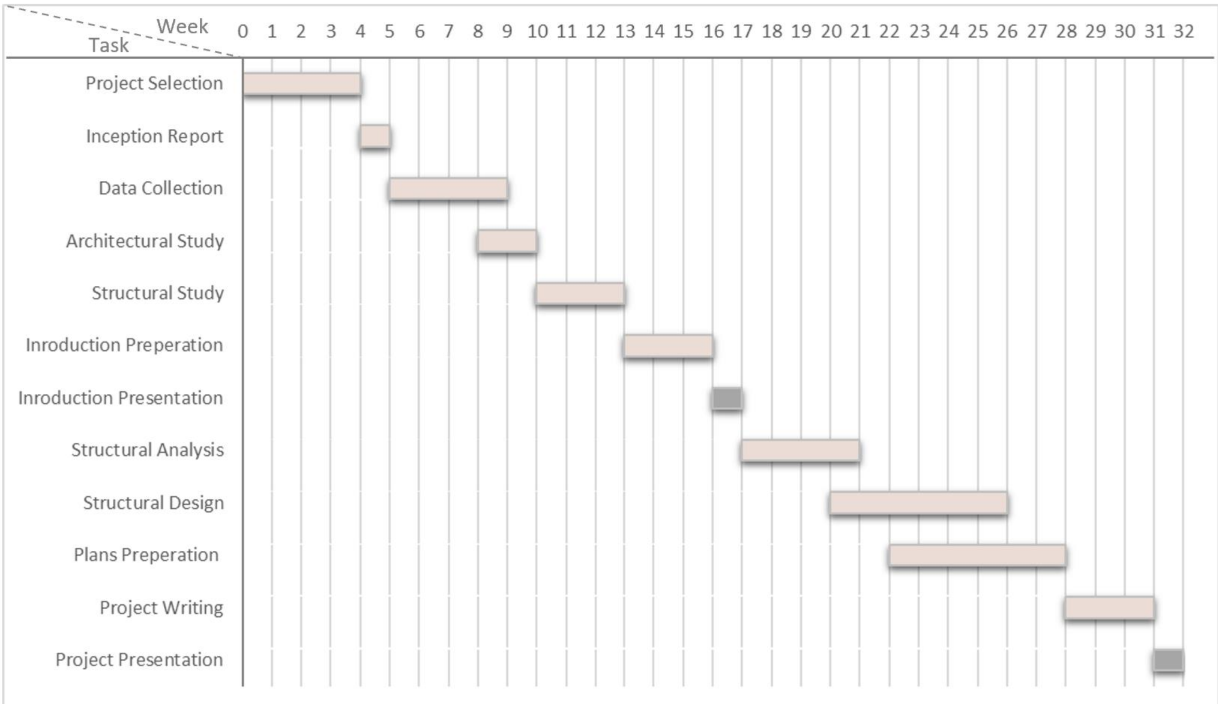
CHAPTER 3: A general description of the structural elements.

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 building with 9 floors one of them is a beauty center, the rest is residential floors with two apartments in each except the roof floor which contains a single apartment.

The building is proposed to be built on 850 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 Dahyat Haram al Ramah to north of Hebron, near to Al-Mizan Hospital, 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 three types of floors: Basement, Ground floor, and 7 residential floors with a total area of 3785 m^2. The following is a brief description of each floor.

1. BASEMENT FLOOR

The basement floor level is 2.3m below the level of Main Street with an area of 598 m^2. It can accommodate 18 parking spaces, an electrical room with an area of 20 m^2 and a mechanical room with an area of 31m^2.

The entrance to the basement is from the southwest side of the building while the exit is from the northeast side, Which is clearly shown in figure (2-2).

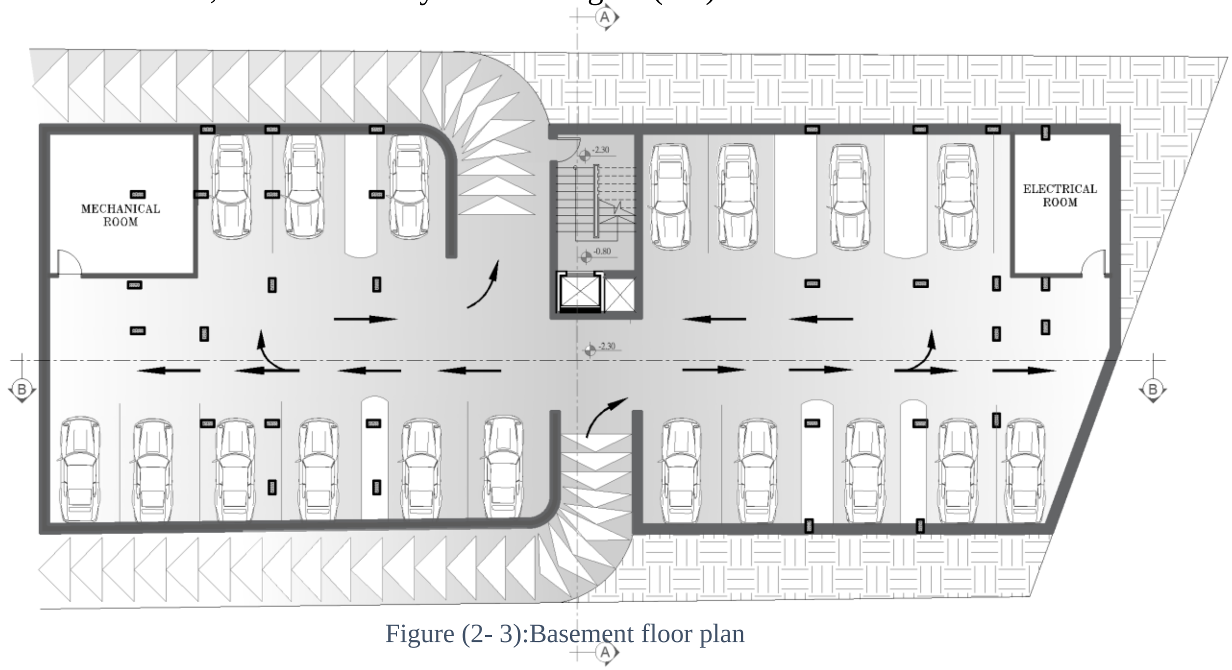


Figure (2- 3):Basement floor plan

2. GROUND FLOOR

It contains a complete beauty center with an area of 438 m2. It has one entrance on the northwest side which is separated from the residential one. The ground floor is divided into spaces which connected in a way to serve the beauty center.

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

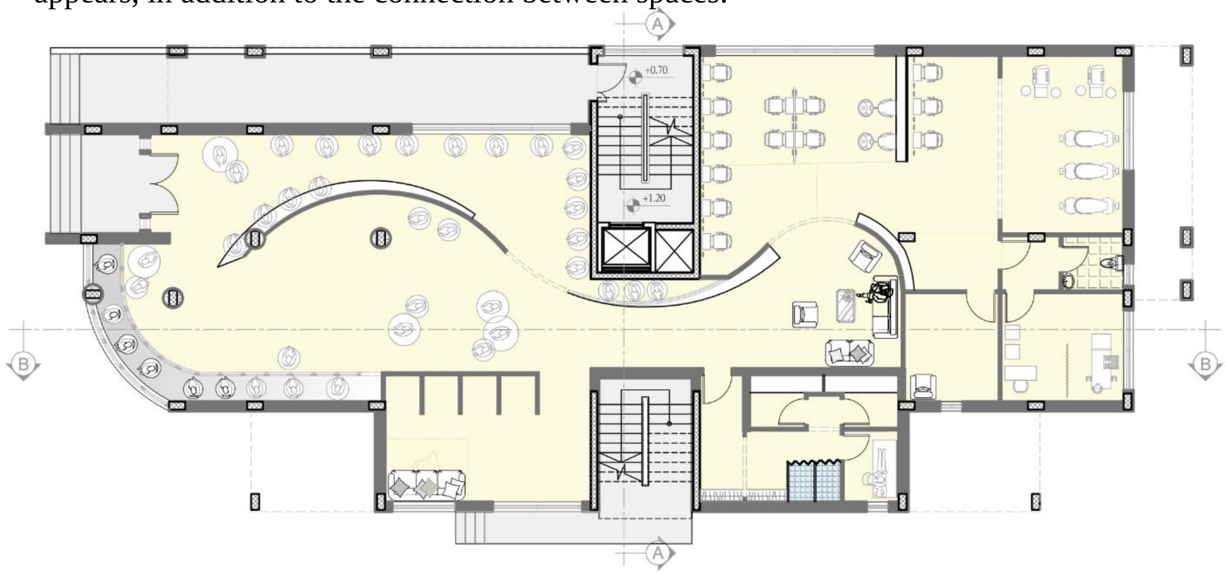


Figure (2- 4):Ground floor Plan

3. TYPICAL FLOORS

There are 6 typical floors, each floor consists of two apartments and has a total area of 438 m^2. There is two access to apartments, the main one is from the northwest side by an elevator or stairs. Apartments can also be accessed through an emergency staircase on the southwestern side of the building. Each apartment consists of:

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

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.

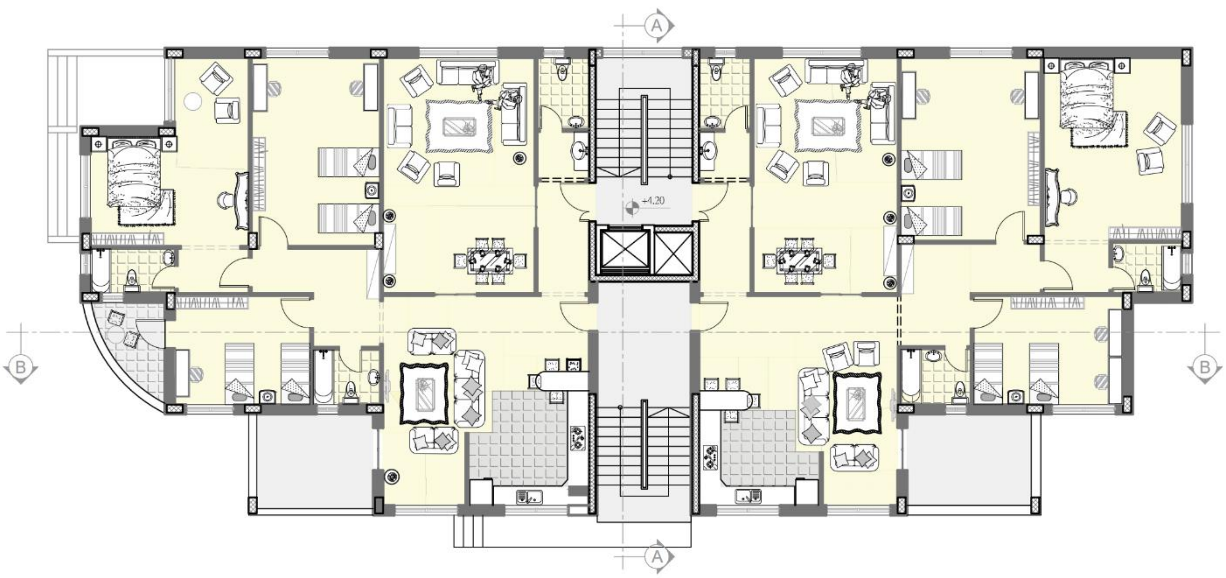


Figure (2- 5): Typical floor Plan

4. ROOF FLOOR

It’s a single apartment with an area equivalent to 198 m^2. It also has two access the main one is from the northwest side of the building, the other is from the southwest side.

Roof floor consist of the following :

- Guest room
- Dining room
- Kitchen
- Living room
- Master bedroom
- Two bedrooms
- 2 bathrooms
- Terrace

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

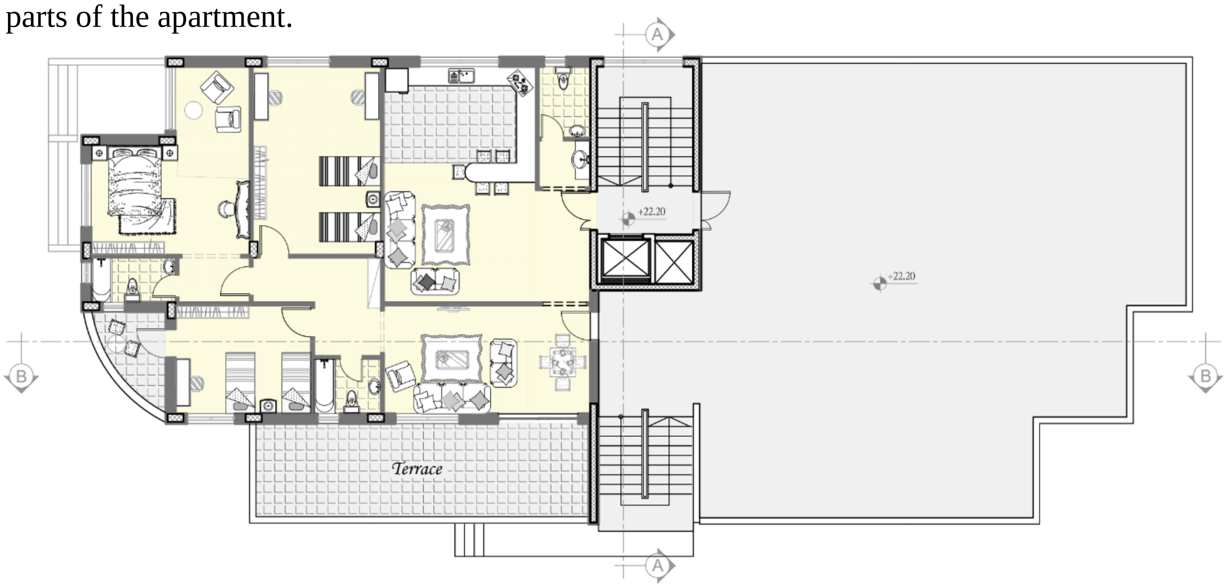


Figure (2- 6):Roof floor Plan

5. MOVEMENT AREAS

This building contains two stairs and one elevator, one of the stairs is the main stair and the other is for an emergency .The main stairs and elevator are located on the northeast side, they are also located in the middle of the building, which makes a connecting point for the two apartments. While emergency stairs are located on the opposite side of the building in a way that can be accessed from both apartments.

2.5 Elevations Description

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

2.3.1 North West elevation

It's the main elevation contains the main entrance to the apartments and the entrance to the beauty center. They are both separated and there is a corridor connecting the main entrance to the apartments with the street. This elevation is characterized by its glass that is integrated with stones, as it appears in figure (2-6).

Figure (2- 7):North West elevation

2.3.2 South West elevation

As shown in figure (2-7), It contains the emergency stairs which are accessible from each apartment, also there is a ramp that provides access for cars from the street to the basement floor.

Figure (2- 8):South West elevation

2.3.3 North East elevation:

In this elevation, the architectural beauty appears in the arrangement of openings and the use of different types of stone to distinguish the openings on one hand and to give a unique appearance to the building on the other hand. Furthermore, concrete columns cladded with stones appear in this elevation these columns serve the building structurally. They also add beauty to the building.

