



**Palestine Polytechnic University**  
**Faculty of Engineering and Technology**  
Departement of Civil Engineering and Architecture  
Specialization in Civil Engineering Branch Building Engineering

**Project Name**

## **Structural Design of General Hospital in Nablus**

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# DEDICATION

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

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

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

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

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

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- *Everyone who extended a hand in the project and was of great help.*

## الملخص

تقوم فكرة المشروع على اساس التصميم الانشائي لمستشفى عام في نابلس. تم تطوير المشروع ليشتمل على جميع متطلبات العمل الضرورية للمستشفى النموذجي حيث ان هناك عدد كافي من غرف النوم، المكاتب، وحدات العلاج الفيزيائي وجميع المتطلبات المرافقة لذلك. تقوم فكرة التصميم على ايجاد الحلول المعمارية والانشائية المناسبة، لذلك يجب مراعاة استخدامات المشروع مما يوفر الخدمة والراحة للمستخدمين.

يتكون المشروع من ٨ طوابق بمساحة تقديرية تبلغ ٣٩٠٠٠ متر مربع تقريبا. وتكمن الناحية الجمالية للمشروع في تراجعات الطوابق المختلفة بالإضافة الى اهتمامه بالطبيعة الامر الذي يضيف رونق معماري جميل.

من الجدير بالذكر ان المشروع سيعتمد في التصميم على الكود الامريكي، بالإضافة الى الكود الاردني لحساب الاحمال، مستعينا ببعض البرامج الحاسوبية المخصصة للتصميم الانشائي.

# Abstract

The idea of this project is the structural design of general hospital in Nablus. The project has been developed to contain all the necessary requirements to work as a typical hospital, where there is a sufficient number of bedrooms, offices, nurses and doctors bedrooms, pharmacy, mechanical and electrical rooms, obstetrics and gynecology and all services needed.

The project design provides optimal architectural and structural solutions, so that it is taking into account the aesthetic and functional purposes and provides comfort use. The project will include the construction elements of slabs, beams, columns, and foundations.

The building consists of eight floors including basement floor with a total area of 39000  $m^2$  approximately. The design will be based on the requirements of the American Code (ACI -318-08)[1], and the Jordanian Code of loads, and we will use some of software like (latex) and (AutoCAD 2015) to project output, and also we use some of programs for structural design and analysis like, Atir, Safe.

After completion of the project we expect to be able to provide structural design of all the structural elements of the project accordance to the requirements of the code.

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# List of Abbreviations

<b>As</b>	area of non-prestressed tension reinforcement.
<b>As'</b>	area of non-prestressed compression reinforcement.
<b>Ag</b>	gross area of section
<b>Av</b>	area of shear reinforcement
<b>b</b>	width of compression face of member
<b>Cc</b>	compression resultant of concrete section.
<b>DI</b>	dead loads.
<b>d</b>	distance from extreme compression fiber to centroid of tension reinforcement.
<b>Fc</b>	compression strength of concrete.
<b>Fy</b>	nspecified yield strength of non-prestressed reinforcement.
<b>h</b>	overall thickness member.
<b>LL</b>	live loads.
<b>M</b>	bending moment.
<b>Mu</b>	factored moment at section.
<b>Mn</b>	nominal moment.
<b>Pn</b>	nominal axial load.
<b>Pu</b>	factored axial load.
<b>S</b>	spacing of shear in direction parallel to longitudinal reinforcement.
<b>Vc</b>	nominal shear strength provided by concrete.
<b>Vn</b>	nominal shear stress.

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<b>V<sub>s</sub></b>	nominal shear strength provided by shear reinforcement.
<b>V<sub>u</sub></b>	factored shear force at section.
<b>W<sub>c</sub></b>	weight of concrete.(Kg/m <sup>3</sup> )
<b>W</b>	width of beam or rib.
<b>W<sub>u</sub></b>	factored load per unit area.
$\phi$	strength reduction factor.
$\epsilon_c$	compression strain of concrete.
$\epsilon_s$	strain of compression steel.
$\rho$	ratio of steel area.
$\epsilon_c$	compression strain of concrete=0.003mm /mm
<b>F<sub>sd,r</sub></b>	total additional tension force above the support.
<b>V<sub>ed,0</sub></b>	shear force at critical section.
<b>N<sub>ed,0</sub></b>	normal tension force at support.
$\alpha$	angel of stirrup.

# Chapter 1

## Introduction

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

All of us knows the importance of hospitals, in the treatment of patients and provide them health services, especially here in Palestine that are under occupation. Which has recently become the number of hospitals in Palestine currently it is unable to provide health services as should be. So it was necessary to think of a hospital includes many functions that supporting the health care in the city, and that contribute to progress the health services in Palestine.

### 1.1 General Identification

The project is a general hospital in Nablus city, it provides all requirements needed for suitable work-place like Nurses and Doctors Offices, Bedrooms, Cafeteria, Pharmacy, Outpatient Clinics, Kitchen, Physical Treatment, Morgue, Operations Rooms, Central Administration and all services needed.

## **1.2 Reasons of Choosing Project**

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

## **1.3 The Project Objectives**

The objectives of this project is divided into architectural and structural objectives.

### **1.3.1 Architectural Objectives**

The main architectural aim is to create a design that is unique in views, representative and break the lack of architectural that Palestine suffers. So the hospital is designed upon the latest architectural views.

### **1.3.2 Structural Objectives**

The structural objectives of this project are:

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

## **1.4 The Problem of Project**

The problem of the project is to find the most appropriate structural system that satisfies the strength and serviceability requirements, and to design and analyze the structural components that consists

the project which is the General Hospital, so we will analyses and design these components like slabs, beams, columns, foundation,etc. After determining the loads on each of the structural member so we can select the required dimensions and reinforcement, after that all of the deign outputs will be presented in the structural drawing that used to transfer the project from being a drawing to the practical field.

### **1.5 The Postulate of the Project**

Our study aims to prepare the required structural drawings of the various structural members existing in the building in such a way that takes the architectural design as the main outlet. The design will be based on the requirements of the American Code (ACI -318-08), and the Jordanian Code of loads.

### **1.6 Chapters of Project**

**Chapter One:** General introduction of the project.

**Chapter Two:** The architectural description of the project.

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

**Chapter Four:** The structural analysis and the design of some structural members like: beams, and slabs.

**Chapter Five:** Results and recommendations.

## 1.7 The Time Table of the Project Stages

The Table below shows the time line table of the project stages.

Table 1.1: The Time Line Table of the Project Stages

Suggested Time	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34		
Project Selection	█	█	█	█	█	█	█																													
Site Study							█	█	█																											
Collect information about the project									█	█																										
Architectural study of the building									█	█	█																									
Structural study of the building										█	█	█																								
Preparation of graduation project introduction														█																						
Make the presentation															█																					
Structural analysis																█	█	█	█																	
Structural design																				█	█	█	█	█	█	█	█	█	█	█	█	█	█	█	█	█
Preparation of construction drawings of the project																					█	█	█	█	█	█	█	█	█	█	█	█	█	█	█	█
Writing the document																												█	█	█	█	█	█	█	█	█
Stand by time																																				
Presentation of the project																																			█	█

# Chapter 2

## Architectural Description

The soul of architecture is to design a structure that is suitable for humans to live in, work in, play in, etc. It is also to give comfort to its residents in order to make them feel comforted, uplifted, and feel that the structure is designed to appeal their requirements of leisure and enjoyment. A good architect does more than just designing buildings, yet he or she understands how people's environment affects their feelings in order to create an atmosphere that meets their needs and desires.

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

### 2.1 Basic Identification of Project

The idea of the project is establishment of General Hospital building in Nablus, so that takes into account all services, and all of comfort and security needed.

The building consists of eight floors including basement floor with a total area of 39000  $m^2$  approximately on an area (12223)  $m^2$  approximately. The project is featured by many declines which adds special architecture beauty to the structure, also by its integration with nature in general.



Figure 2.1: General Picture of Project.

## 2.2 Project Site

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

### 2.2.1 Project Land Location

The project is located in the eastern region of the city of Nablus (near Rujeb and doctor housing).



Figure 2.2: The location of the Project.

### **2.2.2 General Climate of the City**

This area generally enjoys a Mediterranean Climate of a dry summer and mild, rainy winter with occasional snowfall. The recorded average of Nablus's rainfall is about 660 mm (26 in).

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

### **2.2.3 Contour Lines of the Project Land**

- The project land is semi flat area (slope of 3 degree).
- The project land is 460 m above sea level.

## **2.3 Project Components Description**

The designer used many declines which adds special architecture beauty to the structure.

### **2.3.1 Project Plans Description**

The total area of building is 39239  $m^2$ .

#### **1. Basement Floor Plan**

The area of this floor is 7710  $m^2$ , and its consist of Mechanical and Electrical room, Laundry, X- ray rooms, Kitchen, Physical treatment, Offices, and all services needed.



Figure 2.3: Basement Floor.

## 2. Ground Floor Plan

The area of this floor is  $5785 m^2$ , and its consist of Offices, Medicine room, Emergency, Out-patient Clinics, Gift Shop.



Figure 2.4: Ground Floor.

## 3. First Floor Plan

The area of this floor is  $5439 m^2$ , and its consist of Administration department, Conference hall, Sterile department, Operation department.



Figure 2.5: First Floor.

#### 4. Second Floor Plan

The area of this floor is  $5810 m^2$ , and its consist of Incubators department, Taking blood department, Biochemistry department, Bedrooms department, Offices.



Figure 2.6: Second Floor.

#### 5. Third Floor Plan

The area of this floor is  $4366 m^2$ , and its consist of Bedrooms department, Offices, Medicine room, and all services needed.



Figure 2.7: Third Floor.

### 6. Forth Floor Plan

The area of this floor is  $4500 m^2$ , and its consist of Bedrooms department, Offices, Medicine room, and all services needed.

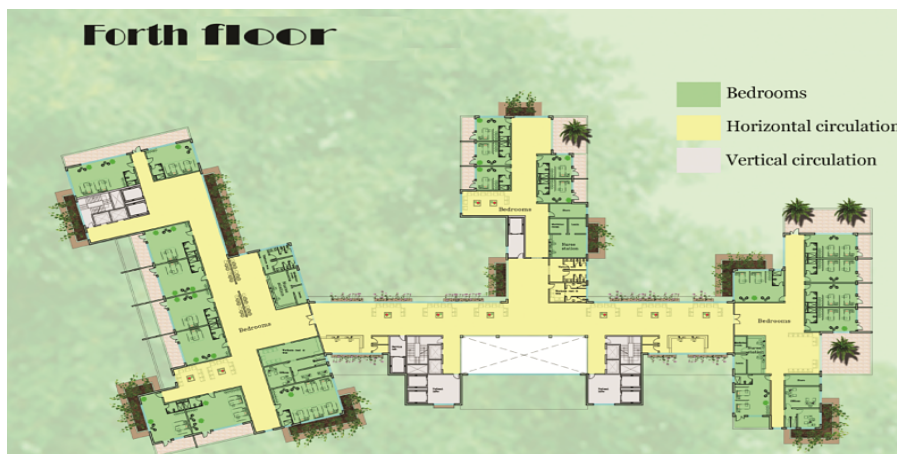


Figure 2.8: Forth Floor.

### 7. Fifth Floor Plan

The area of this floor is  $4500 m^2$ , and its consist of Bedrooms department, Offices, Medicine room, and all services needed.

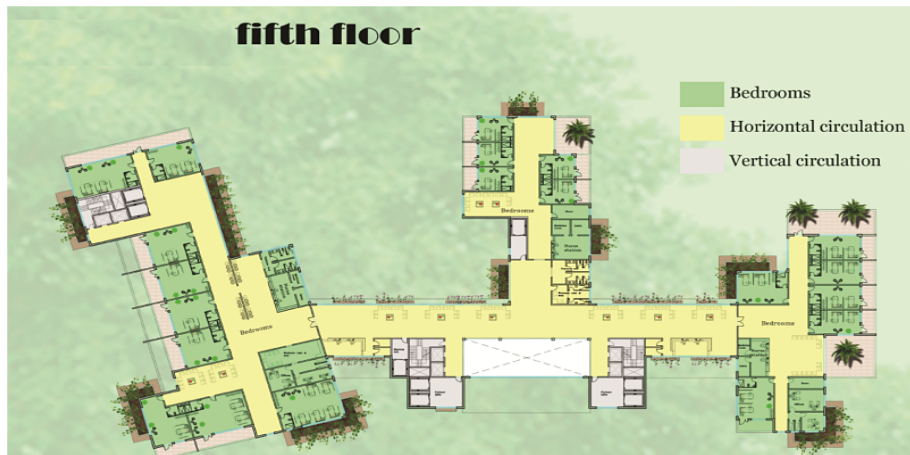


Figure 2.9: Fifth Floor.

### 8. Sixth Floor Plan

The area of this floor is 1300  $m^2$ , and its consist of Bedrooms department, Offices, and all services needed.

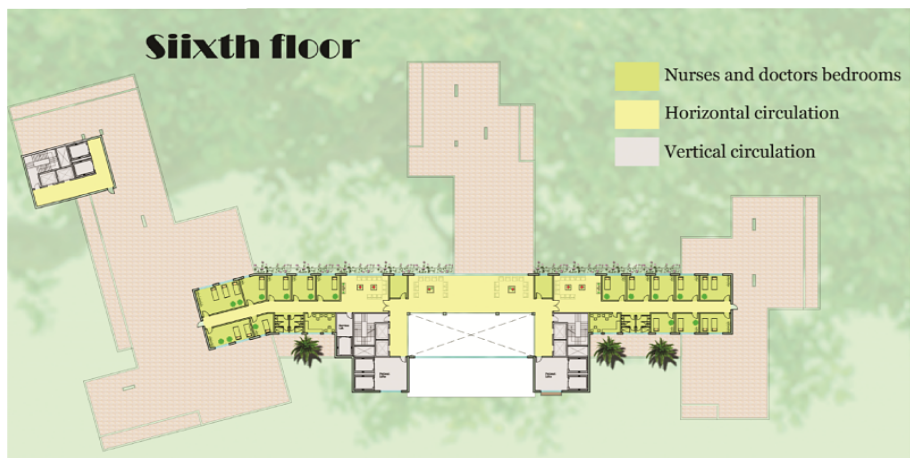


Figure 2.10: Sixth Floor.

### 2.3.2 Project Elevations Description

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

### 1. North Elevation

The main elevation and the main entrance of the building, we can observe the beauty of this elevation that contain glass integrated with stone.



Figure 2.11: North Elevation.

### 2. East Elevation

This elevation shows the inclination of the road, and we can observe the integration between glass and stone in this elevation, and we can observe the setbacks in the building.

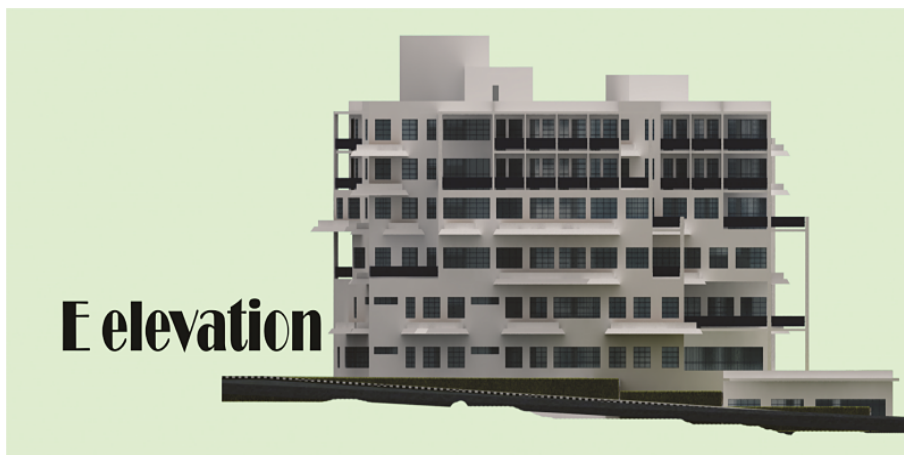


Figure 2.12: East Elevation.

### 3. West Elevation

The back side elevation, in this elevation we can observe some part of basement floor, and this elevation show some of columns, and we can observe the setbacks between floors.



Figure 2.13: West Elevation.

### 4. South Elevation

This elevation shows the entrance to the basement floor, here we can observe two entrance for heavy vehicles to the stores.



Figure 2.14: South Elevation.

### 2.3.3 Description of Vertical Section

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

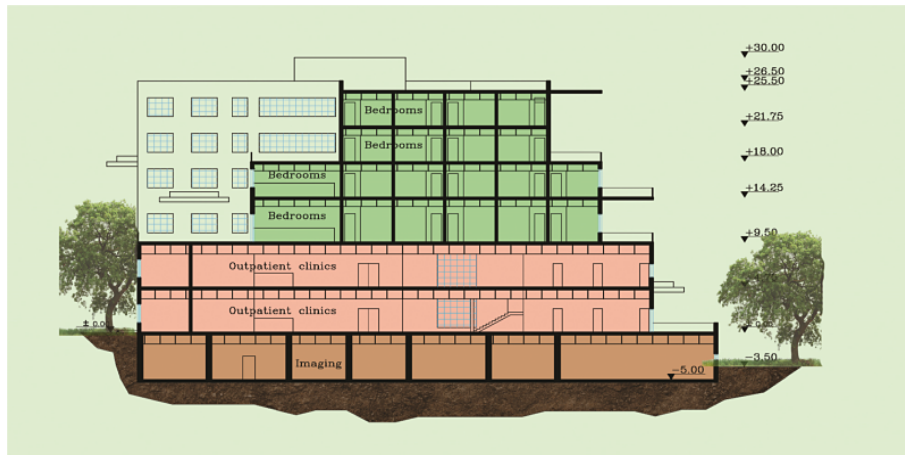


Figure 2.15: Section A-A.

