Palestine Polytechnic University

Faculty of Engineering and Technology

Department of Civil Engineering and Architecture

Specialization in Civil Engineering-Building Engineering Branch

Project Name

Structural Design of Islamic Center for Culture and Media in Ramallah City

Supervisor:

Dr. Mohammad Taha Alsayyed

Team:

Izz Al-Deen Rajai Al-Hashlamoun

Wajdi Mahmoud Rezeq

Omar Jamal Al-Amleh

Palestine – Hebron

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Submitted to the College of Engineering in Partial Fulfillment of Requirements of the Bachelor Degree of Civil engineering/ BuildingEngineering Branch**.**

Faculty of Engineering and Technology

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Certification of Evaluate Graduation Project

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Based on the instructions of Dr.Mohammad Taha Alsayyedand the approval of all members of the committee, this project was introduced to the department of Civil Engineering and Architecture in the Collage of Engineering and Technology for partial fulfillment of the requirements for The Bachelor Degree.

Supervisor signature	Head of the department signature
Dr.Mohammad Taha Alsayyed	Eng.Faydi Shabana

2018

Dedication

Every challenging work needs self-effort as well as guidance of elders especially those who were very close to our heart

Our humble effort we dedicate to our sweet and loving

Father & Mother specially and our Families in general

Along with all hard working and respected supervisor
Dr.Mohammad Taha Alsayyed

Whose affection, love, encouragement and prays of day and night make us able to get such success and honor

Also we dedicate this simple work for our Teachers who tries to simplify the engineer science for us

W.Rezeq, O.Amleh, I.Hashlamoon.

Thanks and Appreciation

Firstly, for our God, for blessing us to finish this project.

For our ancestors who paved the path before us upon whose shoulders we stand. Thank you.

We would like to express our deepest gratitude to our supervisor Dr.Mohammad T.Alsayyed for his unwavering support, collegiality, and mentorship throughout this project.

We would like to extend our thanks to those who collegial guidance and support over this project.

W.Rezeq, O.Amleh, I.Hashlamoon.

Abstract

Structural Design of the Islamic Center for Culture and Media in Ramallah

Supervisor

Dr. Mohammad Taha Alsayyed

Team

Izz Al-Deen Rajai Al-Hashlamoun

Wajdi Mahmoud Rezeq

Omar Jamal Al-Amleh

This project is aimed at the structural design of the Islamic Center for Culture and Media Building located in Ramallah city. The building consists of several types of services distributed in a suitable way. These services are distributed to five floors with total area of 12267.5 m^2.

The main objective was to design all of structural members of this building after clearly understanding all of the architectural aspects in order to provide the suitable structural system. The structural members were designed starting by the slabs to the beams then to columns and walls finally to the foundations which transfer the loads to the foundations soils.

It is very important to know that Jordanian Building Code was used to specify the values of loads that building serves, American Concrete Institute Code (ACI 318-11) used to design all structural members, and the UniformBuilding Code (UBC 1997) for seismic load determination. It is worth mentioning thatsome software packageswere used during this project; such as Office2013,AutoCAD2014, Atir 2016, Safe2016, Etabs2016, Adapt2015 and others.

الملخص

التصميم الإنشائي ل "المركز الإسلامي للثقافة والإعلام في مدينة رام الله"

المشرف

د.محمد طه السيد

فريق العمل

عزالدين رجائي الهشلمون

وجدي محمود رزق

عمر جمل العملة

إن الهدف الأساسي للمشروع هو التصميم الإنشائي للمركز الإسلامي للثقافة والإعلام والمزمع بناؤه في مدينة رام ا . يحتوي المشروع على عدة أقسام خدماتية تلبي الهدف الأساسي للمشروع موزعة بطريقةٍ منظمة تجعلها سلسة التنقل بينها.

من الجدير بالذكر أن المبنى يتكون من خمسة طوابق بمساحة إجمالية للمشروع تساوي ١٢٢٦٧.٥م^{٢}.

جميع العناصر الإنشائية لهذا المبنى تم تصميمها بعد فهم الفكرة المعمارية المبنيّ على أساسها مخططات المشروع، وبالتالي وضع النظام الإنشائي المناسب لها. هذه العناصر الإنشائية تم تصميمها بدءاً بالعقدات ثم الجسور ثم يتبعها الأعمدة والجدران إلى حين الوصول أخيراً إالى القواعد التي بدورها تفرغ الحمل وتوزعه في تربة التأسيس.

من المهم الإشارة إلى أنه تم الإستناد إلى كود الأحمال الأردني ٢٠٠٦ في احتساب قيم الأحمال للمشروع، كود التصميم الأمريكي (ACI 318-11) لتصميم كافة العناصر الإنشائية، كود الأبنية الموحد (UBC 1997) في احتسب أحمل الزلازل. وتجدر الإشارة أيضاً إلى أنه تم استخدام عددا من البرامج الحاسوبية مثل (Office2013, Autocad2014, Atir2016, Safe2016, Etabs2016, Adapt2015) وغيرها من البرامج التي أعانت على إتمام هذا المشروع.

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List of Abbreviations:

- **Ac** = area of concrete section resisting shear transfer.
- **As** = area of non-pre-stressed tension reinforcement.
- **Ag** = gross area of section.
- **Av** = area of shear reinforcement within a distance (S).
- **b** = width of compression face of member.
- **bw** = web width of concrete beam section.
- **DL** = dead load.
- **d** = distance from extreme compression fiber to cancroids of tension reinforcement.
- **Ec** = modulus of elasticity of concrete.
- **Fy** = specified yield strength of non-pre-stressedreinforcement.
- **I** = moment of inertia of section resisting externally applied factored loads.
- **Ln** = length of clear span in long direction of tow-way construction, measured faceto-face of supports in slabs without beams and face to face of beam or other supportsin other cases.
- **LL** = live load.
- **Ld** = development length of steel reinforcement.
- **M** = bending moment.
- **Mu** = factored moment at section.
- **Mn** = nominal moment.
- **Pn** = nominal axial load.
- **S** = spacing of shear or in direction parallel to longitudinal reinforcement.
- **Vc** = nominal shear strength provided by concrete.
- **Vn** = nominal shear stress.
- **Vs** = nominal shear strength provided by shear reinforcement.
- **Vu** = factored shear force at section.
- **Wc** = weight of concrete. (Kg/m^3)**.**

Special Abbreviationsfor Post-tensioned Beams Design:

- A_p: Cross-sectional area of prestressed steel.
- **a**: Tendon sag(the maximum offset from the chord, the line connecting the two highpoints in each span).
- **CGC**: Centroid of concrete cross-section.
- **CGS**: Centroid of gravity of prestressing steel.
- d_p: Distance from extreme compression fiber to centroid of prestressing steel.
- **e**: Eccentricity distance between CGS and the CGC.
- F_i: Initial prestress force.
- F_e: Effective prestress force.
- f'_{ci}: Concrete compression strength at time of stressing.
- f_{pc}: Average concrete compression ($\frac{F}{A}$).
- f_{pe}: Extreme fiber flexural compressive stress caused by equivalent tendon loads at the fiber where tension is caused by applied gravity loads.
- f_{ps}: Stress in prestressing steel at nominal member strength (ultimate strength).

- f_r: Modulus of rupture in concrete, the flexural tensile strength or the stress assumed to produce first cracking.
- f_{se}: Effective stress in prestressing steel after all losses.
- **I** : Moment of inertia.
- M_{cr}: Moment in excess of the unfactored dead load moment.
- $M_{balncing}$: Moment which equilibrates the tendon balanced, or equivalent, loads only (not including the reactions to those loads, which are called the secondary reactions).
- T_{ps}: Tensile force in prestressing steel at nominal member strength (the ultimate prestress force).
- S_t: Section modulus at the top beam fiber.
- S_b: Section modulus at the bottom beam fiber.
- Y_t: Distance from concrete centroid to the extreme fiber where tension is caused by applied gravity loads.
- Y_b: Distance from concrete centroid to the extreme fiber where compression is caused by applied gravity loads.
- β_1: Factor that varies with concrete strengthf'_c.
- γ_p: A factor used in the calculation of f_{ps} for bonded tendons.
- f_{pu}: Specified maximum tensile stress in prestressing steel.
- f_{py}: Specified yield strength in prestressing steel.
- E_p: Modulus of elasticity of prestressing steel.
- f_{ci}: Maximum allowable compressive stress in concrete immediately after transfer and prior to losses.
- f_{ti}: Maximum allowable tensile stress in concrete immediately after transfer and prior to losses.
- f_c**:** Maximum allowable compressive stress in concrete after losses at service load.
- f_t: Maximum allowable tensile stress in concrete after losses at service load.
- A_{ct}: Area of that part of cross section between the flexural tension face and center of gravity of gross section.

Chapter One

Project Introductory

1

1.1 Introduction:

The last century has witnessed a starting of an age of revolution and improvement in all of the life aspects, and the Islamic building becomes one of them to create a conscious young Islamic people to increase their knowledge about the Islamic religion.

It is necessary to know that the Islamic religion is not limited by praying and reading the Quran, it is how to have a brilliant cooperation with others especially whom out of the Islamic religion, the good manners, to show the perfect sides of this religion. In other words, it is very important to promote Islamic people in different sides of their religion, also to provide an Islamic programs which increasing the awareness of the people.

So it was necessary to think of an institution that includes many activities supporting the media, so the idea is to establish a media center was in mind, which includes many branches to serve the idea of the building.

1.2 General Identification:

The project is about Islamic center for cultural and media in Ramallah city. It contains different branches to serve the main idea of this building. Its branches are distributed in a suitable way that makes them cooperate with each other in an organized way. These branches are: Theater, Library, Exhibition, Mosque, Restaurant, Administration center, Radio and TV Centers, Lectures Room and other serving rooms.

1.3 Project Choosing Reasons:

Choosing of project depends on the variety of the building types to enhance the ability of dealing with different service requirements and purposes of the buildings, also to make a wide aspects of structural systems needed to treat the architectural design of the buildings.

1.4 The Project Objectives:

- **Architectural Objectives**
  The main architectural aim is to create a design that is unique in views, representative and breaks the lack of architecture that Palestine suffers. So the Islamic center design shows the ancient Islamic decorations coated by a modern architectural view.

- **Structural Objectives**

  The structural objectives can be summarized in the following points:
  - Increasing the ability to choose a structural system that suits the objectives of the building.
  - To correlate what 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.

1.5 The Problem of the 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 of the project, so all of these components analyzed and designed such like slabs, beams, columns… etc.After determining the loads on each of the structural member so that the required dimensions and reinforcement can be selected. After that, all of the design outputs will be presented in the structural drawing that used to transfer the project from being a drawing to a practical field.

1.6 The Postulate of the Project:

This project aims to prepare the required structural drawings of the various structural members existed in the buildings in such a way that takes the architectural design as the main outlet.

Chapter Two

Architectural Description

2

2.1 Introduction.

2.2 Basic Identification of the Project.

2.3 Project Land Location.

2.4 Project Components Description.

2.1 Introduction:

Architectural description is the most important thing to define and understand the nature of the project and its sections.

The soul of the architecture is to design a structure that will be suited for humans to live in, work in, play in, etc. It is also to give comfort to its users, makes them feel uplifted, and makes them feel that someone cared about their well-being enough to design something that they would enjoy. A good Architect is more than just a designer of buildings, he or she understands how people's surroundings make them feel, and creates an environment that will meet their needs and desires.

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

2.2 Basic Identification of Project:

From the importance of both traditional and new media to the flow of information in the field of religious, national and social, it is worked towards creating a unified media entity able to accommodate the requirements of the future in which it seems that the gap between the possible and impossible will almost fade.

Figure 2- 1: General Picture of the Project.

For that the idea is to establish an integrate "Islamic center for culture and media" that includes a coalition of Journalists, governmental agencies, programs coordinators, theatrical performances and all other organizations involved in the production of radio, television, news, advertising and theatrical shows.

2.3 Project Site:

It's recommended that the project land 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.

2.3.1 Project land location:

The project land locates in Albireh-Ramallah with an area of 15000 m^2, near Al-Waha Roundabout. It is generally away of the city center which make it very desired for the project and it is nature.

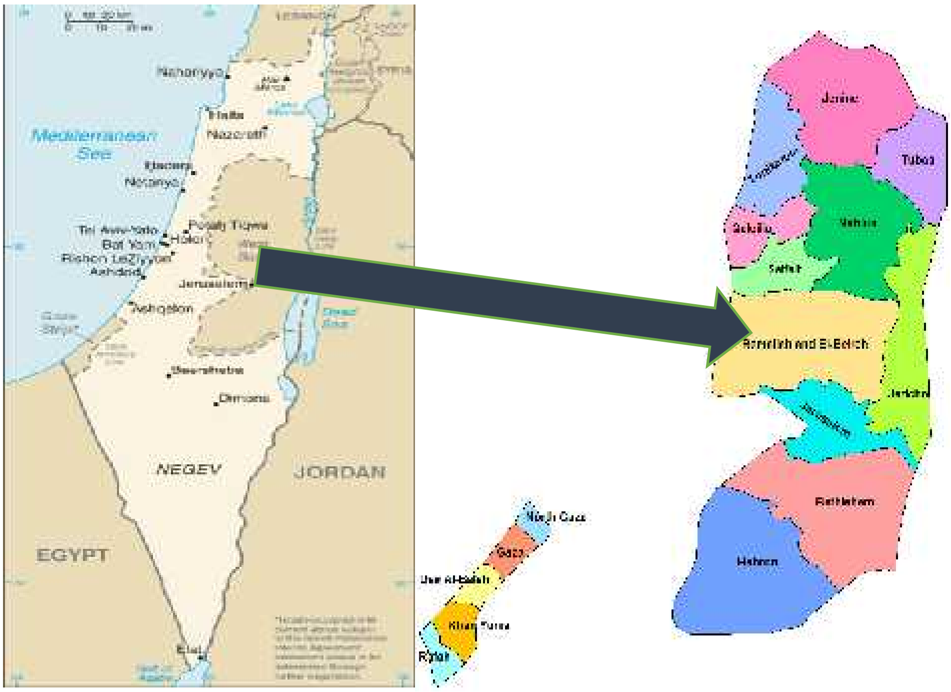


Figure 2- 2: Palestine Map.

Figure 2- 3: Ramallah and Albireh location.

Figure 2- 4: The Project Land Location.

2.3.2 General climate description of the city:

This area generally enjoys a Mediterranean Climate of a dry summer and mild, rainy winter with occasional snowfall. While the western and south western winds dominate, the northern winds are light and the eastern winds are still blow on occasion.

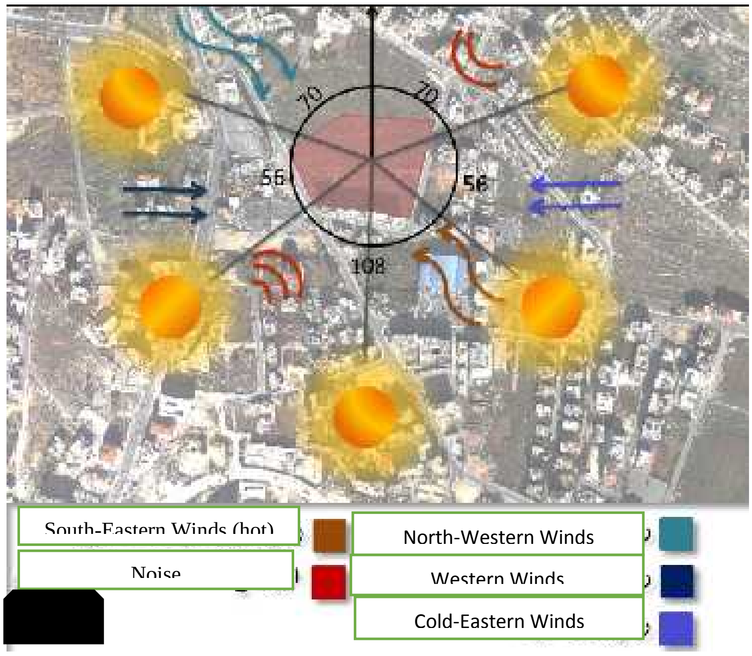
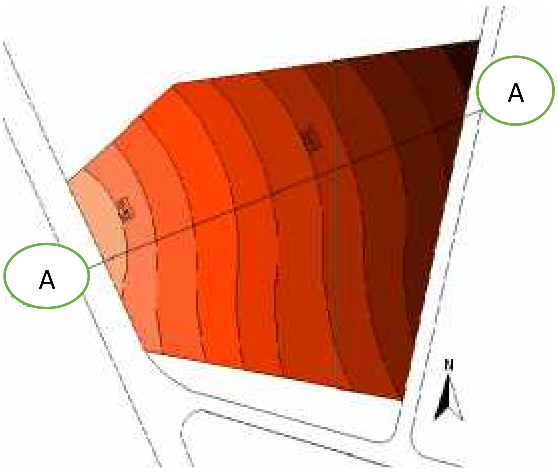


Figure 2- 5: Environmental Analysis of the Project Land.