Figure (2- 9):North East elevation

2.3.4 South East elevation:

This is the backside elevation of the building, it is characterized by its glass and prominent colored stones that give the aesthetic appearance and architectural beauty that reflects the luster of the building. As shown in figure (2-10) cladded concrete columns are appear here too.

Figure (2- 10): South East 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.

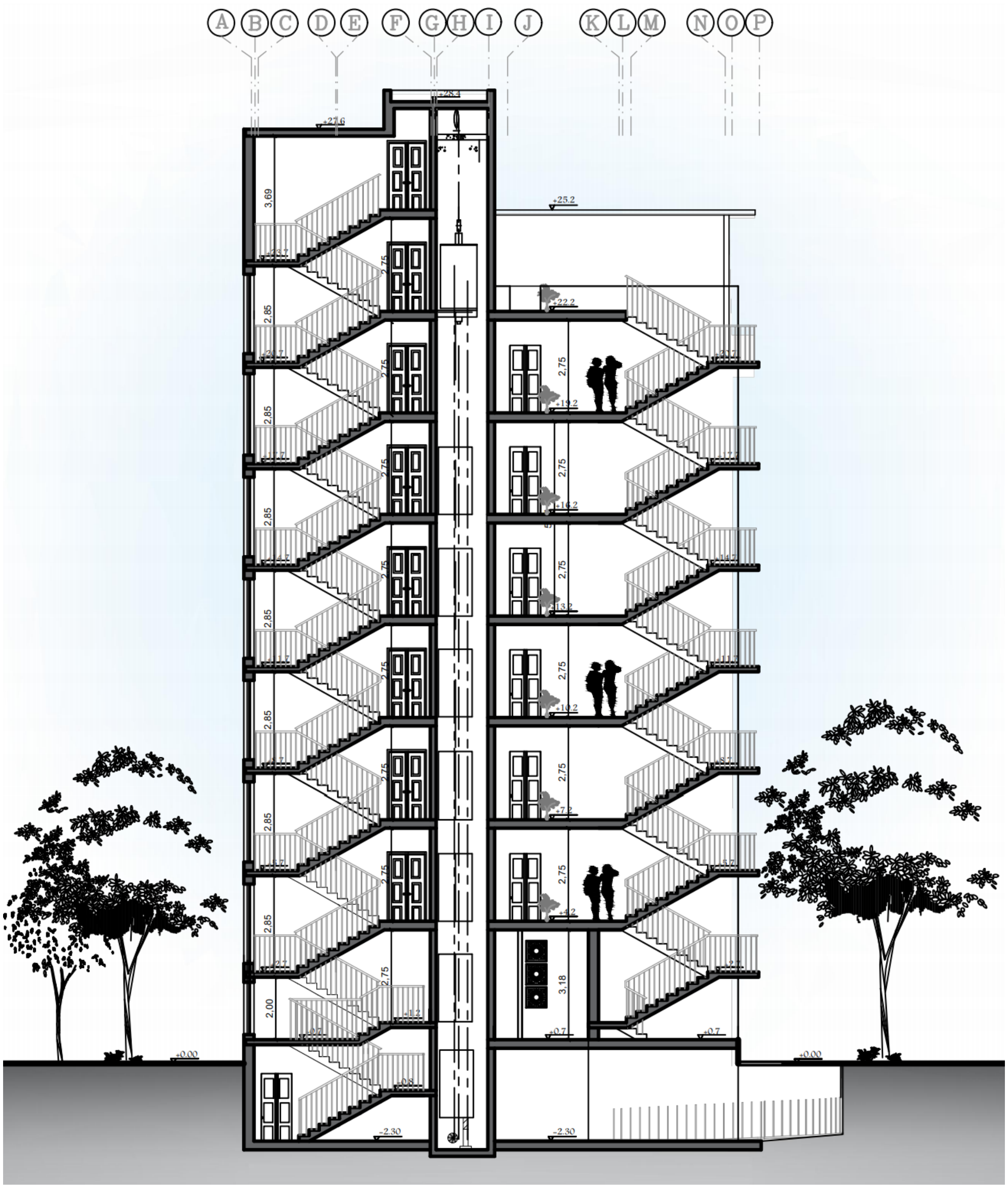


Figure (2- 11):Section A-A

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

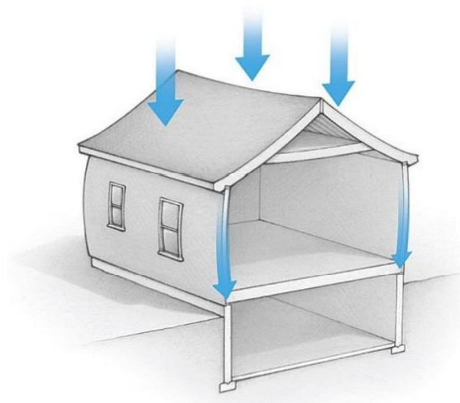
Before the structural study of any building, there is the work of geotechnical studies of the site, which means all work related to exploring the site and studying soil, rocks, and groundwater, then analyzing information and translating it to predict the way the soil behaves when building on it, and 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

Dead loads consist of the weight of all materials of construction incorporated into the building including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes



Figure(3- 1): Dead Load

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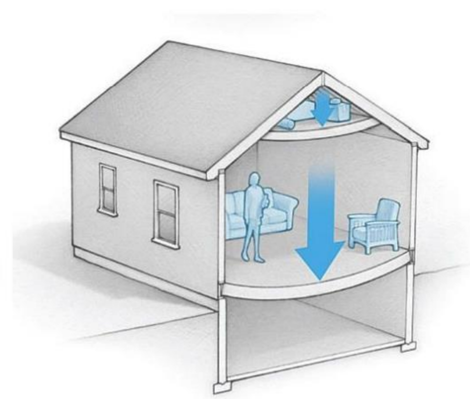
3.4.2 live load

Live loads are those loads produced by the use and occupancy of the building or other structure and do not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load, or dead load.



Figure(3- 2): Live Load

3.3.3 Environmental loads

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 /M²)	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

One of the most important environmental loads that affect the building, 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.

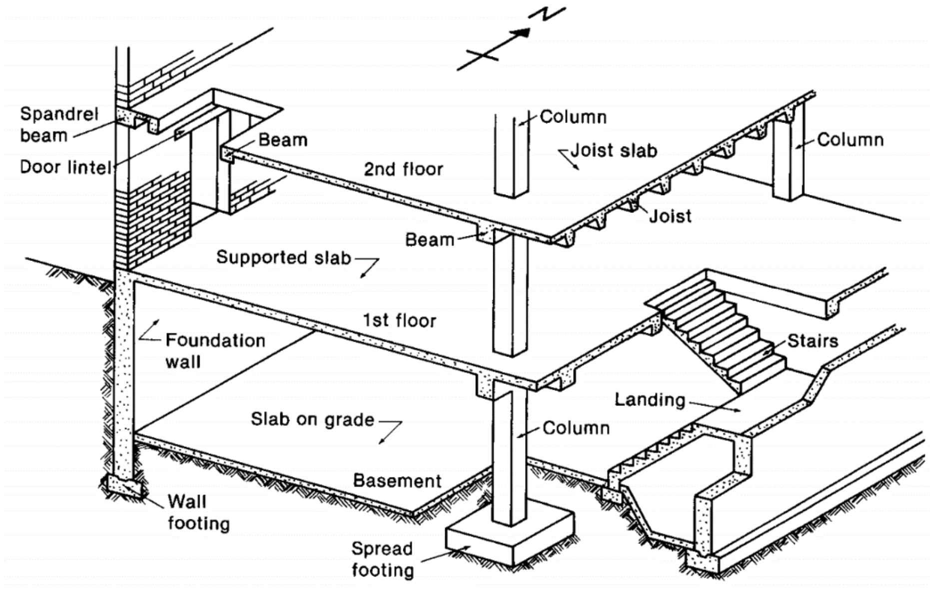
Figure(3- 5):Seismic Loads

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.



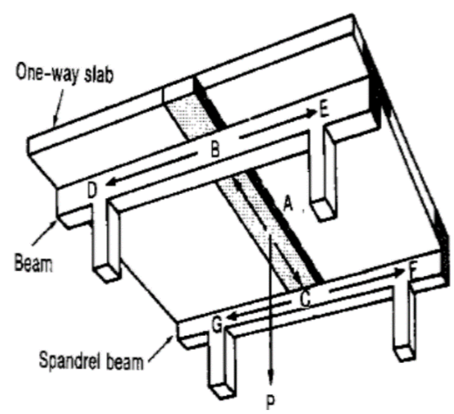
Figure(3- 6):Structural elements of a typical RC structure

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)

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

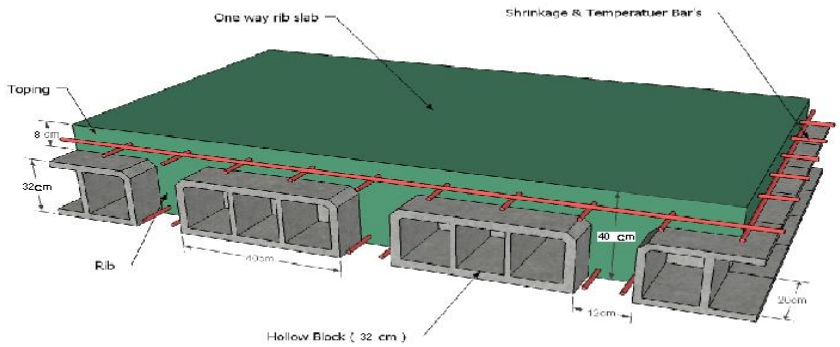


Figure(3- 7): Solid slab

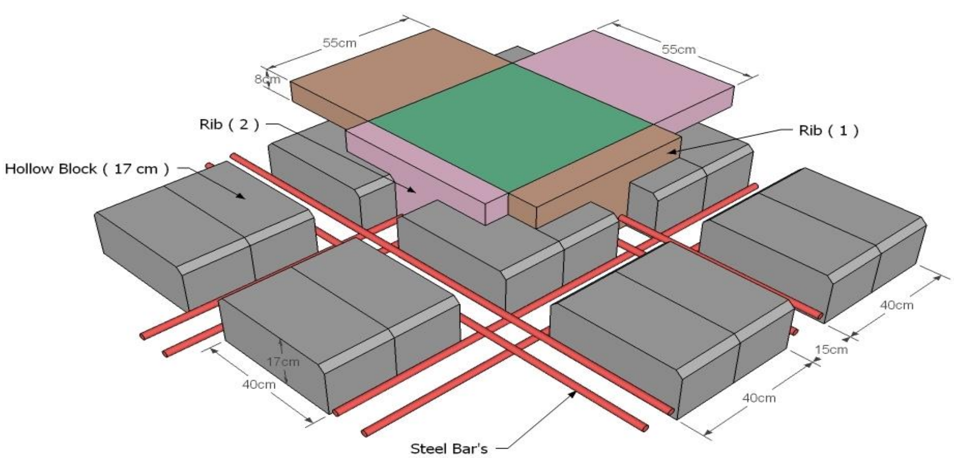
3.5.1.2 Ribbed slab (one or two way)

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.



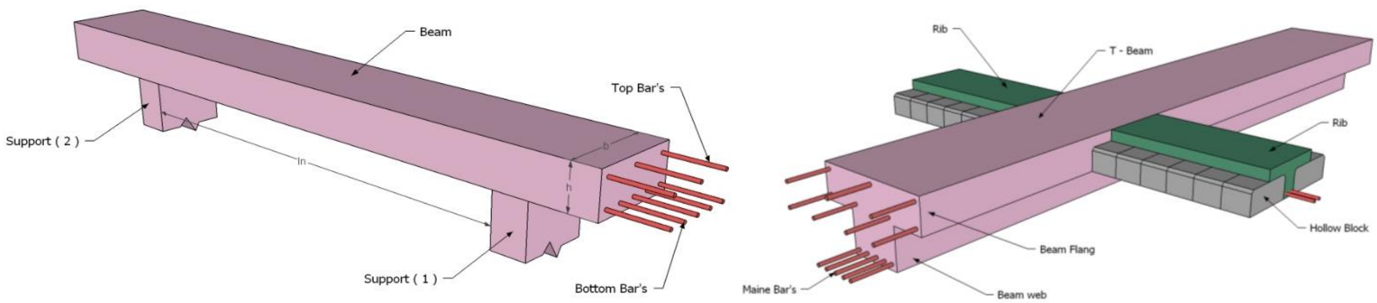
Figure(3- 8):One way ribbed slab.



Figure(3- 9):Two way ribbed slab

3.5.2 Beams

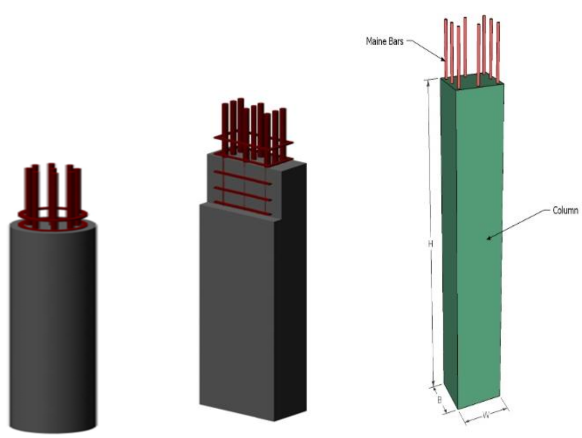
They are basic structural elements in transferring loads from slabs to the columns, and they are of two types, hidden inside the slab and Dropped Beams that emerge from the slab from the bottom.



Figure(3- 10):Beams

3.5.3 Columns

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

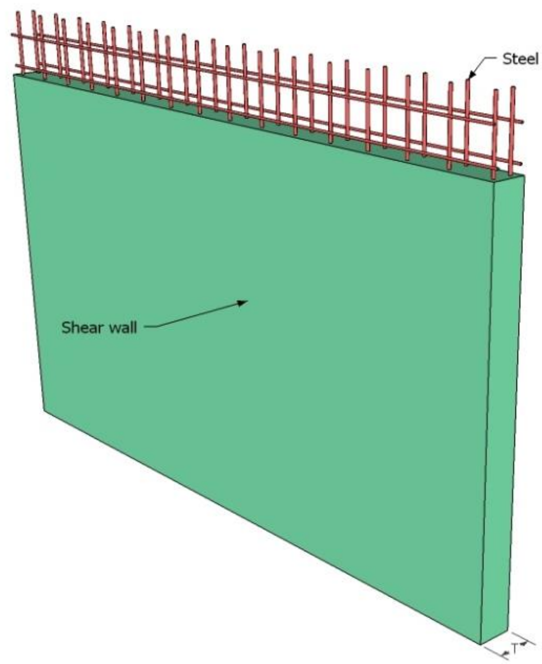


Figure(3- 11):Different types of Columns

3.5.4 Shear walls

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

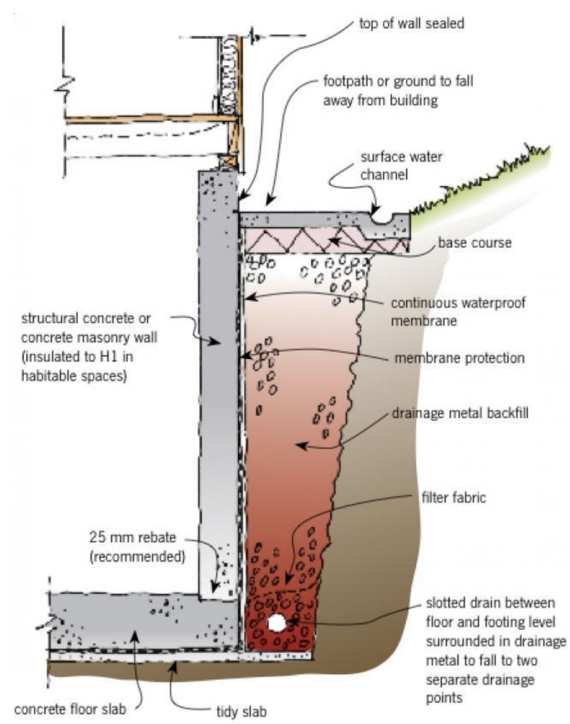
These walls are armed with two layers of steel to increase their efficiency to resist the horizontal forces. The two directions taking into consideration that the distance between the center of resistance formed by the shear walls in each direction and the center of gravity of the building is minimal.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



Figure(3- 12):Shear wall

3.5.5 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.