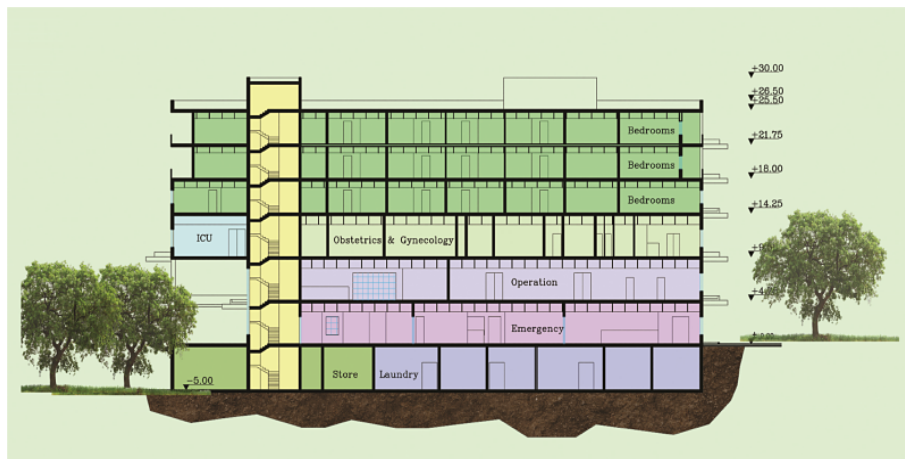


Figure 2.16: Section B-B.

# Chapter 3

## Structural Description

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

In generally, the physical system is the basic system in the building. so, they have the following type:

- Structural system.
- Exterior system.
- Interior subdivisions of space.

### 3.1 The Purpose of Structural Design

The purpose of structural design is the work to find the building is available where all safety requirements, in order to resist all the forces that affect the building in different forms, such as loads of dead

and live or external forces such as earthquakes, wind and landing in the soil. When designing any element of these structural elements should be taken in consideration the following standers:

1. Safety: is the essential element that must be provided in the design, so choosing the appropriate element of each region to resist load that affect them.
2. Cost: must be supplied when working on the selection of appropriate materials, and sufficient for its desired purpose and appropriate quantity.
3. Serviceability : work to avoid any external failures, such as the decline in soil or any cracks in the external shape, or anything that works to increase this failure.
4. Architectural side : work to take into account the architectural elements in the building and try to keep it as much as possible.

### **3.2 Theoretical Studies of the Structural Elements of the Building**

The most important step that should work out of the project before starting the structural design, you must work on a comprehensive study of the project in terms of its size and the nature of its work, and how to work to estimate the loads that effect on the building, and choose items that are exposed to these loads, and identify system construction, which used to resist these loads.

### **3.3 Types of Loads**

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

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

1. Basic loads:

It loads which must be taken into account in the structural design of the building in all cases, it includes : Dead load, Live load and Environmental loads.

2. Secondary loads:

The loads that take them into account in the design in some buildings, depending on the nature of the building and other influences, it includes: Shrinkage load, Thermal load, Snows load, Dynamic load, Seismic load.

### 3.3.1 Dead Load

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

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

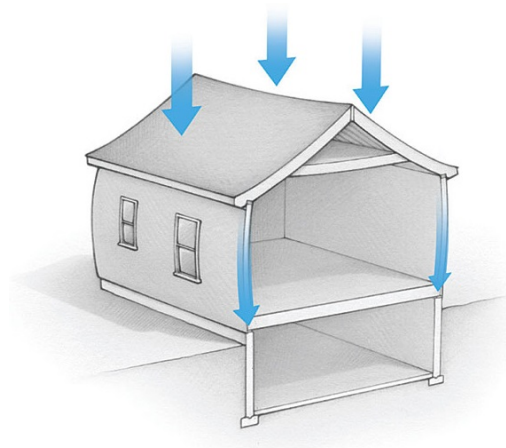


Figure 3.1: Dead Load.

Table 3.1: Specific Density of the Materials Used

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

### 3.3.2 Live load

Live load is imposed loads, are temporary, of short duration, or moving.

These dynamic loads may involve consideration such as impact, momentum, vibration, slosh dynamic of fluids, fatigue, etc. Live loads, sometimes also referred to as probabilistic loads include all the forces that are variable within the object’s normal operation cycle not including construction or environmental loads.

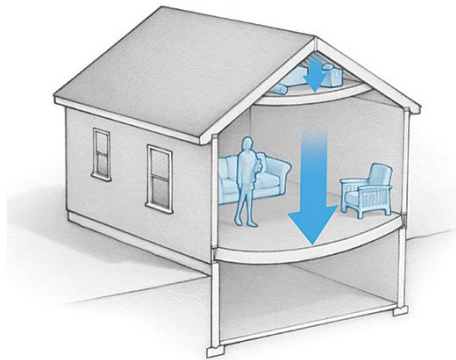


Figure 3.2: Live Load.

Table 3.2: Live Loads [2]

Number	The type of material	The value of live load (KN/m <sup>3</sup> )
1	Kitchens	3
2	Offices	3
3	Corridors and stairs	5
4	Stories	5
5	Roof	1.5

### 3.3.3 Environmental loads

The loads arising from the changes in the environmental such as seismic, wind and snow.

#### 3.3.3.1 Seismic load:

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

#### 3.3.3.2 Wind load:

The forces that affect horizontally on the building, appear especially in high-rise buildings, and it's designed on the basis of wind speed and height of the building, and the amount of buildings surrounding the building.

#### 3.3.3.3 Snow load:

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

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

Note: the building that we will design located in Nablus, so we will take snow in the accounts.

Table 3.3: Loads of Snow by Sea Level

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

### 3.4 Practical Tests

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

### 3.5 Structural Elements

There are many structural elements used in the buildings as the slab, beam, column, stairs, the shear wall and foundations.

#### 3.5.1 The Slabs

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

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

In our project, we will use different types including:

### 3.5.1.1 The rib:

In general, this type is most commonly used in our project, this contain the pipe steel use to transfer the loads , and block and the concrete between this block and the topping of all, and we have two types of this:

- one way rib.
- two way rib.

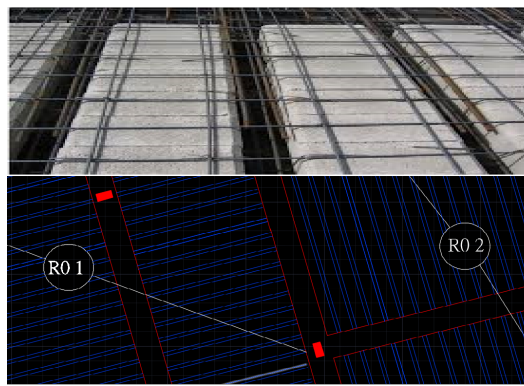


Figure 3.3: Real One Way Rib and Sample in the Project

The two way rib slab is the type use when the length of the two direction in the space approximately is equals, and we used in this type bar of steel in two direction to transfer the load.

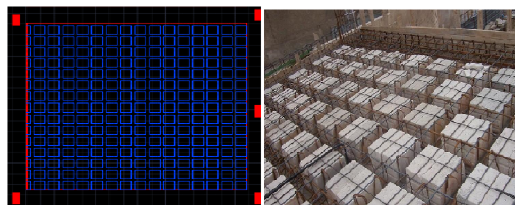


Figure 3.4: Real Two Way Rib and Sample in the Project

### 3.5.1.2 Solid slab:

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

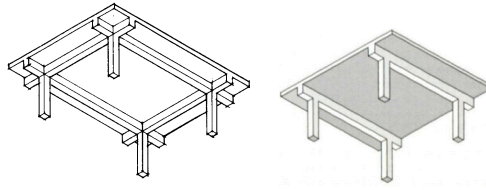


Figure 3.5: Solid Slabs

### 3.5.1.3 Flat plate:

Use this type to transfer the load through it to the column directly, don't use the beam, as shown in the figure:

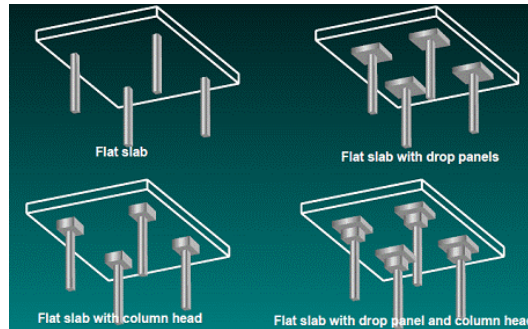


Figure 3.6: Flat Slabs

### 3.5.2 Beams

Use this element to transfer the load from the slab to the column, and have the type as hidden beam when have the same thickness of slab and drop beam when have different thickness. See fig(3.7) and fig(3.8)

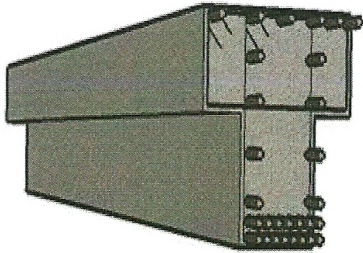


Figure 3.7: Drop Beam

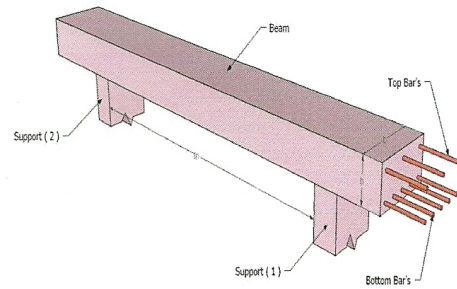


Figure 3.8: Hidden Beam

### 3.5.3 Column

The element is use to transfer the load from the slab to the foundation, and it helps in the stability of the building, and when design we will know the type design if short or slender column. See fig(3.9) and fig(3.10)

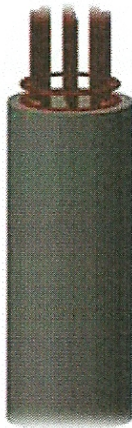


Figure 3.9: Circular Column

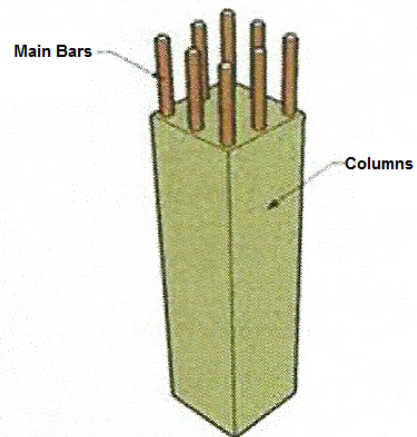


Figure 3.10: Rectangular Column

### 3.5.4 Shear Wall

Shear wall is the important element structure because use to resist the vertical and horizontal load; Shear wall is a type of structural system that provides lateral resistance to the building or structure. It resist loads as the wind and earthquake. When design this wall, we use two layer steel to give it more

strength.

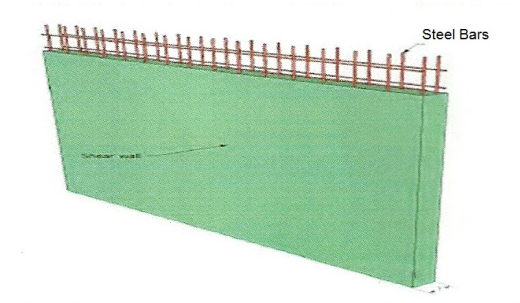


Figure 3.11: Shear Wall

### 3.5.5 The Foundations

The first element we implemented on the ground, but is the last element we design, because all loads are transmitted to it whether the basic load as dead or live load or secondary load. So is the basic element, which receives all the loads and distributed it to the soil. See fig(3.12), fig(3.13), and fig(3.14).

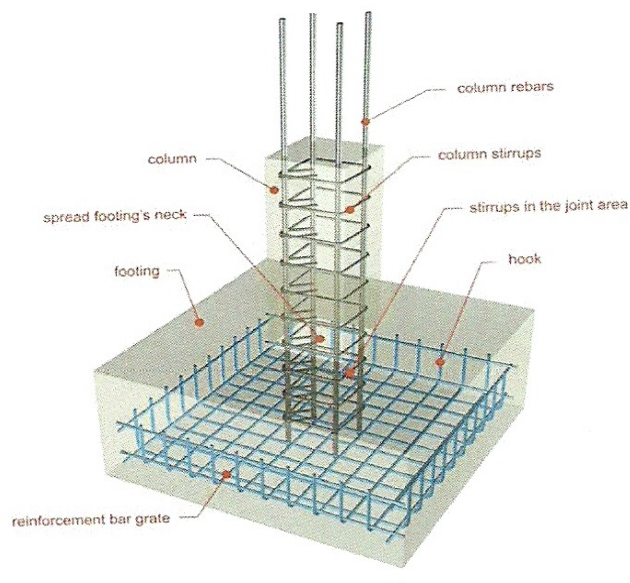


Figure 3.12: Isolated Footing

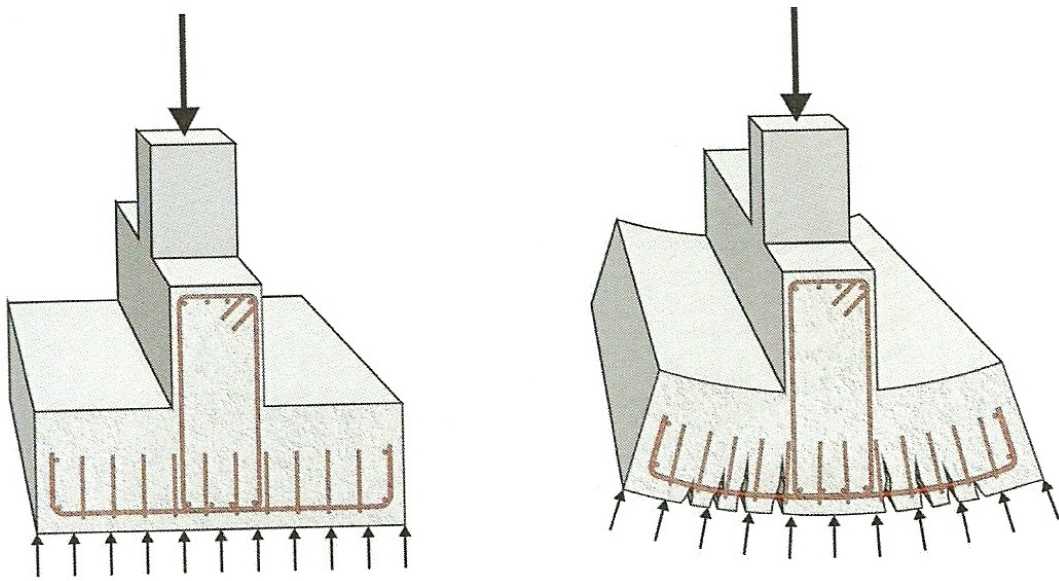


Figure 3.13: Strip Footing

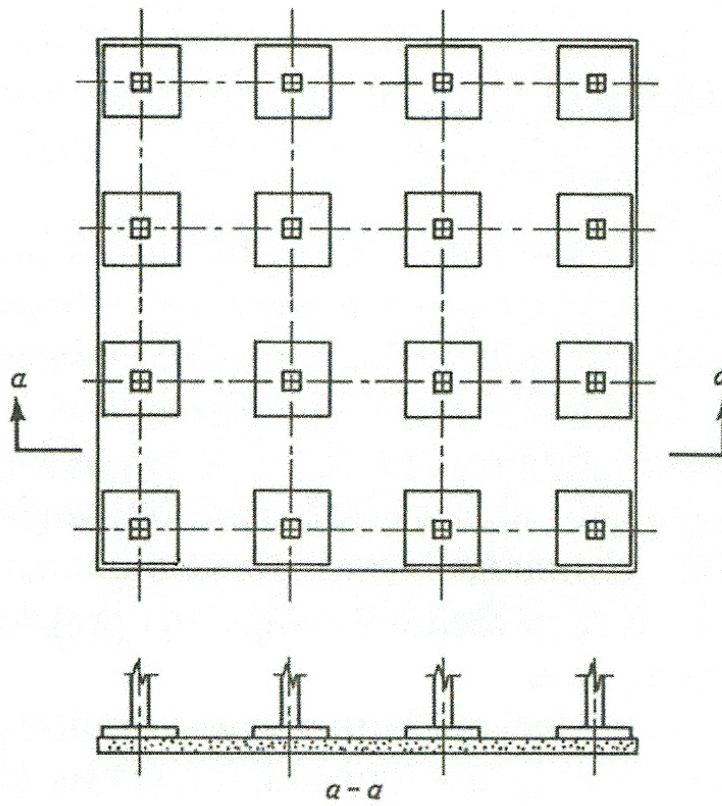


Figure 3.14: Mat Footing

### 3.5.6 Stairs

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

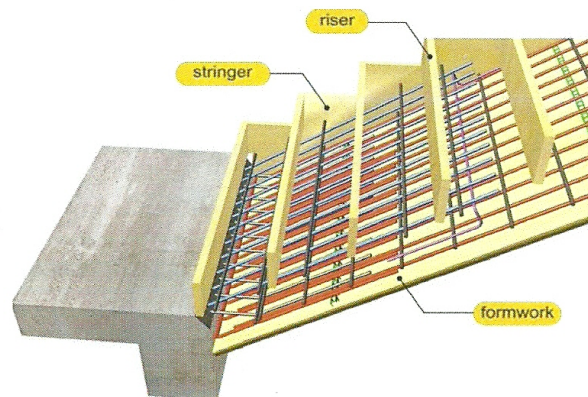


Figure 3.15: Stairs

### 3.5.7 Expansion joints

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

- when you use joints must take into account the vast spaces of the building :
  1. 40m areas with high humidity.
  2. 36m areas with normal humidity.
  3. 32m areas with medium humidity.
  4. 28m with dry areas.

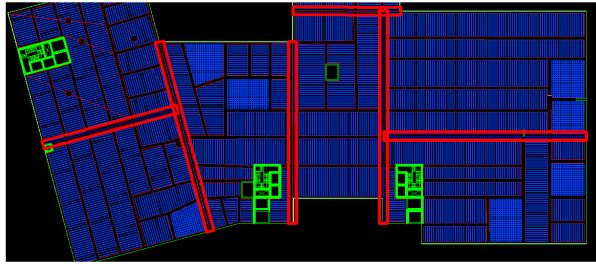


Figure 3.16: Expansion Joints in the Project

- Will have the thickness 5 cm .
- We use 6 expansion joints with 5cm.

## **Chapter 4**

# **Structural Analysis and Design**

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

## 4.1 Determination of Slab Thickness

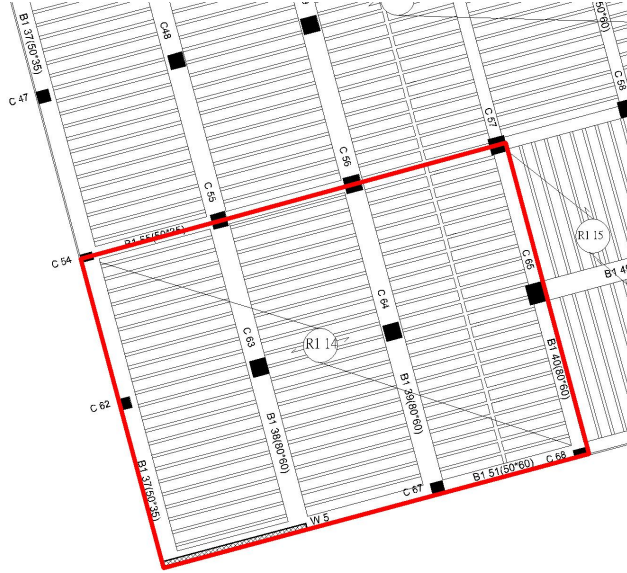


Figure 4.1: First Floor Slab.