2.3.3 Topography of the project land:

The land declines about 10 meters from beginning to the end into Southern-Eastern direction.



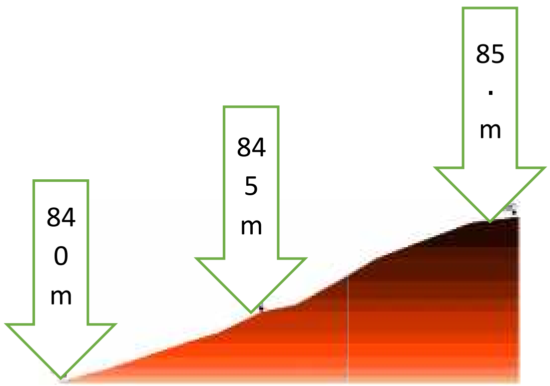


Figure 2- 6: Topography of the Project Land.

Figure 2- 7: Section A-A

2.4 Project Components Description:

The architectural idea based on using rectangular shapes with an aesthetic interlocking and connecting of the project parts which leads to attract attention.

2.4.1 Project plans description:

The total area of the building is **12267.5m^2**.

1- Basement Floor Plan

This floor consists of Car Barks, Theater Entrance, Storage, Electrical Room, Mechanical Room and Exhibition. The total area of this floor is **6138 m^2**.

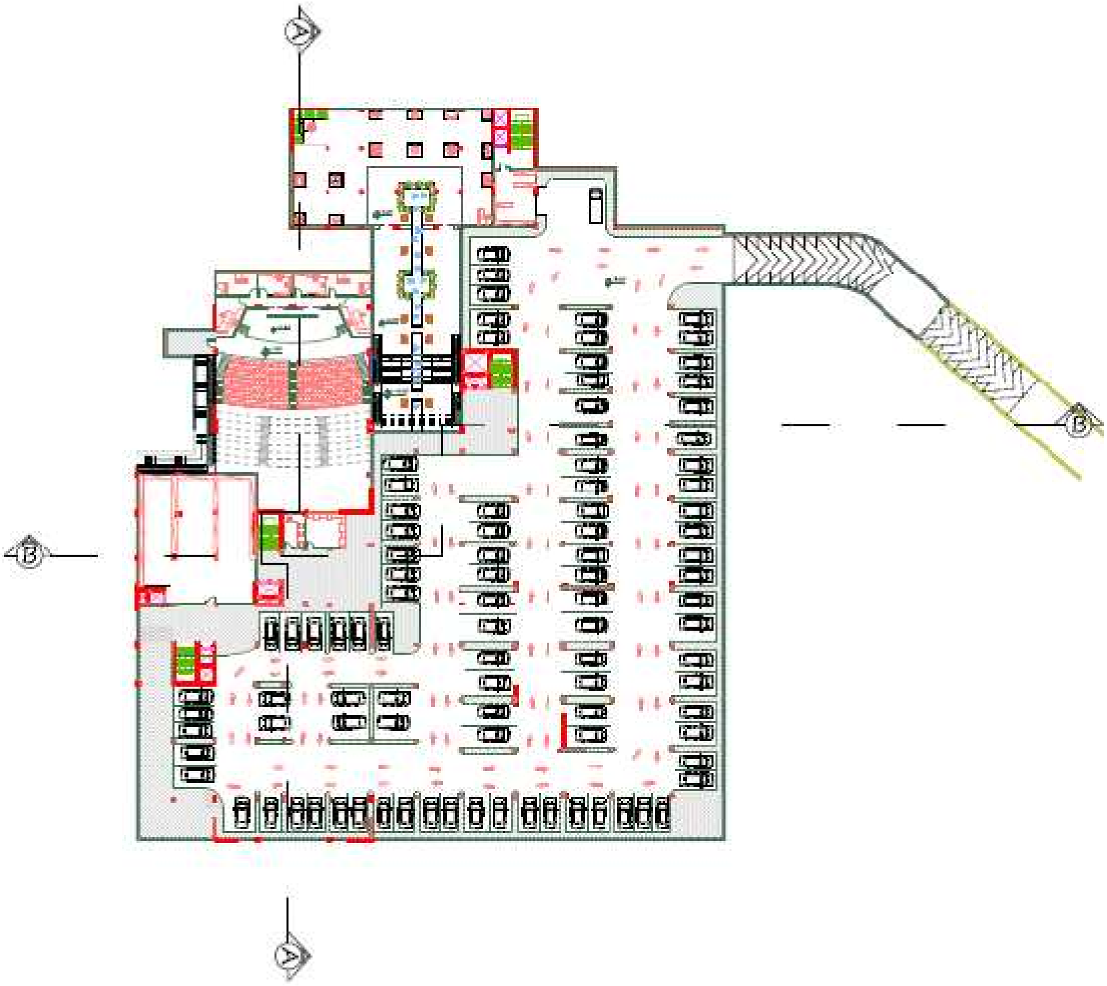


Figure 2- 8: Project's Basement Floor Plan.

2- Ground Floor Plan

The area of this floor is **2418m^2**, and this floor consists of four parts. The first one is Restaurant and its facilities of Kitchen, Store, W.Cs and Reception. The second part is a Studio Floor which consists of Radio Studio, Archives, Control Room, Generator, and Production, Implementation of Programs, Software Development, Stores and Reception. The third part is a Theater lobby, Theater facilities, also there is a Gallery. Finally, the fourth part is a Mosque for Men and its facilities.

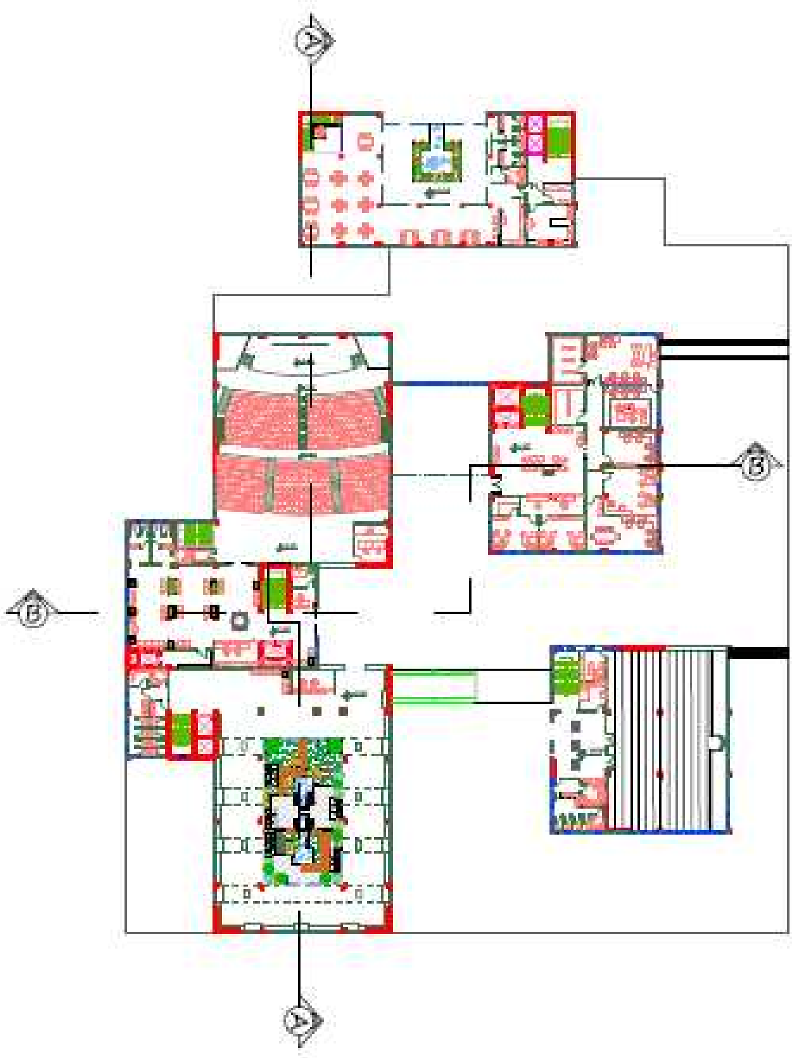


Figure 2- 9: Project's Ground Floor Plan.

3- First Floor Plan

The area of this floor is **2210m^2**, and this floor consists of three parts. The first one is a T.V floor which consists of T.V Studio, Offices, Control, Production, Software Development, Archive and Generator. The other part is the Theater Lobby and second floor of seats and a Library with all required facilities. The last part is a Mosque for women and its facilities.

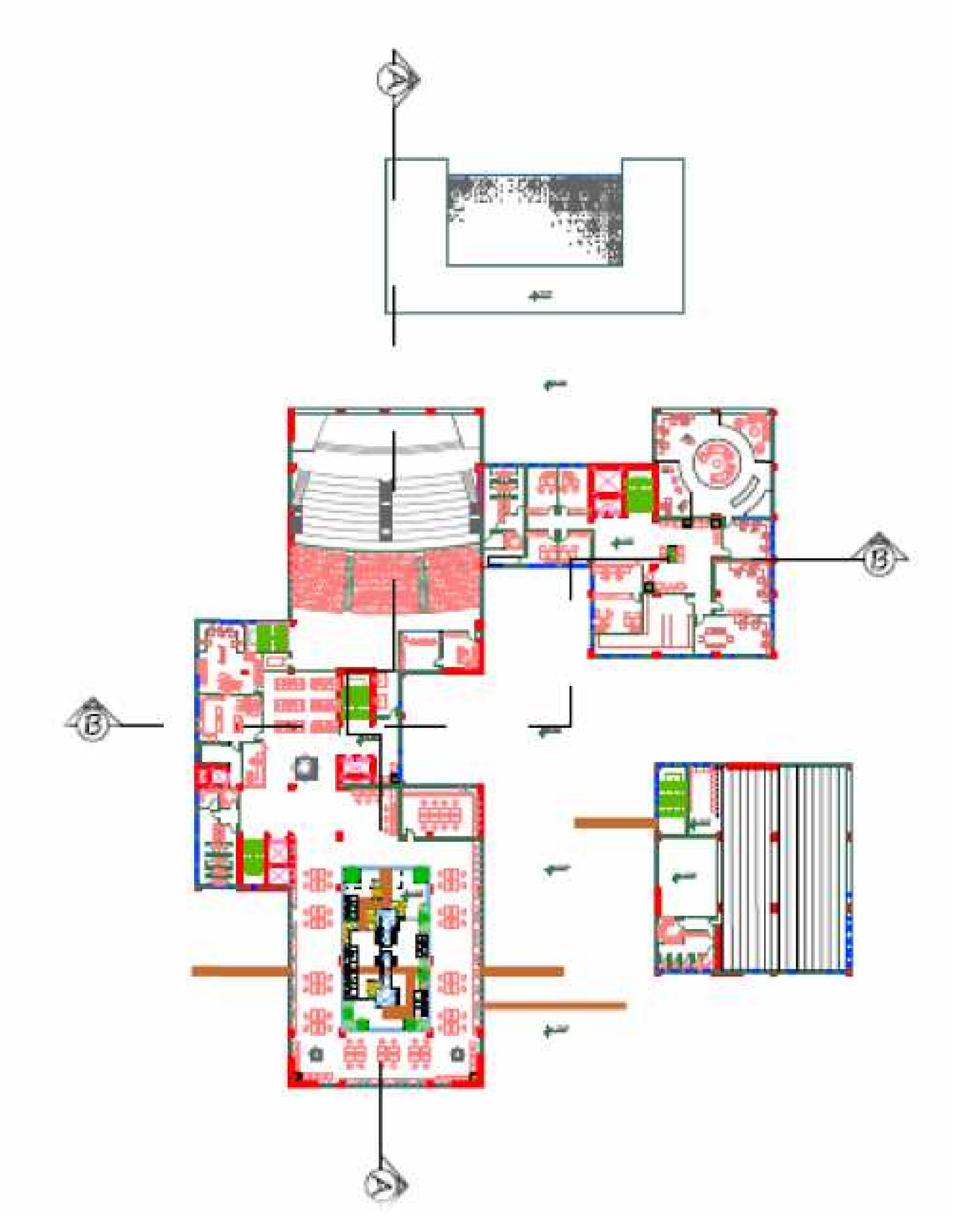


Figure 2- 10: Project's First Floor.

4- Second Floor Plan

The area of this floor is **<u>1197 m²</u>**, and it consists a two part. The first one is a Management Floor which contains an Offices, Archives, Stores, Meeting Hall and W.Cs. The other part is an Educational part which contains a Lecture Halls, Teachers Room, Stores and W.Cs.

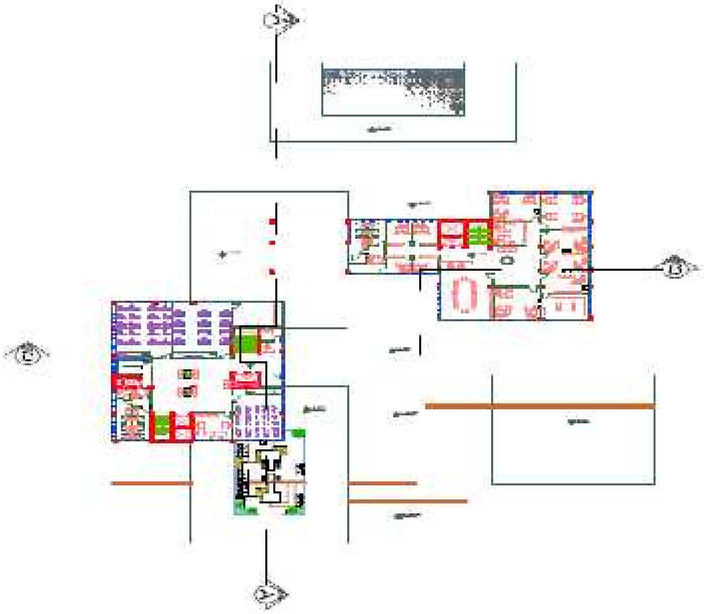


Figure 2- 11: Project's Second Floor Plan.

5- Third Floor Plan

This Floor constitutes the Administration of the building, since it consists of The Manager Room, Deputy Manager Room and Security Room. There is Hall, Stores, Kitchen and W.Cs too. This Floor has an area of **304.5 m^2**.

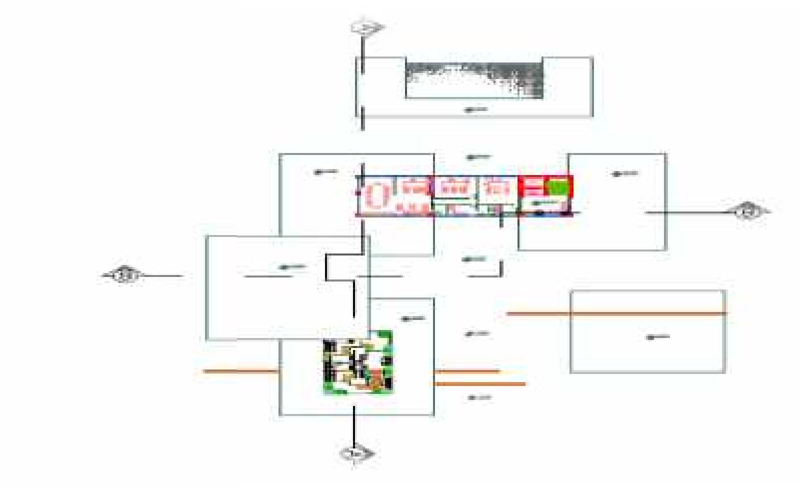


Figure 2- 12: Project's Third Floor.

2.4.2 Project elevation's description:

The interest of elevations for any architect is great as the elevations appearance should besuitable 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- South Elevation:

The main elevation and the main entrance of the building, it can observed the creativity ofthis elevation that there is an amazing setbacksand contains glass with embossed decoration integrated with stone to provide an Islamic vision of the elevation.

Figure 2- 13: Project's South Elevation.

2- East Elevation:

This elevation shows a beautiful side of the building, since the setbacks are shown comfortably and arranged carefully, also a glass with embossed decoration integrated with stone is used. It is clear that a decoration column used to provide more aesthetic show for the elevation and the building in general.

Figure 2- 14: Project's East Elevation.