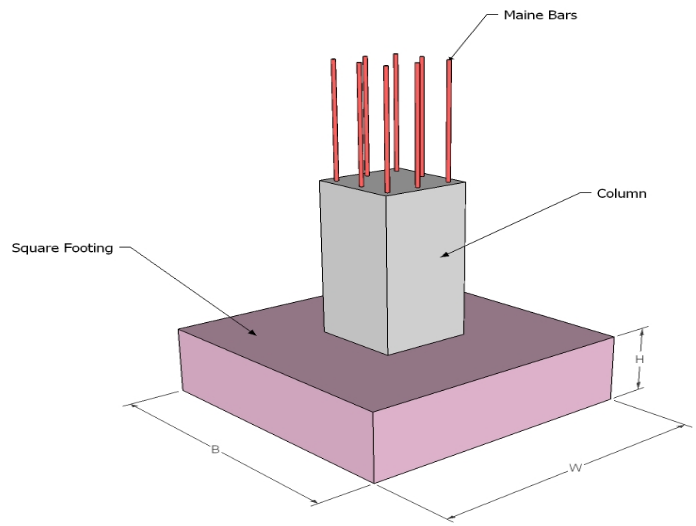
Figure(3- 13): Basemet Wall

3.5.6 Foundations

Although the foundations are the first to start with the construction of the structure, their design takes place after the completion of the design of all structural elements in the building.

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

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

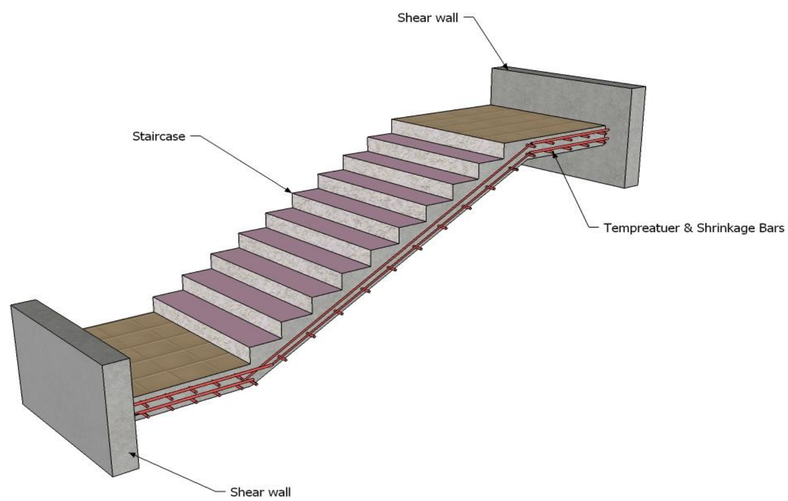


Figure(3- 14): Isolated Footing

3.5.7 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 Section 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 B30
4.5. Design of Column C14
4.6. Design of Shear Wall
4.7. Design of Basement Wall
4.8. Design of Basement Footing
4.9. Design of Isolated 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	$= \frac{500}{16} = 31.25$
One end continues	L/18.5	1	3	$= \frac{620}{18.5} = 33.5$
Both ends continuous	L/21	3	5	$= \frac{630}{21} = 30$

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

∴ **Select slab thickness = 32cm with 24cm block & 8cm topping.**

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

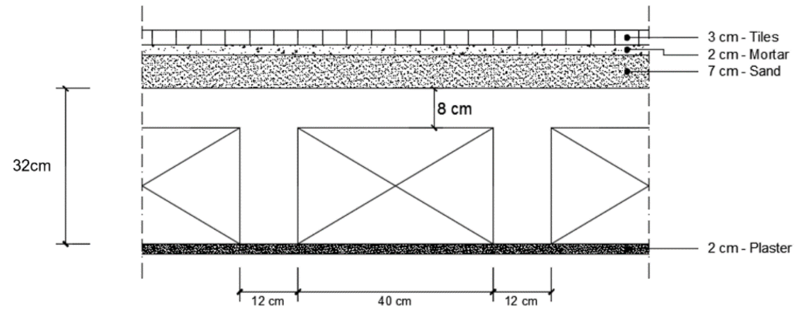


Figure (4- 1) :Typical section of one-way ribbed slab

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.

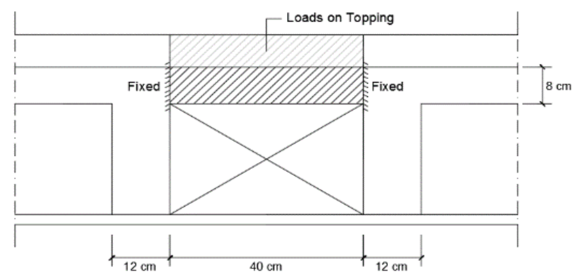


Figure (4- 2):System of topping

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 = **2.0 × 1 = 2.0 kN/m**

→ <u>Factored load (W_u)</u> = 1.2 ×D.L + 1.6 ×L.L = 1.2 * 6.1 + 1.6 * 2.0 = **<u>10.52 kN/m.</u>**

4.3.1.2. Analysis of topping

- $Vu = \frac{W_u \times L}{2} = \frac{10.52 \times 0.4}{2} = \mathbf{2.1\ kN}$
- $Mu = \frac{W_u \times L^2}{12} = \frac{10.52 \times 0.4^2}{12} = \mathbf{0.14\ kN.m}$

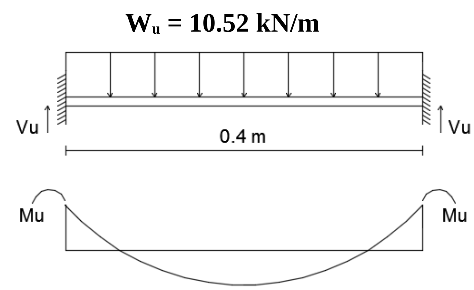


Figure (4- 3):System and analysis of topping

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 → 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 → SAFE**

∴ **Plain Concrete Section is SAFE #**

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

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

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

= 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 (R17)

Rib (R17) is selected to be designed , the following figure shows its location in ground floor slab:

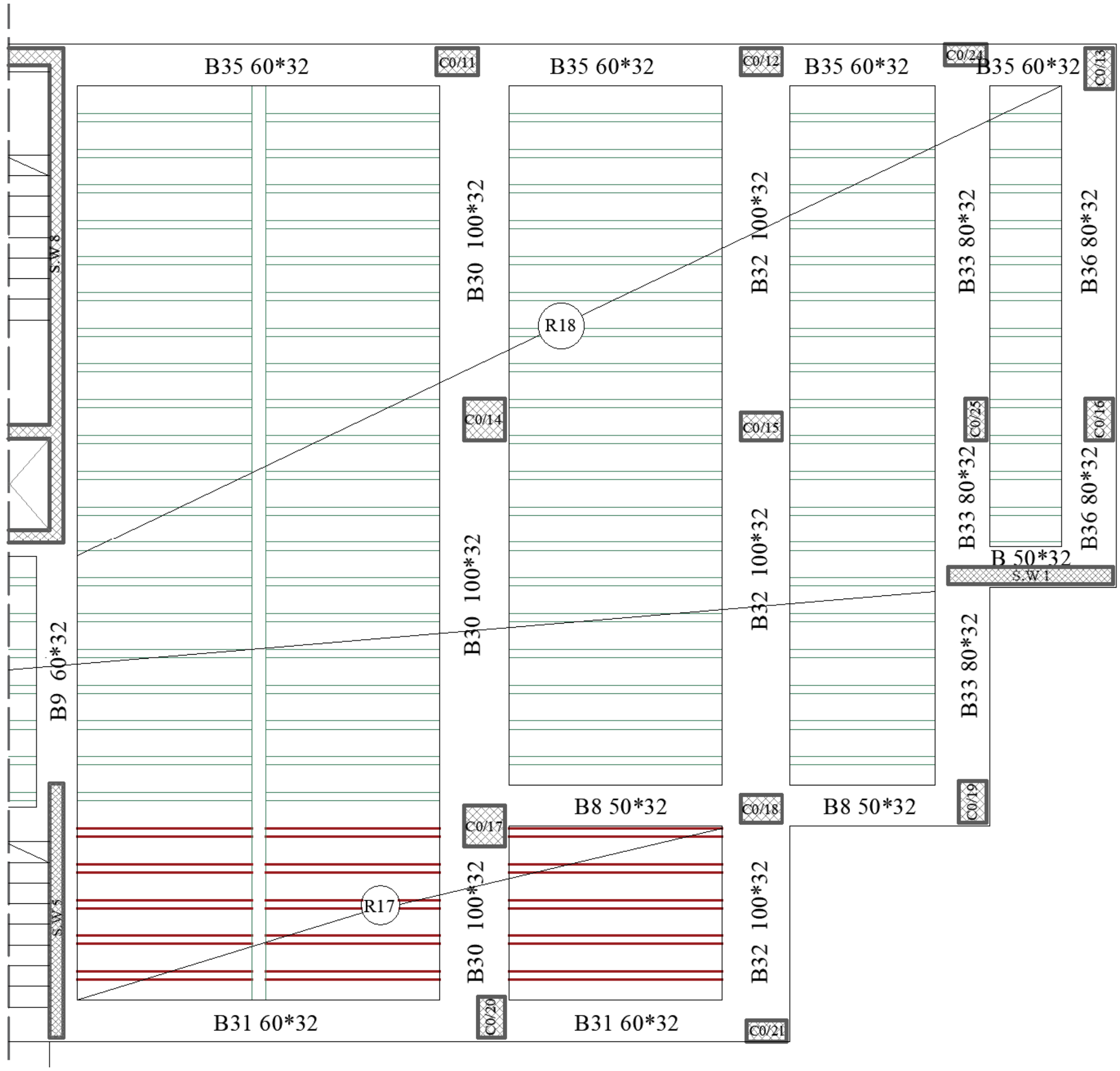


Figure (4- 4):Rib (R17) Location in Ground Floor Slab

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 (R17)

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 :

→ **<u>Dead loads :</u>**

Table(4- 3):3 Dead Load Calculation for rib (R17)

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			

→ **<u>Live loads</u>** = 2.0 × 0.52 = **1.04 kN/m/rib**

→ **<u>Factored Load (W_u)</u>** = 1.2×D.L + 1.6×L.L

WuD = 1.2× 5.27 = **6.32 kN/m/rib**

WuL = 1.6*1.04 = **1.66 kN/m/rib**

4.3.2.3. Analysis

Figure (4-5)& (4-6) shows the shear and Moment envelope of the rib (R17) obtained from Atir 2018 software .

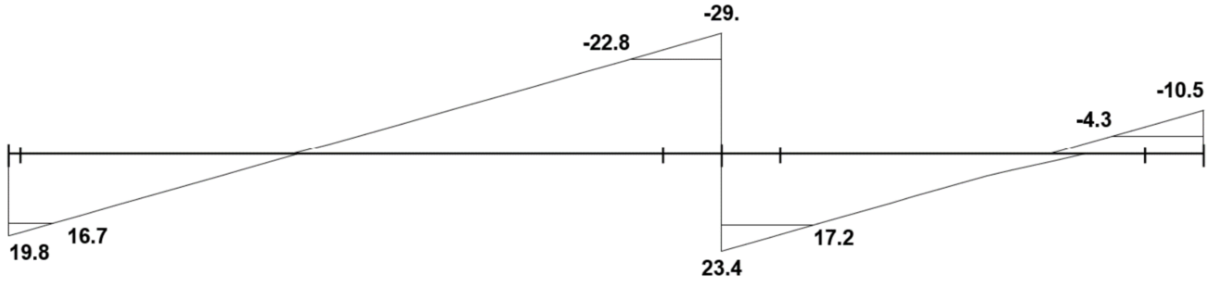


Figure (4- 5):Shear envelope of rib (R17) – [kN]

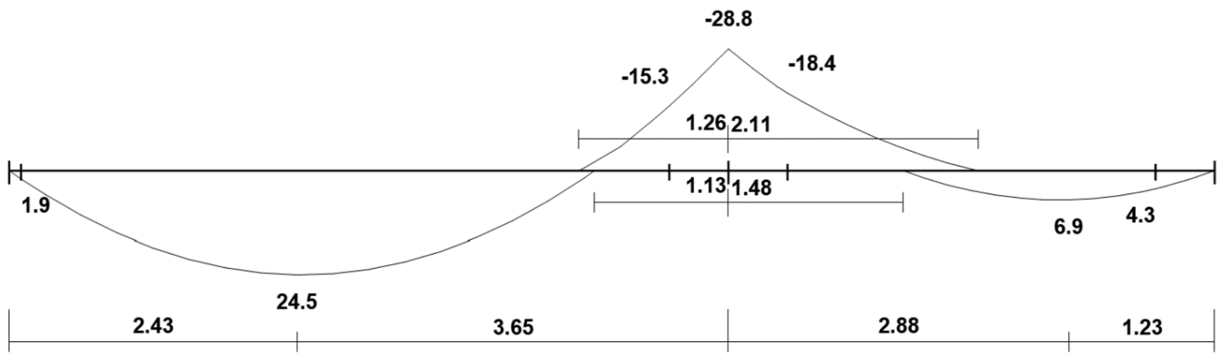


Figure (4- 6): Moment envelope of rib (R17) – [kN.m]

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 .