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

$$h_{min} \text{ for one end continuous (for Ribs)} = \frac{L}{18.5} = \frac{580}{18.5} = 31.35 \text{ cm}$$

$$h_{min} \text{ for both end continuous (for Ribs)} = \frac{L}{21} = \frac{550}{21} = 26.20 \text{ cm}$$

$$h_{min} \text{ for one end continuous (for Beams)} = \frac{L}{18.5} = \frac{645}{18.5} = 34.86 \text{ cm}$$

$$h_{min} \text{ for both end continuous (for Beams)} = \frac{L}{21} = \frac{607}{21} = 28.90 \text{ cm}$$

The controller slab thickness is 34.86 cm.

But by deflection checked it was controlled at 35 cm thickness, and use drop beams in the project.

Select Slab thickness  $h = 35 \text{ cm}$  with hollow block 27 cm and Topping 8 cm.

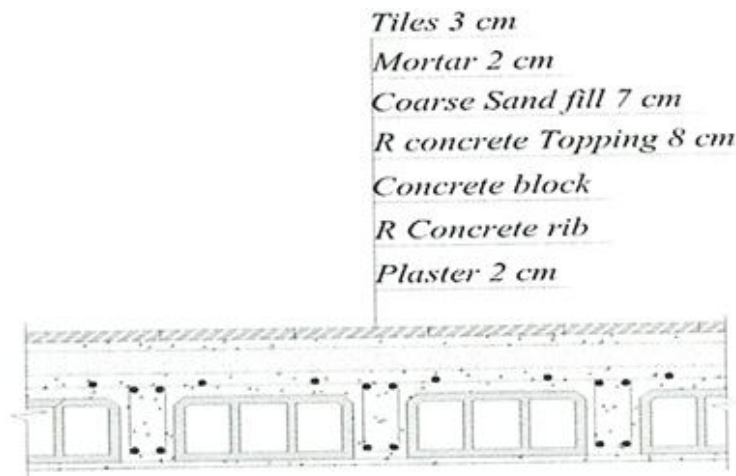


Figure 4.2: Typical Section in Ribbed slab.

## 4.2 Determination of Loads of Ribs

### 4.2.1 Determination of dead load

Table 4.1: Dead Loads of the One Way Rib Slab

Type	$\gamma \times b \times h$	$KN/m/rib$
Tiles	$0.03 \times 0.52 \times 23$	0.359
Mortar	$0.03 \times 0.52 \times 22$	0.343
Course Sand	$0.07 \times 0.52 \times 17$	0.619
Topping	$0.08 \times 0.52 \times 25$	1.04
Hollow block	$0.4 \times 0.27 \times 10$	1.08
Plaster	$0.03 \times 0.52 \times 22$	0.343
R.C rib	$0.12 \times 0.24 \times 25$	0.72
Partitions	$2.3 \times 0.52$	1.196
Sum		5.7

### 4.2.2 Determination of live load

Nominal Total live load for studies =  $5 \times 0.52 = 2.6 \text{ kN/m/rib}$

### 4.2.3 Determination of factored dead and live load

Factored dead load =  $1.2 \times \text{Deadload} = 1.2 \times 5.7 = 6.84 \text{ kN/m}$ .

Factored Live load =  $1.6 \times \text{live load} = 1.6 \times 2.6 = 4.16 \text{ kN/m}$ .

## 4.3 Design of Topping

### 4.3.1 Determination of dead load of topping

Table 4.2: Dead Loads of the Topping

Type	$\gamma \times b \times h$	$\text{KN/m}$
Tiles	$0.03 \times 1 \times 23$	0.69
Mortar	$0.02 \times 1 \times 22$	0.44
Sand	$0.07 \times 0.17 \times 1$	1.19
Topping	$0.08 \times 1 \times 25$	2
Partitions	$2.3 \times 1$	2.3
Sum		6.84

Live Load =  $5 \text{ kN/m}$ .

$wu = 1.2DL + 1.6LL = 1.2 \times 6.84 + 1.6 \times 5 = 16.208 \text{ kN/m}$ . (Total Factored Load).

$$M_u = \frac{(w_u * l^2)}{12} = \frac{16.208 * 0.4^2}{12} = 0.216 \text{ kN.m.}$$

$(\phi M)_n > M_u$  [Strength Condition, where  $\phi = 0.55$ ] for plane concrete.

$$M_n = 0.42 \times \sqrt{f'c} \times \frac{bh^2}{6}$$

Where  $S_m$  for rectangular section of the slab :

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

$$M_n = 0.42 \times \sqrt{24} \times \frac{1000 \times (80)^2}{6} = 2.19 \text{ kN.m}$$

$$\phi M_n = 0.55 \times 2.19 = 1.2 \text{ kN.m.}$$

$$\phi M_n = 1.2 > M_u = 0.216 \text{ kN.m.}$$

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

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

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

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

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

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

Step(s) is the smallest of :

1.  $S = 3h = 3 \times 80 = 240 \text{ mm}$

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

3.  $S = 380 \times \frac{280}{f_s} - 2.5C_c = 380 \times \frac{280}{(2/3 \times 400)} - 2.5 \times 20 = 349 \text{ mm}$  , But  
 $S \leq 300 \times \frac{280}{f_s} = 300 \times \frac{280}{(2/3 \times 400)} = 315 \text{ mm}$

$$S = 200 \text{ mm} < S_{max} = 240 \text{ mm}$$

Take  $\phi 8@200$  in both directions.

## 4.4 Design of Rib 1-14

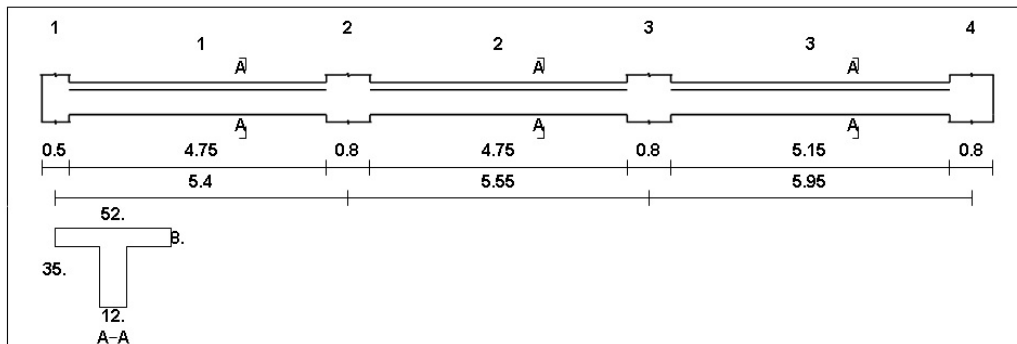


Figure 4.3: Rib 1-14 Geometry.

CHAPTER 4. STRUCTURAL ANALYSIS AND DESIGN

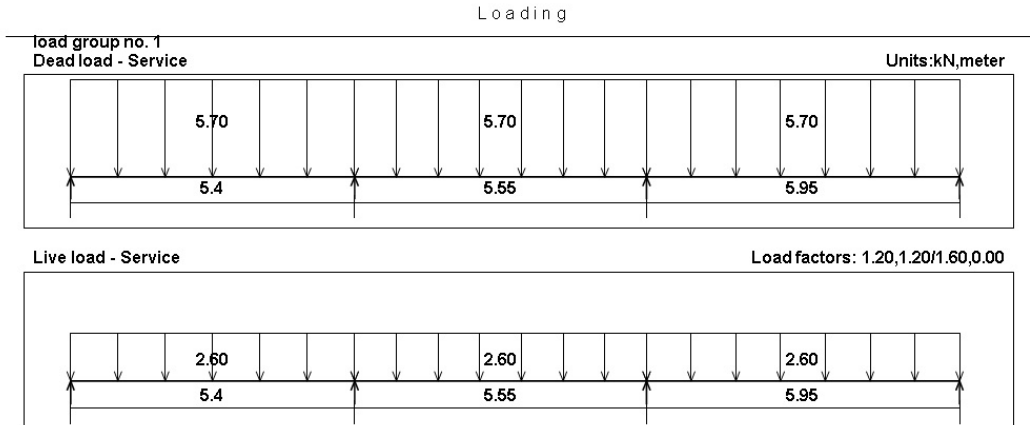


Figure 4.4: Loading of Rib 1-14

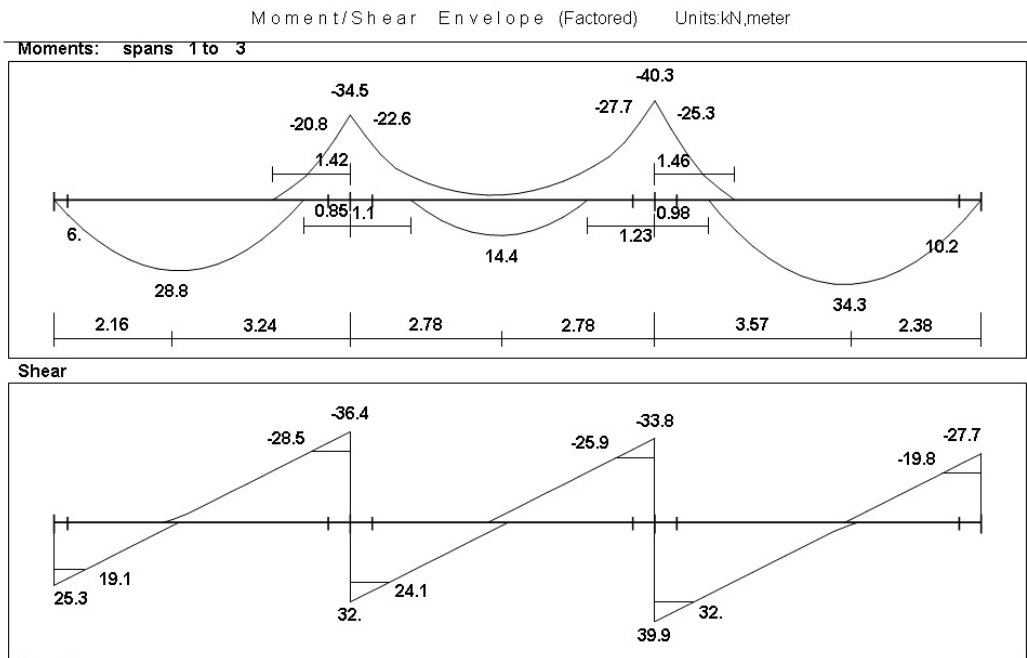


Figure 4.5: Moment and Shear Envelope of Rib1-14

### 4.4.1 Design of flexure

#### 4.4.1.1 Design of positive moment of rib 1-14

**Assume  $2\phi 14\text{ mm}$  (+ve moment)**

$$d = 350 - 20 - 8 - 7 = 315\text{ mm.}$$

$$A_{1\phi 14} = 153.93\text{ mm}^2$$

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

$$A_{s,min} = \frac{\sqrt{24}}{4(420)}(120)(315) \geq \frac{1.4}{420}(120)(315)$$

$$A_{s,min} = 110.2 < 126$$

$$A_{s,min} = 126\text{ mm}^2$$

$$307.8\text{ mm}^2 > A_{s,min} = 126\text{ mm}^2$$

Tension = Compression

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

$$307.8 \times 420 = 0.85 \times 520 \times 27 \times a$$

$$a = 10.83\text{ mm.}$$

$$c = \frac{a}{\beta_1} = \frac{10.83}{0.85} = 12.74\text{ mm}$$

$$\varepsilon_s = \frac{(315 - 12.74)}{12.74} \times 0.003 = 0.07 > 0.005$$

$$\phi = 0.9$$

$$Mn = A_s \times f_y \times (d - \frac{a}{2}) = 307.8 \times 420 \times (315 - \frac{10.83}{2}) = 40.02\text{ kN.m}$$

$$\phi Mn = Mu = 0.9 \times 40.02 = 36.02\text{ kN.m.}$$

Select  $2\phi 14\text{ mm}$  At Spans (1 and 3) with  $A_s > A_{s\text{ req}}$

**Assume  $2\phi 10\text{ mm}$  (+ve moment)**

$$d = 350 - 20 - 8 - 5 = 317\text{ mm.}$$

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

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

$$A_{s,min} = \frac{\sqrt{24}}{4(420)}(120)(317) \geq \frac{1.4}{420}(120)(317)$$

$$A_{s,min} = 110.90 < 126.8$$

$$A_{s,min} = 126.8 \text{ mm}^2$$

$$157.08 \text{ mm}^2 > A_{s,min} = 126.8 \text{ mm}^2$$

Tension = compression

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

$$157.08 \times 420 = 0.85 \times 520 \times 27 \times a$$

$$a = 5.5 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{5.5}{0.85} = 6.5 \text{ mm}$$

$$\varepsilon_s = \frac{(317 - 6.5)}{6.5} \times 0.003 = 0.14 > 0.005$$

$$\phi = 0.9$$

$$M_n = A_s \times f_y \times \left(d - \frac{a}{2}\right) = 157.08 \times 420 \times \left(317 - \frac{5.5}{2}\right) = 20.73 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 \times 20.73 = 18.66 \text{ kN.m.}$$

Select 2 $\phi$ 10 mm At Span (2) with  $A_s > A_{s \text{ req}}$

#### 4.4.1.2 Design of negative moment of rib 1-14

Assume bars diameter of 14mm (-ve moment)

$$d = 350 - 20 - 8 - 7 = 315 \text{ mm.}$$

$$A_{1\phi 14} = 153.93 \text{ mm}^2$$

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

$$A_{s,min} = \frac{\sqrt{24}}{4(420)}(120)(315) \geq \frac{1.4}{420}(120)(315)$$

$$A_{s,min} = 110.2 < 126$$

$$A_{s,min} = 126 \text{ mm}^2$$

$$307.87 \text{ mm}^2 > A_{s,min} = 126 \text{ mm}^2$$

Tension = Compression

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

$$307.87 \times 420 = 0.85 \times 120 \times 27 \times a$$

$$a = 46.95 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{46.95}{0.85} = 55.23 \text{ mm}$$

$$\varepsilon_s = \frac{(315 - 55.23)}{55.23} \times 0.003 = 0.014 > 0.005$$

$$\phi = 0.9$$

$$M_n = A_s \times f_y \times \left(d - \frac{a}{2}\right) = 307.87 \times 420 \times \left(315 - \frac{46.95}{2}\right) = 37.7 \text{ kN.m}$$

$$\phi M_n = M_u = 0.9 \times 37.7 = 33.9 \text{ kN.m.}$$

Select  $2\phi 12 \text{ mm}$  At supports (1,3) with  $A_s > A_{s \text{ req}}$

#### 4.4.2 Design of shear of rib 1-14

1.  $V_{ud} = 28.5 \text{ kN}$  (at Support 2)

$$\phi V_c = \phi \times \frac{\sqrt{f_c'}}{6} b w \times d = 0.75 \times \frac{\sqrt{24}}{6} \times 120 \times 0.315 = 23.147 \text{ kN}$$

$$1.1 \times \phi V_c = 1.1 \times 23.147 = 25.46 \text{ kN.}$$

- $V_{s,max} = \frac{2}{3} \times \sqrt{24} \times 120 \times 315 \times 10^{-3} = 123.45 \text{ kN}$

- $V_s = V_n - V_c = \frac{28.5}{0.75} - 33.94 = 4.06 \text{ kN}$

The section is large enough

Check for items :

**Case I :**

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

$$28.5 > 12.73 \text{ (Not OK)}$$

**Case II :**

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

$$12.73 < 28.5 > 25.46 \text{ (Not OK)}$$

**Case III :**

$$V_{s,min} = \frac{1}{16} \times \sqrt{f_c'} \times bw \times d = \frac{1}{16} \times \sqrt{24} \times 120 \times 315 \times 10^{-3} = 11.57 \text{ kN.}$$

$$V_{s,min} = \frac{1}{3} \times bw \times d = \frac{1}{3} \times 120 \times 315 \times 10^{-3} = 12.6 \text{ kN}$$

$$\phi(V_c + V_{s,min}) = 34.9 \text{ kN.}$$

$$\phi V_c = 25.46 \leq V_u = 28.5 \leq \phi(V_c + V_{s,min}) = 34.9 \text{ kN} \dots\dots \text{ OK}$$

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

Or

$$\frac{A_v}{s} = \frac{1}{3} \times \frac{bw}{f_{yt}} = \frac{1}{3} \times \frac{120}{420} = 0.095 \dots\dots\dots \text{ control}$$

Use stirrups 2U–shape (2–lg stirrups)  $\phi 8 \text{ mm}$  with  $A_v = 100.53 \text{ mm}^2$

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

$$S \leq 600 \quad \text{or} \quad S \leq \frac{d}{2} = \frac{315}{2} = 157.5 \text{ mm}$$

Take  $S = 150 \text{ mm}$ .

Use stirrups 2U–shape (2–lg stirrups)  $\phi 8 \text{ mm} @ 150 \text{ mm}$ .

2.  $V_{ud} = 32 \text{ kN}$  (at Support 3)

**Case I :**

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

$$32 > 12.73 \text{ (Not OK)}$$

**Case II :**

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

$$12.73 < 32 > 25.46 \text{ (Not OK)}$$

**Case III :**

$$V_{s,min} = \frac{1}{16} \times \sqrt{f_c} \times bw \times d = \frac{1}{16} \times \sqrt{24} \times 120 \times 315 \times 10^{-3} = 11.75 \text{ kN}.$$

$$V_{s,min} = \frac{1}{3} \times bw \times d = \frac{1}{3} \times 120 \times 315 \times 10^{-3} = 12.6 \text{ KN}$$

$$\phi(V_c + V_{s,min}) = 34.9 \text{ kN}.$$

$$\phi V_c = 25.46 \leq V_u = 32 \leq \phi(V_c + V_{s,min}) = 34.9 \text{ kN} \dots\dots \text{OK}$$

$$\frac{A_v}{s} = \frac{1}{16} \sqrt{f_c} \times \frac{bw}{f_{yt}} = \frac{1}{16} \times \sqrt{24} \times \frac{120}{420} = 0.087$$

Or

$$\frac{A_v}{s} = \frac{1}{3} \times \frac{bw}{f_{yt}} = \frac{1}{3} \times \frac{120}{420} = 0.095 \dots\dots\dots \text{control}$$

Use stirrups 2U–shape (2–lg stirrups)  $\phi 8 \text{ mm}$  with  $A_v = 100.53 \text{ mm}^2$

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

$$S \leq 600 \quad \text{or} \quad S \leq \frac{d}{2} = \frac{315}{2} = 157.5 \text{ mm}$$

Take  $S = 150 \text{ mm}$ .