3- North Elevation:

The backside elevation where the entrance of the Restaurant shown. In this elevation the third floor, which the last floor, is shown approximately at the middle of the elevation. Glass with embossed decoration integrated with stone is used in this elevation too.

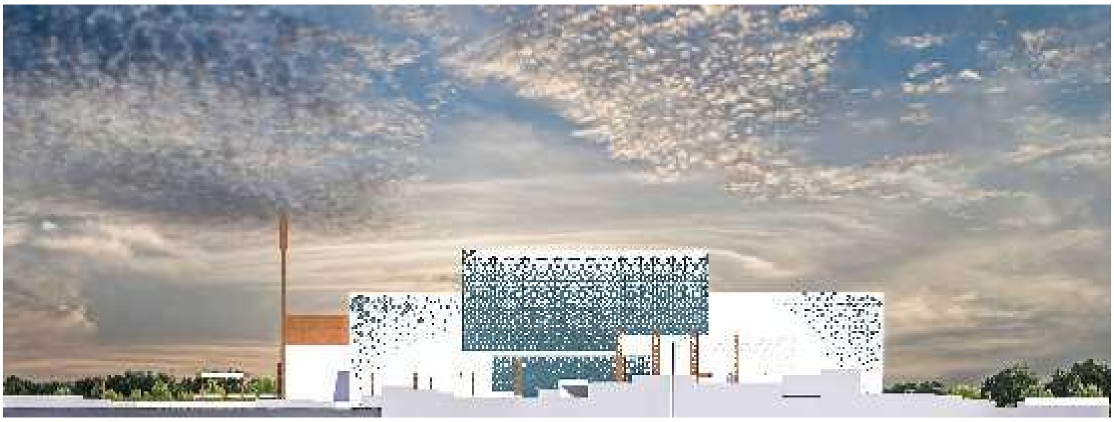


Figure 2- 15: Project's North Elevation.

4- West Elevation:

In this elevation, a regular setbacks and conglomerates provide a special beauty of the elevation, also a glass with embossed decoration integrated with stone is used.

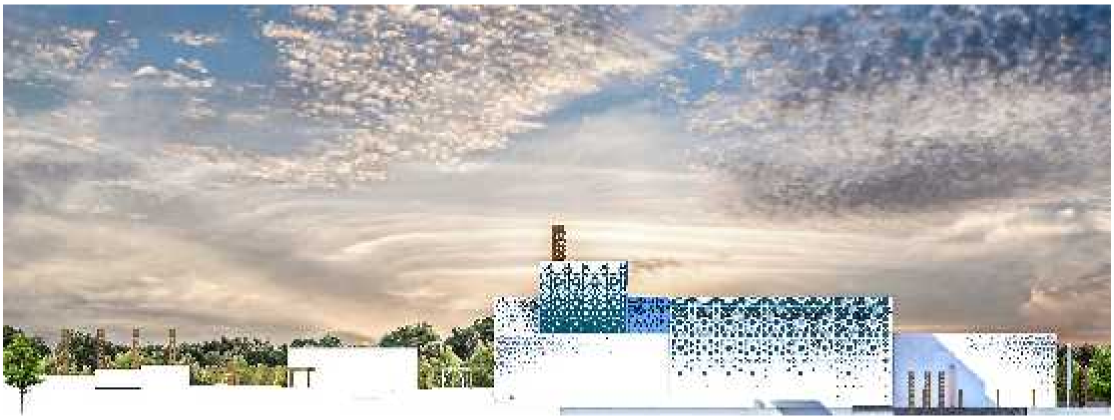


Figure 2- 16: Project's West Elevation.

2.4.3 Vertical movement description:

The designer distributed the movement through the horizontal and vertical axes through stairsand 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.

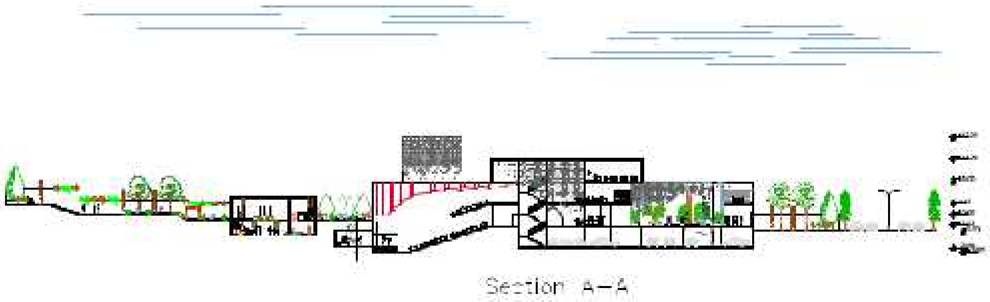


Figure 2- 17: Project's Section A-A.

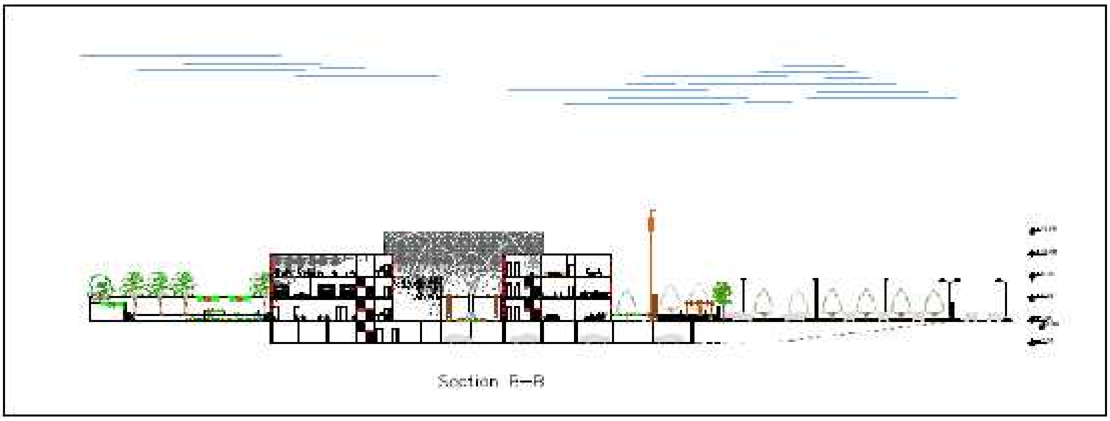


Figure 2- 18: Project's Section B-B.

Chapter Three

Structural Description

3

3.1 Introduction.

3.2 The Purpose of Structural Design.

3.3 Theoretical Study of the Structural Elements of the Buildings.

3.4 Types of Loads.

3.5 Practical Tests.

3.6 Structural Elements Used in this Project.

3.1 Introduction:

After the completion of the process of the project architect explanation of all details, the construction phase starts by structural studying of the architectural analysis of the project, in order to choose the appropriate structural system for each element in the building so that provides all the requirements and work on the design of the elements necessary for that system. This requires to take the loads affecting the project elements into consideration, and show how to deal with them and work to resist. So, all structural elements must be understood in a very small details in order to be customized and analyzed accurately.

In general, the physical systems is the basic system broken down in the building. They have the following types:

- STRUCTURAL SYSTEM.
- EXTERIOR SYSTEM.
- INTERIOR SUBDIVISION OF SPACE.

3.2 The Purpose of Structural Design:

Structural design is a systematic and methodical study of the stability of the structure, strength and rigidity of structure to resist forces. Therefore, the basic intention of design is to ensure that the structure/building withstands the entire probable loads for a stipulated period of time. The design also takes care of durability issues like cracks, leakage, excessive vibrations and deflections. This means carefully structurally designed buildings takes care of safety and well-being of its occupants, and this can be and only be achieved by engaging consultants having relevant experience and practice.

In structural terms, planning involves working out various alternative structural systems considering the various functional requirements. Aesthetics, economics and environment and constructability is also considered. So, when designing any element of the structural elements the following points describe the most important factors to be taken in consideration:

- <u>Safety</u>: is the essential element that must be provided in the design, so choosing the appropriate element of each region to resist loads that affecting them.
- <u>Economy</u>: must be supplied when working on the selection of appropriate materials, and sufficient for its desired purpose and appropriate quantity, with lowest cost and highest quantity.
- <u>Serviceability</u>: 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.
- <u>Architectural side</u>: work to take into account the architectural elements in the building and try to keep it as much as possible.

3.3 Theoretical Study of the Structural Elements of the Building:

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

3.4 Types of Loads:

Loads are the base of design process, so they must have great consideration and special identifying and study. So the difference of building to another depends on the architectural design, project site and materials used in construction and other influences, therefore loads can be classified as follows:

- Basic loads:

  The 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.

- Secondary loads:

  The loads that should be take 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, and Seismic load.

3.4.1 Dead load:

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.

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.

Table 3. 1: Specific gravity of materials used.

Type of Material	Specific Gravity kN/m^3
Tiles	23
Mortar	22
Course sand	17
Reinforced Concrete	25
Hollow Block	10
Plaster	22

3.4.2 Live load:

Live load is imposed loads which 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.

This project depend on Jordanian Code for Loads and Forces (2006), and 4kN/m^2 was selectedas a live load value in designing purposes after studying all services that the building contains.

3.4.3 Environmental loads:

Environmental Loads are structural loads caused by natural forces such as wind, rain, snow, earthquake or extreme temperatures.

3.4.3.1 Wind load:
The forces that affect horizontally on the building appear especially in high-rise buildings, and it should be designed on the basis of wind speed and height of the building, and the amount of buildings surrounding the building.

3.4.3.2 Snow load:
The building must be designed to resist snow loads and to be taken into account in 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 snow-load to be used in the case of design.

Table 3. 2: Snow Load according to Jordanian Code (2006)

Building height above sea level	Snow load on surface (kN/m^2)
h < 250	0
250 < h < 500	(h-250)/800
500 < h < 1500	(h-400)/320

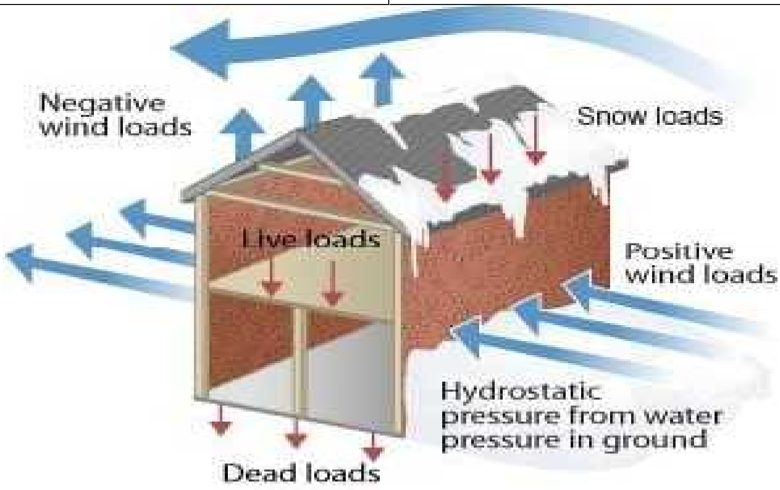


Figure 3. 1: Loads affecting on building.

3.4.3.3 Seismic load:

Is the 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.

Figure 3. 2: Seismic Load

The next figure explains the path that the load follow in its travel through the building until reaching the foundation soil.

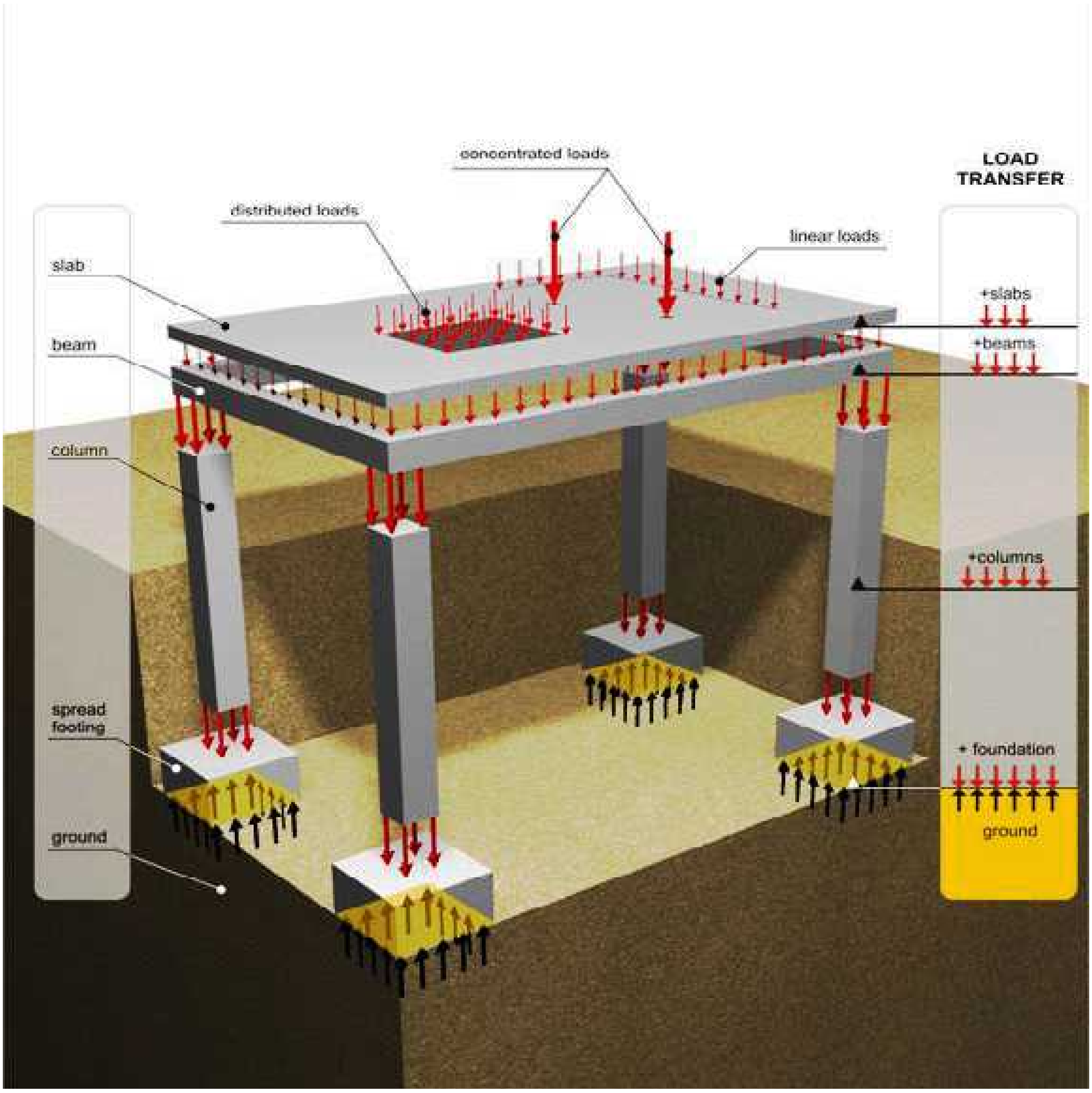


Figure 3. 3: Loads Path.

3.5 Practical tests:

Before beginning of the process of design and construction, some of necessary tests should be done 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, as soil test.

3.6 Structural Elements Used in this Project:

This project have many types of structural elements which work together to construct and serve the objectives of the building, such as slabs, beams, columns, stairs, shear and basement wall and foundations.

3.6.1 The slabs:

Concrete slab, a very common and important structural element, are constructed to provide flat, useful surfaces. It is a horizontal structural component, with top and bottom surfaces parallel or near so. Is an element which transfers the loads that are exposed to other structural elements such as column, beam and wall.

Concrete Slab's Support:

The concrete slab may be supported by:

- Masonry or reinforced concrete Walls
- Monolithically casted reinforcement concrete beams
- Structural steel members
- Columns
- The ground

In projects, there are different types of slabs can be used including:

3.6.1.1 One way ribbed slab:

A one-way joist floor slab consists of a series of small, reinforced concrete T beams that are connected with girders that in turn carried by the building column. T beams are known as joists which are formed by setting steel pan at a constant spacing. Concrete is cast between those spacing to make those ribs and in this way, the slab also cast and the slab becomes the flange of T beam. In general, this type is most commonly used in this project, this contains the steel bars use to transfer the loads, and block and the concrete between this block and the topping of all.