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

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

$$= \mathbf{22.88\ kN}$$

Ø.Vc=**22.88 kN** > Vu max = 20.2 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 ($\boldsymbol{be}$) :

According to (ACI 318-14) (be) is the smallest of:
- be ≤ Span/4 ≤ (608/4) = 152 cm
- be ≤ 16*hf + bw = 16*8 + 12 = 140 cm
- be ≤ bw + ½ Lc = 12 + ½ * 40 + ½ * 40 = **52 cm** **« Cont.**

⇒ **Design of span 1 - Max Mu^+ = 24.5 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^+ = 24.5 kN.m

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

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

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

   $\rightarrow kn = \frac{Mu/Ø}{b*d^2} = \frac{24.5*10^6/0.9}{520*283^2} = 0.654$ MPa

   $\rightarrow \rho = \frac{1}{m} * (1- \sqrt{1 - \frac{2*KN*m}{Fy}}) = \frac{1}{20.59} * (1- \sqrt{1 - \frac{2*0.654*20.59}{420}}) = 0.001583$

   $\rightarrow Asreq = \rho * b * d = 0.001583 * 520 * 283 = 232.95\ mm^2$

∴ **Select 2Ø14 with As = 308 mm^2**

3. Check As min :

As (min) = $0.25 * \frac{\sqrt{Fc'}}{Fy} * bw * d = 0.25 * \frac{\sqrt{24}}{420} * 120 * 283 = 99.03$ mm²

Or

As (min) = $\frac{1.4}{Fy} * bw * d = \frac{1.4}{420} * 120 * 283 =$ **113.2 mm²** « **Controlled**

∴ **Use 2Ø14 with As = 308 mm² > Asmin =113.2 mm²**

4. Check Strain :

C=T

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

0.85 * 24 * a * 520 = 308* 420

a=12.19 mm ⇒ X = a / β = 12.19 / 0.85 = 14.34 mm

$\varepsilon_s = \frac{0.003d}{x} - 0.003 = \frac{0.003*283}{14.34} - 0.003 = 0.0562 > 0.005 \Rightarrow Ø = 0.9$ … (OK)

⇒ Design of span 2 - Max Mu+ = 6.9 kN.m

1. Check if (a ≤ t) or (a > t)

Assume a=t=8cm

Ø * Mn = **185.6 kN.m** > Mu⁺ = **6.9** kN.m

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

2. Design as Rectangular Section with b=bE

$\rightarrow kn = \frac{6.9*10^6/0.9}{520*283^2} = 0.184$ MPa

$\rightarrow \rho = \frac{1}{20.59} * (1 - \sqrt{1 - \frac{2*0.184*20.59}{420}}) = 0.00044$

$\rightarrow$ Asreq = 0.00044 ∗ 520 ∗ 283 = 64.75 mm²

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

3. Check Strain :

C=T

0.85 * 24 * a * 520 = 226* 420

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

$\varepsilon_s = \frac{0.003*283}{10.53} - 0.003 = 0.0776 > 0.005 \Rightarrow Ø = 0.9$ … (OK)

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

Max Mu^- = 18.4 kN.m
(Compression zone in web ⇒ design as rectangular RC section)

→ kn = $\frac{18.4*10^6/0.9}{120*283^2}$ = 2.13 MPa

→ $\rho = \frac{1}{20.59} * (1- \sqrt{1-\frac{2*2.13*20.59}{420}}) = 0.00537$

→ Asreq = $\rho * b * d = 0.00537 * 120 * 283 = 182.4$ mm^2

∴ **Select 2Ø12 with As = 226 mm2 > As min = 113.2 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 (R17)

Span No.	Span Length (mm)	Δlimit (mm)	ΔCalculated (mm)	Check
Span 1	6080	6080/240 =25.3	6080/259 =23.47	ΔCalculated < Δlimit (OK)
Span 2	4110	4110/240 =17.1	4110/4081 =1.00	

4.4. Design of Beam B30

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

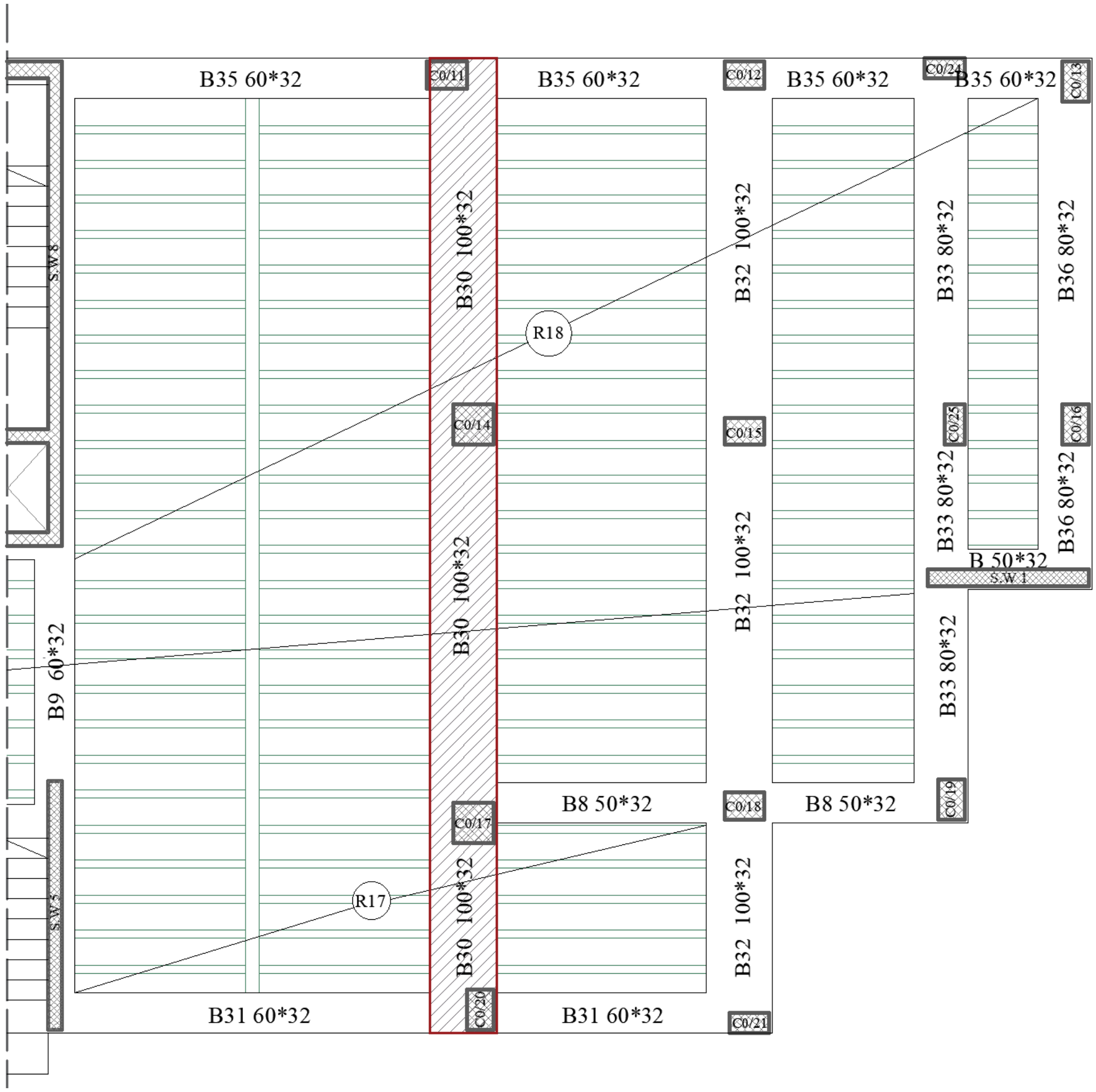


Figure (4- 7):Beam (B30) Location In Ground Floor Slab

4.4.1. Load Calculation for beam

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

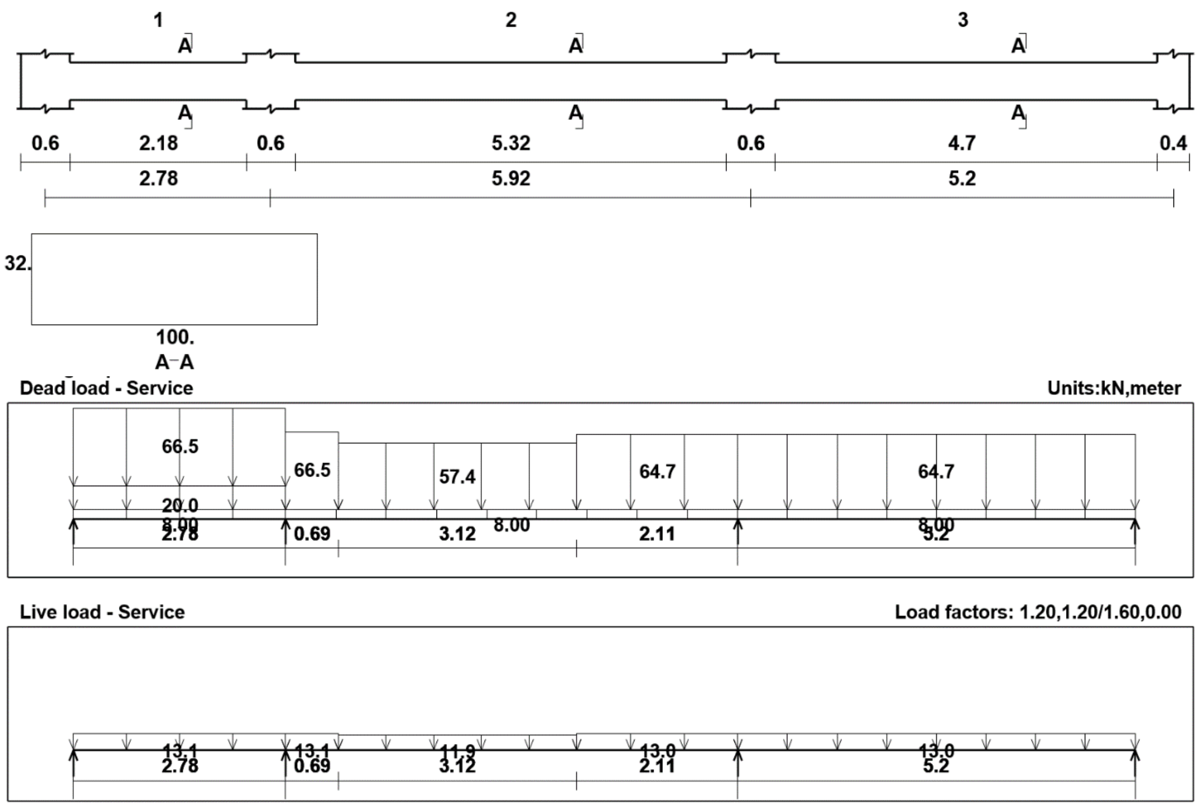


Figure (4- 8) : Beam B30 Geometry and loads

Calculation of Loads that acts on beam B30 :

1. Own weight of the beam :
   Own wt. = 25*0.32*1.00 = **8.0 kN/m**

2. Reactions of ribs that acting on it .
   The following table shows calculation of loads that act on B30 from ribs .

Table(4- 5):Loads on B30 from ribs

	Rib(R17)	Rib(R15)	Rib (R18)
quD(kN/m)	34.61/0.52=66.5	29.85/0.52 = 57.4	33.68/0.52 = 64.7
quL (kN/m)	6.83/0.52 = 13.1	6.21/0.52=11.9	6.77/0.52 = 13.0

4.4.2. Design of beam B16 for Flexure

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

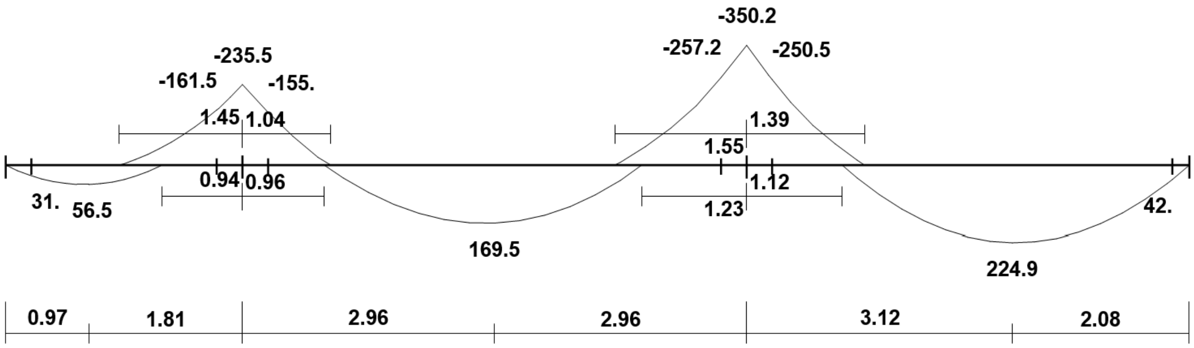


Figure (4- 9):Moment Envelope of beam (B30) – [kN.m]

4.4.3.1 Design of Negative Moment – Top Reinforcement

⇒ **Design of negative moment Mu- = 257.2 kN.m @ support (3)**

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

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

$\boldsymbol{d}$ = 320 – 40 -10 - 20/2 = $\boldsymbol{60\ mm}$

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

$\rightarrow$ Mn req = $\frac{257.2}{0.9}$ = 286 kN.m

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

$\rightarrow$ kn = $\frac{\text{Mn req}}{\text{b}*\text{d}^2} = \frac{286*10^6}{1000*260^2}$ = 4.23 Mpa

$\rightarrow$ ρreq = $\frac{1}{m} * (1-\sqrt{1-\frac{2*KN*m}{Fy}}) = \frac{1}{20.59} * (1-\sqrt{1-\frac{2*4.23*20.59}{420}}) =$ 0.0114

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

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

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

Assume rectangular & tension control section.