**Use stirrups 2U–shape (2–lg stirrups)  $\phi 8 \text{ mm} @ 150 \text{ mm}$ .**

## 4.5 Design of Beam: Beam (1-27)

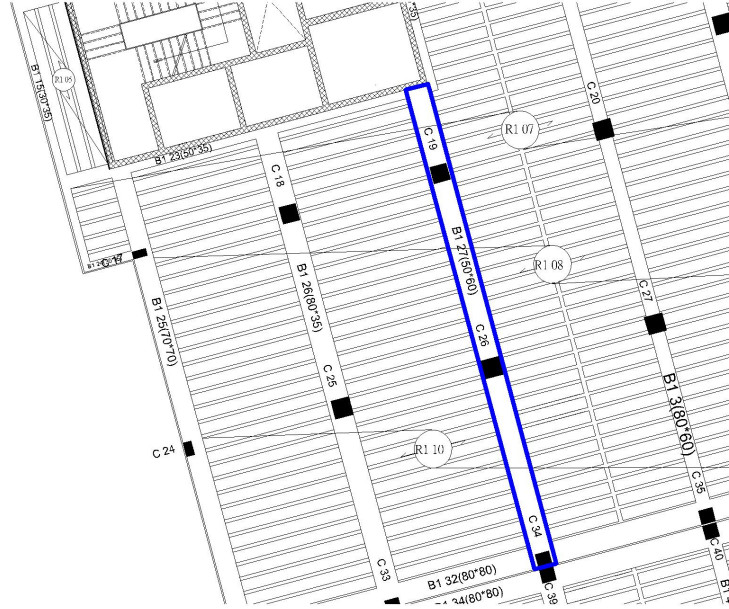


Figure 4.6: Beam 1-27 Plan.

### 4.5.1 Determination of dead load of beam

$$\text{From rib 1 - 7} = \frac{33.46}{0.52} = 64.33 \text{ kN/m.}$$

$$\text{From rib 1 - 8} = \frac{32.68}{0.52} = 62.84 \text{ kN/m.}$$

$$\text{From rib 1 - 10} = \frac{32.04}{0.52} = 61.61 \text{ kN/m.}$$

### 4.5.2 Determination of live load of beam

$$\text{From rib 1 - 7} = \frac{17.65}{0.52} = 33.94 \text{ kN/m.}$$

$$\text{From rib 1 - 8} = \frac{17.3}{0.52} = 33.27 \text{ kN/m.}$$

$$\text{From rib 1 - 10} = \frac{17.01}{0.52} = 32.71 \text{ kN/m.}$$

### 4.5.3 Determination of factored dead and live load

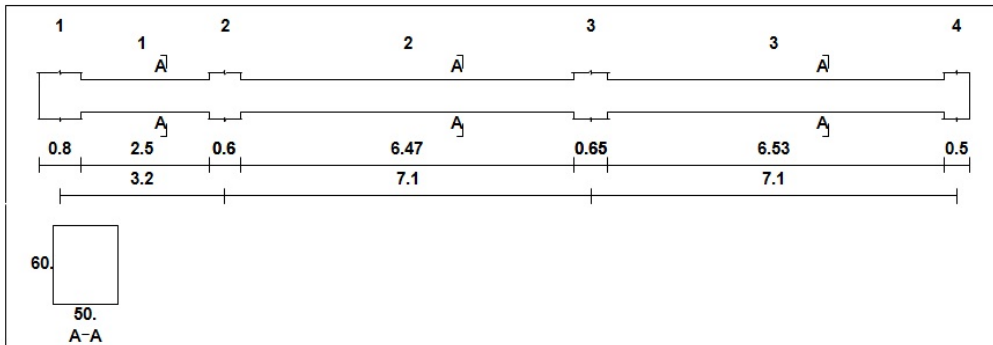


Figure 4.7: Beam (1-27) Geometry.

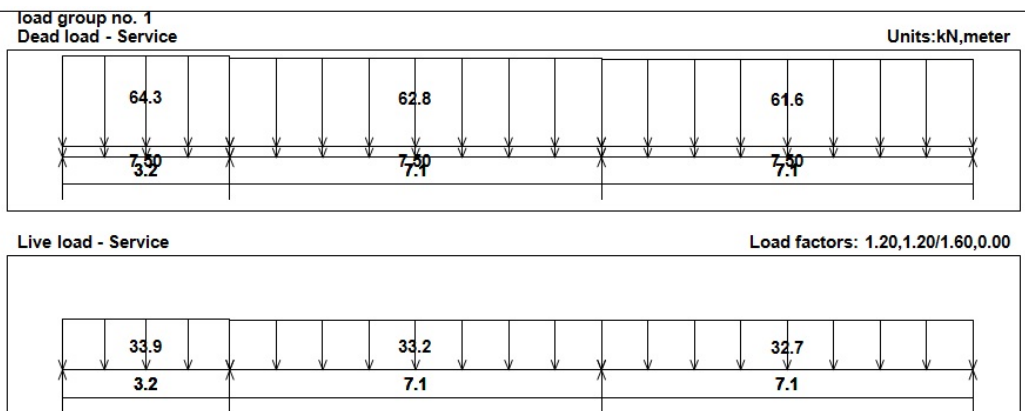


Figure 4.8: Beam (1-27) Loading.

CHAPTER 4. STRUCTURAL ANALYSIS AND DESIGN

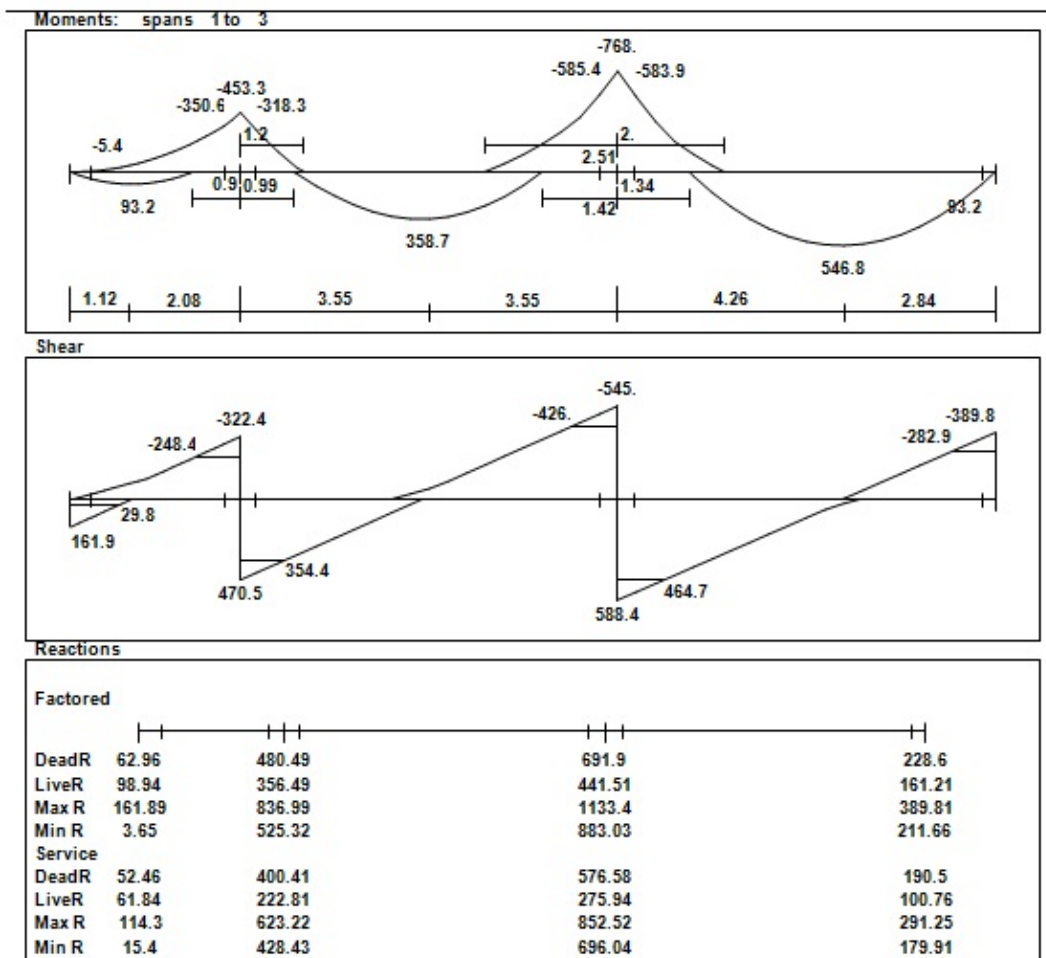


Figure 4.9: Beam (1-27) Shear and Moment diagram.

## 4.5.4 Design of flexure

### 4.5.4.1 Design of positive moment

Assume bars of  $\phi 25$ ,  $b = 50 \text{ cm}$ ,  $h = 60 \text{ cm}$

1.  $M_u = 358.7 \text{ kN.m}$

$$d = 600 - 40 - \frac{25}{2} - 10 = 537.5 \text{ mm.}$$

$$C_{max} = \frac{3}{7} \times d = 3 \times \frac{537.5}{7} = 230.35 \text{ mm.}$$

$$a = 0.85 C = 0.85 \times 230.35 = 195.8 \text{ mm}$$

$$\phi M_n \text{ max} = \phi \times 0.85 f_c' \times a \times b \times \left(d - \frac{a}{2}\right)$$

$$0.82 \times 0.85 \times 24 \times 195.8 \times 500 \times \left(537.5 - \frac{195.8}{2}\right) \times 10^{-6} = 719.9 \text{ kN.m} > M_u = 358.7 \text{ kN.m.}$$

• **Design as Singly :**

$$M_n = \frac{M_u}{0.9} = \frac{358.7}{0.9} = 398.55 \text{ kN.m}$$

$$R_n = \frac{M_n}{(b \times d^2)} = \frac{398.55 \times 10^6}{(500 \times 537.5^2)} = 2.76 \text{ mpa}$$

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

$$\rho = \frac{1}{20.6} \left(1 - \sqrt{1 - \frac{2(2.76)(20.6)}{420}}\right) = 0.00709$$

$$A_s = 0.00709(500)(537.5) = 1905.44 \text{ mm}^2.$$

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

$$A_{s,min} = \frac{\sqrt{24}}{4(420)}(500)(537.5) \geq \frac{1.4}{420}(500)(537.5)$$

$$A_{s,min} = 895.83 \text{ mm}^2$$

$$1905.44 \text{ mm}^2 > A_{s,min} = 895.83 \text{ mm}^2$$

$$\# \text{ of bars} = \frac{A_s}{A_{s,bar}} = \frac{1905.44}{490.87} = 3.88 \text{ bars}$$

Note  $A_{\phi 25} = 490.87 \text{ mm}^2$ .

Select  $4\phi 25 \text{ mm}$  with  $A_s = 1963.5 > A_{s \text{ req}}$

• **Check for strain**

Tension = compression

$$A_s \times f_y = 0.85 \times f'_c \times b \times a$$

$$a = \frac{(1963.5 \times 420)}{(0.85 \times 24 \times 500)} = 80.85 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{80.85}{0.85} = 95.11 \text{ mm}$$

$$\varepsilon_s = \frac{d - c}{c} \times 0.003 = \frac{(537.5 - 95.11)}{107.86} \times 0.003 = 0.014 > 0.005$$

$$\phi = 0.9$$

$$M_n = 0.85 \times f'_c \times a \times b \times \left(d - \frac{a}{2}\right)$$

$$M_n = 0.85 \times 24 \times 80.85 \times 500 \times \left(537.5 - \frac{80.85}{2}\right) = 409.92 \text{ kN.m}$$

$$\phi M_n = 0.9 \times 409.92 = 368.9 \text{ kN.m} > 358.7 \text{ kN.m}$$

• **Check for spacing**

$$S = \frac{(500 - 40 \times 2 - 2 \times 10 - 4 \times 25)}{3} = 100 \text{ mm} > 25 \text{ mm.}$$

2.  $M_u = 546.8 \text{ kN.m}$

$$d = 600 - 40 - \frac{25}{2} - 10 = 537.5 \text{ mm.}$$

$$C_{max} = \frac{3}{7} \times d = 3 \times \frac{537.5}{7} = 230.35 \text{ mm.}$$

$$a = 0.85 C = 0.85 \times 230.35 = 195.8 \text{ mm}$$

$$\phi M_n \text{ max} = \phi \times 0.85 f'_c \times a \times b \times \left(d - \frac{a}{2}\right)$$

$$0.82 \times 0.85 \times 24 \times 195.8 \times 500 \times \left(537.5 - \frac{195.8}{2}\right) \times 10^{-6} = 719.9 \text{ kN.m} > M_u = 546.8 \text{ kN.m.}$$

• **Design as Singly**

$$M_n = \frac{M_u}{0.9} = \frac{546.8}{0.9} = 607.55 \text{ kN.m}$$

$$R_n = \frac{M_n}{(b \times d^2)} = \frac{607.55 \times 10^6}{(500 \times 537.5^2)} = 4.2 \text{ mpa}$$

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

$$\rho = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2(4.2)(20.6)}{420}} \right) = 0.011$$

$$A_s = 0.011(500)(537.5) = 2956.25 \text{ mm}^2.$$

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

$$A_{s,min} = \frac{\sqrt{24}}{4(420)}(500)(537.5) \leq \frac{1.4}{420}(500)(537.5)$$

$$A_{s,min} = 895.83 \text{ mm}^2$$

$$895.83 \text{ mm}^2 < A_{s,min} = 2956.25 \text{ mm}^2$$

$$\# \text{ of bars} = \frac{A_s}{A_{sbar}} = \frac{2956.25}{490.87} = 7 \text{ bars}$$

$$\text{Note } A_{\phi 25} = 490.83 \text{ mm}^2.$$

Select  $7\phi 25 \text{ mm}$  with  $A_s = 3436.11 > A_s \text{ req}$

• **Check for strain**

Tension = compression

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

$$a = \frac{(3436.11 \times 420)}{(0.85 \times 24 \times 500)} = 141.5 \text{ mm}$$

$$c = 166.45 \text{ mm}$$

$$\epsilon_s = \frac{d - c}{c} \times 0.003 = \frac{(537.5 - 166.45)}{166.45} \times 0.003 = 0.0066 > 0.005$$

$$\phi = 0.9$$

$$Mn = 0.85 \times f_c \times a \times b \times \left( d - \frac{a}{2} \right)$$

$$Mn = 0.85 \times 24 \times 141.5 \times 500 \times \left( 537.5 - \frac{141.5}{2} \right) = 673.6 \text{ kN.m}$$

$$\phi Mn = 0.9 \times 673.6 = 606 \text{ kN.m} > 546.8 \text{ kN.m}$$

• **Check for spacing**

$$S = \frac{(500 - 40 \times 2 - 2 \times 10 - 7 \times 25)}{4} = 37.5 \text{ mm} > 25 \text{ mm}.$$

3.  $M_u = 96.6 \text{ kN.m}$

Select  $4\phi 18 \text{ mm}$  with  $A_s = 1017.87 > A_s \text{ req}$

#### 4.5.4.2 Design of negative moment

Assume bars of  $\phi 25$

$$b = 50 \text{ cm} \quad h = 60 \text{ cm}$$

$$d = 600 - 40 - \frac{25}{2} - 10 = 537.5 \text{ mm.}$$

$$C_{max} = \frac{3}{7} \times d = \frac{3}{7} \times 537.5 = 230.35 \text{ mm}$$

$$a = 0.85 \times C = 0.85 \times 230.35 = 195.8 \text{ mm}$$

$$\phi M_n \text{ max} = \phi \times 0.85 f_c' \times a \times b \times \left(d - \frac{a}{2}\right)$$

$$0.82 \times 0.85 \times 24 \times 195.8 \times 500 \times \left(537.5 - \frac{195.8}{2}\right) \times 10^{-6} = 719.9 \text{ kN.m} > M_u = 350.6 \text{ kN.m.}$$

- Design as Singly

1.  $M_u = 350.6 \text{ kN.m.}$

Select  $4\phi 25 \text{ mm}$  with  $A_s = 1963.5 > A_s \text{ req}$

2.  $M_u = 585.4 \text{ kN.m.}$

Select  $7\phi 25 \text{ mm}$  with  $A_s = 3436.12 > A_s \text{ req}$

#### 4.5.4.3 Design of shear

1.  $V_u = 354.7 \text{ kN}$  (at Support 1)

$$V_c = \frac{1}{6} \times 1 \times \sqrt{24} \times 500 \times 537.5 \times 10^{-3} = 219.43 \text{ kN}$$

$$0.5 \times \phi V_c = 0.5 \times 0.75 \times 219.43 = 82.29 \text{ kN.}$$

**Case I :**

$$V_u \leq \frac{\phi V_c}{2} \quad (\text{Not OK})$$

**Case II :**

$$\phi \times V_c = 0.75 \times 219.43 = 164.57 \text{ kN}$$

$$\frac{\phi V_c}{2} \leq V_u \leq \phi V_c \quad (\text{Not OK})$$

**Case III :**

$$V_{s,min} = \frac{1}{16} \times \sqrt{24} \times 500 \times 537.5 \times 10^{-3} = 82.3 \text{ kN.}$$

$$V_{s,min} = \frac{1}{3} \times 500 \times 537.5 \times 10^{-3} = 89.58 \dots \dots \dots \text{ Control}$$

$$\phi(V_c + V_{s,min}) = 0.75(219.43 + 89.58) = 231.76 \text{ kN.}$$

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

$$164.57 < 354.7 \leq 231.76 \dots \dots \dots \text{ Not OK}$$

**Case IV :**

$$\dot{V}_s = \frac{1}{3} \times \sqrt{f_c'}(bw)(d)$$

$$\dot{V}_s = \frac{1}{3} \times \sqrt{24} \times 500 \times 537.5 \times 10^{-3} = 438.87 \text{ kN.}$$

$$\phi(V_c + \dot{V}_s) = 0.75(438.87 + 219.43) = 493.725 \text{ kN.}$$

$$219.43 < 354.7 \leq 493.725 \dots \dots \dots \text{ OK}$$

$$\frac{A_v}{s} = \frac{V_s}{(f_{yt} \times d)}$$

Use stirrups U-shape (2-leg stirrups)  $\phi 10 \text{ mm}$  with  $A_v = 2 \times 78.54 = 157.08 \text{ mm}^2$

$$V_s = V_n - V_c = \frac{354.7}{0.75} - 219.43 = 253.5 \text{ kN}$$

$$\frac{A_v}{s} = \frac{V_s}{(f_{yt} \times d)} \rightarrow \frac{157.08 \times 10^{-3}}{s} = \frac{253.5}{420 \times 537.5}$$

$$S = 139.88 \text{ mm}$$

$$S_{max} \leq 600 \quad \text{or} \quad S_{max} \leq \frac{d}{2} = \frac{537.5}{2} = 268.75 \text{ mm}$$

Take  $S = 10 \text{ mm}$ .