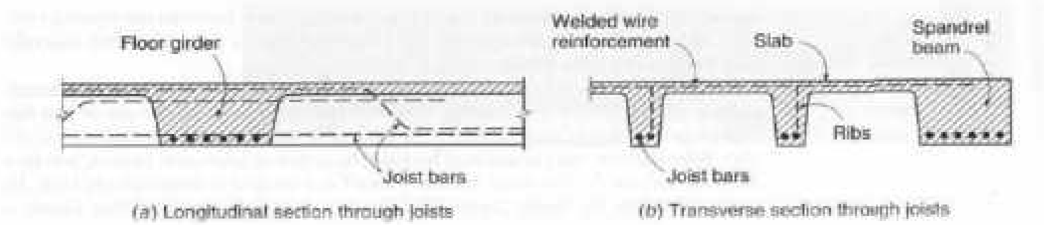


Figure 3. 4: One Way Ribbed Slab.

3.6.1.2 Two way ribbed slab:

Two way ribbed slab is the type used when the length of the two direction in the space approximately is equals, and the reinforcing bars of steel in two direction to transfer the load.

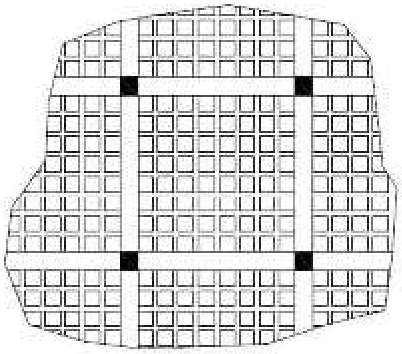


Figure 3. 5: Two Way Ribbed Slab.

3.6.1.3 Solid slab:

This type of slab typically used with drop beams, it requires less thickness than ribbed slab with the same spans and criteria, which provide more clear height when the spaces are important. This slab transfers the load into a beams then to columns, and it has two types one way and two way solid slab.

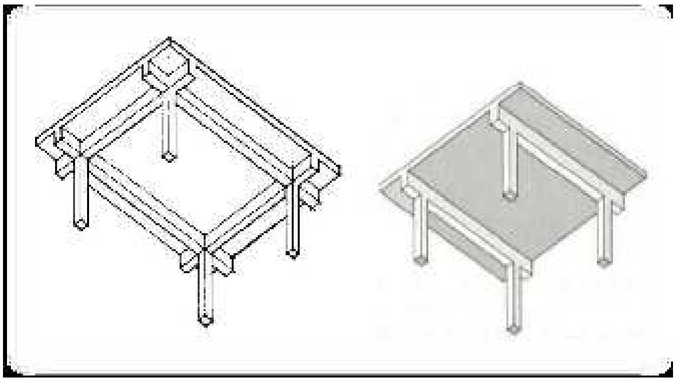


Figure 3. 6: Two Way and One Way Solid Slab.

3.6.1.4 Flat plate:

It is a two way slab system consisting of a uniform slab that rests directly on columns and does not have beams or columns capitals. This type has not been used during this project.

If capitals or drop panels are used in resisting the punching shear, then the name of the slab will be flat slab instead of flat plate.

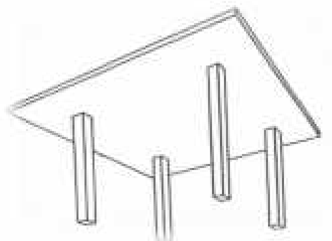


Figure 3. 7: Flat Plate.

3.6.2 Beams:

Structural elements capable of withstanding load primarily by resisting bending and shear. Bending moment is the bending force induced into the material of the beam as a result of external loads, own weight, span and external reactions to these loads.The loads carried by a beam are transferred to columns, walls, or girders, which then transfer the force to adjacent structural compression members.

Beams are commonly used as hidden beam with the same depth of the slab that it supports, or drop beam that the required thickness of beam is large and the thickness of the slab is not enough.

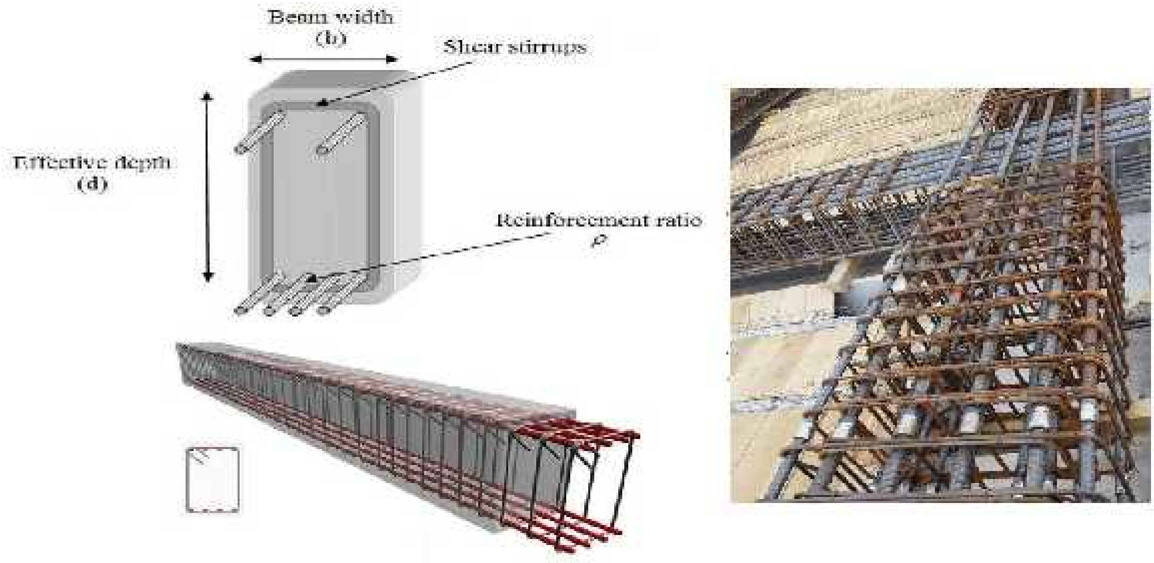


Figure 3. 8: Rectangular Beam.

There is type of beam were used in this project with an identification of post-tensioned beam, which can serve for long spans and a large value of load using a reinforcing cables with a yielding stress can reaches to 1860 Mpa which is so larger than typically reinforcing steel bars with yielding stress can reaches to 420 Mpa.

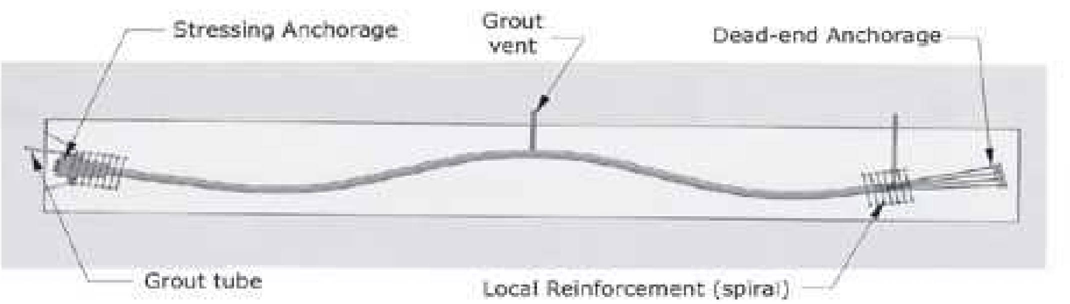


Figure 3. 9: Multistrand Post-tension Schem.

3.6.3 <u>Columns:</u>

It is a structural element that transmits, through compression, the weight of the structure above to other structural elements below. In other words, a column is a compression member, and it has deferent shapes, the most common are rectangular and circular shapes. In order to design the column, it is very important to specify wither the column is short or long (check of slenderness).

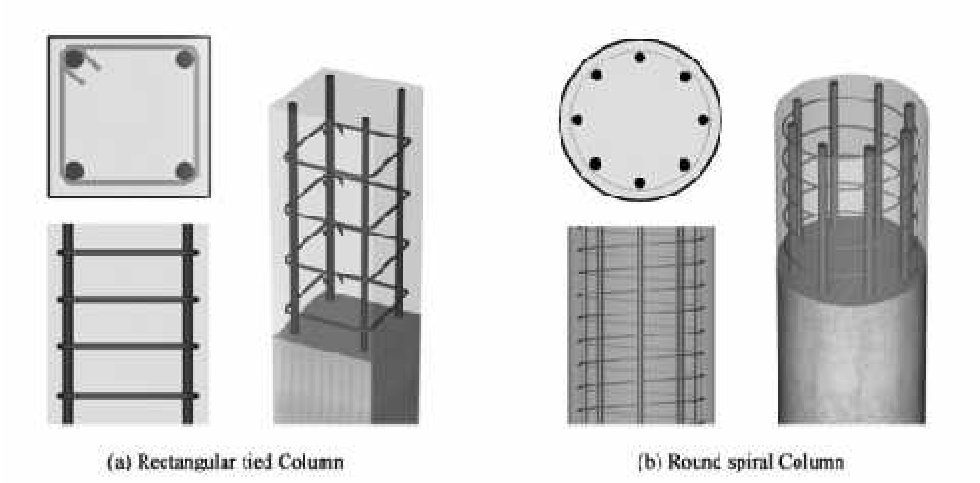
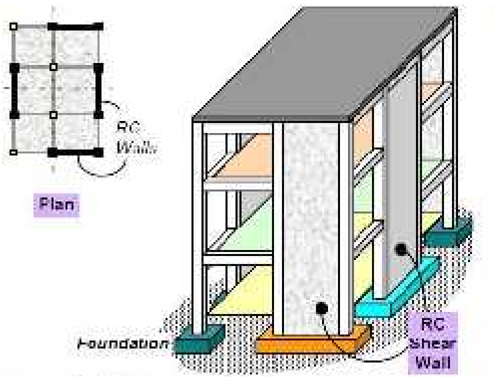


Figure 3. 10: Rectangular and Circular Columns.

3.6.4 <u>Shear wall:</u>

Shear wall is a very important structural element which resists vertical and horizontal loads, but specially used to resist shear, wind force or the lateral force that causes the damage to the building during earthquakes.



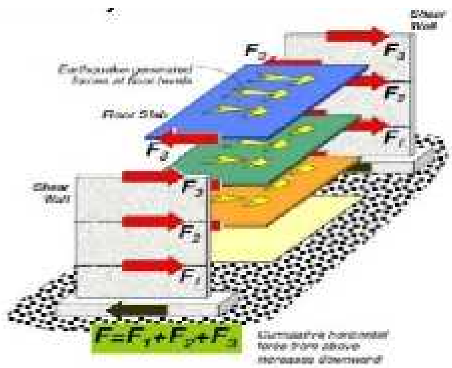


Figure 3. 11: Shear Wall System in buildings. Figure 3. 12: Shear Force Transfer.

3.6.5 Basement wall:

In the case that a part of building exists underground, a reinforced concrete wall is required to resist the lateral force from the soil in a name of basement wall. It should be noted that the basement wall can also loaded vertically but it is not must.

3.6.6 Staircase:

Staircase is an important component of a building providing access to different floors and roof of the building. It consists of a flight of steps (stairs) and one or more intermediate landing slabs between the floor levels. Different types of staircases can be made by arranging stairs and landing slabs.

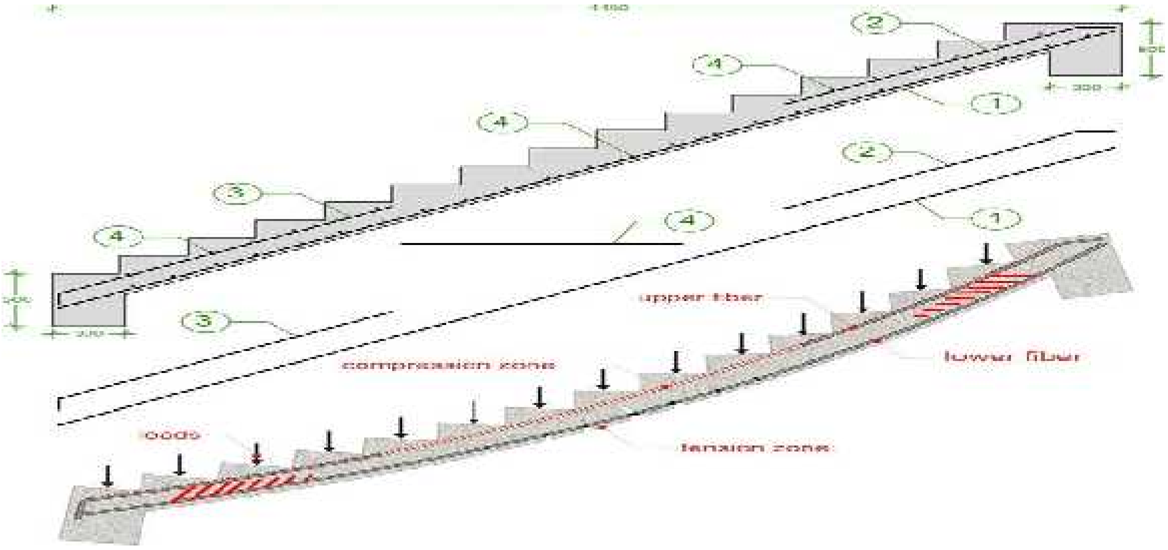


Figure 3. 13: Stair Description and Reinforcing Details.

3.6.7 Foundations:

The last element is to design after collecting all of the loads of the building, so that this element receive the loads and distributes them to the soil. All foundations are divided into two categories: shallow foundations and deep foundations, and in regards to shape, there are a lot of shapes depend on the distance between columns and the bearing capacity of the soil and others. The most important shapes of foundations and were used in this project: Isolated Footing, Combined Footing, Strap Footing and Strip Footing.

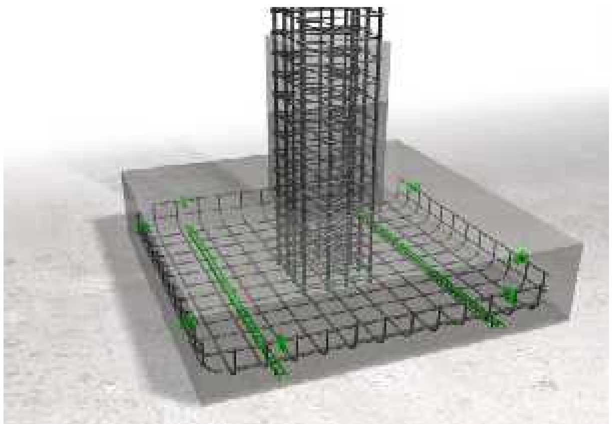


Figure 3. 14: Isolated Footing.

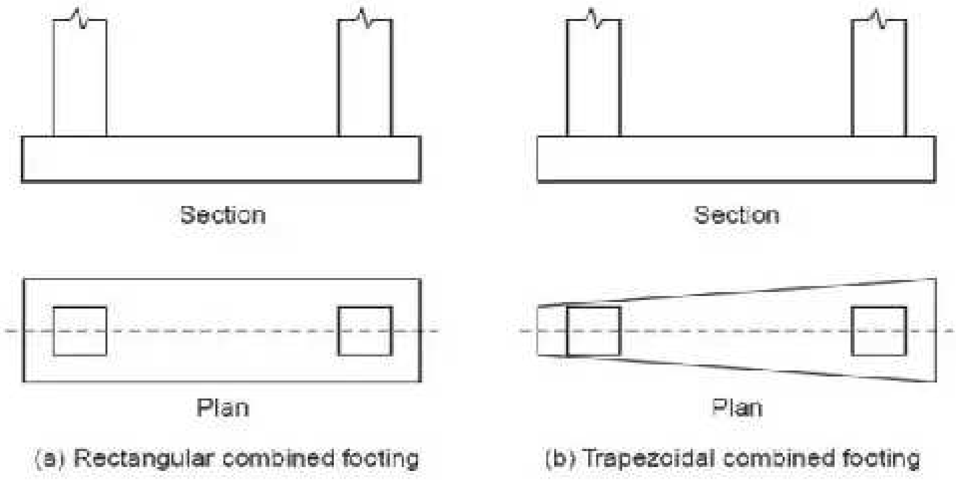
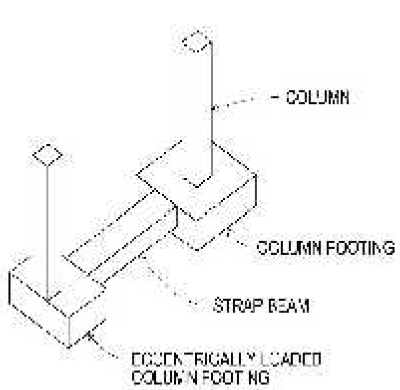


Figure 3. 15: Combned Footing.