$\rightarrow$ Asreq = $0.0114 * 1000 * 260 = 2964$ mm^2

∴ **Select 15Ø16 with As = 3015 mm^2 .**

3. Check As min :

As (min) = 0.25 * $\frac{\sqrt{Fc'}}{Fy}$ * bw * d = 0.25 * $\frac{\sqrt{24}}{420}$ * 1000 * 260 =758 mm²

Not less than :

As (min) = $\frac{1.4}{Fy}$ * bw * d = $\frac{1.4}{420}$ * 1000 * 260 = **867 mm²** « **Controlled**

As = 3015 mm²> Asmin =867mm² … (OK)

4. Check Strain for Ø and Asmax

C=T

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

0.85 * 24 * a *1000 = 3015* 420

a=62 mm

X = a / β = 62 / 0.85 = 72.9 mm

$\varepsilon_s = \frac{0.003d}{x} - 0.003 = \frac{0.003*260}{72.9} - 0.003 = 0.0077$

$\therefore\ \varepsilon_s = 0.0077 > 0.005$ then Ø = 0.9 … (OK)

also, $\varepsilon_s = 0.0077 > 0.004$ then As < Asmax ... (OK)

5. Check for spacing

S= $\frac{1000-2(40)-2(10)-15(16)}{14}$ = 47.1 mm >25 mm … (OK)

> db=16mm … (OK)

⇒ <u>Design of negative moment Mu- = 161.5 kN.m @ support (2)</u>

Since Mu =161.5 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=161.5/0.9 =179.4 kN.m

→ m = 20.59

→ kn = $\frac{179.4*10^6}{1000*260^2}$ = 2.65 MPa

→ $\rho = \frac{1}{20.59} * (1-\sqrt{1-\frac{2*2.65*20.59}{420}}) = 0.00678$

→ Asreq = $\rho * b * d = 0.00678 * 1000 * 260 = 1762.8$ mm²

∴ **<u>Select 9 Ø16 with As = 1809 mm²</u>**

→ As = 1809 mm^2> Asmin =867 mm² … (OK)

→ **Check Strain for Ø and Asmax**

C=T

0.85 * 24 * a *1000 = 1809* 420

a=37.2 mm , X = 37.2 / 0.85 = 43.8 mm

$\varepsilon_s = \frac{0.003*260}{43.8} - 0.003 = 0.0148$

$\therefore\ \varepsilon_s = 0.0148 > 0.005\ \ then\ Ø = 0.9$ … (OK)

also, $\varepsilon_s = 0.0148 > 0.004$ then As < Asmax ... (OK)

→ **Check for spacing:**

S= $\frac{1000-2(40)-2(10)-9(16)}{8}$ = 94.5 mm >25 mm … (OK)

> db=16mm … (OK)

4.4.3.2 Design of Positive Moment – Bottom Reinforcement

⇒**Design of span 1 - Max Mu+ = 56.5 kN.m**

Since max Mu in this span < 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=56.5 /0.9 =62.8 kN.m

→ m = 20.59

→ kn = $\frac{62.8*10^6}{1000*260^2}$ = 0.929MPa

→ $\rho req = \frac{1}{20.59} * (1- \sqrt{1 - \frac{2*0.929*20.59}{420}}) = 0.00226$

→ Asreq = $0.00226 * 1000 * 260 = 559$ mm^2

∴ **Select 4Ø16 with As = 804 mm^2**

→ As =804 mm^2> Asmin =867 mm² … (OK)

→ **Check Strain for Ø and Asmax :**

C=T

0.85 * 24 * a *1000 = 804* 420

a=16.55 mm , X = 16.55 / 0.85 =19.47 mm

$\varepsilon_s = \frac{0.003*260}{19.47} - 0.003 = 0.037$

$\therefore\ \varepsilon_s = 0.037 > 0.005\ \ then\ Ø = 0.9$ … (OK)

also, $\varepsilon_s = 0.037 > 0.004$ then As < Asmax ... (OK)

→ **Check for spacing:**

$S = \frac{1000-2(40)-2(10)-4(16)}{3}$ = 227 mm >25 mm … (OK)

> db=16mm … (OK)

⇒ Design of span 2 - Max Mu+ = 169.5 kN.m

Since max Mu in this span < 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=169.5 /0.9 =188.3 kN.m

→ m = 20.59

→ kn = $\frac{188.3*10^6}{1000*260^2}$ = 2.79MPa

→ $\rho req = \frac{1}{20.59} * \left(1-\sqrt{1-\frac{2*2.79*20.59}{420}}\right) = 0.00717$

→ Asreq = $0.00717 * 1000 * 260 = 1864.8$ mm^2

∴ **Required 10Ø16,but select 11Ø16 with As=2211 mm^2 for deflection reasons .**

→ As =2211 mm^2> Asmin =867 mm^2 … (OK)

→ **Check Strain for Ø and Asmax :**

C=T

0.85 * 24 * a *1000 = 2211* 420

a=415.5 mm , X = 45.5 / 0.85 =53.6 mm

$\varepsilon_s = \frac{0.003*260}{53.6} - 0.003 = 0.0116$

$\therefore\ \varepsilon_s = 0.0116 > 0.005\ \ then\ Ø = 0.9$ … (OK)

also, $\varepsilon_s = 0.0116 > 0.004$ then As < Asmax ... (OK)

→ **Check for spacing:**

$S = \frac{1000-2(40)-2(10)-11(16)}{10}$ = 72.4 mm >25 mm … (OK)

> db=16mm … (OK)

Design of span 3 - Max Mu+ = 224.9 kN.m

Since max Mu in this span < 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=224.9 /0.9 =250 kN.m

→ m = 20.59

→ kn = $\frac{250*10^6}{1000*260^2}$ = 3.7 MPa

→ $\rho req = \frac{1}{20.59} * (1- \sqrt{1 - \frac{2*3.7*20.59}{420}}) = 0.00979$

→ Asreq = $0.00979 * 1000 * 260 = 2548$ mm^2

∴ **Required 13Ø16,but select 16 Ø16 with As=3216 mm^2 for deflection reasons .**

→ As =3216 mm^2> Asmin =867 mm^2 … (OK)

→ **Check Strain for Ø and Asmax :**

C=T

0.85 * 24 * a *1000 = 3216* 420

a=66.2 mm , X = 66.2 / 0.85 =77.9 mm

$\varepsilon_s = \frac{0.003*260}{77.9} - 0.03 = 0.007$

∴ $\varepsilon_s = 0.007 > 0.005$ *then* Ø = 0.9 … (OK)

also, $\varepsilon_s = 0.007 > 0.004$ then As < Asmax ... (OK)

→ **Check for spacing:**

S= $\frac{1000-2(40)-2(10)-16(16)}{15}$ = 42.9 mm >25 mm … (OK)

> db=16mm … (OK)

4.4.4 Design Beam B30 for Shear

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

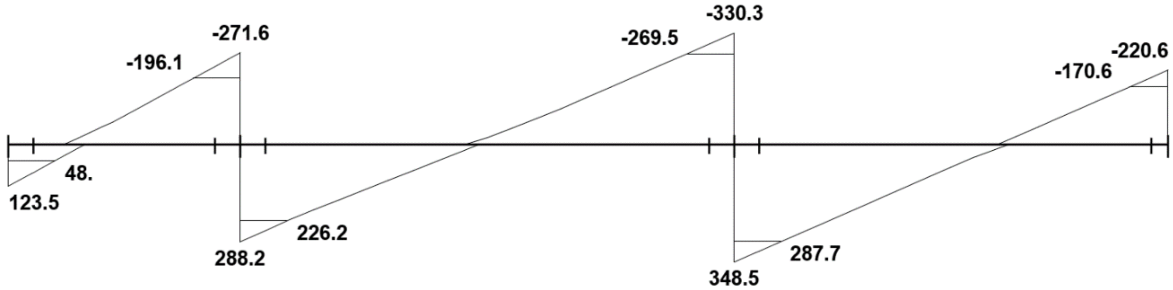


Figure (4- 10):Shear envelope of beam B30 – [kN]

The following are steps of shear force design :

1. **Check for dimensions:**

   If Vu max $\le$ 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 = 287.7 kN as shown in figure (4-10).

   $$Ø.Vc = Ø * \frac{1}{6} * \sqrt{Fc'} * bw * d$$
   $$= 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 260 * 10^{-3}$$
   $$= \mathbf{159.22\ kN}$$

   $$Ø\frac{2}{3}\sqrt{Fc'} * bw * d = 0.75 * \frac{2}{3}\sqrt{24} * 1000 * 260 * 10^{-3} = \mathbf{636.87\ kN}$$

   $$\mathbf{Ø.Vc + Ø\frac{2}{3}\sqrt{Fc'} * bw * d = 796.02\ kN > Vu\ max = 287.7} \quad ...(OK)$$

   ∴ **Section is adequate .**

2. **Category (III) :**

   $$Ø.Vc < Vu \le Ø.Vc + Ø.Vs\ min$$

   Ø .Vs min is the maximum between :

   $\rightarrow$ Ø .Vs min = $0.75 \times \frac{1}{16} \times \sqrt{fc'} \times bw \times d = 0.75 \times \frac{\sqrt{24}}{16} \times 1000 \times 260 \times 10^{-3}$ = **59.7 kN**

   OR

   $\rightarrow$ Ø .Vs min=$0.75 \times \frac{1}{3} \times bw \times d = 0.75 \times \frac{1}{3} \times 1000 \times 260 \times 10^{-3}$= **65 kN** « **Cont.**

   $$Ø.Vc + Ø.Vs\ min = \mathbf{159.22 + 65 = 224.22\ kN}$$

   ∴ For all shear values that is $\le$ 224.22 kN , minimum shear reinforcement is required .

→ Minimum Shear Reinforcement :

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

→ $Sreq = \frac{0.75*200*420*260}{65*10^3} =$ **252 mm**

But , **Smax** ≤ d/2 → 260/2 = **130 mm« Cont**

Or , **Smax ≤ 600 mm**

∴ **Select Ø8/10cm ,4legs**

Note :
Assume Ø8 stirrups with 4 legs are used ,
then Av = 4 $* \frac{\pi * 8^2}{4}$ = 200 mm^2

3. Category (IV) :

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

→ $Ø \times \frac{1}{3} \times \sqrt{fc'} \times \mathrm{bw} \times \mathrm{d} = 0.75 \times \frac{1}{3} \times \sqrt{24} \times 1000 \times 260 \times 10^{-3} =$ **318.44 kN**

→ $Ø.\mathrm{Vc} + Ø \times \frac{1}{3} \times \sqrt{fc'} \times \mathrm{bw} \times \mathrm{d} = \mathbf{477.66\ kN} > \mathrm{Vu\ max} = \mathbf{287.7\ kN}$

$Sreq = \frac{Av*Fyt*d}{\mathrm{Vs}}$, where Vs = $\frac{Vu - Ø.\mathrm{Vc}}{Ø} = \frac{287.7 - 159.22}{0.75}$ = 171.3 kN

→ $Sreq = \frac{200*420*260}{171.3*10^3} =$ **127.5 mm « Cont**

But , **Smax** ≤ d/2 → 260/2 = **130 mm**

Or **Smax ≤ 600 mm**

∴ **Select Ø8/10cm ,4legs**

4.5. Design of Column (C14)

4.5.1. Calculation of Loads act on Column (C14)

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

Loads acting on column (C14) are as follows:

Dead Load = (Service Dead reaction from B10) + (Service Dead reaction from B30) + (Service Dead reaction from B38 x 6) + (Self weight of the column x 8)

= (455.5) + (454..8) + (432.9 x 6) + (0.6 x 0.6 x 3 x 25 x 8) = 3724 kN

Live Load =(Service Live reaction from B10) + (Service Live reaction from B30) + (Service Live reaction from B38 x 6)

=(83) + (83) + (83 x 6) =664 kN

Factored loads (Pu) = 1.4 DL = 1.4 x 3724 = 5213 kN.

OR Pu = 1.2 DL + 1.6 LL = 1.2 x 3724+ 1.6 x 664 = **5530 kN** << Cont.

4.5.2. Calculation of Required Dimension of Column (C14)

Total load Pu =5530 KN

Pn = 5530 /(0.65) = 8508 KN

Assume $\rho g = 2.0\ \%$

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

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

$$Ag = 0.37\ m^2$$

∴**Select 60*60cm with Ag = 3600 cm².**

- **Check Slenderness Effect :**

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

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

Where :

Lu: Actual unsupported (unbraced) length = 3.18 m

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

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

System about X

$\rightarrow \lambda = \frac{1 * 3.18}{0.3 * 0.6} = 17.67$

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

$\lambda = 17.67 > 22$ ∴ Short about X .

System about Y

$\rightarrow \lambda = \frac{1 * 3.18}{0.3 * 0.6} = 17.67$

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

$\lambda = 17.67 > 22$ ∴ 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 :

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

Where , Pu =5530 KN

$5530 * 10^3 = 0.65 * 0.8 * 600 * 600\{0.85 * 24 + \rho\text{g}(420 - 0.85 * 24)\}$

$\Rightarrow \rho_g = 0.021 > \rho_{min} = 0.01 \ \& < \rho_{max} = 0.08$

As req = 0.021 *600 *600 =7560 mm^2

Use Φ 25 >> $\# \, of \, bar = \frac{7560}{491} = 15.4$

∴ **Use 16 Ø 25 with As = 7856 mm^2 >As $_{req}$ = 7560 mm^2**

- **Check spacing between the bars :**

$S = \frac{600-2*40-2*10-5*25}{4} = 93.75\ mm$

$S = 93.75\ \text{mm} \geq \frac{4}{3}\text{M.A.S}$

$\geq 40mm$

$\geq 1.5\text{db} = 37.5\ mm$

4.5.4. Determination of Stirrups Spacing

<u>According to ACI :</u>

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

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

$\text{S} \leq$ Least dimension.

$\text{Spacing} \leq 16 \times \text{d}_\text{b}(\text{Longitudinal. bar. diameter}) = 16 \times 2.5 = 40\text{cm}.$

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

$\text{Spacing} \leq \text{Least. dim e nsion} = 60\ \text{cm}$

∴ **<u>Select Ø 10/20cm</u>**

Column (C14) Section is shown in figure(4-11) where bars arrangmenet and stirrups detailing appear :

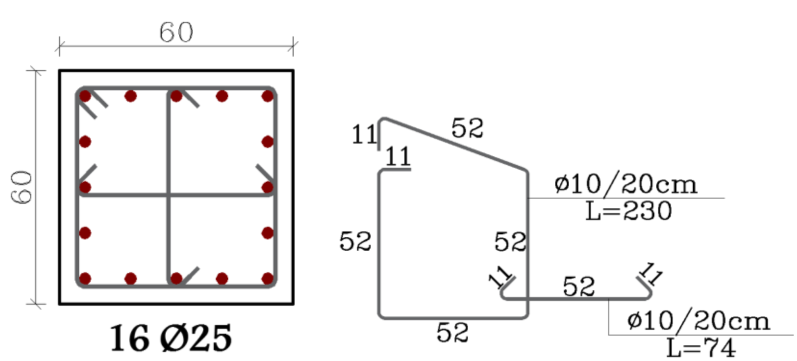


Figure (4- 11): C14 Reinforcement Details

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 :

- Shear Wall thickness = h = 20 cm
- Shear Wall length Lw = 3.4 m
- Building height Hw=27.5 m
- Critical section shear : Lw<hw → d =0.8*Lw =2.72 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- $Vc = \frac{1}{6}\sqrt{fc'} \times b \times d$

$= \frac{1}{6}\sqrt{24} \times 200 \times 2720 = \mathbf{444.17\ kN} \ll$ Controlled

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

$= \frac{\sqrt{24} \times 200 \times 2720}{4} + 0 = 666.26\ kN$

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

Where :

- Mu1=415.9 kN.m

- $\frac{Mu1}{Vu} - \frac{Lw}{2} = \frac{415.9}{670.52} - \frac{3.4}{2} = -1.08 < 0$ → This equation is not applicable .

$\therefore Vc = 444.17$ kN → $\emptyset$Vc < Vumax[1] = 670.52 kN →Horizontal Reinforcement is Required.

$\rightarrow Vs = \frac{Vu}{\emptyset} - Vc = \frac{670.52}{0.75} - 444.17 = 449.86$ kN

$\rightarrow \frac{Avh}{s} = \frac{Vs}{fy*d} = \frac{449.86*10^3}{420*2720} = 0.394$

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

[1] For shear and moment diagrams see appendix C

→ Avh : For 2 layers of Horizontal Reinforcement

Select Ø10 :

Avh = 2 *79 = 158 mm^2

$$\frac{Avh}{s} = 0.5 \rightarrow Sreq = \frac{158}{0.5} = 316\ mm$$

Smax= Lw/5 = 3400/5 = 680 mm

= 3h = 3*200 =600mm

$= 45cm \ll$ Controlled.