2.  $V_u = 464.7 \text{ kN}$  (at Support 2)

$$V_c = \frac{1}{6} \times 1 \times \sqrt{24} \times 500 \times 537.5 \times 10^{-3} = 219.43 \text{ kN}$$

$$0.5 \times \phi V_c = 0.5 \times 0.75 \times 219.43 = 82.29 \text{ kN}.$$

**Case I :**

$$V_u \leq \frac{\phi V_c}{2} \quad (\text{Not OK})$$

**Case II :**

$$\phi \times V_c = 0.75 \times 219.43 = 164.57 \text{ kN}$$

$$\frac{\phi V_c}{2} \leq V_u \leq \phi V_c \quad (\text{Not OK})$$

**Case III :**

$$V_{s,min} = \frac{1}{16} \times \sqrt{24} \times 500 \times 537.5 \times 10^{-3} = 82.3 \text{ kN}.$$

$$V_{s,min} = \frac{1}{3} \times 500 \times 537.5 \times 10^{-3} = 95.58 \dots \dots \dots \text{Control}$$

$$\phi(V_c + V_{s,min}) = 0.75(219.43 + 95.58) = 290.25 \text{ kN}.$$

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

$$164.57 < 464.7 \leq 290.25 \dots \dots \dots \text{Not OK}$$

**Case IV :**

$$\dot{V}_s = \frac{1}{3} \times \sqrt{f_c} (bw)(d)$$

$$\dot{V}_s = \frac{1}{3} \times \sqrt{24} \times 500 \times 537.5 \times 10^{-3} = 438.87 \text{ kN}.$$

$$\phi(V_c + \dot{V}_s) = 0.75(219.43 + 438.87) = 493.725 \text{ kN}.$$

$$164.57 < 464.7 \leq 493.725 \dots \dots \dots \text{OK}$$

$$\frac{A_v}{s} = \frac{V_s}{(f_{yt} \times d)}$$

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

$$V_s = V_n - V_c = \frac{464.7}{0.75} - 219.43 = 400.17 \text{ kN}$$

$$\frac{A_v}{s} = \frac{V_s}{(f_{yt} \times d)} \rightarrow \frac{314.159 \times 10^{-3}}{s} = \frac{400.17}{420 \times 537.5}$$

$$S = 177.2 \text{ mm}$$

$$S_{max} \leq 600 \quad \text{or} \quad S_{max} \leq \frac{d}{2} = \frac{537.5}{2} = 268.75 \text{ mm}$$

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

#### 4.5.4.4 Cut of bars calculations

According to German code (DIN 1045-1) :

Required Reinforcement At end Supports:

- The first support

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

$$V_{Ed, 0} = 87.4 \text{ kN}$$

$$a_1 = 0.54 \times d = 0.54 \times 87.4 = 47.2 \text{ mm}$$

$$z = 0.9 \times 87.4 = 78.66 \text{ mm}.$$

$$F_{sd, R} = 87.4 \times \frac{47.2}{78.66} + 0.0 \geq \frac{87.4}{2}$$

$$F_{sd, R} = 52.44 \geq 43.7$$

$$A_{sreq} = \frac{F_{sd, R}}{0.75 \times f_y} = \frac{52.44}{0.75 \times 42} = 1.66 \text{ cm}^2$$

$$A_{s, prov} = 2\phi 25 = 2 \times 4.91 = 9.82 > 1.66 > 0.25 \times A_{sPos} = 0.25 \times 19.64 = 4.91 \text{ cm}^2$$

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

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

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

$$C_b = \frac{1}{2} \times 125 = 62.5 \text{ mm}. \quad - \text{control}$$

$$\text{or, } = 40 + 10 + \frac{25}{2} = 62.5 \text{ mm}$$

$$L_{dt} = \frac{9}{10} \times \frac{420}{\sqrt{24}} \times \frac{1 \times 1 \times 0.8}{2.5} \times 25 = 617.3 \text{ mm}$$

$$\alpha = \frac{A_{s, req}}{A_{s, prov}} = \frac{1.66}{9.82} = 0.17$$

$$L_{b, bir} = \frac{2}{3} \times 617.3 \times 0.17 = 69.96 \text{ mm} > 150 \text{ mm} > 6 \times db = 150 \text{ mm}$$

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

• **Lab Splices**

$$L_{dt} = \frac{9}{10} \times \frac{f_y}{\sqrt{f_c}} \times \frac{\psi_t \times \psi_e \times \psi_s}{\frac{cb + Ktr}{db}} \times db$$

$$C_b = \frac{1}{2} \times 125 = 62.5 \text{ mm.} - \text{control}$$

$$\text{or, } = 40 + 10 + \frac{25}{2} = 62.5 \text{ mm}$$

$$L_{dt} = \frac{9}{10} \times \frac{420}{\sqrt{24}} \times \frac{1 \times 1 \times 0.8}{2.5} \times 25 = 617.3 \text{ mm}$$

$$L_{s, t} = 1.30 \times L_{dt} = 802.5 \text{ mm}$$

## 4.6 Design of Two way Rib Slab (R1 - 38)

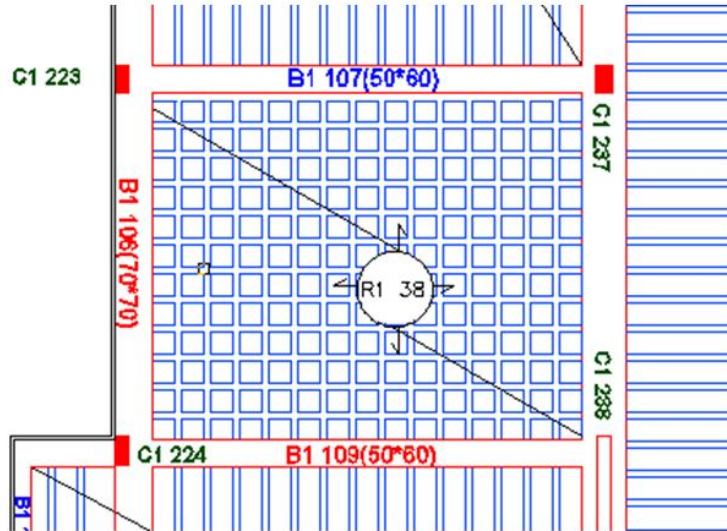


Figure 4.10: Two Way Slab R1-38

Design of Two way Rib Slab:

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

$$\bar{Y} = \frac{40 \times 8 \times 4 + 35 \times 12 \times 17.5}{40 \times 8 + 35 \times 12} = 11.66 \text{ cm}$$

$$I_{rib} = \frac{52 \times 11.66^3}{3} - \frac{40 \times 0.34^3}{3} - \frac{2 \times 23.66^3}{3} = 18647.16 \text{ cm}^4$$

- Exterior Beam:

$$\bar{Y} = \frac{45 \times 35 \times 17.5 + 25 \times 70 \times 35}{45 \times 35 + 70 \times 25} = 26.7 \text{ cm}$$

$$I_{B106} = \frac{70 \times (35 + 8.2)^3}{3} - \frac{35 \times 8.2^3}{3} + \frac{25 \times 26.71^3}{3} = 7598347.3 \text{ cm}^4$$

- Interior Beams:

$$I_{B109} = I_{107} = \frac{1}{12} b h^3 = \frac{1}{12} \times 50 \times 60^3 = 900000 \text{ cm}^4$$

$$I_{B114} = \frac{1}{12} b h^3 = \frac{1}{12} \times 80 \times 35^3 = 285833.33 \text{ cm}^4$$

**The exterior Beam: (B1-106):**

- The short direction = 6.2m = 620cm

$$I_{s1} = \frac{(\frac{620}{2} + 25) \times 35^3}{12} = 1196927.08 \text{ cm}^4$$

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- The long direction = 7.68m = 768 cm

$$I_{s2} = \frac{(\frac{768}{2} + 25) \times 35^3}{12} = 1461322.92 \text{ cm}^4$$

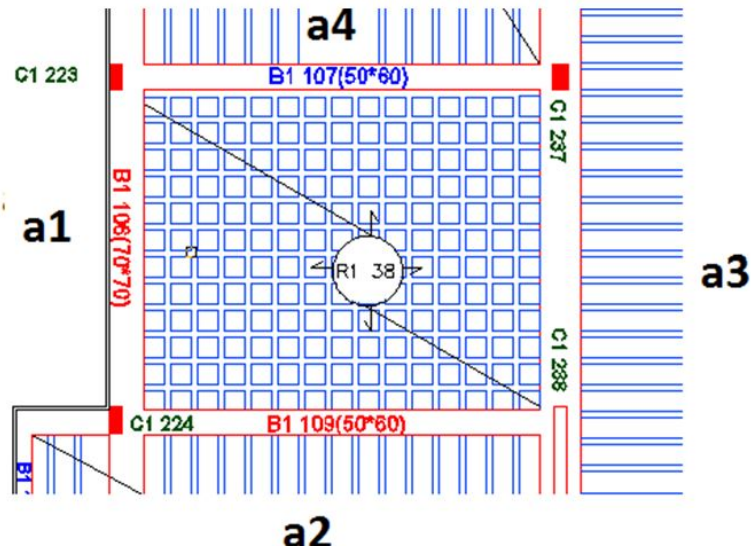
**The interior Beams(B1- 107 , B1- 109 , B1- 114):**

- The short direction = 6.2m = 620 cm

$$I_{s3} = \frac{(\frac{620}{2} + 50) \times 35^3}{12} = 1286250 \text{ cm}^4$$

- The long direction = 7.68m = 768cm

$$I_{s4} = \frac{(\frac{768}{2} + 50) \times 35^3}{12} = 1550645.83 \text{ cm}^4$$



$$a_1 = \frac{Ib}{I_s} = \frac{7598347.3}{1461322.92} = 5.2$$

$$a_2 = \frac{Ib}{I_s} = \frac{900000}{1550645.83} = 0.58$$

$$a_3 = \frac{Ib}{I_s} = \frac{285833.33}{1550645.83} = 0.184$$

$$a_4 = \frac{Ib}{I_s} = \frac{900000}{1286250} = 0.7$$

$$a_{fm} = \frac{a_1 + a_2 + a_3 + a_4}{4} = \frac{5.2 + 0.58 + 0.184 + 0.7}{4} = 1.666$$

$$0.2 < a < 2.0$$

$$0.2 < 1.666 < 2.0$$

**According to ACI-code:**

$$h_m = \frac{L(0.8+fy/1400)}{36+5B(afm-0.2)}$$

$$B = \frac{La}{Lb} = \frac{6.2}{6.68} = 0.928$$

$$H_m = \frac{668 \times (0.8+420/1400)}{36+5 \times 0.928 \times (0.2355-0.2)} = 171.67mm > h_{min} = 125mm$$

Type	$\gamma \times b \times h$	KN/Rib
Tiles	0.03*0.52 <sup>2</sup> *23	0.187
Mortar	0.02*0.52 <sup>2</sup> *22	0.119
Sand	0.07*0.52 <sup>2</sup> *17	0.322
Topping	0.08*0.52 <sup>2</sup> *25	0.541
Hollow block	0.28*0.4 <sup>2</sup> *10	0.448
Plaster	0.03*0.52 <sup>2</sup> *22	0.178
R.C rib	0.12*0.28*25*(0.54+0.4)	0.7728
Partitions	2.3*0.52 <sup>2</sup>	0.621
Sum		3.19

Nominal total dead load = 3.2 KN/Rib

$$3.2/0.52^2 = 11.83KN/m^2$$

Nominal Total Live Load for Hospital = 5KN/m<sup>2</sup>

– Determination of Factored Dead & live load

$$\text{Factor dead load} = 1.2 \times \text{Dead load} = 1.2 \times 11.83 = 14.2KN/m^2$$

$$\text{Factor live load} = 1.6 \times \text{Live load} = 1.6 \times 5 = 8KN/m^2$$

$$\text{Total factor load} = 14.2 + 8 = 22.2KN/m^2$$

**Design for moment :**

The slab is discontinuous from 4 sides , so it will be assumed as :

(case 1 in analysis for moments)

Short - direction (a) case 4:

Max positive Moment :

Assume  $\phi 12$  :

$$d = \frac{350-20-12}{2} = 324mm$$

$$\frac{La}{Lb} = \frac{6.2}{6.68} = 0.92$$

$$\frac{C_{a\text{pos}}}{d_l} = 0.043$$

$$\frac{C_{a\text{pos}}}{l_l} = 0.043$$

$$Ma(+Ve) = (C_{a\text{dl}} \times W_{dl} \times La^2 \times 0.52) + (C_{a\text{ll}} \times W_{ll} \times La^2 \times 0.52)$$

$$Ma(+Ve) = (0.043 \times 14.2 \times 6.2^2 \times 0.52) + (0.043 \times 8 \times 6.2^2 \times 0.52)$$

$$Ma(+Ve) = 19.08KN.m/Rib$$

Positive moment for a direction =+19.08 KN.m/Rib

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

$$R_n = \frac{M_n}{b \times d^2} = \frac{(19.08/0.9) \times 10^6}{520 \times (324)^2} = 0.39 \text{ mpa}$$

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

$$p = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{220.60.39}{420}} \right) = 0.00096$$

$$A_s = pbd = 0.00096 \times 520 \times 324 = 161.74 \text{ mm}^2$$

$$A_{s.min} = \frac{\sqrt{f_c}}{4(f_y)} bwd \geq \frac{1.4}{f_y} bwd$$

$$A_{s.min} = \frac{\sqrt{24}}{4 \times 420} 120 \times 324 \geq \frac{1.4}{420} 120 \times 324$$

$$A_{s.min} = 113.37 < 129.6$$

As from eq= 129.2 ,but its < 161.74

So :  $A_{s.min} = 161.74 \text{ mm}^2$

2 $\phi$ 12 give  $A_s = 226.2 \text{ mm}^2 > 161.74 \text{ mm}^2$

– Check for strain:

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{226.2 \times 420}{0.85 \times 24 \times 520} = 8.955mm$$

$$c = \frac{8.955}{0.85} = 10.53mm$$

$$E_s = \frac{323-10.53}{10.53} \times 0.003 = 0.089 > 0.005$$

Design for negative moment for a direction at the discontinues edge =  $\frac{1}{3} Ma(Ve+)$

Assume  $2\phi 10/\text{Rib}$

$$d = \frac{350-20-10}{2} = 325mm$$

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

$$Rn = \frac{Mn}{b \times d^2} = \frac{(6.36/0.9) \times 10^6}{120 \times (325)^2} = 0.557 mpa$$

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

$$\rho = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.557}{420}} \right) = 0.00134$$

$$As = \rho bd = 0.00134 \times 120 \times 325 = 52.45mm^2$$

$$As.min = \frac{\sqrt{fc}}{4(fy)} bwd \geq \frac{1.4}{fy} bwd$$

$$As.min = \frac{\sqrt{24}}{4 \times 420} 120 \times 325 \geq \frac{1.4}{420} 120 \times 325$$

$$As.min = 113.72 < 130$$

$$As.min \text{ from eq} = 130 > 52.4mm^2$$

$$\text{So : } As.min = 130 mm^2$$

$$2\phi 10 \text{ give } As = 157.08 mm^2 > 130 mm^2$$

– Check for strain:

$$a = \frac{Asfy}{0.85fcb} = \frac{157.08 \times 420}{0.85 \times 24 \times 120} = 26.95mm$$

$$c = \frac{26.95}{0.85} = 31.7mm$$

$$Es = \frac{325-31.7}{31.7} \times 0.003 = 0.027 > 0.005$$

**long - direction (a) case 4:**

Max positive Moment :

Assume  $2\phi 10$  :

$$d = \frac{350-20-10}{2} = 325mm$$

$$\frac{La}{Lb} = \frac{6.2}{6.68} = 0.92$$

$$\frac{C_{a_{pos}}}{dl} = 0.0306$$

$$\frac{C_{a_{pos}}}{ll} = 0.0306$$

$$Ma(+Ve) = (C_{bdl} \times W_{dl} \times Lb^2 \times 0.52) + (C_{bll} \times W_{ll} \times Lb^2 \times 0.52)$$

$$Ma(+Ve) = (0.0306 \times 14.2 \times 6.68^2 \times 0.52) + (0.0306 \times 8 \times 6.68^2 \times 0.52)$$

$$Ma(+Ve) = 15.76 \text{ KN.m/Rib}$$

**Positive moment for a direction = +15.76 KN.m/Rib**

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

$$R_n = \frac{M_n}{b \times d^2} = \frac{(15.76/0.9) \times 10^6}{520 \times (325)^2} = 0.32 \text{ mpa}$$

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

$$\rho = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.32}{420}} \right) = 0.00077$$

$$A_s = \rho b d = 0.00077 \times 520 \times 325 = 129.8 \text{ mm}^2$$

$$A_{s.min} = \frac{\sqrt{f_c}}{4(f_y)} b w d \geq \frac{1.4}{f_y} b w d$$

$$A_{s.min} = \frac{\sqrt{24}}{4 \times 420} 120 \times 325 \geq \frac{1.4}{420} 120 \times 325$$

$$A_{s.min} = 113.7 < 130$$

$$A_s \text{ from eq} = 130 > 129.8$$

$$\text{So : } A_{s.min} = 130 \text{ mm}^2$$

$$2\phi 10 \text{ give } A_s = 157.08 \text{ mm}^2 > 130 \text{ mm}^2$$

– Check for strain:

$$a = \frac{A_s f_y}{0.85 f_c b} = \frac{157.08 \times 420}{0.85 \times 24 \times 520} = 6.22 \text{ mm}$$

$$c = \frac{6.22}{0.85} = 7.3 \text{ mm}$$

$$E_s = \frac{325 - 7.32}{7.32} \times 0.003 = 0.13 > 0.005$$

Design for negative moment for a direction at the discontinues edge =  $\frac{1}{3} Ma(Ve+)$