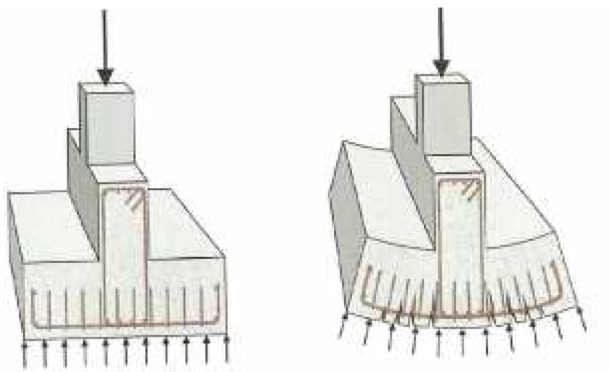


Figure 3. 16: Strap Footing.

Figure 3. 17: Strip Footing.

3.6.8<u>Settlement Joint</u>**:**

A joint between adjacent parts of a building, structure, or concrete work that permits the adjoining masses to settle at slightly different rates. In this project a 5 cm settlement joint width were used.

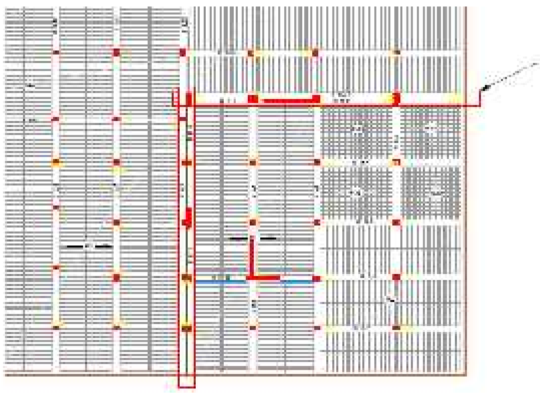


Figure 3. 18: Settlement Joint Location in the Project.

Chapter Four

Structural Analysis & Design

4

4.1 Introduction:

The project consists of several structural elements that will be designed according to ACI code using many of computer software programs, such as ATIR, SP column, Safe and Etabs to find the internal forces, deflections, shear and moments for all structural elements in order to design them.

4.2 Materials Properties were used:

For concrete, it was used a B300 ($fc' = 30*0.8 = 24MPa$) concrete compressive strength for slabs, beams and columns and B350 ($fc' = 35*0.8 = 28MPa$) for foundations.

For reinforcement steel, it is used a 420Mpa steel yielding strength.

For post-tensioned concrete beam, it was used $fc' = 35$ Mpa and prestressing steel type with Low-Relaxation 7-Wire Strand Grade 270 with the following information:

$f_{pu} = 1860\ MPa$

$f_{py} = 0.9 f_{pu} = 1674\ MPa$

$E_p = 196500\ MPa$

$A_p = 98.71\ mm^2$

4.3 Design of Rib 1-14:

Figure 4. 1: Rib 1-14 in the Project.

4.3.1 Determination of slab thickness:

According to ACI-Code-318-11, the minimum thickness of nonprestressed beams or one way slabs unless deflections are computed as follows:

Table 4. 1: Minimum Thickness Requirement of Nonprestressed Beams and One Way Slabs.

Minimum slab thickness(h)				
Member state	Simply Supported	One End Continuous	Both End Continuous	Cantilever
Solid One Way Slabs	L/20	L/24	L/28	L/10
Beams or One Way Ribbed Slabs	L/16	L/18.5	L/21	L/8

The largest spans to be checked are as follows:

- h_{min} for both end continuous = L/21 = 618/21 = 29.43 cm.

The control is 32.108 cm, so select slab thickness of 32cm.

Note: h = 32 cm (24 cm Hollow Block + 8 cm Topping), that all deflection values checked using ATIR program when the slab thickness in other places required more than 32 cm.

❖ **Section :-**

⇒ B = 520 mm

⇒ b_w = 120 mm

⇒ h = 320 mm

⇒ t = 80 mm

The Effective Flange Width (b_e) according to ACI 8.12.2:

b_e is the smallest of:

a) $b_e \leq \frac{L}{4} = \frac{284}{4}$ = 71 cm, (L = 354 – 80/2 – 60/2 = 284 cm; the smallest span)

b) $b_e \leq b_w + 16h_f$ = 12 + 16 * 8 = 140 cm.

c) $b_e \leq$ center to center spacing between adjacent beams = 40 + 12 = 52 cm.

Take b_e = 52 cm.

4.3.2 Determination of ribs loads:

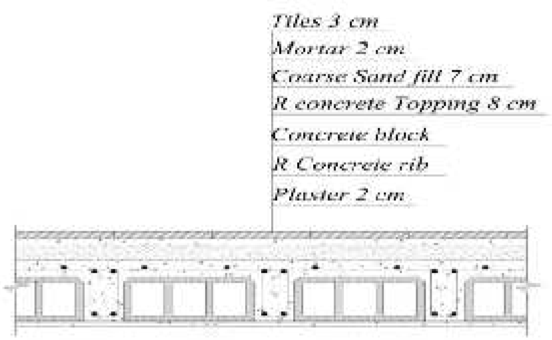


Figure 4. 2: Typical Section in Ribbed Slab.

4.3.2.1 Determination of dead load:

Table 4. 2: Dead Load Determination of Rib.

Dead load from:	δ×γ×b	kN/m/rib
Tiles	0.03*23*0.52	0.359
Mortar	0.03*22*0.52	0.343
Course Sand	0.07*17*0.52	0.619
Topping	0.08*25*0.52	1.04
RC Rib	0.24*25*0.120	0.72
Hollow Block	0.24*10*0.4	0.96
Plaster	0.03*22*0.52	0.343
Interior Partition	2.3*0.52	1.196
	Sum	5.58

4.3.2.2 Determination of live load:

From Jordanian Loads and Forces Code (2006), the live load for education centers = 4 kN/m^2

For rib, live load = 4*0.52 = 2.08 kN/m/rib.

4.3.2.3 Determination of factored loads of ribs:

Wu = 1.2 * 5.58 + 1.6 * 2.08 = 10.024 kN/m/rib.

4.3.3 <u>Design of topping:</u>

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

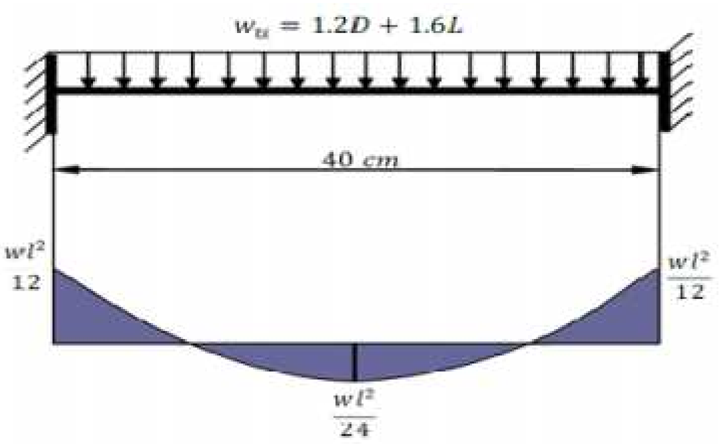


Figure 4. 3: Both Fixed End Static System of the Topping.

4.3.3.1 Determination of dead load of topping:

Table 4. 3: Dead Load Calculation for Topping.

Dead Load From:	δ×γ×1	kN/m
Tiles	0.03*1*23	0.69
Mortar	0.03*22	0.66
Sand	0.07*17	1.19
Topping	0.08*25	2
Partition	2.3	2.3
	Sum	6.84

Live load calculation: 4*1 = 4 kN/m.

Factored load:

W_U = 1.2 ×6.84+ 1.6×4 =14.61 kN/m.

$M_u = \frac{W_u L^2}{12}$

$$Mu = \frac{14.61 * 0.4^2}{12} = 0.195 kN.m/m\ of strip width$$

4.3.3.2 Check the strength condition for plain concrete of topping:

$ØM_n \geq M_u$, where Ø= 0.55

$M_n = 0.42\ \lambda\ \sqrt{f'_c}\ S_m$ (ACI 22.5.1, equation 22-2)

$$S_m = \frac{b.h^2}{6} = \frac{1000.80^2}{6} = 1066666.67\ mm^2.$$

$ØM_n$ =0.55×1×$\sqrt{24}$ ×1066666.67 ×10^{-6} =1.21 kN.m

$ØM_n >> M_u$ = 0.195 kN. m

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

$\rho_{shrinkage}$ = 0.0018 **ACI 7.12.2.1**

$A_s = \rho \times b \times h_{topping}$=0.0018 ×1000×80 = 144 mm^2/m.

Try bars Ø8 with A_s = 50.27 mm^2

Bar numbers n = $\frac{As}{As\ Ø8} = \frac{144}{50.27} = 2.87$

Take 3 Ø 8/m with A_s = 150.8 mm^2 /m strip or Ø 8 @ 300 mm in both directions.

Check for maximum step, (s) is the smallest of:

1. 3h = 3×80 =240 mm. ***control*** **ACI 10.5.4**
2. 450mm.
3. S =380 $\frac{280}{f_s} - 2.5C_c = 380\ \frac{280}{\frac{2}{3}*420} - 2.5\ .20 = 330mm$

   but

   $S \le 300\ \frac{280}{fs} = 300\ \frac{280}{\frac{2}{3}*420} = 300mm$ **ACI 10.6.4**

Take Ø 8 @ 200 mm in both direction, S = 200 mm < S_{max} = 240 mm ………. OK

4.3.4 Design of rib 1-14 for flexure:

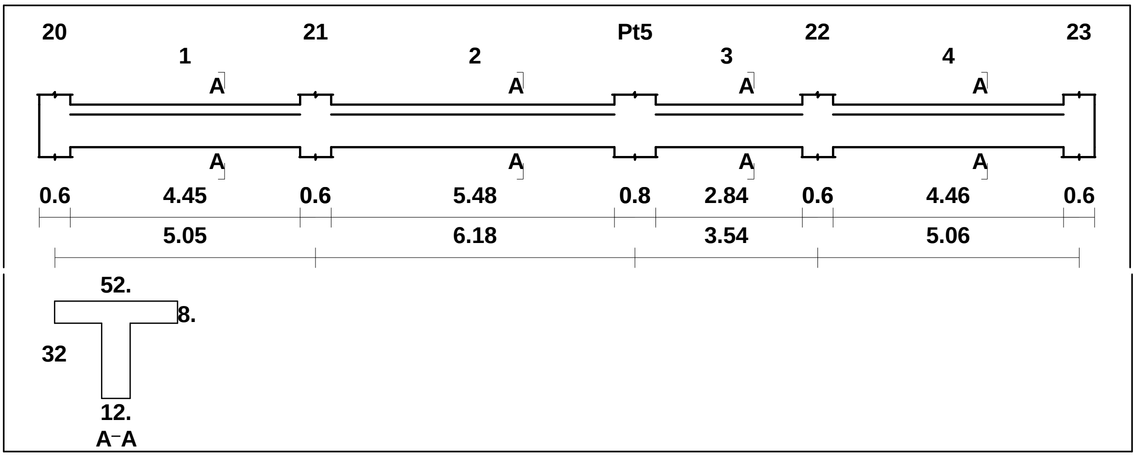


Figure 4. 4: Rib 1-14 Geometry and Spans.

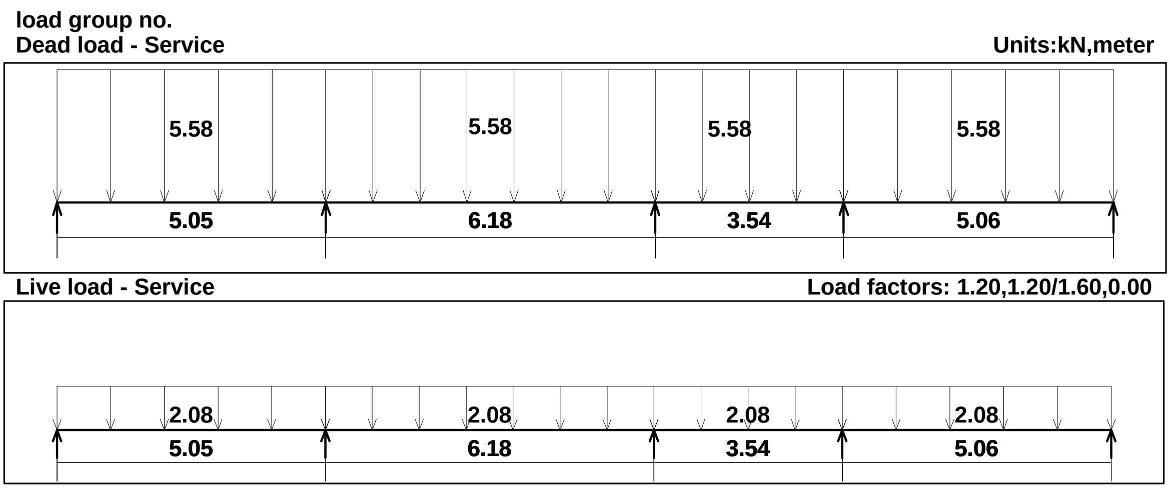


Figure 4. 5: Loading of Rib 1-14.

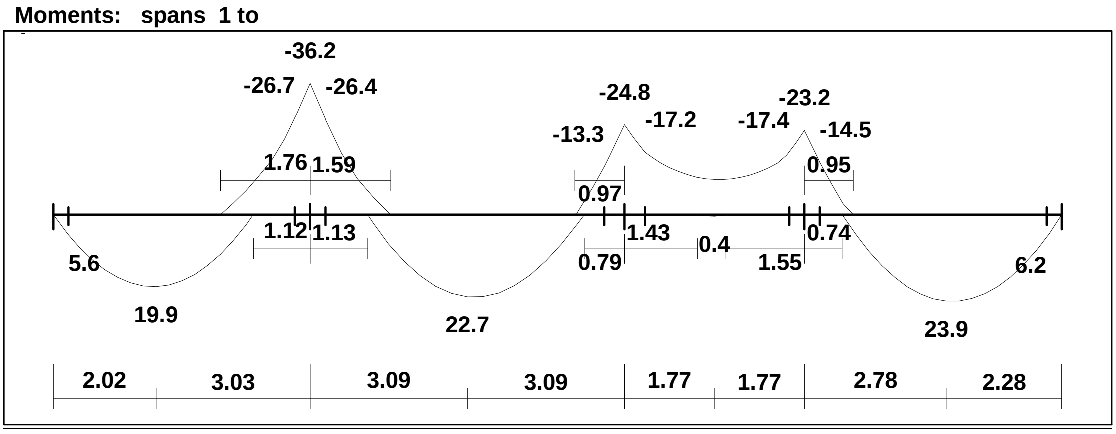


Figure 4. 6: Rib 1-14 Moment Envelop (Factored) Diagram.

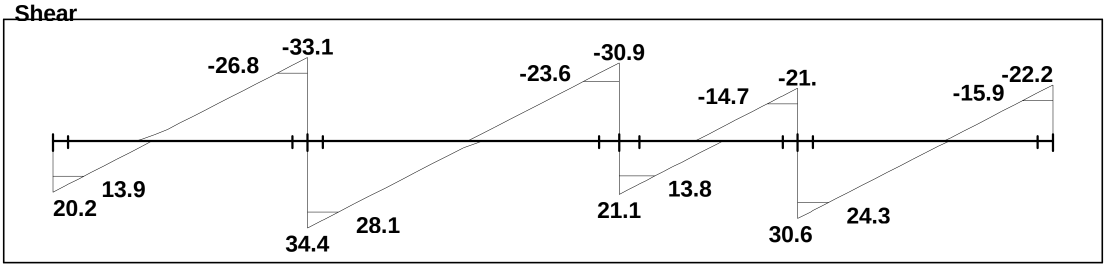


Figure 4. 7: Shear Envelop (Factored) Diagram of Rib 1-14.

4.3.4.1 Design of rib 1-14 for positive moments:

Assume bar diameter Ø 12 for main positive reinforcement.

$$d = h - cover - d_{stirrup} - d_b/2 = 320 - 20 - 10 - 12/2 = 284 \text{ mm.}$$

$A_{s,min}$:-

$$A_{s,min} = 0.25\frac{\sqrt{fc'}}{(fy)}(bw)(d)$$ **ACI-318 (10.5.1)**

$$A_{s,min} = 0.25*\frac{\sqrt{24}}{(420)}(120)(284) = 99.38mm^2$$

$$A_{s,min} = \frac{1.4}{(fy)}(bw)(d)$$

$$A_{s,min} = \frac{1.4}{420}(120)(284) = 113.6mm^2$$ **controls**

The maximum positive moment in all spans of Rib 1-14 M_u = +23.3 kN.m

Check if a > h_f to determine whether the section will act as rectangular or T- section

M_{nf} =0.85. $f_c'.b_e.h_f.(d-\frac{h_f}{2})$

$= 0.85 \times 24 \times 520 \times 80 \times \; 284 - \frac{80}{2} \;\times 10^{-6} = 207.07 \text{ kN.m}$

M_{nf} = 207.07 kN.m >> $\frac{Mu}{\varnothing} = \frac{23.3}{0.9} = 25.9$ kN.m which means a < h_f

So, the section will be designed as rectangular section with b = 520 mm.