∴Select Ø10 @ 250 mm at each side .

4.6.2. Design of Vertical Reinforcement

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

$$\frac{hw}{lw} = \frac{27.5}{3.4} = 8.09 > 2.50$$

→ $\frac{\text{Avv}}{\text{Sver}}$ = [0.0025 +0.5 (0)($\frac{2*79}{250*200} - 0.0025$)] $* 200$

∴ $\frac{\text{Avv}}{\text{Sver}}$= 0.5

Smax = Lw/3 = 3400/3 = 1133 mm

= 3h = 3*200 =600mm

$= 450\ mm \ll$ Controlled.

Select Ø12 :

Avv = 2 *113 = 226 mm^2

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

∴Select Ø12 @ 200 mm at each side .

4.6.3. Design of Bending Moment

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

→ Max Mu =1975.7 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 *$\frac{3400}{200} = 3842$ mm^2
- $\frac{Z}{Lw} = \frac{1}{2+\frac{0.85*\beta 1*fc'*Lw*h}{Asv*fy}} = \frac{1}{2+\frac{0.85*0.85*24*3400*200}{3842*420}} = 0.107$

∴ Muv = 0.9 [0.5*3842*420*3400 (1- $\frac{0.107}{2}$)] = 2336.78 kN.m

Muv = 2336.78 kN.m > Mu =1975.7 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.

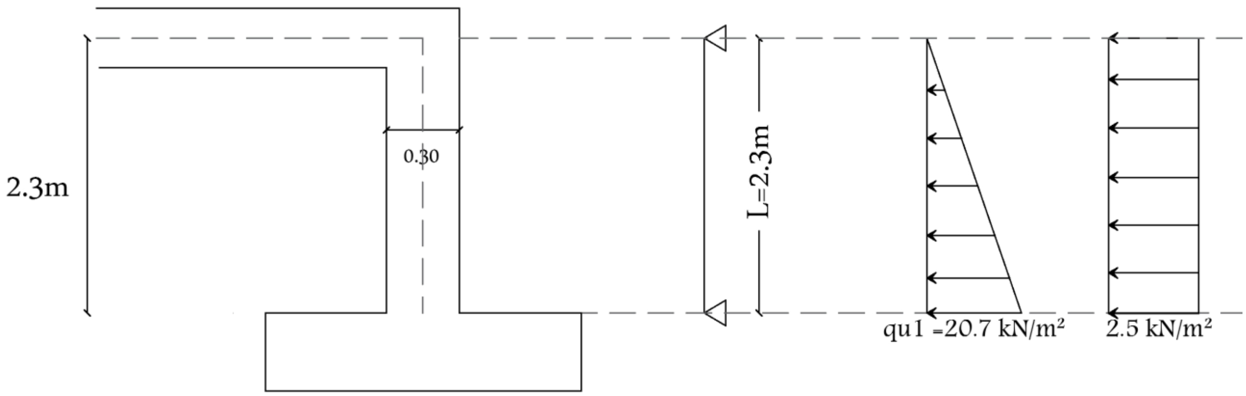


Figure (4- 12):Basement Wall system and loads

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*2.3=20.7 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.

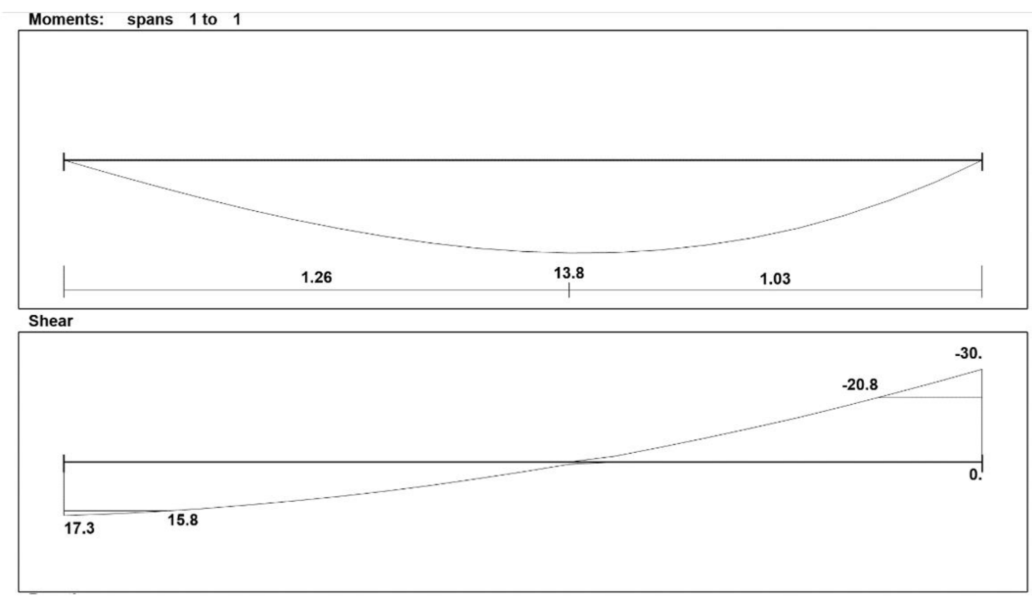


Figure (4- 13): Moment and Shear Envelope of Basement wall

4.7.2. Design of Shear Force

Max value shear force is obtained from figure(4-13) , Vu= 20.8 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 = 13.8 kN.m

→ $m = \frac{420}{0.85*24} = 20.6$

→ Mn = 13.8 /0.9= 15.3 kN.m

→ $kn = \frac{Mn}{b*d^2} = \frac{15.3*10^6}{1000*260^2} = 0.23$ MPa

→ $\rho = \frac{1}{20.6} * (1- \sqrt{1 - \frac{2*0.23*20.6}{420}}) = 0.00055$

→ $Asreq = \rho * b * d = 0.00055 * 1000 * 260 = 143$ 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=65.56 kN/m & qL=5.15 kN.m
- Total Service Loads : qtot =65.56+5.15 = 70. 71 kN/m
- Total Factored Loads : qu =1.4 *65.56 = 91.78 kN/m

4.8.1. Check if footing width is correct

$$\sigma_b = {qtot}/{Areq} \leq \sigma_{b\,(allow\,.\,net)}$$

$$\therefore \ \mathbf{70.71}/{1.0 * 1.0} = 70.71 < \sigma_{b(allow\,.\,net)} = 400 \text{ kN/m}^2$$

∴ **a=1.0m is correct#**

4.8.2. Design of one way shear

→ Assume h =30cm
→ d=300-50-20 = 230 mm
→ Vu=91.78*0.12*1m = 11 Kn
→ $Ø * Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 230 = 140.8 \text{ kN} > Vu$
∴ **h =30 cm (SAFE) .**

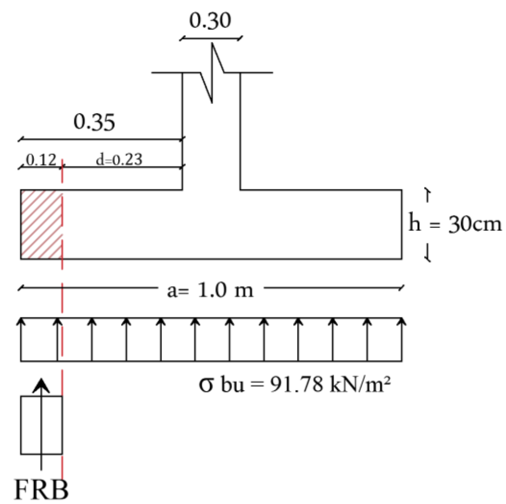


Figure (4- 14): Critical Section of Shear force

4.8.3. Design of Bending Moment

➢ **Main Steel:**

Mu =91.78*0.35*1*(0.35/2) = 5.62 kN.m

→ Mn = 5.62 /0.9= 6.94 kN.m
→ $kn = \frac{Mn}{b*d^2} = \frac{6.94*10^6}{1000*230^2} = 0.13 \text{ MPa}$
→ $\rho = \frac{1}{20.6} * (1- \sqrt{1 - \frac{2*0.13*20.6}{420}}) = 0.000314$
→ $Asreq = 0.000314 * 1000 * 230 = 72 \text{ mm}^2/\text{m}$
→ As (min) = 0.0018*1000 * 300 = 540 mm^2/m

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

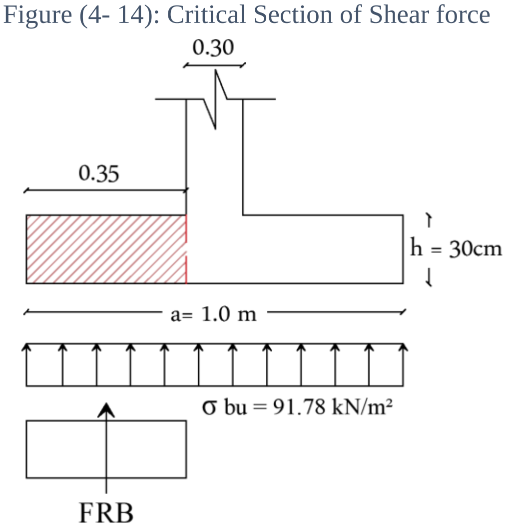


Figure (4- 15):Critical Section of Bending Moment

➢ **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.

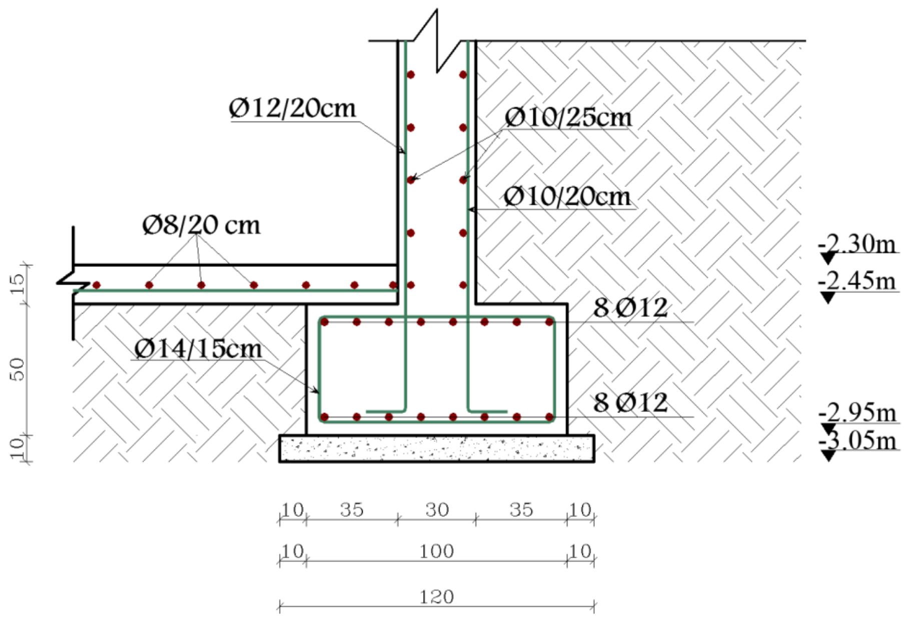


Figure (4- 16): Basement wall Reinforcement Details

4.9. Design of Isolated Footing (F5)

Loads that act on footing F5 are :

- PD =3724 kN , PL = 664 kN → Pu = 1.2 * 3724 + 1.6*664 = 5530 kN

The following parameters are used in design :

- γ $_{concrete}$ = 25 kN/m^3
- γ $_{soil}$ = 18 kN/m^3
- σ_{allow} = 400 kN/m^2
- clear cover =5cm

4.9.1.Determination of footing dimension (a)

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

→ Assume h = 85 cm

→ $\sigma_{b(allow)net}$ = 400 – 25*0.85 = 378.75 kN/m²

→ σ $_{bu(\text{ allow . net})}$ = 1.4* 378.75 = 530.25 kN/m²

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

∴ $5530/a^2$ = 530.25 → **a=3.23m** → **Select a=3.3m**

→ $Bearing\ Pressure\ \sigma bu$ = $\text{Pu}/\text{A} = \frac{5531.2}{3.3*3.3}$ = **508 kN/m^2** ≤ 530.25 kN/m² **…. (SAFE)**

4.9.2. Determination of footing depth (h)

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

4.9.2.1. Design of one way shear

→ $d = \text{h} - cover - \emptyset = 850 - 50 - 16 = 784\ mm$

→ Vu at distance d from the face of column :

Vu = $FRB = \sigma bu \times 0.566 \times b$

= 508 × 0.566 × 3.3 = 949 kN

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

$= 0.75 * \frac{1}{6} * \sqrt{24} * 3300 * 784$ =1584.33 kN

>Vu

∴ **h = 85 cm is correct ✓**

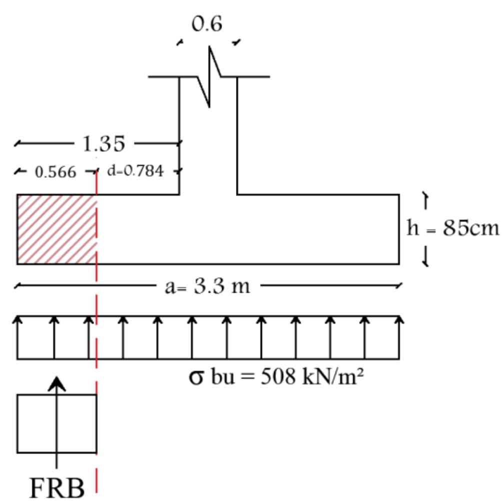


Figure (4- 17): Critical Section of Shear Force

4.9.2.2. Design of Punching (two way shear)

→ $d = 784\ mm$

→ $b_o = 4 \times 1384 = 5536$ mm

→ Bc = 1

→ αs = 40 (interior column)

Vu = 5531.2 – (508 *1.384*1.384) = **4558.15 kN**

<u>Ø×Vc is the smallest of :</u>

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

$= \left(2+\frac{4}{1}\right)\times\frac{\sqrt{24}}{12}\times 5536\times 784$

$= 10631.33$ kN

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

$= \left(\frac{40\times 784}{5536}+2\right)\times\frac{\sqrt{24}}{12}\times 5536\times 784$

$= 13581.1$ kN

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

$= 4\times\frac{\sqrt{24}}{12}\times 5536\times 784 =$ **7087.56 kN ..** ⮜ **cont.**