Assume 2 $\phi$ 10/Rib

$$d = \frac{350-20-10}{2} = 325mm$$

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

$$Rn = \frac{Mn}{b \times d^2} = \frac{(5.25/0.9) \times 10^6}{120 \times (325)^2} = 0.46 mpa$$

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

$$\rho = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.46}{420}} \right) = 0.0011$$

$$As = \rho bd = 0.0011 \times 120 \times 325 = 43.2mm^2$$

$$As.min = \frac{\sqrt{fc}}{4(fy)} bwd \geq \frac{1.4}{fy} bwd$$

$$As.min = \frac{\sqrt{24}}{4 \times 420} 120 \times 325 \geq \frac{1.4}{420} 120 \times 325$$

$$As.min = 113.72 < 130$$

$$As \text{ from eq} = 130 > 43.2 mm^2$$

$$\text{So : } As.min = 130 mm^2$$

$$2\phi 10 \text{ give } As = 157.08 mm^2 > 130 mm^2$$

– Check for strain:

$$a = \frac{Asfy}{0.85fcb} = \frac{157.08 \times 420}{0.85 \times 24 \times 120} = 26.95mm$$

$$c = \frac{26.95}{0.85} = 31.7mm$$

$$Es = \frac{325-31.7}{31.7} \times 0.003 = 0.02 > 0.005$$

### Design for shear :

The Shear in the slab calculated by using tributary Area for shear:

$$w_u = 1.2DL + 1.6LL = 1.2 \times 11.83 + 1. \times 5 = 22.2KN/m^2$$

$$V_u d = wu \times bf \left( \frac{ln}{2-d} \right)$$

$$V_u d = 22.2 \times 0.52 \times \left( \frac{6.2}{2-0.324} \right) = 32.05KN$$

$$Vc = \frac{1.1}{6} \sqrt{fcbwd}$$

$$V_c = \frac{1.1}{6} \sqrt{24} \times 120 \times 324 \times 10^{-3} = 34.92 \text{KN}$$

$$\Phi V_c = 0.75 \times 34.92 = 26.2 \text{KN}$$

$$\Phi V_c = 26.2 < 32.05$$

$$V_{s_{min}} = \frac{1}{3} bwd \geq \frac{1}{16} \sqrt{f_c} bwd$$

$$V_{s_{min}} = \frac{1}{3} 120 \times 324 \times 10^{-3} \geq \frac{1}{16} \sqrt{24} \times 120 \times 324 \times 10^{-3}$$

$$V_{s_{min}} = 12.96 > 11.9$$

$$\Phi V_c = 26.2 < V_u = 32.05 < \Phi(V_c + V_{s_{min}}) = 39.16$$

Provide minimum shear reinforcement

Use 2Φ 8 for stirrup  $A_v = 2 \times 50.26 = 100.53 \text{mm}^2$

$$\frac{A_{s_{min}}}{s} = \frac{b_w}{3F_{yt}} = \frac{1}{3} \frac{120}{420} = 0.0952$$

$$\frac{100.53}{s} = 0.0952, \quad s = 1055.99 \text{mm}$$

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

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

use 28@150mm

## 4.7 Design of Columns

### 4.7.1 Design of short column

Column (C-1 95) in the Basement floor

- **Load calculation**

$$DL = 4000 \text{KN}$$

$$LL = 1812 \text{KN}$$

$$P_u = 1.2 \times DL + 1.6 \times LL = 1.2 \times 4000 + 1.6 \times 1812 = 7700 \text{KN}$$

$$P_u(\text{design}) = 8000 \text{KN} > 7700 \text{KN}$$

$$\Phi P_n = P_u, \quad \Phi = 0.65 \text{ for tied column}$$

Let  $\rho = 0.02$

$$A_s = 0.02A_g$$

$$\Phi P_{max} = \Phi 0.8[0.85 \times f_c(A_g - A_s) + A_s \times F_y]$$

$$\Phi P_{max} = 0.6 \times 0.8[0.85 \times 24(A_g - 0.02 \times A_g) + 0.02 \times 420 \times 0.2A_g]$$

$$8000 \times 10^3 = 14.76A_g$$

$$A_g, req = 542005.42mm^2$$

Assume  $A_g = a \times a$ , then  $A_g = a^2$

$$a(req) = 736.2mm$$

take  $a(design) = 800mm$        $A_g = 640000mm^2 > 54005.42mm^2$

• **Check slenderness Effect**

In both direction:

$$\frac{Klu}{r} \leq 34 - 12 \frac{M1}{M2}$$

Lu : Actual unsupported length.

r: Radius of Gyration =  $0.3 \times h = \sqrt{\frac{1}{A}}$

K = 1.0, According to ACI318-08(10.12.2) the effective length factor (K) shall be permitted to be taken as 1.0

$$Lu = 5.1 - 0.36 = 4.74m$$

$$\frac{M1}{M2} = 1.0$$

For the width  $a = 0.8m$

$$r = 0.3 \times 0.8 = 0.24$$

$$\frac{1.0 \times 4.74}{0.24} \leq 34 - 12 \times 1.0$$

$$19.8 \leq 22$$

Short column in both direction.

• **Select the longitudinal bars**

$$\Phi P_{max} = \Phi 0.8[0.85 \times f_c(A_g - A_s) + A_s \times F_y]$$

$$8000 \times 10^3 = 0.65 \times 0.8[0.85 \times 24(640000 - A_s) + 0.02 \times 420 \times A_s]$$

$$A_s(req) = 5827.37mm^2 \quad \text{try } \Phi 25 \text{ with } A_s = 490.87mm^2$$

$$\text{Use } 12\Phi 25 \text{ with } A_s = 5890.5mm^2 > 5827.37mm^2$$

$$P(\rho) = \frac{5890.5}{640000} = 0.0092 < 0.01$$

$$\text{try } \Phi 28 \text{ with } A_s = 615.75mm^2$$

$$\text{Use } 14\Phi 28 \text{ with } A_s = 8620.6mm^2 > A_s(req)$$

$$P(\rho) = \frac{8620.6}{640000} = 0.013$$

• **Design for ties**

Use ties  $\Phi 10$  with spacing shall not exceed the smallest of:

- $48 \times ds = 48 \times 10 = 480mm$
- $16 \times db = 16 \times 28 = 448mm$
- *The least dim. of the column = 800mm*

Use  $\Phi 10 @ 300$  mm.

- **Check for code requirements**

$$* \text{ Clear spacing between longitudinal bars} = \frac{800-402-102-628}{5} = 114.4mm$$

$$\text{Gross reinforcement ratio} = 0.01 < 0.0134 < 0.08$$

\* No of bars = 14 > 4 for square columns.

\* Min ties diameter :  $\Phi 10$  for  $\Phi 32$  longitudinal bars and smaller.

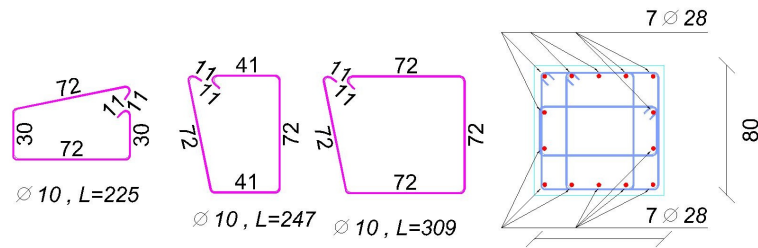


Figure 4.11: Column C95 Details

## 4.7.2 Design of long column(C 205) in basement floor

### – Load calculation

$$DL = 1160 \text{ KN}$$

$$LL = 520 \text{ KN}$$

$$P_u = 1.2 \times DL + 1.6 \times LL = 1.2 \times 1160 + 1.6 \times 520 = 2224 \text{ KN}$$

$$P_u(\text{design}) = 2500 \text{ KN} > 2224 \text{ KN}$$

$$\Phi P_n = P_u \quad \Phi = 0.65 \text{ for tied column}$$

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

$$A_s = 0.02 A_g$$

$$\Phi P_{max} = \Phi 0.8 [0.85 \times f'_c (A_g - A_s) + A_s \times F_y]$$

$$\Phi P_{max} = 0.65 \times 0.8 [0.85 \times 24 (A_g - 0.02 \times A_g) + 0.02 \times 420 \times 0.2 A_g]$$

$$A_{g, req} = 169376.7 \text{ mm}^2$$

$$\text{Assume } A_g = a \times a, \quad \text{then } A_g = a^2$$

$$a(\text{req}) = 411.55 \text{ mm}$$

$$\text{take } a(\text{design}) = 500 \text{ mm}, \quad A_g = 250000 \text{ mm}^2 > 169376.7 \text{ mm}^2$$

### – Check slenderness Effect

In both direction:

$$\frac{Klu}{r} \leq 34 - 12 \frac{M_1}{M_2}$$

$L_u$  : Actual unsupported length.

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

$K = 1.0$ , According to ACI318-08(10.12.2) the effective length factor ( $K$ ) shall be permitted to be taken as 1.0

$$L_u = 5.1 - 0.36 = 4.74 \text{m}$$

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

For the width  $a = 0.5 \text{m}$

$$r = 0.3 \times 0.5 = 0.15 \text{m}$$

$$\frac{1.0 \times 4.74}{0.15} \leq 34 - 12 \times 1.0$$

$31.6 > 22$ , the column is slender.

long column in both direction.

– Calculate  $e(\min)$  and  $M(\min)$ :

$$e(\min) = 15 + 0.03 \times h = 15 + 0.03 \times 500 = 30 \text{mm}$$

$$p_u = 2500 \text{KN}$$

$$M(\min) = p_u \times e(\min) = 0.03 \times 2500 = 75 \text{KN.m}$$

– Calculate  $EI$ :

$$I_g = \frac{bh^3}{12} = \frac{500 \times 500^3}{12} = 5.2 \times 10^9 \text{mm}^4$$

$$E_c = 4700 \sqrt{f'_c} = 23025.2 \text{mpa}$$

$$d_{ns} = \frac{1.2D(\text{sustained})}{1.2D+1.6L} = \frac{1.2 \times 1160}{2224} = 0.626$$

$$EI = \frac{0.4E_c I_g}{1+d_{ns}} = \frac{0.4 \times 23025.2 \times 5.2}{1+0.626} = 29454.13 \text{KN.m}^2$$

– **Determine the Euler buckling load**

$$P_c = \frac{\pi^2 EI}{K L_u^2} = \frac{\pi^2 \times 29454.13}{1.0 \times 4.74^2} = 12938.66 \text{KN}$$

\* **Calculate moment magnifier factor**

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} = 0.6 + 0.4 \times 1 = 1$$

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$$\delta n s = \frac{Cm}{1 - \frac{Pu}{0.75Pc}} = \frac{1.0}{1 - \frac{2500}{0.75 \times 12938.66}} = 1.34 > 1.0$$

The magnified (e) and (M):

$$e = e(\min) \times ns = 1.34 \times 30 = 40.2$$

$$M = ns \times M(\min) = 1.34 \times 75 = 100.5 \text{ K N.m}$$

By using interaction diagram for square tied columns :

$$F_c = 24 \text{ MPa} = 3.5 \text{ Ksi}$$

$$f_y = 420 \text{ Mpa} = 60 \text{ Ksi}$$

Assume  $\Phi 20$

$$\gamma = \frac{d-d'}{h} = \frac{500-240-210-20}{500} = 0.75$$

$$\frac{e}{h} = \frac{40.2}{500} = 0.08$$

$$\frac{\Phi P n}{A_g} = \frac{P_u}{A_g} = \frac{2500 \times 10^3}{500 \times 500} = 0.145 = 1.45 \text{ Ksi}$$

Use A-9b & A-10b.

A-9b:

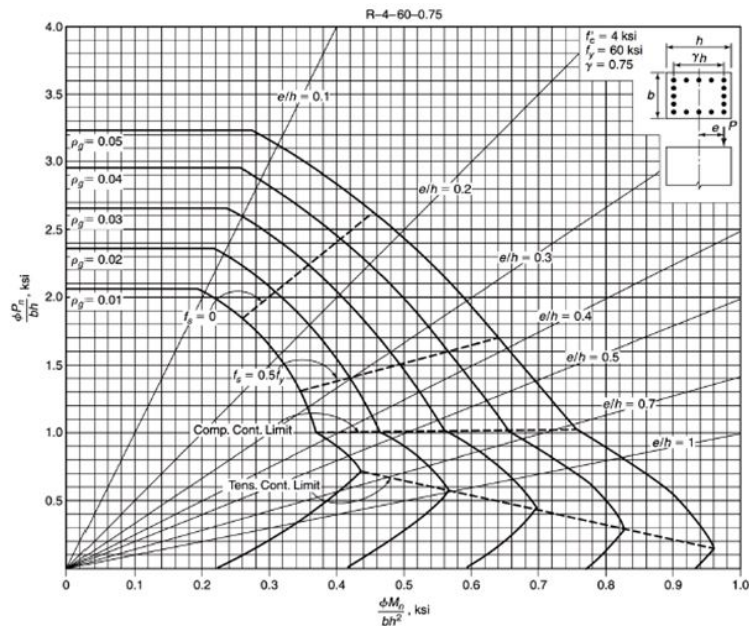


Figure 4.12: Interaction Diagram

From  $\frac{e}{h} = 0.08$  and  $\frac{\Phi P n}{A_g} > \rho_{req}$  less than 0.01

$$\rho = 0.01$$

A-10b :

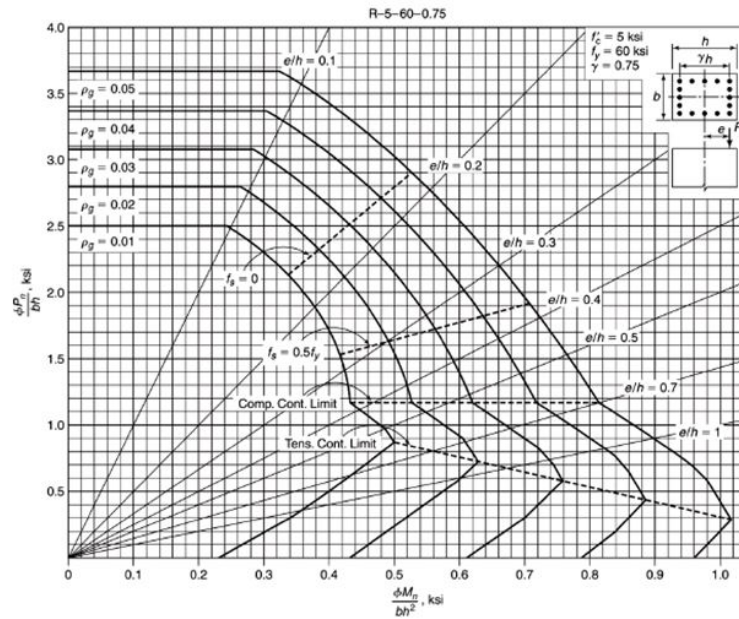


Figure 4.13: Interaction Diagram

From  $\frac{e}{h} = 0.08$  and  $\frac{\Phi P_n}{A_g} > \rho_{req}$  less than 0.01

$$\rho = 0.01$$

Using interpolation to solve it for  $f_c = 3.5$  Ksi

$$\frac{0.01 - 0.01}{5 - 4} = \frac{0.01 - x}{4 - 3.5} = 0, \quad x = 0.01$$

$$\rho = 0.01$$

\* **Select reinforcement**

$$A_{req} = \rho \times A_g = 0.01 \times 500 \times 500 = 2500 \text{ mm}^2$$

$$\text{Select } \Phi 20, \quad A_s = 314.16 \text{ mm}^2$$

$$\text{Use } 8\Phi 20 \text{ with } A_s = 2513.3 \text{ mm}^2 > A_s( req ) = 2500 \text{ mm}^2$$

\* **Design for ties**

Use ties  $\Phi 10$  with spacing shall not exceed the smallest of:

- $48 \times d_s = 48 \times 10 = 480 \text{ mm}$
- $16 \times d_b = 16 \times 20 = 320 \text{ mm}$
- The least dim of the column = 500 mm

Use  $\Phi 10$  @300 mm.

\* **Check for code requirements**

- Clear spacing between longitudinal bars =  $\frac{500-402-102-320}{2}=170$  mm >25 mm
- Gross reinforcement ratio =  $0.01 \leq 0.08$
- No of bars =  $8 > 4$  for squar columns.
  
- Min ties diameter :  $\Phi 10$  for  $\Phi 32$  longitudinal bars and smaller.