$R_n = \frac{Mu}{\varnothing bd^2} = \frac{23.3\times10^6}{0.9\times520\times284^2} = 0.617\ Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\;1-\;\sqrt{1-\frac{2.m.R_n}{420}} = \frac{1}{20.6}\;1-\;\sqrt{1-\frac{2\times20.6\times0.617}{420}} = 0.001493$

$A_{s,req}$ = ρ.b.d = 0.001493 ×520×284 = 220.432 mm^2

As_{req}= 220.432 mm^2 > As_{min}= 113.6 mm^2 **OK**

Use 2 Ø 12, $A_{s,\ provided}$ = 226 mm^2> $A_{s,\ required}$ = 220.432 mm^2 …. Ok

$S = \frac{120-40-20-(2\times12)}{1} = 36\ mm > d_b = 12 > 25\ \ mm \qquad \boldsymbol{OK}$

Check for strain:

$ß_1$ = 0.85 – 0.007(f_c' - 28) **ACI 10.2.7.3**

but f_c'= 24 < 28, so $ß_1$ = 0.85

Compression = Tension
0.85*f_c'*b*a = A_s*f_y

$a = \frac{As.f_y}{0.85b\,f_c'} = \frac{226\times420}{0.85\times520\times24} = 8.948\ mm$

$c = \frac{a}{ß_1} = \frac{8.948}{0.85} = 10.527mm$

$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{284\ -10.527}{10.527} = 0.078\ > 0.005 \qquad \boldsymbol{0k}$

- M_n for 2 Ø 12 is

  $ØM_n$ = 0.9*A_s*f_y*(d-a/2) = 0.9 * 226 * 420 * (284 – $\frac{8.948}{2}$) * 10^{-6}= 23.88 kN.m

❖ M_n for 2 Ø 10 is

d = h – cover – $d_{stirrup}$ – d_b/2 = 320 – 20 – 10 – 10/2 = 285 mm.

$A_{s,min}$:-

$$A_{s,min} = 0.25\frac{\sqrt{fc'}}{(fy)}(bw)(d)$$ **ACI-318 (10.5.1)**

$$A_{s,min} = 0.25*\frac{\sqrt{24}}{(420)}(120)(285) = 99.73mm^2$$

$$A_{s,min} = \frac{1.4}{(fy)}(bw)(d)$$

$$A_{s,min} = \frac{1.4}{420}(120)(285) = 114mm^2 \textbf{ controls}$$

A_s = 157.08 mm^2.> A_{smin} = 114 mm^2

Compression = Tension
0.85*f_c'*b*a = A_s*f_y

$$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{157.08\times420}{0.85\times520\times24} = 6.219\ mm$$

$$c = \frac{a}{\beta_1} = \frac{6.219}{0.85} = 7.317mm$$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{285\ -7.317}{7.317} =$$

$$0.1139 > 0.005 \qquad \mathbf{Ok}$$

$\text{Ø}M_n$ = 0.9*A_s*f_y*(d-a/2) = 0.9 * 157.08 * 420 * (285 – $\frac{6.219}{2}$) * 10^{-6}

= 16.738 kN.m

$$S = \frac{120-40-20-(2\times10)}{1} = 40\ mm > d_b = 12 > 25\ mm \qquad \mathbf{OK}$$

Usually, no reinforcement less than 2 Ø 10 can be used. So, for spans 1, 2 and 4 it is used 2 Ø 12, and 2 Ø 10 for span 3.

4.3.4.2 Design of rib 1-14 for negative moments:

❖ M_n for 2 Ø 14 is
$A_{s,\ 2\ Ø\ 14}$ = 307.88 mm^2
d = 320 – 20 – 10 – 14/2 = 283 mm.

$A_{s,min}$:-

$$A_{s,min} = 0.25\frac{\sqrt{fc'}}{(fy)}(bw)(d)$$ **ACI-318 (10.5.1)**

$$A_{s,min} = 0.25 * \frac{\sqrt{24}}{(420)}(120)(283) = 99.03mm^2$$

$$A_{s,min} = \frac{1.4}{(fy)}(bw)(d)$$

$$A_{s,min} = \frac{1.4}{420}(120)(283) = 113.2mm^2$$ **controls**

$A_{s,\,2\,Ø\,14}$ = 307.88 mm^2.> A_{smin} = 113.2 mm^2

Compression = Tension

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

a =$\frac{A_s f_y}{0.85 b f_c'} = \frac{307.88\times420}{0.85\times120\times24} = 52.82\ mm$

c=$\frac{a}{\beta_1} = \frac{52.82}{0.85} = 62.144\ mm$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{283\ -62.144}{62.144} = 0.0106 > 0.005 \qquad 0k$$

ØM$_n$ = 0.9*A$_s$*f$_y$*(d-a/2) = 0.9 * 307.88 * 420 * (283 –$\frac{52.82}{2}$) * 10^{-6}
= 29.86 kN.m

❖ M_n for 2 Ø 12 is

$A_{s,\,2\,Ø\,12}$ = 226.2 mm^2
d = 320 – 20 – 10 – 12/2 = 284 mm.

$$A_{s,min} = 0.25\frac{\sqrt{fc'}}{(fy)}(bw)(d)$$ **ACI-318 (10.5.1)**

$$A_{s,min} = 0.25 * \frac{\sqrt{24}}{(420)}(120)(284) = 99.38mm^2$$

$$A_{s,min} = \frac{1.4}{(fy)}(bw)(d)$$

$$A_{s,min} = \frac{1.4}{420}(120)(284) = 113.6mm^2$$ **controls**

$A_{s,2 Ø 12}$ = 226.2 mm^2.> A_{smin} = 113.6 mm^2
Compression = Tension

$0.85*f_c'*b*a = A_s*f_y$
$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{226.2\times420}{0.85\times120\times24} = 38.8mm$
$c = \frac{a}{B_1} = \frac{38.8}{0.85} = 45.657\ mm$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{284-45.657}{45.657} = 0.01566 > 0.005 \quad Ok$$

$ØM_n = 0.9*A_s*f_y*(d-a/2) = 0.9 * 226.2 * 420 * (284 - \frac{38.8}{2}) * 10^{-6}$
= 22.62 kN.m

❖ M_n for 2 Ø 10 is

$A_{s, 2 Ø 10}$ = 226.2 mm^2
d = 320 – 20 – 10 – 10/2 = 285 mm.

$A_{s,min}$:-

$A_{s,min} = 0.25\frac{\sqrt{fc'}}{(fy)}(bw)(d)$ **ACI-318 (10.5.1)**

$A_{s,min} = 0.25*\frac{\sqrt{24}}{(420)}(120)(285) = 99.73mm^2$

$A_{s,min} = \frac{1.4}{(fy)}(bw)(d)$

$A_{s,min} = \frac{1.4}{420}(120)(285) = 114mm^2$ **controls**

$A_{s, 2 Ø 10}$ = 157.08 mm^2.> A_{smin} = 114 mm^2

Compression = Tension
$0.85*f_c'*b*a = A_s*f_y$
$a = \frac{A_s f_y}{0.85 b f_c'} = \frac{157.08\times420}{0.85\times120\times24} = 26.95\ mm$
$c = \frac{a}{B_1} = \frac{26.95}{0.85} = 31.70\ mm$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{285-31.70}{31.70} = 0.024 > 0.005 \quad Ok$$

$ØM_n = 0.9*A_s*f_y*(d-a/2) = 0.9 * 157.08 * 420 * (285 - \frac{26.95}{2}) * 10^{-6} = 16.13$ kN.m

Select 2 Ø 10 at supports 1&2, 2 Ø 12 at supports 3&4 and 2 Ø 14 at support 2.

4.3.5 Design of rib 1-14 for shear:

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

- ✓ Determining the excepted shear values:

  Assume Ø 12 flexural reinforcement in all spans, so

  d = 320 – 20 – 10 – 6 = 284 mm.

  $V_c = \frac{1.1}{6} \sqrt{f_c'} b_w d = \frac{1.1}{6} \sqrt{24} \times 120 \times 284 \times 10^{-3} = 30.61\ kN$

  $ØV_c = 0.75 \times 30.61 = 22.96$ kN (Ø = 0.75 for shear)

  $0.5\ Ø\ V_c = 0.5 \times 22.96 = 17.2$ kN.

  All shear values ≤ 17.2 kN needn't shear reinforcement, but as permits by ACI Code that for concrete joist construction excepted of providing minimum shear reinforcement when $0.5\ Ø\ V_c \leq V_u \leq Ø\ V_c$ (case 2), so all shear values less than 22.96 kN are excepted of shear reinforcement.

- ✓ Determining of shear values require minimum shear reinforcement:

  $Vs_{min} = \frac{1}{16} \sqrt{f_c'} bw\ d \geq \frac{1}{3}\ bw\ d$

  $Vs_{min} = \frac{1}{16} \sqrt{24} * 120 * 284 = 10.435 kN$

  Or

  $Vs_{min} = \frac{1}{3}\ bw\ d = \frac{1}{3} * 120 * 284 = 11.3 k$N **(Control).**

  $Ø\ (V_C + Vs_{min}) = 0.75(30.61+11.3) = 31.43$ kN

  Case 3 in shear check requirements $(Ø\ V_C \leq V_u \leq Ø\ (V_C + Vs_{min}))$ define that minimum shear reinforcement must be provided ($A_{v,\ min}$) with:

  $S_{max} \leq \frac{d}{2}$ or $S_{max} \leq 600$ mm.

  So, at supports 2, left of 3 and right of 4 minimum shear reinforcement is to be provided.

Minimum shear reinforcement:

Use stirrups 2 leg Ø 8, $A_v = 2 \times 50.24 = 100.5\ mm^2$

$V_s = \frac{Av * fyt * d}{S}$, but $V_s = 11.3$ kN(minimum), so

$11.3 * 10^3 = \frac{100.5 * 420 * 284}{S}$>>> S = 1060.85 mm.

$S_{max} = 600$ mm. Or $S_{max} = 284 \div 2 = 142$ mm (control).

So, use 2legØ8 @ 125 mm spacing.

4.4 Design of Beam (B, F 5):

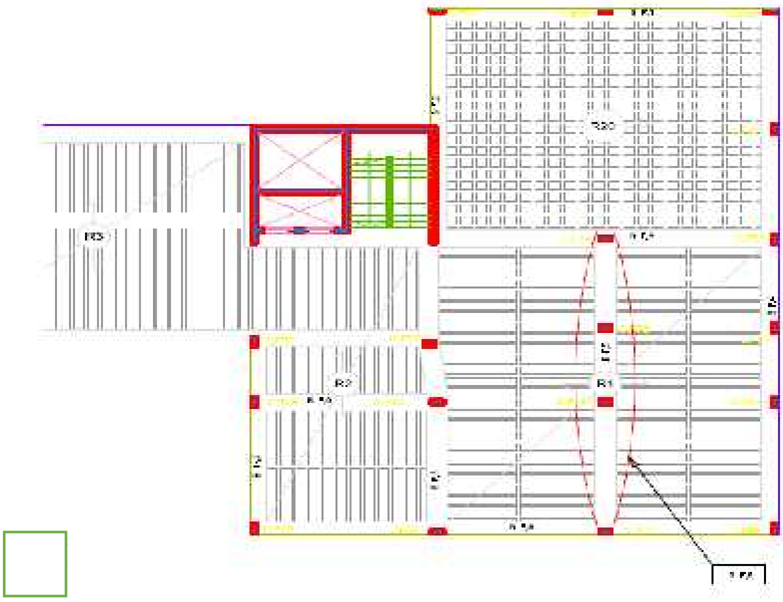


Figure 4. 8: Beam (B, F 5) in the Project

4.4.1 <u>Beam loading and envelop diagrams:</u>

BEAMD program was used for loading and getting the envelop diagrams for bending moment and shear force as shown below:

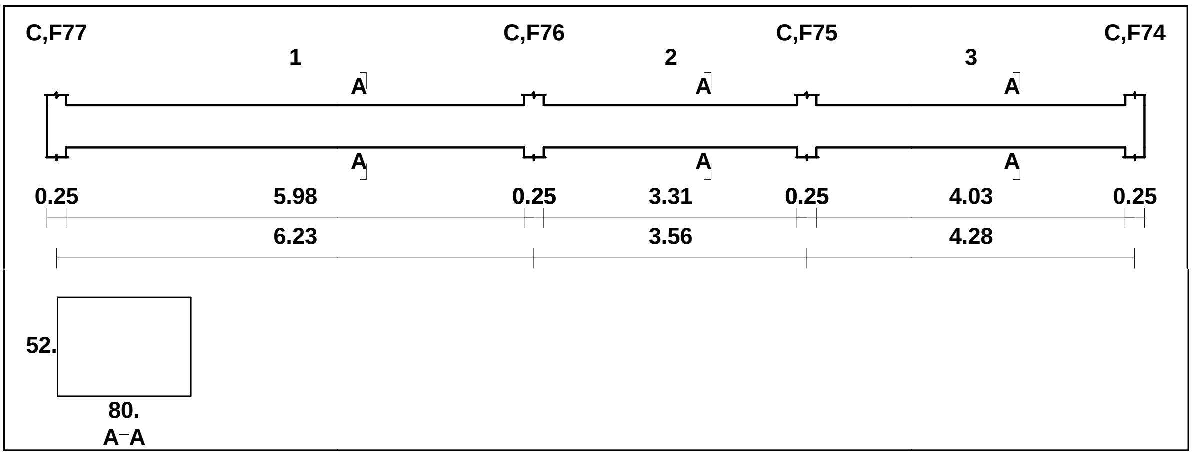


Figure 4. 9: Beam (B, F 5) Spans and Geometry.

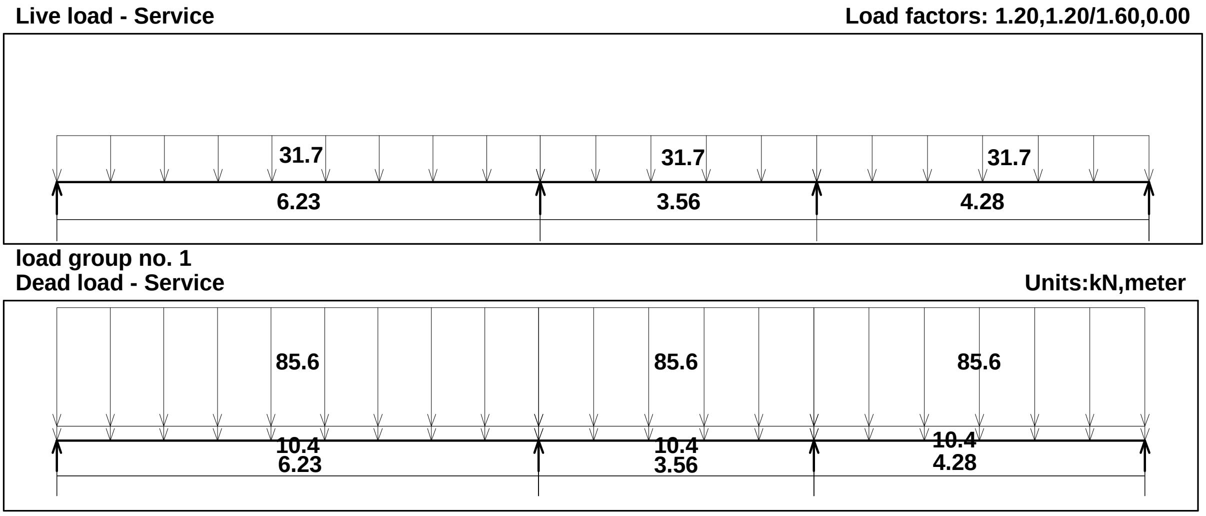
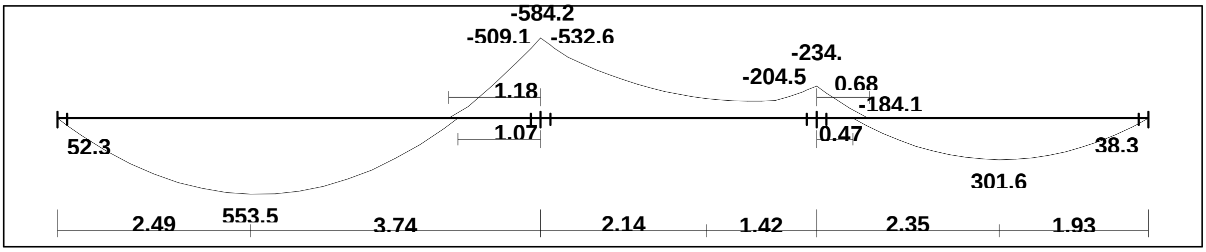


Figure 4. 10: Beam (B, F 5) Loading.