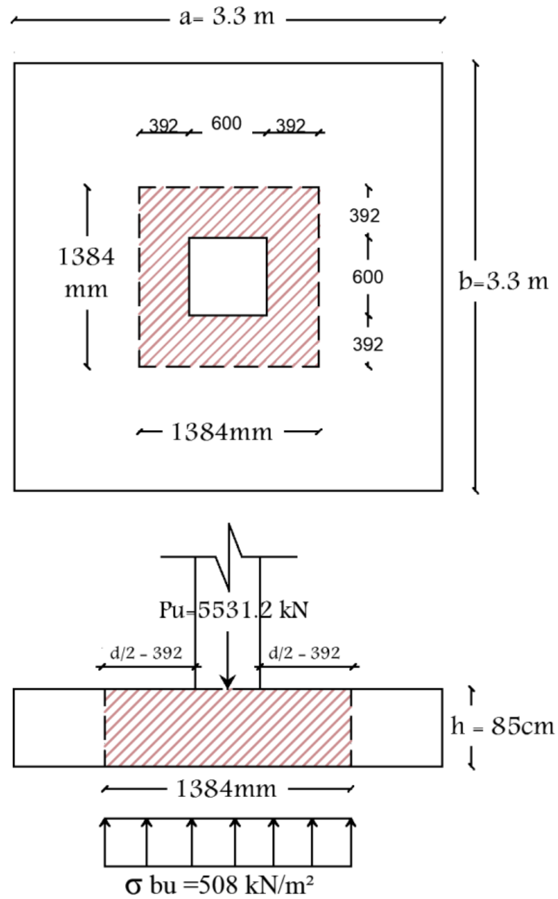


Figure (4- 18):Punching Shear Critical Section

→ Ø×Vc = 0.75 × 7087.56 = **5315.66** kN > Vu = **4558.15** kN

∴ <u>**h = 85 cm is correct ✓**</u>

4.9.3.Design of Reinforcement

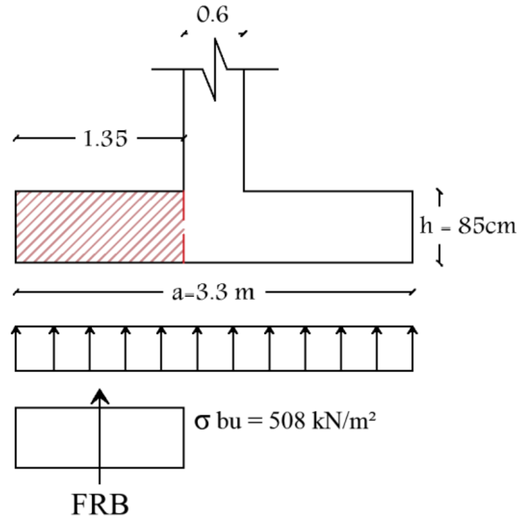


Figure (4- 19):Critical Section of Bending Moment

Mu = 508 *1.35*3.3*(1.35/2) = 1527.62kN.m

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

→ Mn = 1527.62/0.9= 1697.35 kN.m

→ $kn = \frac{Mn/\emptyset}{b*d^2} = \frac{1697.35*10^6}{3300*784^2}$ = 0.836 MPa

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

$= \frac{1}{20.6} * (1- \sqrt{1-\frac{2*0.836*20.6}{420}}) = 0.002033$

→ Asreq = $\rho * b * d = 0.002033 * 3300 * 784 =$ 5259.77 mm^2

→ As (min) = 0.0018*b*h = 0.0018*3300 * 850 = 5049 mm^2

→ Asreq > As (min)

∴ **Select for both directions: 27Ø16 with As = 5428.67 mm^2 > Asreq … (ok)**

4.9.4.Design the Connection between Column & Footing

→ **Design of bearing pressure at section of column :**
$\emptyset \times Pnb = 0.65 \times 0.85 \times fc' \times A1 \geq Pu$
$= 0.65 \times 0.85 \times 24 \times 600 \times 600 = 4773.6$ kN < Pu = 5531.2 kN
∴ **Dowels are required to transfer the load between column and footing**

→ **Design of Dowels :**
The dowels will carry the difference between ($\emptyset \times Pn$) and (Pu) .

- $Fy \cdot Asreq = \Delta P$
- $420 \cdot Asreq = \frac{(5531.2-4773.6)}{0.65}$
- $Asreq = 2775.1$ mm^2
- As min for dowels = $0.005 \times A1 = 0.005 \times 600 \times 600 = 1800$ mm²
- $Asreq$ > As min

∴ **Select 16Ø25 which is just like the reinforcement of column.**

→ **Check Compression lap splice between steel of column and dowels (Lsc) :**

$Lsc\ creq = 0.071 \times fy \times db = 0.071 \times 420 \times 25 = 745.5$ mm >300mm

$\therefore$ **Select $Lsc = 78.2$ cm > $Lsc\ req = 74.6$cm**

→ **Design of compression development length (Ldc) :**

- $Ldc = 0.24 \times \frac{\text{fy}}{\sqrt{\text{fc}'}} \times db = 0.24 \times \frac{420}{\sqrt{24}} \times 25 = 514.4$ mm .. ≺ cont.
- $Ldc = 0.043 \times fy \times db = 0.043 \times 420 \times 25 = 451.5$ mm

  $\therefore$ **Ldc req = 514.4 mm**

- $Available\ Ldc = 850 - 50 - 16 - 16 = \mathbf{768}\ mm > Ldc$ req $= 514.4\ mm$ .. ok

→ **Check tension development length using simplified method (Ldt) :**

Since we have a footing , it must satisfy two conditions to be considered under category A , otherwise it will considered as category B :

1- $Clear\ lateral\ spacing = \frac{3300-(2*50)-(27*16)}{26} = 106.46$ mm > 2db = 32mm ✓

2- $Clear$ cover = 50 mm > 1 db = 16 mm ✓

⇨ Category A

Design of tension development length (Ldt):

- Ld , req $= \frac{12}{20} \times \frac{\text{fy}}{\text{fc}'} \times \frac{\varphi t \times \varphi e}{\lambda} \times db = \frac{12}{20} \times \frac{420}{24} \times \frac{1 \times 1}{1} \times 25 = 262.5$ mm
- Ld , available $= \frac{3300-600}{2}$ - 50 = 1570mm > Ldt , req …. (ok)

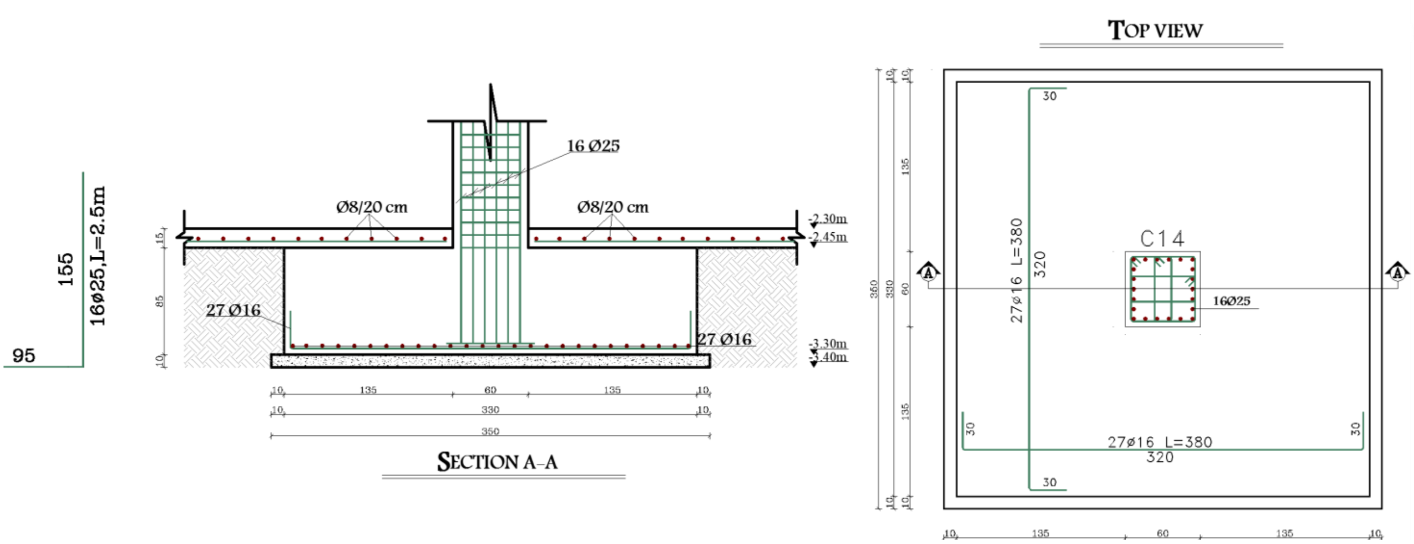


Figure (4- 20):F5 Reinforcement Details

4.10. Design of Stairs

The following figure shows a top view of the stairs :

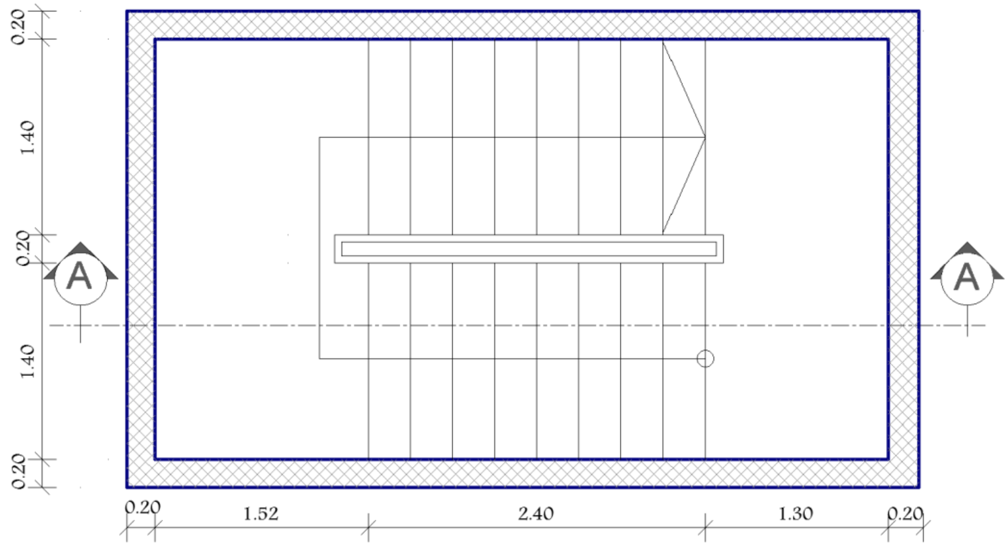


Figure (4- 21): Stairs Top View

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) = 16.7/30 → α = 29°

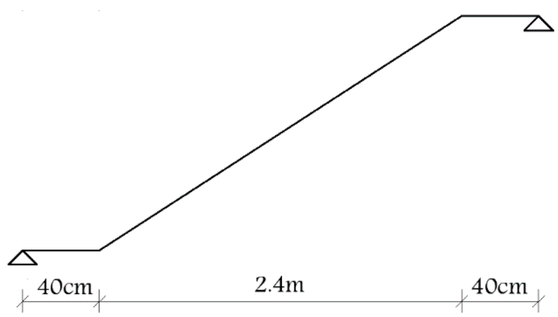


Figure (4- 22): Structural system of flight

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

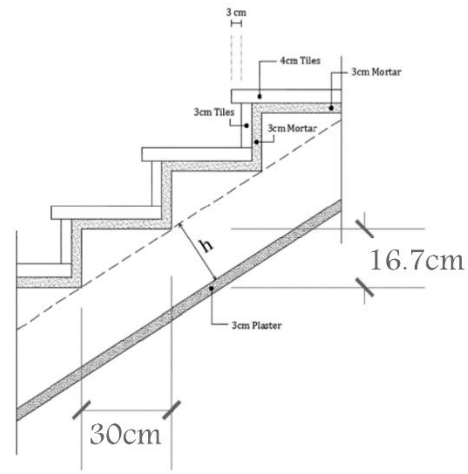


Figure (4- 23): Section of The Flight

Factored Loads :

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

Au= 14.2*2.4/2 = 17.52 kN

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

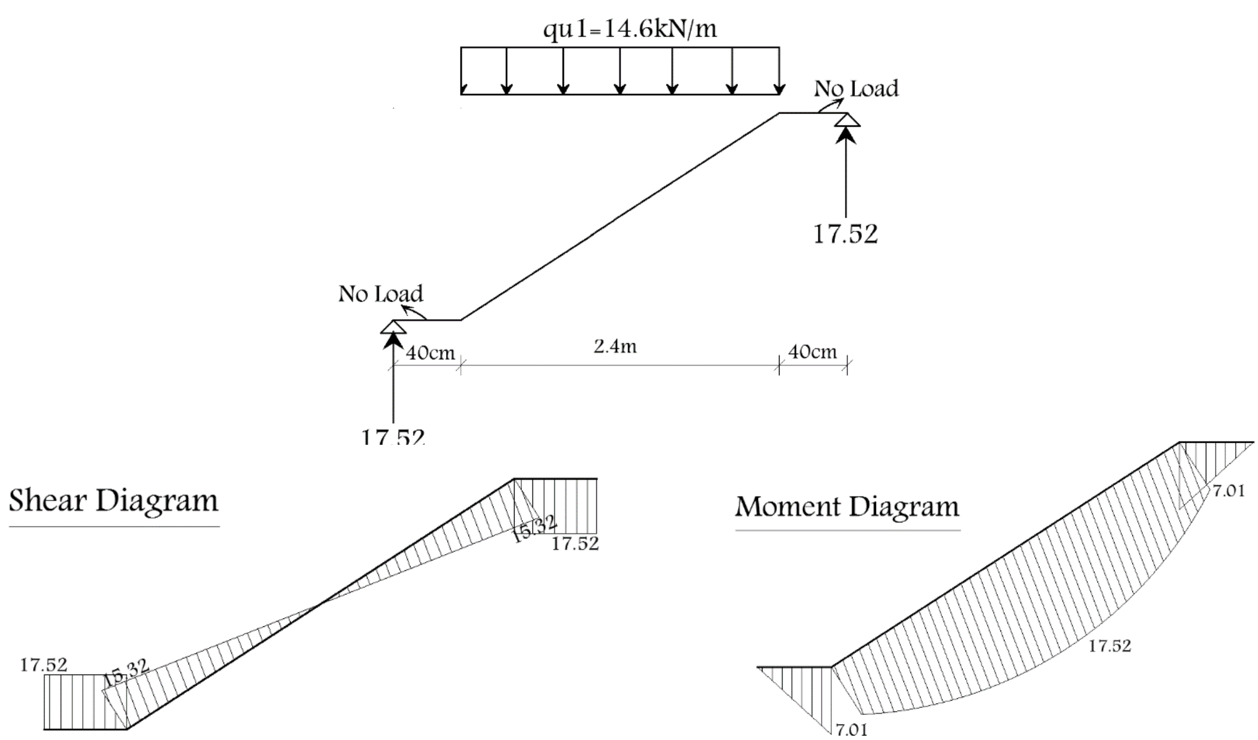


Figure (4- 24) :Analysis of the flight

4. Design :

- **Design of Shear Force :**

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

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

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

= 75.9 kN > Vu max = 15.32 kN

∴ No Shear Reinforcement is Required#

- **Design of Bending Moment :**

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

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

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

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

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

∴ Select **Ø12/20 with As = 565 mm²** > As req …. For Main Reinforcement

For secondary Reinforcement select **Ø10 /20 with As=395 mm²** = As min

→ Check Spacing :