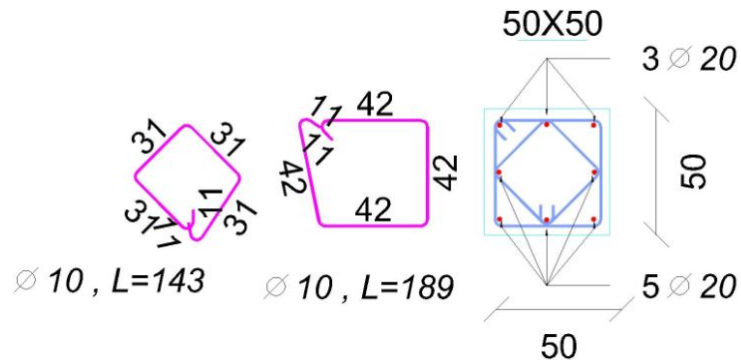


Figure 4.14: Column C205 Details

## 4.8 Design of Shear Wall

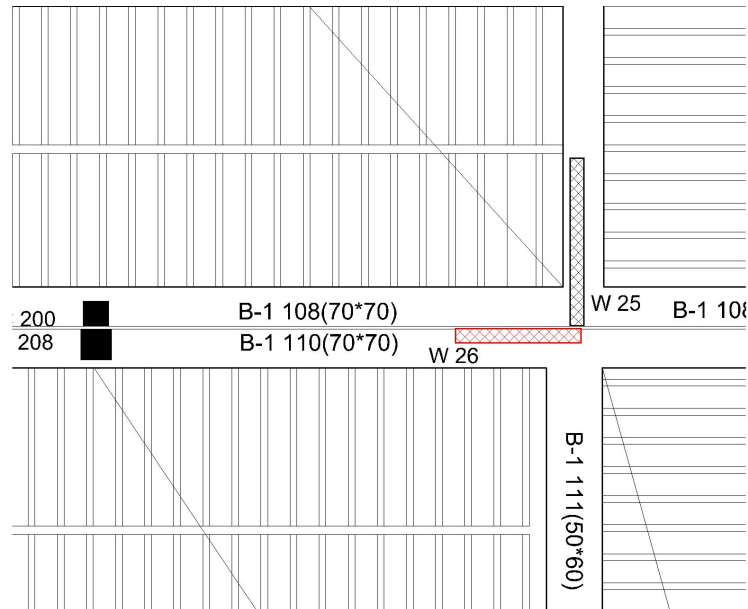


Figure 4.15: Shear Wall W26

Shear wall thickness ( $h$ ) = 25 cm

Shear wall length ( $L_w$ ) = 2.25 m

Story high = 5.1 m

Shear force ( $V_u$ ) = 41 KN

Bending moment = 709 KN.m

Vertical normal force = 3117.2 KN

\* **Check max shear strength permitted**

$$\Phi V_{n, max} = 0.75 \times 0.83 \times \sqrt{f_c'} \times h \times d$$

$$d = 0.8 \times L_w = 0.8 \times 2250 = 1800mm = 1.8m$$

$$\Phi V_{n, max} = 0.75 \times 0.83 \times \sqrt{24} \times 1.8 \times 225 = 1235.1KN > V_u = 41KN$$

\* **Calculate shear wall strength provided by concrete**

Critical section for concrete is the smallest of :

$$\cdot \frac{L_w}{2} = \frac{225}{2} = 1.125m$$

$$\cdot \frac{\Sigma hw}{2} = \frac{5.1 \times 1 + 4.85 \times 3 + 3.85 \times 4}{2} = 17.525m$$

$$\cdot \text{Story height} = 5.1m$$

Vc is the smallest of:

$$\cdot Vc = \frac{1}{6} \sqrt{f_c'} \times h \times d = \frac{1}{6} \sqrt{24} \times 1.8 \times 225 = 330.68KN$$

$$\cdot Vc = 0.27 \sqrt{f_c'} \times h \times d + \frac{Nu \times d}{4 \times Lw} = 0.27 \sqrt{24} \times 1.8 \times 225 + \frac{3117.2 \times 1.8}{4 \times 2.25} = 1159.14KN$$

$$\cdot Vc = \left[ 0.05 \lambda \sqrt{f_c'} + \frac{Lw(0.1 \times \sqrt{f_c'} + 0.2 \frac{Nu}{hLw})}{\frac{Mu}{Vu} - \frac{Lw}{2}} \right] hd$$

Mu at critical section = 208.1 KN.m

$$\frac{208.1}{3117.2} - \frac{2.25}{2} = -1.06 < 0.0$$

This equation failed and won't be in control.

\* **Determine required horizontal reinforcement**

Vu=41KN.

$$0.5 \times \theta \times Vc = 0.5 \times 0.75 \times 330.68 = 124.005KN > Vu = 41KN$$

No need for reinforcement

Horizontal reinforcement at the minimum

$$\text{Let } (\rho) = 0.025 = \frac{As}{s \times h}$$

try  $\Phi 12$

let  $s = 300mm$

$$\rho = \frac{2 \times 113.1}{300 \times 250} = 0.0035 > 0.0025$$

$$0.0025 = \frac{2 \times 113.1}{s \times 250} > S(req) = 362mm$$

Take  $s = 400mm$

Max spacing is the least of:

$$\cdot \frac{Lw}{5} = \frac{2.25}{5} = 0.45m$$

$$\cdot 3h = 3 \times 0.25 = 0.75m$$

$$\cdot 450mm$$

For horizontal reinforcement use  $\Phi 12 @ 200mm$

\* **Determine required vertical reinforcement**

$$\frac{\Sigma hw}{Lw} = \frac{5.1+3 \times 4.85+3.85 \times 4}{2.25} = 15.57$$

$$\rho v, \min > 0.0025 + 0.5(2.5 - \frac{\Sigma hw}{Lw}) [\rho - 0.0025] \geq 0.0025$$

$$\text{Take } (\rho)v, \min = 0.0025$$

Max spacing is the least of :

$$\cdot \frac{Lw}{3} = \frac{2.25}{3} = 0.75m$$

$$\cdot 3h = 3 \times 0.25 = 0.75m$$

$$\cdot 450mm$$

Use  $\Phi 12@300mm$

\* **Design for flexural(uniformly distributed flexural reinforcement**

Check moment strength based on required vertical reinforcement for shear

The uniformly distributed vertical reinforcement  $12@300mm$

$$Ast = \frac{2250}{300} \times 2 \times 113.1 = 1696.5mm^2$$

$$\omega = \left(\frac{Ast}{Lw \times h}\right) \frac{fy}{fc} = \frac{1696.5}{2250 \times 250} * \frac{420}{24} = 0.053$$

$$a = \frac{Pu}{Lw \times fc \times h} = \frac{3117.2 \times 1000}{2250 \times 24 \times 250} = 0.23$$

$$\frac{c}{Lw} = \frac{w+a}{2 \times w + 0.85\beta 1} = \frac{0.23+0.053}{2 \times 0.053 + 0.85 \times 0.85} = 0.342$$

$$\Phi Mn = \Phi [0.5Ast \times fy \times Lw (1 + \frac{Pu}{As/fy}) (1 - \frac{c}{Lw})]$$

$$\Phi Mn = 0.9 [0.5 \times 1696.5 \times 420 \times 2250 (1 + \frac{3117.2}{420 \times 24}) (1 - 0.342)] \times 10^{-6}$$

$$\Phi Mn = 621.5KN.m < 709KN.m$$

$$\Phi Mn < Mu$$

The uniformly distributed vertical reinforcement  $\Phi 12@300mm$  is not adequate for flexure, therefore the amount of vertical reinforcement must be increased:

Try  $\Phi 12@100mm$

$$Ast = \frac{2250}{100} \times 2 \times 113.1 = 5089.5mm^2$$

$$\omega = \left(\frac{Ast}{Lw \times h}\right) \frac{fy}{fc} = \frac{2035.8}{2250 \times 250} \times \frac{420}{24} = 0.15$$

$$\frac{c}{Lw} = \frac{w+a}{2 \times w + 0.85\beta 1} = \frac{0.23+0.063}{2 \times 0.063 + 0.85 \times 0.85} = 0.37$$

$$\Phi Mn = 0.9 [0.5 \times 5089.5 \times 420 \times 2250 (1 + \frac{3117.2}{420 \times 24}) (1 - 0.37)] \times 10^{-6} = 1753.7KN.m$$

Use  $\Phi 12 @ 100$  mm for vertical reinforcement.

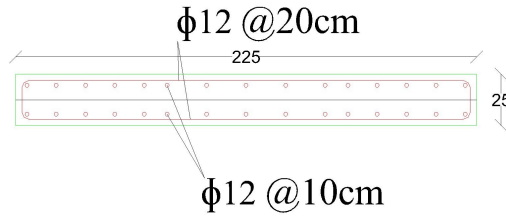


Figure 4.16: Shear Wall Reinforcement

## 4.9 Design of Isolated Footing

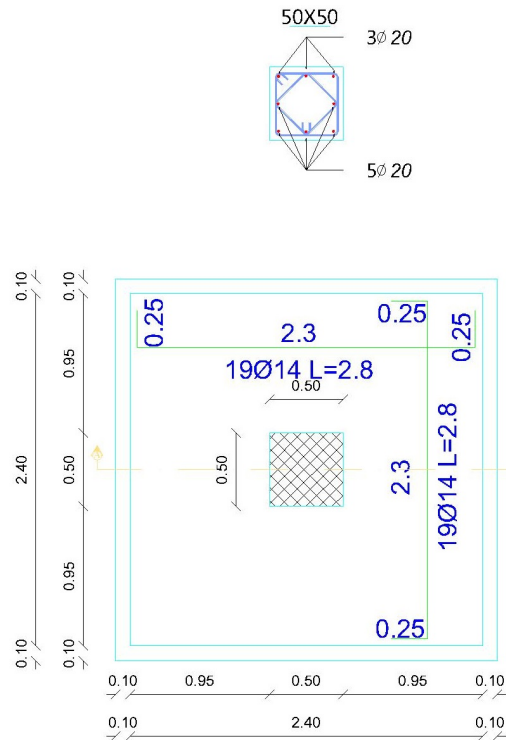


Figure 4.17: Isolated Footing Plan

$$P_d = 1483\text{KN}(\text{Service})$$

$$P_L = 662\text{KN}(\text{Service})$$

$$Pu = 2839KN(\text{Factored})$$

$$Pu(\text{design}) = 3000KN(\text{factored})$$

Column Dimensions :  $50 \times 50cm$ .

Surcharge =  $0.5m$ .

Allowable bearing capacity  $q_{all} = 400KN/m^2$ .

Soil Density =  $18KN/m^3$

**Area of footing:** Assume  $h = 60cm$ .

$$q_{all,net} = 400 - 0.6 \times 25 - 0.6 \times 18 = 374.2KN/m^2$$

$$Area(A) = \frac{PD+PL}{q_{all,net}} = \frac{1483+662}{374.2} = 5.73m^2$$

use  $L = B = 2.45m$  ,  $A = 6m^2 > 5.73m^2$

**Depth of footing:**

Assume  $h = 60cm$ .

\* **Check one-way shear**

$$q_{all} = \frac{Pu}{Area} = \frac{3000}{6} = 500 KN/m^2$$

Assume  $\Phi 14$

$$d = 600 - 75 - 14 = 511mm$$

$$\Phi V_c = \Phi \frac{1}{6} \times \sqrt{f_c'} \times bw \times d = 0.75 \times \frac{1}{6} \sqrt{24} \times 2450 \times 0.511 = 766.66KN$$

$$V_{ud} = q_{all} \times \left( \frac{B-a}{2} - d \right) \times L = 500 \times \left( \frac{2.45-0.464}{2} - 0.511 \right) \times 2.45 = 590.45KN.$$

$$\Phi V_c = 766.66KN > V_{ud} = 590.45KN$$

One way shear is safe.

\* **Check two-way shear**

To calculate  $V_u$  at critical section which take square shape :

$$\text{Inner area} = (0.5 + 0.511) \times (0.5 + 0.511) = 1.02m^2$$

$$\text{Outer area} = \text{area of footing} = 6m^2$$

$$V_u = qu \times (\text{outer area} - \text{inner area}) = 500 \times (6 - 1.02) = 2490KN$$

According to *ACI*,  $V_c$  shall be the smallest of :

- $V_c = \frac{1}{6}(1 + \frac{2}{\beta_c})\sqrt{f_c'} \times b \times d = 0.5\sqrt{f_c'} \times b_0 \times d$
- $V_c = \frac{1}{12}(\frac{a_s}{b_0/d} + 2)\sqrt{f_c'} \times b_0 \times d = 1.0\sqrt{f_c'} \times b_0 \times d$
- $V_c = \frac{1}{3}\sqrt{f_c'} \times b_0 \times d \dots \dots \text{control}$

Where :

$$c = \frac{a}{b} = \frac{50}{50} = 1$$

$$b_o = 4 \times (0.5 + 0.511) = 4.04m.$$

$a_s = 40$  for interior column.

$$\cdot V_c = \frac{1}{3}\sqrt{24} \times 4.04 \times 511 = 3671.62KN$$

$$\Phi V_c = 0.75 \times 3671.62 = 2753.72KN > V_u = 2490KN$$

Two-way shear is safe.

**\* Design for flexural reinforcement**

$$M_u = 500 \times 2.45 \times 0.975 \times \frac{0.975}{2} = 582.23KN.m$$

$$M_n = \frac{582.23}{0.9} = 646.95KN.m$$

$$R_n = \frac{M_n}{b \times d^2} = \frac{646.95 \times 10^{-6}}{2.45 \times 0.511^2} = 0.001mpa$$

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

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2 \times m \times R_n}{f_y}}) = \frac{1}{20.59}(1 - \sqrt{1 - \frac{2 \times 20.59 \times 0.001}{420}}) = 2.38 \times 10^{-6}$$

$$A_{req} = \rho \times b \times d = 2.38 \times 10^{-6} \times 2450 \times 511 = 2.98mm^2$$

$$A_{s_{min}} = 0.0018 \times 2450 \times 600 = 2646mm^2$$

Use  $20\Phi 14$  with  $A_s = 3078.76mm^2 > A_{s_{req}}(min) = 2646mm^2$

Use  $20\Phi 14$  in both direction with  $A_s = 3078.76mm^2$

**\* Check of strain**

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

$$3078.76 \times 420 = 0.85 \times 24 \times 600 \times a$$

$$a = 105.64mm$$

$$x = \frac{a}{\beta_1} = \frac{105.64}{0.85} = 124.28mm$$

$$\epsilon = \frac{975 - 124.28}{124.28} \times 0.003 = 0.02 > 0.005, \quad \theta = 0.9$$



## 4.10 Design of Basement Wall and Strip Footing

### \* Design of Basement Wall

· Load Calculation

$f_c$	$f_y$	$Y_s$	$q_{all}$	$\phi$	surcharge
24Mpa	420 Mpa	18 KN/m <sup>3</sup>	400 KN/m <sup>2</sup>	30	5KN/m <sup>2</sup>

$$C_a = 1 - \sin\theta = 1 - \sin30 = 0.5 \text{ ( Static Earth Pressure)}$$

$$P_a = C_a \times h \times \gamma = 0.5 \times 5.8 \times 18 = 52.2KN/m^2$$

$$h_s = \frac{W_s}{W} = \frac{5}{18} = 0.278m$$

$$P_s = C_a \times h_s \times \gamma = 0.5 \times 0.278 \times 18 = 2.5KN/m^2$$

Ca	Pa	hs	Ps
0.5	52.2 KN/m <sup>2</sup>	0.278 m	2.5 KN/m <sup>2</sup>

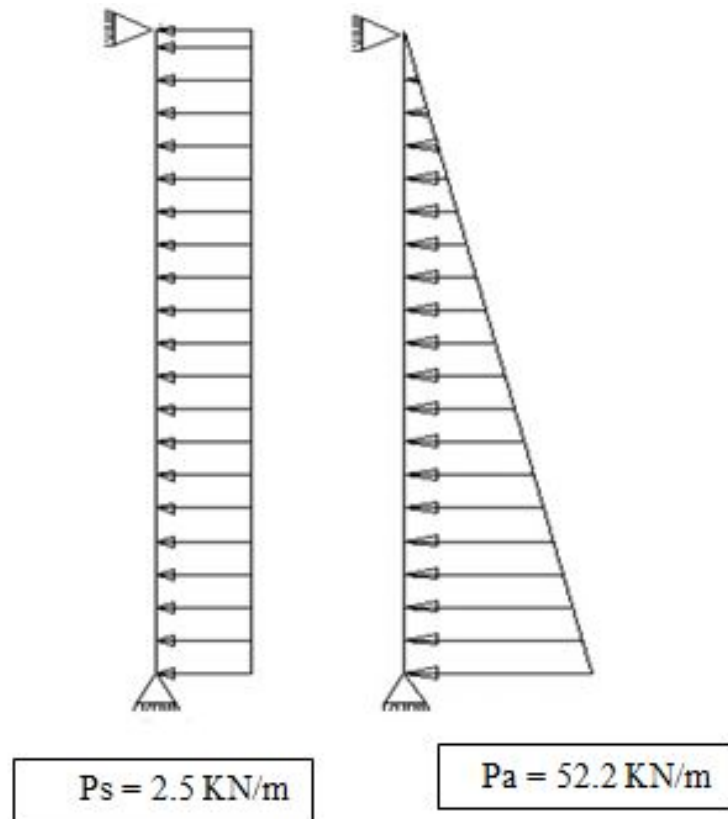


Figure 4.19: Static System

· Design of Bending Moment

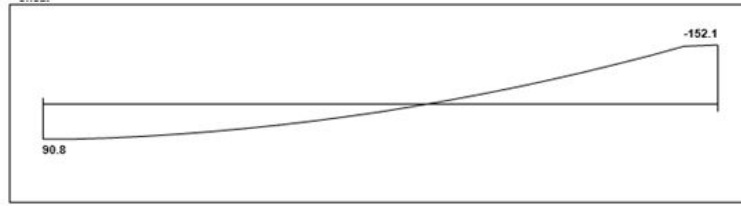


Figure 4.20: Shear Envelope Diagram of Basement Wall

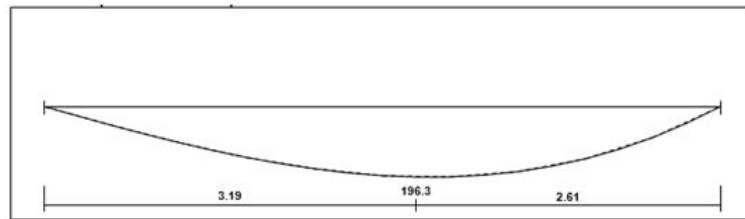


Figure 4.21: Moment Envelope Diagram of Basement Wall

$$M_u = +196.3 \text{ kN.m}$$

$$d = 320 - 50 - \frac{20}{2} = 260 \text{ mm}$$

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

$$R_n = \frac{1963 \times 10^6}{0.9 \times 1000 \times 260^2} = 3.22 \text{ mpa}$$

$$m = \frac{f_y}{0.85 \times f'_c}$$

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

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right) = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 3.22}{420}} \right) = 0.0084$$

$$A_{s,req} = 0.0084 \times 1000 \times 260 = 2184 \text{ mm}^2/\text{m}$$

Use  $\Phi 20 @ 14 \text{ cm}$

with  $A_{s,provided} = 2242 \text{ mm}^2/\text{m} > A_{s,req} = 2184 \text{ mm}^2/\text{m}$

$A_{s,min}$  for vertical bars:

$$- 0.0015 \times b \times h = 0.0015 \times 1000 \times 320 = 480 \text{ mm}^2/\text{m}$$

$$- 0.25 \frac{\sqrt{24}}{420} \times 1000 \times 260 = 758.1 \text{ mm}^2/\text{m}$$

$$- \frac{1.4}{420} \times 1000 \times 260 = 866.6 \text{ mm}^2/\text{m}$$

Use  $\Phi 12 @ 14 \text{ cm}$

with  $A_{s,provided} = 1100 \text{ mm}^2/m > A_{s,req} = 866.6 \text{ mm}^2/m$

Use  $\Phi 20@14\text{cm}$  with  $A_{s,provided} = 2242 \text{ mm}^2/m > A_{s,req} = 2184 \text{ mm}^2/m$