Span1 Span2 Span3

Figure 4. 11: Beam (B, F 5) Moment Envelop (Factored) Diagram.

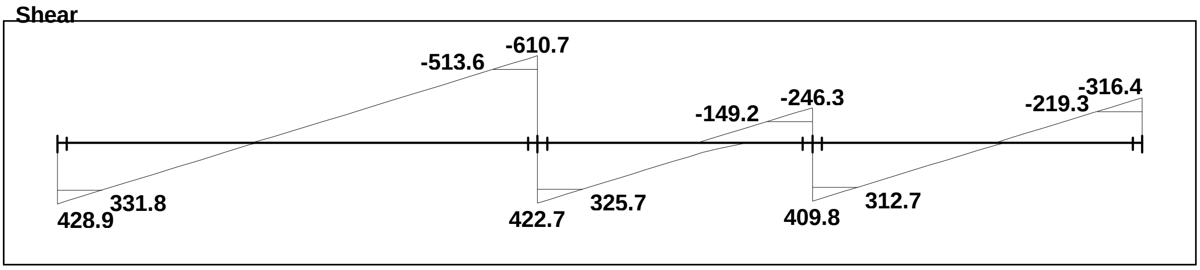


Figure 4. 12: Beam (B, F 5) Shear Force Envelop (Factored) Diagram.

Load calculations for beam (B, F 5):

The distributed Dead and Live loads acting upon (B,F 5) can be defined from the support reactions of the(Rib 1) and the beam self-weight.

⇒ Service dead load from rib1 = 44.52 ÷ 0.52 = 85.62 kN/m.
Weight of the beam = h * b * γ = 0.52 * 0.8 * 25 = 10.4 kN/m.
Total dead load on the beam = 96.02 kN/m.
Service live load from rib1 = 16.5 ÷ 0.52 = 31.7 kN/m.

4.4.2 Design the beam for bending moments:

Assume Ø 25 for main reinforcement of the beam,

d_t = depth of beam – cover – diameter of stirrup – bar diameter/2

= 520 – 40 – 10 – 25/2 = 457.5mm. (for singly sections, $d = d_t$).

Maximum moment in span 1 with M_u = 553.5 kN.m.

- Check whether the section is singly or doubly.

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

$c = \frac{3}{7} * d_t = \frac{3}{7} * 457.5 = 196.07mm$,

$\beta_1 = 0.85 - 0.007(f_c' - 28) = 0.85 - 0.007(24 - 28)$, take $\beta_1 = 0.85$.

$a = \beta_1 * c = 0.85 * 196.07 = 166.66mm$.

$M_{n,max} = 0.85 * f_c' * a * b(d - \frac{a}{2}) = 0.85 * 24 * 166.66 * 800 * (457.5 - 166.66/2) * 10^{-6}$

= 1017.7 kN.m.

$\Phi = 0.82$

$M_u = 553.5$ kN.m < $\Phi M_n = 0.82 * 1017.7 = 834.514$ kN.m,

So, design the beam as singly reinforcement concrete section.

$A_{s,min}$:-

$$A_{s,min} = \frac{\sqrt{fc'}}{4(fy)}(bw)(d) = \frac{\sqrt{24}}{4*420}*800*457.5 = 1067.3 \text{ mm}^2$$

$$A_{s,min} = \frac{1.4}{(fy)}(bw)(d) = \frac{1.4}{420}*800*457.5 = 1220 \text{ mm}^2$$ **Controls**

4.4.2.1 Design for positive moments:

- Design for M_u = 553.5 kN.m:

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{553.5 \times 10^6}{0.9 \times 800 \times 457.5^2} = 3.673Mpa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 20.6 \times 3.673}{420}}\right) = 0.00972$$

$A_s = \rho.b.d = 0.00972 \times 800 \times 457.5 = 3556.6 \text{ mm}^2 > A_{s,min} = 1220 \text{ mm}^2$.

Use 8Ø20 as bottom bars, $A_{s,provided} = 3960 \text{ mm}^2 > A_{s,required} = 3556.6 \text{ mm}^2$… Ok

Check spacing:

$$S = \frac{800-40*2-20-8\times25}{7} = 71.43\ mm > d_b = 25mm > 25\ mm \quad OK$$

Check for strain:-

$$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{3960\times420}{0.85\times800\times24} = 101.91\ mm$$

$$c = \frac{a}{B_1} = \frac{101.91}{0.85} = 119.9\ mm$$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{457.5-119.9}{119.9} = 0.00845 > 0.005 \qquad Ok$$

- Design for M_u = 553.5 kN.m:

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{553.5\times10^6}{0.9\times800\times457.5^2} = 3.673Mpa$$

$$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times3.673}{420}}\right) = 0.00972$$

$A_s = \rho.b.d = 0.00972\times800\times457.5 = 3556.6\ mm^2 > A_{s,min} = 1220\ mm^2$.

Use 8Ø25 as bottom bars, $A_{s,provided} = 3960\ mm^2 > A_{s,required} = 3556.6\ mm^2$… Ok

Check spacing:

$$S = \frac{800-40*2-20-8\times25}{7} = 71.43\ mm > d_b = 25mm > 25\ mm \quad OK$$

Check for strain:-

$$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{3960\times420}{0.85\times800\times24} = 101.91\ mm$$

$$c = \frac{a}{B_1} = \frac{101.91}{0.85} = 119.9\ mm$$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{457.5-119.9}{119.9} = 0.00845 > 0.005 \qquad Ok$$

- Design for span 2:
  In span 2, the bending moment envelop diagram shows that there is no positive moment in this span 2, so it must provide the minimum reinforcement in this region.
  $A_{s,min} = 1220\ mm^2$,

Use 4Ø20 as bottom bars, $A_{s,provided} = 1256\ mm^2 > A_{s,min} = 1220\ mm^2$… OK

Check spacing:

$$S = \frac{800-40*2-20-4\times20}{3} = 206.67\ mm > d_b = 25mm > 25\ mm \quad OK$$

Check for strain:

a $=\frac{A_s.f_y}{0.85b\ f_c'}=\frac{1256\times420}{0.85\times800\times24}=32.32\ mm$

c $=\frac{a}{B_1}=\frac{32.32}{0.85}=38.03\ mm$

d = 520 – 40 – 10 – 20/2 = 460 mm.

$$\varepsilon_s=0.003\ \frac{d-c}{c}\ =0.003\ \frac{460-38.03}{38.03}\ =0.0333>0.005\qquad Ok$$

- Design for M_u = 301.6 kN.m:

$Rn=\frac{M_u}{\emptyset bd^2}=\frac{301.6\times10^6}{0.9\times800\times457.5^2}=2.001Mpa$

m$=\frac{f_y}{0.85f_c'}=\frac{420}{0.85\times24}=20.6$

ρ$=\frac{1}{m}\ \left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right)=\frac{1}{20.6}\ \left(1-\sqrt{1-\frac{2\times20.6\times2.001}{420}}\right)=0.005025$

A_s = ρ.b.d = 0.005025×800×457.5 = 1839.2 mm^2>$A_{s,min}$ = 1220 mm^2.

Use 4Ø25 as bottom bars, $A_{s,provided}$ = 1980 mm^2> $A_{s,required}$ = 1839.2 mm^2… Ok

Check spacing:

S $=\frac{800-40*2-20-4\times25}{3}=200\ mm>d_b=25mm>25\ mm\quad OK$

Check for strain:

a $=\frac{A_s.f_y}{0.85b\ f_c'}=\frac{1980\times420}{0.85\times800\times24}=50.96\ mm$

c $=\frac{a}{B_1}=\frac{50.96}{0.85}=59.95\ mm$

$$\varepsilon_s=0.003\ \frac{d-c}{c}\ =0.003\ \frac{457.5-50.96}{50.96}\ =0.0199>0.005\qquad Ok$$

4.4.2.2 Design the beam for negative moments:

- Design for M_u = -532.6 kN.m:

$Rn=\frac{M_u}{\emptyset bd^2}=\frac{532.6\times10^6}{0.9\times800\times457.5^2}=3.534Mpa$

m$=\frac{f_y}{0.85f_c'}=\frac{420}{0.85\times24}=20.6$

ρ$=\frac{1}{m}\ \left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right)=\frac{1}{20.6}\ \left(1-\sqrt{1-\frac{2\times20.6\times3.534}{420}}\right)=0.0093$

A_s = ρ.b.d = 0.0093×800×457.5 = 3406.12 mm^2>$A_{s,min}$ = 1220 mm^2.

Use 7Ø25 as top bars, $A_{s,provided}$ = 3465 mm^2> $A_{s,required}$ = 3406.12 mm^2… Ok

Check spacing:

$S = \frac{800-40*2-20-7\times25}{6} = 87.5mm > d_b = 25mm > 25\ mm \quad \boldsymbol{OK}$

Check for strain:

$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{3465\times420}{0.85\times800\times24} = 89.17\ mm$

$c = \frac{a}{B_1} = \frac{89.17}{0.85} = 104.91\ mm$

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{457.5-104.91}{104.91} = 0.01 > 0.005 \qquad \boldsymbol{Ok}$$

- Design for M_u = -204.5 kN.m:

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{204.5\times10^6}{0.9\times800\times457.5^2} = 1.357Mpa$

$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times1.357}{420}}\right) = .00335$

A_s = ρ.b.d = 0.00335×800×457.5 = 1224.74 mm^2>$A_{s,min}$ = 1220 mm^2.

Use 4Ø20 as top bars, $A_{s,provided}$ = 1256 mm^2> $A_{s,required}$ = 1224.74 mm^2… Ok

Check spacing:-

$S = \frac{800-40*2-20-4\times20}{3} = 206.67\ mm > d_b = 25mm > 25\ mm \quad \boldsymbol{OK}$

Check for strain:-

$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{1256\times420}{0.85\times800\times24} = 32.32\ mm$

$c = \frac{a}{B_1} = \frac{32.32}{0.85} = 38.03\ mm$

d = 520 – 40 – 10 – 20/2 = 460 mm.

$$\varepsilon_s = 0.003\ \frac{d-c}{c} = 0.003\ \frac{460-38.03}{38.03} = 0.0333 > 0.005 \qquad \boldsymbol{Ok}$$

4.4.3 Design of beam for shear forces:

Beams must be designed to resist shear forces acting on, but the shear values should be checked in which case it exists in order to provide the required shear reinforcement. The cases are described as follows:

- Case 1:

  If $V_u < \frac{1}{2}\Phi V_c$, no shear reinforcement is required.

- Case 2:

  If $\frac{1}{2}\Phi V_c < V_u < \Phi V_c$, minimum shear reinforcement must be provided, and the maximum spacing between stirrups is the lowest of 600 mm or d/2.

  Minimum shear reinforcement is the larger of:

  $V_{s,min} = \frac{1}{3} b_w * d$ or $\frac{1}{16}\sqrt{fc'}\, b_w * d$ (which is greater).

- Case 3:

  If $\Phi V_c < V_u$, shear reinforcement is required and can computed as:

  $V_s = \frac{Vu}{\Phi} - V_c$,

  - ✓ If $V_s < V_{s,min}$, minimum shear reinforcement is to be provided
  - ✓ If $V_{s,min} < V_s < V_s'$, which $V_s' = \frac{1}{3}\sqrt{fc'}\, b_w * d$, so design for the V_s value and the maximum spacing is the lowest of 600mm or d/2.
  - ✓ If $V_s' < V_s < V_{s,max}$, which $V_{s,max} = \frac{2}{3}\sqrt{fc'}\, b_w * d$, so design for the V_s value and the maximum spacing is the lowest of 300mm or d/4.
  - ✓ If $V_s > V_{s,max}$, the section dimensions must be changed.

☒ The section capacity for shear force is:

$V_c = \frac{1}{6}\sqrt{fc'}\, b_w * d = \frac{1}{6}\sqrt{24} * 800 * 457.5 * 10^{-3} = 298.84$ kN.

$\Phi\ V_c = 0.75 * 298.84 = 224.13$ kN.

$\frac{1}{2}\Phi\ V_c = 112.06$ kN.

☒ For minimum shear reinforcement:

$V_{s,min} = \frac{1}{3} * 800 * 457.5 * 10^{-3} = 122$ kN. **(Control)**

Or $V_{s,min} = \frac{1}{16}\sqrt{24} * 800 * 457.5 * 10^{-3} = 112.06$ kN.

Assume 2legØ10 with $A_v = 2*78.54 = 157.08\ mm^2$,

$$S = \frac{A_v f_{yt} d}{v_s} = \frac{157.08 * 420 * 457.5}{122 * 1000} = 247.4\ mm$$

Maximum spacing,
S_{max} = 600 or 457.5/2 = 228.75 (Controls), use 2legØ10 at 225mm.
The maximum shear value that minimum shear reinforcement can resist:
$Vu = \Phi\ (Vs_{,min} + V_c) = 0.75 * (122 + 224.13) = 259.6$ kN.

Check the shear values under the three cases described before:

- There is no shear value exists in the first case.

- Case 2:
  Shear force values 219.3 and 149.2 are exist in this case,
  112.06 < (149.2/219.3) < 224.13.
  Minimum shear reinforcement is required.

- Case 3:
  - ✓ $V_s' = \frac{1}{3} \sqrt{fc'}\ b_w*d = \frac{1}{3}\sqrt{24} * 800 * 457.5 * 10^{-3} = 597.7$ kN.
  - ✓ $V_{s,max} = \frac{2}{3} \sqrt{fc'}\ b_w*d = \frac{2}{3}\sqrt{24} * 800 * 457.5 * 10^{-3} = 1195.35$ kN.

  For shear values (312.7, 325.7 and 331.8 kN), check for the greatest value of 331.8 kN.
  $V_s = 331.8/0.75 - 224.13 = 218.27$ kN, $V_{s,min} = 122$ kN < 218.27 kN < $V_s' = 597.7$ kN.

  Assume 2legØ10 with $A_v = 2*78.54 = 157.08$ mm^2,

$$S = \frac{A_v f_{yt} d}{v_s} = \frac{157.08 * 420 * 457.5}{218.27 * 1000} = 138.3\ mm$$

  Maximum spacing,
  S_{max} = 600 or S_{max} = 457.5/2 = 228.75 (Controls) >138.3 mm, use 2legØ10 at 125mm.

  For shear value 513.6 kN:
  $V_s = 513.6/0.75 - 224.13 = 459.95$ kN, $V_{s,min} = 122$ kN < 459.95 kN < $V_s' = 597.7$ kN.
  Assume 4legØ10 with $A_v = 4*78.54 = 314.16$ mm^2,

$$S = \frac{A_v f_{yt} d}{v_s} = \frac{314.16 * 420 * 457.5}{459.95 * 1000} = 131.24\ mm$$

  Maximum spacing,
  S_{max} = 300 or S_{max} = 457.5/4 = 114.375 (Controls) <131.24 mm, use 4legØ10 at 100mm.