20cm > **S min** = 2.5 + 1.0 = **3.5 cm** or 2*(1.0)= **2.0 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 = 300.6 *420

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

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

∴ $\boldsymbol{\varepsilon_s}$ **= 0.057 > 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:

  h ≥ minimum h

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

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

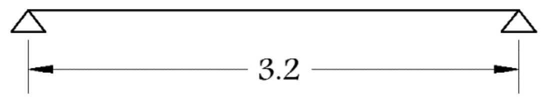


Figure (4- 25):Structural system of landing

- **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

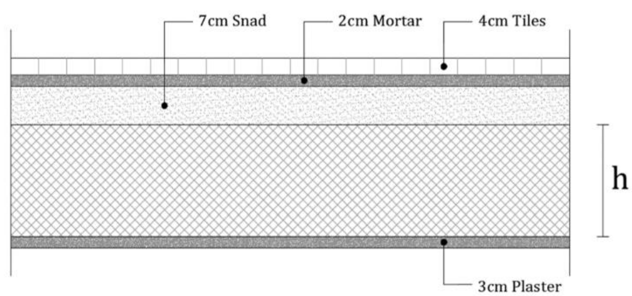


Figure (4- 26):Section of The Landing

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

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

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

→ **Analysis :**

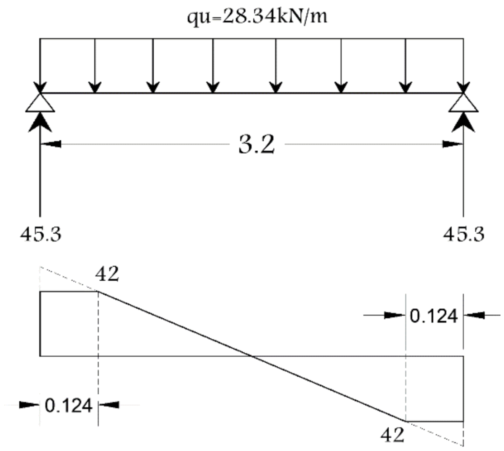


Figure (4- 27):Analysis of Landing

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

Vumax= 45.3 – (28.34*0.124) = **42 kN**

Mu max = $\frac{28.34*3.2^2}{8}$ = **36.3 kN.m**

→ **Shear Force Design :**

d=124mm & Vu max=42 kN

$Ø \times Vc = 0.75 * \frac{1}{6} * \sqrt{24} * 1000 * 124 = 75.9kN > Vu\ max = 42\ kN$

∴No Shear Reinforcement is Required#

→ **Bending Moment Design : (Mu max = 36.3 kN.m)**

- m = 20.6
- kn = $\frac{36.3 * 10^6/0.9}{1000*124^2}$ = 2.6 MPa
- $\rho = \frac{1}{19.6} * (1- \sqrt{1 - \frac{2*2.6*20.6}{420}}) = 0.0066$
- $Asreq = 0.0066 * 1000 * 124 = 818.4\ mm^2$
- As min = 0.0018 *1000*150 = 270 mm^2

∴ Select **Ø14 /15cm with As = $\frac{\pi*14^2}{4} * \frac{100}{15}$ = 1026 mm^2** > As req …. For Main Reinforcement

- Check Spacing :

  15cm > **S min** = 2.5 + 1.0 = **3.5 cm** or 2*(1.0)= **2.0 cm … ok**
  15cm < **S max** = 3 * 15 = **45 cm … ok**

- Check Strain:

$$C = T$$

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

$$0.85*24*a*1000 = 1026*420$$

$$a = 20.11 \text{ mm} \rightarrow \quad X = a/\beta = 20.11/0.85 = 23.66 \text{ mm}$$

$$\varepsilon_s = \frac{0.003*124}{23.66} - 0.003$$

$$\therefore \; \boldsymbol{\varepsilon_s = 0.012 > 0.005 \; \ldots. \; Ø = 0.9 \; (OK)}$$

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

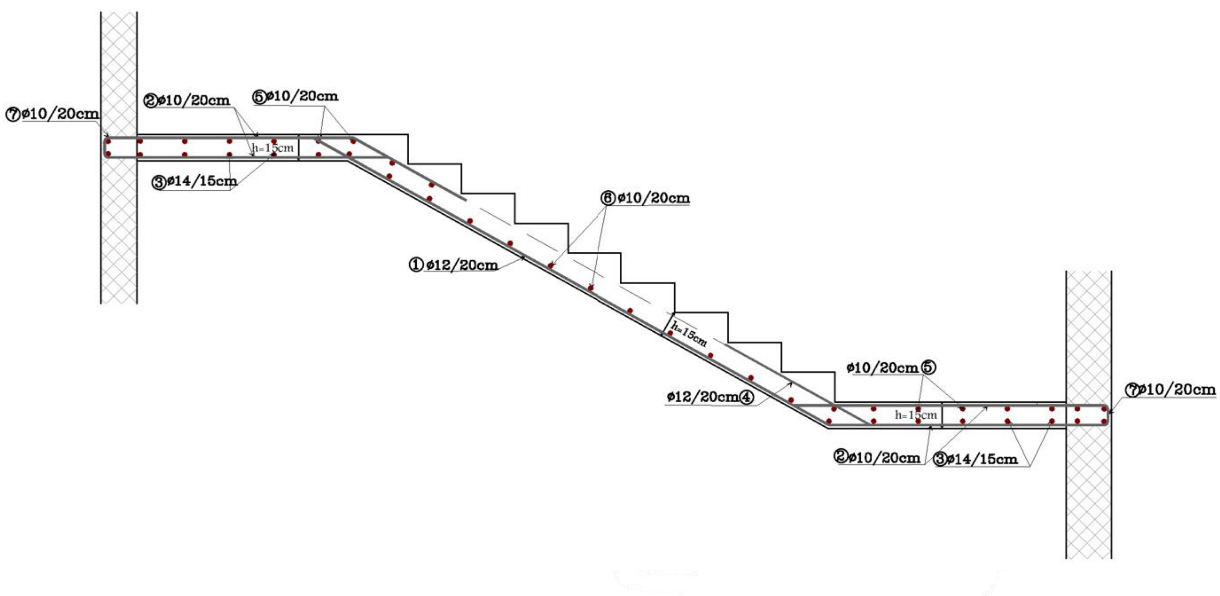


Figure (4- 28):Reinforcement Details of Stairs

CHAPTER 5

RESULTS AND RECOMMENDATIONS

5.1 INTRODUCTION

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 الأردني, كود الأحمال والقوى, عمان,الأردن: مجلس البناء الوطني الأردني, 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]	$C_t = 0.02ft$
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]

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

[ASCE 12.8.1.1, Eq. 12.8-3]

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

[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]

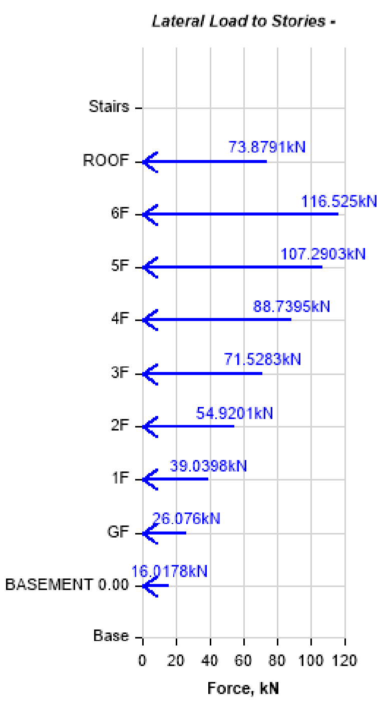
$$C_{S,min} = 0.5 \frac{S_1}{(\frac{R}{I})} \text{for} S_1 = 0.6g$$

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

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	255.5838	0
6F	21.52	405.6778	0
5F	18.82	376.2204	0
4F	16.12	313.7637	0
3F	13.42	255.405	0
2F	10.72	198.4772	0
1F	8.02	143.2979	0
GF	5.32	97.842	0
BF	2.62	62.4265	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
BASEMENT 0.00	P1	20918.2	15058	3195.4	200	0.818

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	19320.5	15058	22515.9	15058	3195.4	200
Bottom	Leg 1	19320.5	15058	22515.9	15058	3195.4	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	1598	0.0025	0.006	DWal26	99.5591	-1.5205	-45.6601	639085
Bottom	2720	0.0043	0.006	DWal19	-201.2612	1.4019	942.9165	639085

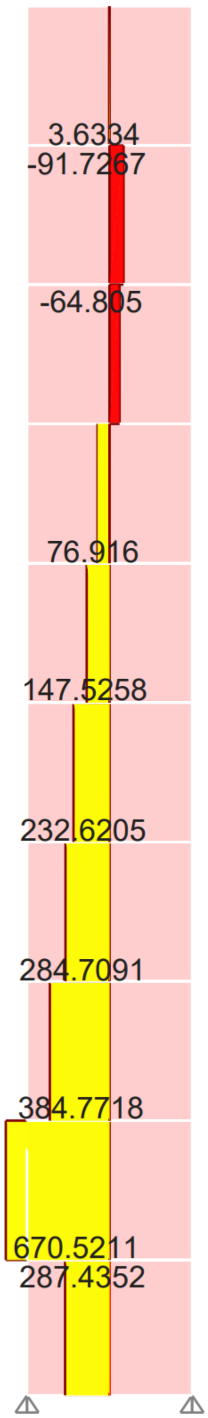
Shear Design

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	DWal7	106.7881	100.2842	287.4352	530.7592	927.3293
Bottom	Leg 1	500	DWal7	161.2061	853.3644	287.4352	538.9219	935.492

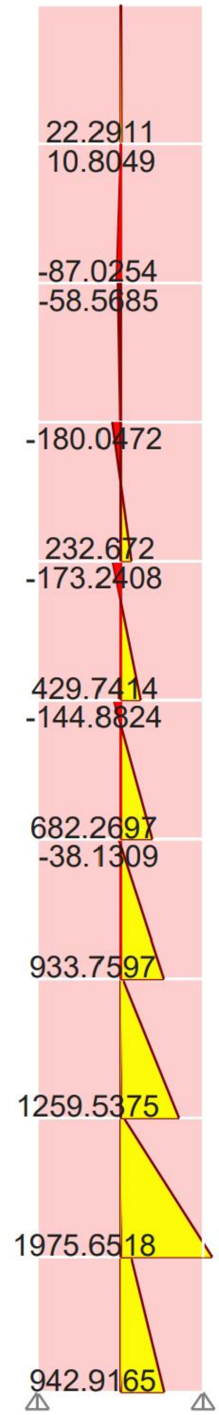
Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

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	295.2	DWal8	1477.1884	-665.2223	4.27	4.8	590.4	710.1
Top–Right	Leg 1	Not Required	DWal8	106.7881	100.2842	0.46	4.8		
Bottom–Left	Leg 1	303	DWal8	1531.6065	-1261.1963	4.31	4.8	606.1	710.1
Botttom–Right	Leg 1	Not Required	DWal8	663.6726	73.801	1.26	4.8		

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 irregulation check for output case EQX+5

Story	Output Case	Ratio	Ok?
ROOF	EQX+5	1.05	OK
6F	EQX+5	1.06	OK
5F	EQX+5	1.06	OK
4F	EQX+5	1.07	OK
3F	EQX+5	1.08	OK
2F	EQX+5	1.08	OK
1F	EQX+5	1.08	OK
GF	EQX+5	1.07	OK
BF	EQX+5	1.06	OK

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

Story	Output Case	Ratio	Ok?
ROOF	EQX-5	1.1	OK
6F	EQX-5	1.11	OK
5F	EQX-5	1.12	OK
4F	EQX-5	1.13	OK
3F	EQX-5	1.14	OK
2F	EQX-5	1.14	OK
1F	EQX-5	1.15	OK
GF	EQX-5	1.14	OK
BF	EQX-5	1.03	OK

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

Story	Output Case	Ratio	Ok?
ROOF	EQY+5	1.03	OK
6F	EQY+5	1.03	OK
5F	EQY+5	1.01	OK
4F	EQY+5	1.05	OK
3F	EQY+5	1.09	OK
2F	EQY+5	1.14	OK
1F	EQY+5	1.2	OK
GF	EQY+5	1.29	OK
BF	EQY+5	1.12	OK

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

Story	Output Case	Ratio	Ok?	Eccentricity length
ROOF	EQY-5	1.3	N.Ok	0.85
6F	EQY-5	1.73	N.Ok	1.51
5F	EQY-5	1.76	N.Ok	1.56
4F	EQY-5	1.78	N.Ok	1.60
3F	EQY-5	1.8	N.Ok	1.63
2F	EQY-5	1.79	N.Ok	1.62
1F	EQY-5	1.75	N.Ok	1.53
GF	EQY-5	1.58	N.Ok	1.25
BF	EQY-5	1.01	Ok	0.51

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	…	…

Table(6):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	114215.701	2.51	…
6F	EQX-5	286671.614	1.52	…
5F	EQX-5	434678.223	1.30	2.02
4F	EQX-5	563827.633	1.23	1.62
3F	EQX-5	694359.954	1.23	1.52
2F	EQX-5	856069.109	1.30	1.58
1F	EQX-5	1115745.32	…	…
GF	EQX-5	0	…	…
BF	EQX-5	0	…	…

Table(7):Stiffness check for output case EQY+5

Story	**Output Case**	**Stiff Y kN/m**	Ki/ki+1	Ki/(Ki+1+Ki+2 ki+3)/3
ROOF	EQY+5	276080.418	2.79	…
6F	EQY+5	769410.262	1.52	…
5F	EQY+5	1171197.04	1.30	2.07
4F	EQY+5	1528258.08	1.24	1.64
3F	EQY+5	1890911.44	1.23	1.52
2F	EQY+5	2330384.68	1.29	1.56
1F	EQY+5	2998255.2	…	…
GF	EQY+5	0	…	…
BF	EQY+5	0	…	…

Table(8):Stiffness check for output case EQY-5

Story	**Output Case**	**Stiff Y kN/m**	Ki/ki+1	Ki/(Ki+1+Ki+2 ki+3)/3
ROOF	EQY-5	251929.648	3.08	…
6F	EQY-5	775716.385	1.52	…
5F	EQY-5	1181186.07	1.31	2.09
4F	EQY-5	1541852.72	1.24	1.64
3F	EQY-5	1909194.64	1.23	1.53
2F	EQY-5	2357534.3	1.29	1.57
1F	EQY-5	3048462.47	…	…
GF	EQY-5	0	…	…
BF	EQY-5	0	…	…

c. Mass Check

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

Table(9):Mass Check

Story	**Mass X** kg	**Mass Y** kg	mi/mi+1	mi/mi-1
ROOF	61767.6	88767.6	1.48	
6F	131302.9	131302.9	1.08	0.47
5F	141565.5	141565.5	1.00	0.92
4F	141565.5	141565.5	1.00	1
3F	141565.5	141565.5	1.00	1
2F	141565.5	141565.5	1.00	1
1F	141565.5	141565.5	1.11	1
GF	157614.36	157614.36	0.82	0.89
BF	128942.97	128942.97	0.00	1.22