For horizontal bars

$$0.002 \times b \times h = 0.002 \times 320 \times 1000 = 640 \text{ mm}^2/m$$

Use  $\Phi 14@20\text{cm}$  with  $A_{s,provided} = 769.6 \text{ mm}^2/m > A_{s,req} = 640 \text{ mm}^2/m$

· Check for shear

$$d = 320 - 50 - 10 = 260\text{mm}$$

$$\phi V_c = 0.75 \times \frac{1}{6} \times \sqrt{f'_c} \times bw \times d$$

$$\phi V_c = 0.75 \times \frac{1}{6} \times \sqrt{24} \times 1000 \times 260 = 159.2\text{KN}$$

$$V_u = 152.1\text{KN} < \phi V_c = 159.5$$

The thickness is enough

**\* Design of Strip footing**

· Load Calculation

$$H(\text{slab}) = 0.3\text{m}$$

$$H(\text{BC}) = 0.1\text{m}$$

$$\text{Weight of wall (D.L.)} = \text{height} \times \text{thickness} \times 1\text{m wide} \times \gamma_c$$

$$\text{Weight of wall (D.L.)} = 5.8 \times 0.32 \times 25 = 46.4\text{KN/m}$$

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

$$Q_{net} = 400 - 0.8 \times 18 - 0.3 \times 25 = 378.1\text{KN/m}^2$$

$$A = \frac{P_n}{q_{net}} = \frac{46.6}{378.1} = 0.122\text{m}^2$$

$$B = 0.122\text{m}$$

Take  $B = 70\text{cm}$

$$P_u = 1.4 \times 46.4 = 64.96\text{KN/m}$$

$$q_u = \frac{P_u}{A} = \frac{64.96}{1 \times 1} = 64.96\text{KN/m}^2$$

· Check of Shear

$$h = 300\text{mm}$$

$$d = 300 - 50 - 10 = 240\text{mm}$$

$$V_u = 1 \times (0.5 - 0.16 - 0.24) \times 64.96 = 6.496 \text{ KN}$$

$$\Phi V_c = 0.75 \frac{1}{6} \times \sqrt{f'_c} \times bw \times d = 0.75 \times \frac{1}{6} \sqrt{24} \times 240 \times 10^3 = 146.9 \text{ KN}$$

$$\Phi V_c \gg V_u$$

No Shear Reinforcement required

$$M_u = 64.96 \times 0.34 \times 1 \times \frac{0.34}{2} = 3.75 \text{ KN.m}$$

$$R_n = \frac{M_n}{b \times d^2} = \frac{3.75 \times 10^6}{1000 \times 240^2} = 0.065 \text{ mpa}$$

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

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

$$\rho = \frac{1}{20.6} \left( 1 - \sqrt{1 - \frac{2 \times 20.6 \times 0.065}{420}} \right) = 0.00015$$

$$A_{s( req)} = 0.00015(1000)(240) = 36 \text{ mm}^2$$

$$A_{s, min} = 0.0018 \times b \times h = 0.0018 \times 1000 \times 300 = 540 \text{ mm}^2$$

Select  $\Phi 12 @ 20 \text{ cm/m}$  with  $A_{s, prov.} 565 \text{ mm}^2/\text{m}$

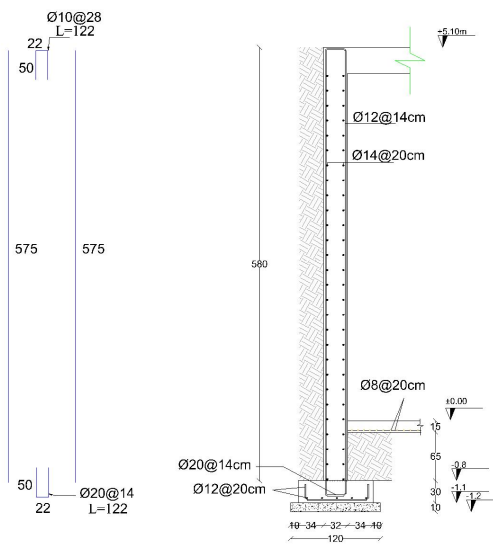


Figure 4.22: Basement Wall Details

### 4.11 Design of Stairs

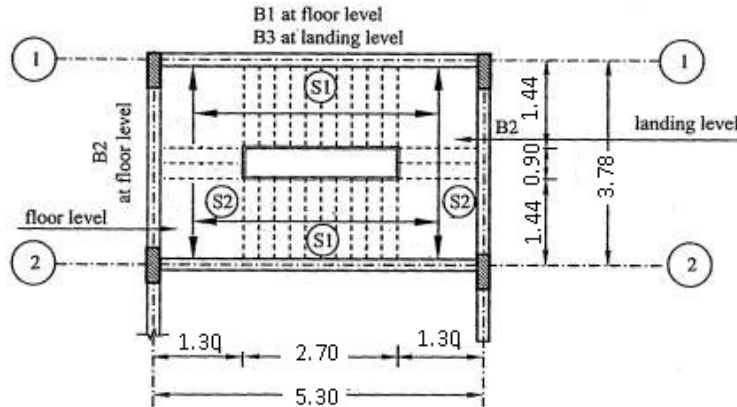


Figure 4.23: Stair Details

**- Determination of Slab Thickness**

$$h_{min} = \frac{L}{20} = \frac{4}{20} = 0.2m = 20cm$$

Take  $h_{min} = 250mm$ .

**- Loads**

\* Flight Dead Load computation:

$$\theta = \tan^{-1}\left(\frac{rise}{run}\right) = \tan^{-1}\left(\frac{176}{300}\right) = 30.4^\circ$$

Material	Quality Density $KN/m^3$	W $KN/m$
Tiles	23	$23 \times \left(\frac{0.176+0.35}{0.3}\right) \times 0.03 \times 1 = 1.21$
mortar	22	$22 \times \left(\frac{0.15+0.3}{0.3}\right) \times 0.02 \times 1 = 0.7$
Stair steps	25	$\frac{25}{0.3} \times \left(\frac{0.15+0.3}{2}\right) \times 1 = 2.2$
Reinforced Concrete solid slab	25	$\frac{25 \times 0.25 \times 1}{\cos 30.4} = 7.25$
Plaster	22	$\frac{22 \times 0.03 \times 1}{\cos 30.4} = 0.76$
Total Dead Load, $KN/m$		12.12

\* Landing Dead Load Computation:

Material	Quality Density $KN/m^3$	$\gamma \times h \times 1$ $KN/m$
Tiles	22	$22 \times 0.03 \times 1 = 0.66$
mortar	22	$22 \times 0.02 \times 1 = 0.44$
Reinforced Concrete Solid Slab	25	$25 \times 0.025 \times 1 = 6.25$
Plaster	22	$22 \times 0.03 \times 1 = 0.66$
Total Dead Load		8.01

\* Live Load:  $LL = 5KN/m^2$

\* Total factored Load:  $w = 1.2D + 1.6L$

for flight:  $w = 1.2 \times 12.12 + 1.6 \times 5 = 22.54KN/m$

for landing:  $w = 1.2 \times 8.01 + 1.6 \times 5 = 17.61KN/m$

– Design of slab S1:

Slab S1 is supported at the centerline of slabs S2.

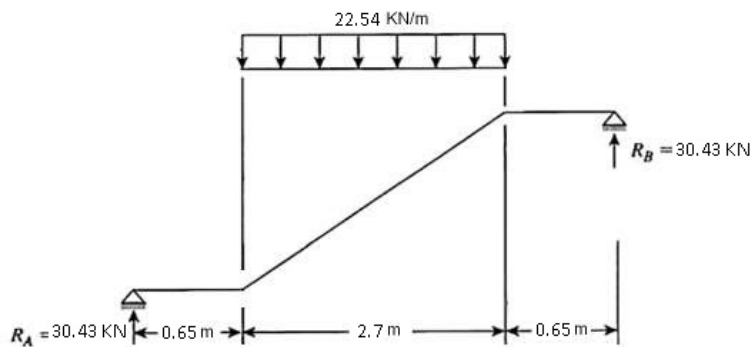


Figure 4.24: Static System of Stair  $S_1$

The reaction at each end

$$R = \frac{wl}{2} = \frac{22.54 \times 2.7}{2} = 30.43KN$$

\* Check for shear strength:

Assume bar diameter  $\phi 14$  for main reinforcement.

$$d = h - 20 - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 223mm$$

Take the maximum shear as the support reaction:

$$V_u = 30.43KN$$

$$V_c = \frac{1}{6}\sqrt{f'_c} \times b_w \times d = \frac{1}{6}\sqrt{24} \times 1000 \times 223 \times 10^{-3} = 182.1 \text{ KN/m strip}$$

$$\phi V_c = 0.75 \times 182.1 = 136.56 \text{ KN/m strip}$$

$$V_{u,max} = 30.43 \text{ KN} < \frac{1}{2}\phi V_c = \frac{136.56}{2} = 68.3 \text{ KN}$$

The thickness of the slab is adequate enough.

\* Calculate the maximum bending moment and steel reinforcement:

$$M_u = 30.43 \times (0.65 + 1.35) - 22.54 \times \frac{1.35^2}{2} = 46.4 \text{ KN.m}$$

$$M_n = \frac{M_u}{\phi} = \frac{46.4}{0.9} = 51.56 \text{ KN.m/m}$$

Assume bar diameter  $\Phi 14$  for main reinforcement.

$$R_n = \frac{M_n}{bd^2} = \frac{51.56 \times 10^6}{1000 \times 223^2} = 1.04 \text{ mpa}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{400}{0.85 \times 24} = 19.61$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2R_n m}{f_y}} \right) = \frac{1}{19.61} \left( 1 - \sqrt{1 - \frac{2 \times 1.04 \times 19.61}{400}} \right) = 0.00267$$

$$A_s = \rho b d = 0.00267 \times 1000 \times 223 = 595.41 \text{ mm}^2$$

$$A_{s,min} = 0.0018 b h = 0.0018 \times 1000 \times 250 = 450 \text{ mm}^2$$

$$A_s = 595.41 \text{ mm}^2 > A_{s,min} = 450 \text{ mm}^2$$

Use  $\Phi 10$  then

$$n = \frac{A_s}{A_{s\phi 10}} = \frac{595.41}{79} = 7.63$$

$$s = \frac{1}{n} = \frac{1}{7.63} = 0.13 \text{ m}$$

Take  $8\Phi 10/m$

$\Phi 10 @ 100 \text{ mm}$

Step(s) is the smallest of:

1.  $3h = 3 \times 250 = 750 \text{ mm}$ .

2.  $450 \text{ mm}$ .

3.  $s = 380 \left( \frac{280}{f_s} \right) - 2.5 \times 20 = 337.4 \text{ mm}$ , but

$$s \leq 300 \left( \frac{280}{f_s} \right) = 300 \left( \frac{280}{\frac{2}{3} \times 420} \right) = 300 \text{ mm}$$

$$s = 100 \text{ mm} < s_{max} = 300 \text{ mm}$$

\* Temperature and shrinkage reinforcement.

$$A_s = 0.0018 b h = 0.0018 \times 1000 \times 250 = 450 \text{ mm}^2$$

$$n = \frac{A_s}{A_{s\phi14}} = \frac{450}{79} = 5.76$$

$$s = \frac{1}{n} = \frac{1}{5.76} = 0.17 \text{ m}$$

Take  $6\Phi10/m$

$\Phi10@150 \text{ mm}$

Step (s) is the smallest of :

1.  $5h = 5 \times 250 = 1250 \text{ mm}$

2.  $450 \text{ mm}$

$$s = 150 \text{ mm} < s_{max} = 450 \text{ mm}$$

– Design of slab S2:

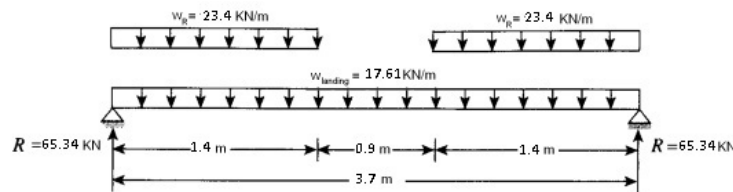


Figure 4.25: Static System of Stair  $S_2$

\* Check for shear strength:

Assume bar diameter  $\phi14$  for main reinforcement.

$$d = h - 20 - \frac{d_b}{2} = 250 - 20 - \frac{14}{2} = 223 \text{ mm}$$

Take the maximum shear as the support reaction:

$$V_u = 65.34 \text{ KN}$$

$$V_c = \frac{1}{6} \sqrt{f'_c} \times b_w \times d = \frac{1}{6} \sqrt{24} \times 1000 \times 223 \times 10^{-3} = 182.1 \text{ KN/m strip}$$

$$\phi = 0.75$$

$$\phi V_c = 0.75 \times 182.1 = 136.56 \text{ KN/m strip}$$

$$V_{u,max} = 65.34 \text{ KN} < \frac{1}{2} \phi V_c = \frac{136.56}{2} = 68.3 \text{ KN}$$

The thickness of the slab is adequate enough.

\* Calculate the maximum bending moment and steel reinforcement:

$$M_u = 65.34 \times 2 - 17.61 \times \frac{1.85^2}{2} - 23.4 \times 1.4 \times \left( \frac{1.4}{2} + 0.3 \right) = 67.78 \text{ KN.m}$$

$$M_n = \frac{M_u}{\phi} = \frac{67.78}{0.9} = 75.31 \text{ KN.m/m}$$

Assume bar diameter  $\Phi 14$  for main reinforcement.

$$d = 223 \text{ mm}$$

$$R_n = \frac{M_n}{bd^2} = \frac{75.31 \times 10^6}{1000 \times 223^2} = 1.51 \text{ mpa}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{400}{0.85 \times 24} = 19.61$$

$$\rho = \frac{1}{m} \left( 1 - \sqrt{1 - \frac{2R_n m}{f_y}} \right) = \frac{1}{19.61} \left( 1 - \sqrt{1 - \frac{2 \times 0.933 \times 19.61}{400}} \right) = 0.00392$$

$$A_s = \rho b d = 0.00392 \times 1000 \times 223 = 875.53 \text{ mm}^2$$

$$A_{s,min} = 0.0018 b h = 0.0018 \times 1000 \times 250 = 450 \text{ mm}^2$$

$$A_s = 875.53 \text{ mm}^2 > A_{s,min} = 450 \text{ mm}^2$$

Use  $\Phi 10$  then

$$n = \frac{A_s}{A_{s\phi 10}} = \frac{875.53}{113} = 7.74$$

$$s = \frac{1}{n} = \frac{1}{7.74} = 0.129 \text{ m}$$

Take  $8\Phi 12/m$

$\Phi 12 @ 100 \text{ mm}$

Step(s) is the smallest of:

1.  $3h = 3 \times 250 = 750 \text{ mm}$
2.  $450 \text{ mm}$
3.  $s = 380 \frac{280}{f_s} - 2.5 \times 20 = 337.4 \text{ mm}$   
 $s \leq 300 \frac{280}{f_s} = 300 \left( \frac{280}{\frac{2}{3} 412} \right) = 305.8 \text{ mm}$   
 $s = 100 \text{ mm} < s_{max} = 305.8 \text{ mm}$

\* Temperature and shrinkage reinforcement.

$$A_s = 0.0018 b h = 0.0018 \times 1000 \times 250 = 450 \text{ mm}^2$$

$$n = \frac{A_s}{A_{s\phi 10}} = \frac{450}{79} = 5.76$$

$$s = \frac{1}{n} = \frac{1}{5.76} = 0.17 \text{ m}$$

Take  $6\Phi 10/m$

$\Phi 10 @ 150 \text{ mm}$

Step (s) is the smallest of :

1.  $5h = 5 \times 250 = 1250 \text{ mm}$

2.  $450 \text{ mm}$

$s = 150 \text{ mm} < s_{max} = 450 \text{ mm}$

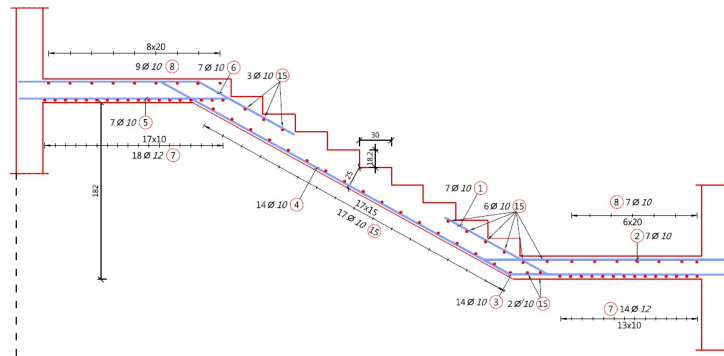


Figure 4.26: Reinforcement of Stair

# Chapter 5

## Results and Recommendations

The Architectural and Structural Drawings shown in Appendix A and B, respectively.

### 5.1 The Result

1. Each student or structural designer should be able to design manually so he can get the experience and knowledge in using the computer software.
2. One of the factors that must be take in consideration is the environment factors surrounding the building, the site terrains, and the forces effects on the site.
3. One of the important steps of the structural design is how to connect the structural members to work together, then to divide these members and design them individually, and should take the surrounding condition in the consideration.
4. The soil strength capacity is  $400 \text{ KN}/m^2$ .
5. Various types of slabs have been used: two way and one way ribbed slabs, in some slabs that have a regular or nearly regular distribution of columns and beams. One way solid slabs mainly in the stairs, because it has high resistance to the concentrated forces.
6. The used software programs:

- AutoCAD 2015, to draw the detail of drawings for structural drawings.
  - ETABS, to analysis and design the shear wall.
  - ATIR, to analysis and design the structural members.
7. We have used the live loads using the Jordanian code of loads.

## 5.2 The Recommendations

This project has an important role in widening and enhancing our understanding to the nature of the structural project including all the details, analysis, and designs.

We want here through this experience- to introduce a group of recommendations, we hope it to be useful for planning to select a structural project.

At the beginning, the architectural drawings have to be prepared and ordered and the construction material and the structural system have to be choose alongside. And it's essential at this stage to have information about the project site, the soil, the soil strength capacity at the site from the geotechnical report, after that the bearing walls and the columns is going to be set up alongside the architectural team in a compatible manner. The civil engineer tries at this stage to plant as much as possible the reinforced concrete walls, which should be use after that in resisting the earthquake loads and other lateral loads.

# **Appendices**

# **Appendix A**

## **Architectural Drawings**

# **Appendix B**

## **Structural Drawings**

# **Bibliography**

- [1] American Concrete Institute (A.C.I). Building code requirement for structural concrete.
- [2] National Jordanian Construction loads. The national jordanian construction council.