4.5 Design of Post TensionedBeam (PTB2, F1):

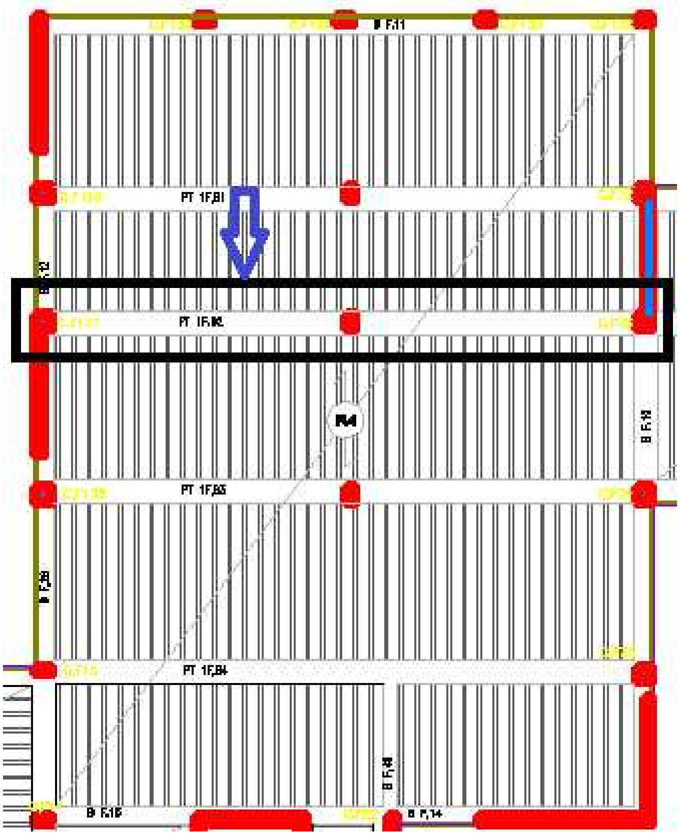
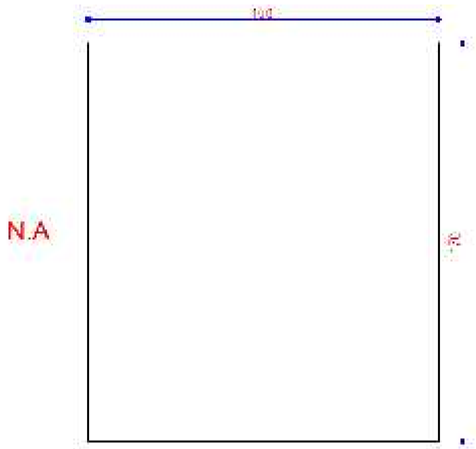


Figure 4. 13: Beam (PTB2, F1) in Project Plan.



- Slab information:
  - Slab type is one-way ribbed slab
  - Slab thickness is 32 cm
  - Material properties:

    $f_{c'} = 24\ MPa$

    $f_y = 420\ MPa$

- Beam information:
  - Concrete properties:

    $f'_c = 35\ MPa$

    $f'_{ci} = 0.75 f_{c'} = 26.25\ MPa$

  - Steel Reinforcement:

Figure 4. 14: Cross Section of the Beam.

    $f_y = 420\ MPa$

    $f_{yt} = 420\ MPa$

- Prestressing steel:

  Prestressing steel type is Low-Relaxation 7-Wire Strand Grade 270

  $f_{pu} = 1860\ MPa = 1860\ MPa$

$f_{py} = 0.9 f_{pu} = 1674\ MPa$
$E_p = 196500\ MPa$
$A_p = 98.71\ mm^2$

- Cross section of the beam:
  - Width = 150 cm and Depth = 170 cm
  - $A_g = bh = 18400\ cm^2$
  - $y_t = 85\ cm$ and $y_b = 85\ cm$
  - $I_x = \frac{1}{12} bh^3 = 61412500\ cm^4$
  - $S_t = \frac{I_x}{y_y} = 722500\ cm^3$

$$S_b = \frac{I_x}{y_b} = 722500\ cm^3$$

4.5.1 Load Calculations:

Live load =2 KN/m2 (Roof live load)

Table 4. 4: Determination of Loads Acting in the Beam.

Load Name	Type of load	Load(KN/m)	Load(KN)
slab's self weight	Uniform load	25*0.32*3.4 = 27.2	-----
beam's self weight	Uniform load	25*1.5*1.7 = 63.75	-----
Live load	Uniform load	2*4.9 = 9.8	-----
Superimposed from the column	Concentrated load	------	1028.02
Live load from the column	Concentrated load	------	146.7

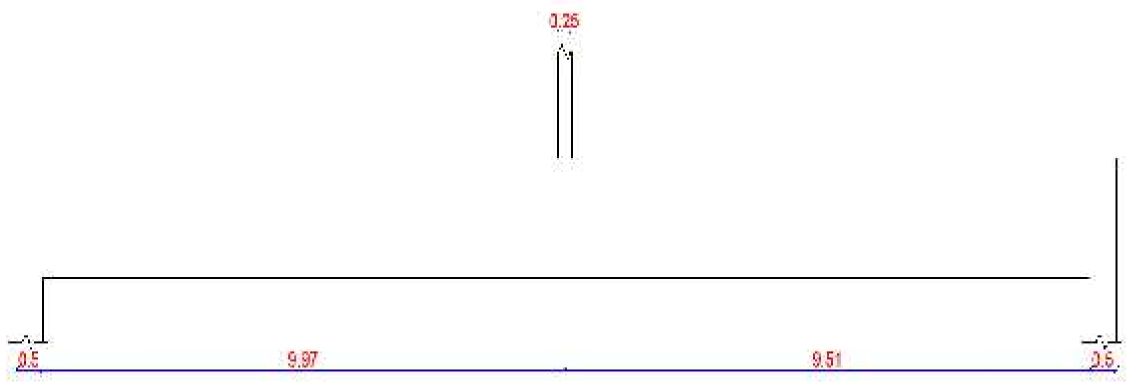


Figure 4. 15: Beam General Information.

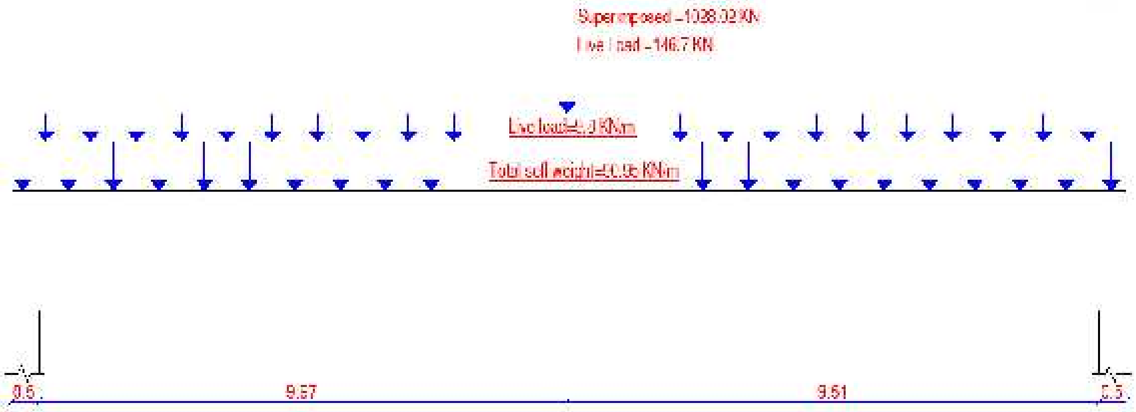


Figure 4. 16: The Beam loads.

4.5.2 Design of the PTBeam:

4.5.2.1 Assumptions to start the design processes:

- Balancing 90% of self weight
- Assume Drape (a) = 36 cm The balancing load is :
  $W_{balancing} = 0.9\, W_{total\, self-weight} = 81.86\ KN/m$

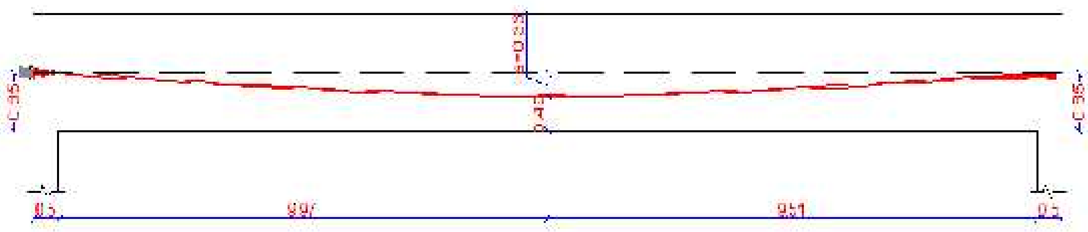


Figure 4. 17: Tendon Profile of the Beam.

- The effective force in tendons (F_e) is:
  $W_{balancing} = \dfrac{8\, F_e\, a}{l^2} \xrightarrow{yields} F_e = \dfrac{W_{balancing} l^2}{8\, a}$

  $F_e = 11346.04\ KN$

- Find the usable stress to each tendon (F_e/tendon):

  According to ACI, stress and seat an anchor at 70% of f_{pu}, so :

  $f_{pi} = 0.7\, f_{pu} = 0.7 * 1860 = 1302\ MPa$

  Assume a loss that applies to all reasons is 100 MPa, and it's going conservatively cover them, so:

  $f_{pe} = 1302 - 100 = 1202\ MPa$

$$\frac{F_e}{tendon} = f_{pe} * A_{ps} = 1202 * 98.71 * 10^{-3} = 118.65 \frac{KN}{tendon}$$

$$\frac{F_i}{tendon} = f_{pi} * A_{ps} = 1302 * 98.71 * 10^{-3} = 128.52 \frac{KN}{tendon}$$

>>> So, the number of tendons required is:

$$\# \, of \, tendons = \frac{F_e}{\frac{F_e}{tendon}} = \frac{11346.04}{118.65} = 95.63 \, tendons$$

So, use 95 tendons

$$F_e = \frac{F_e}{tendon} * \# \, of \, tendons = 11271.75 \, KN$$

$$F_i = \frac{F_i}{tendon} * \# \, of \, tendons = 12209.4 \, KN$$

So, $W_{balancing} = \frac{8 F_e a}{l^2} = 81.32 \, KN/m$

4.5.2.2 Check for Stresses:

1. Stresses At transfer:

$$f_t = -\frac{F_i}{A_g} \pm \frac{M_{net}}{S_t}$$

$$f_b = -\frac{F_i}{A_g} \pm \frac{M_{net}}{S_b}$$

Assumed the compression stress is negative (-), and the tension stress is positive (+).

- Allowable stresses based on ACI:

The allowable of maximum compression stress in the cross section of the beam is:

$$f_{ci} = 0.6 \, f'_{ci} = 0.6 * 26.25 = 15.75 \, MPa$$

The allowable of maximum tension stress in the cross section of the beam is:

$f_{ti} = 0.25 \sqrt{f_{ci'}} = 1.28 \, MPa$ “At mid of the span”

$f_{ti} = 0.5 \sqrt{f_{ci'}} = 2.56 \, MPa$ “At edges of the simply supported”

- $W_{net} = W_{total\, self-weight} - W_{balancing} = 9.63 \, KN/m$

For simply supported with uniform load, the moment is:

$$M_{net} = \frac{W_{net} l^2}{8} = \frac{9.63 * 19.98^2}{8} = 480.54 \, KN.m$$

Find stress at the left support, $M_{net} = 0$:

$$f_t = -\frac{12209.4}{1.84}\ 10^{-3} = -6.6\ MPa < -15.75\ MPa$$

$$f_b = -\frac{12209.4}{1.84}\ 10^{-3} = -6.6\ MPa < -15.75\ MPa$$

Find stress at the left support, $M_{net} = 480.54\ KN.m$: :

$$f_t = -\frac{12209.4}{1.84} - \frac{480.54}{0.7225}\ 10^{-3} = -7.3\ MPa < -15.75\ MPa$$

$$f_b = -\frac{12209.4}{1.84} + \frac{480.54}{0.7225}\ 10^{-3} = -5.97\ MPa < -15.75\ MPa$$

Find stress at the Right support, $M_{net} = 0$:

$$f_t = -\frac{12209.4}{1.84}\ 10^{-3} = -6.6\ MPa < -15.75\ MPa$$

$$f_b = -\frac{12209.4}{1.84}\ 10^{-3} = -6.6\ MPa < -15.75\ MPa$$

2. Check for effective stresses after losses:

$$f_t = -\frac{F_e}{A_g} \pm \frac{M_{net}}{S_t}$$

$$f_b = -\frac{F_e}{A_g} \pm \frac{M_{net}}{S_b}$$

- Allowable stresses based on ACI:

The allowable of maximum compression stress in the cross section of the beam is:

$$f_c = 0.45\ f'_c = 0.45 * 35 = 15.75\ MPa$$

The allowable of maximum tension stress in the cross section of the beam is:

$f_t = \sqrt{f'_c} = 5.92\ MPa$ “Class cracked“

Find stress at the left support, $M_{net} = 0$:

$$f_t = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

$$f_b = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

Find stress at the left support, $M_{net} = 480.54\ KN.m$: :

$$f_t = -\frac{11271.75}{1.84} - \frac{480.54}{0.7225}\ 10^{-3} = -6.8\ MPa < -15.75\ MPa$$

$$f_b = -\frac{11271.75}{1.84} + \frac{480.54}{0.7225}\ 10^{-3} = -5.5\ MPa < -15.75\ MPa$$

Find stress at the right support, $M_{net} = 0$:

$$f_t = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

$$f_b = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

3. Check for stresses at service loads:

$$f_t = -\frac{F_e}{A_g} \pm \frac{M_{total}}{S_t}$$

$$f_b = -\frac{F_e}{A_g} \pm \frac{M_{total}}{S_b}$$

- Allowable stresses based on ACI:

The allowable of maximum compression stress in the cross section of the beam is:

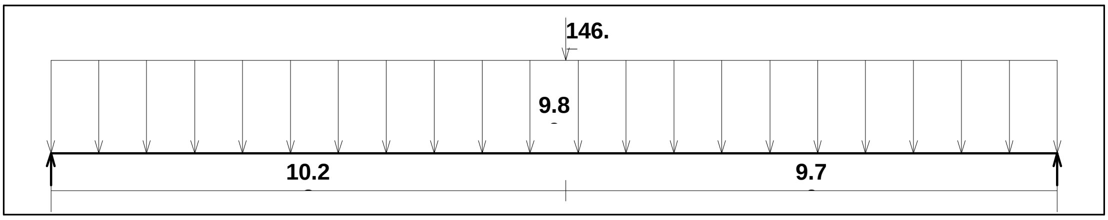
$$f_c = 0.45\ f'_c = 0.45 * 35 = 15.75\ MPa$$

The allowable of maximum tension stress in the cross section of the beam is:

$$f_t = \sqrt{f'_c} = 5.92\ MPa$$ "Class cracked"

Find M_{total}:

Live load - Service



Dead load - Service Units:kN,met

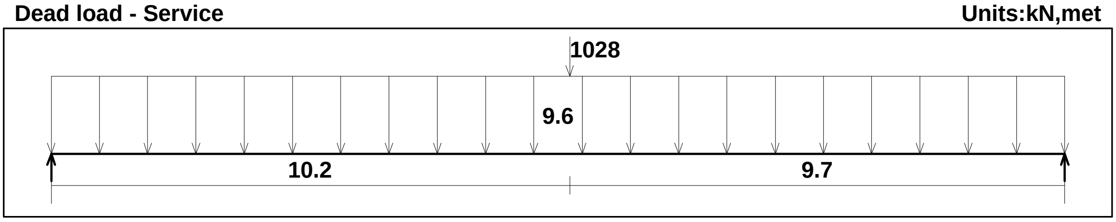


Figure 4. 18: Beam (PTB2, F1) Loading System.

Moments: spans 1 to

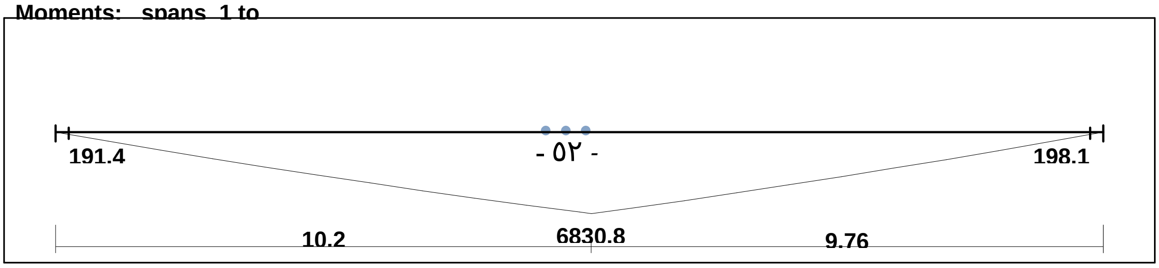


Figure 4. 19: Beam (PTB2, F1) Bending Moment Diagram (service).

Find stress at the left support, $M_{total} = 0$:

$$f_t = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

$$f_b = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

Find stress at the left support, $M_{total} = 6830.8\ KN.m$:

$$f_t = -\frac{11271.75}{1.84} - \frac{6830.8}{0.7225}\ 10^{-3} = -15.6\ MPa < -15.75\ MPa$$

$$f_b = -\frac{11271.75}{1.84} + \frac{6830.8}{0.7225}\ 10^{-3} = +3.33\ MPa < +5.92\ MPa$$

Find stress at the right support, $M_{total} = 0$:

$$f_t = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

$$f_b = -\frac{11271.75}{1.84}\ 10^{-3} = -6.13\ MPa < -15.75\ MPa$$

So the assumptions are okay for stresses check.

4.5.2.3 Design for flexural strength:

- Factored Mu:

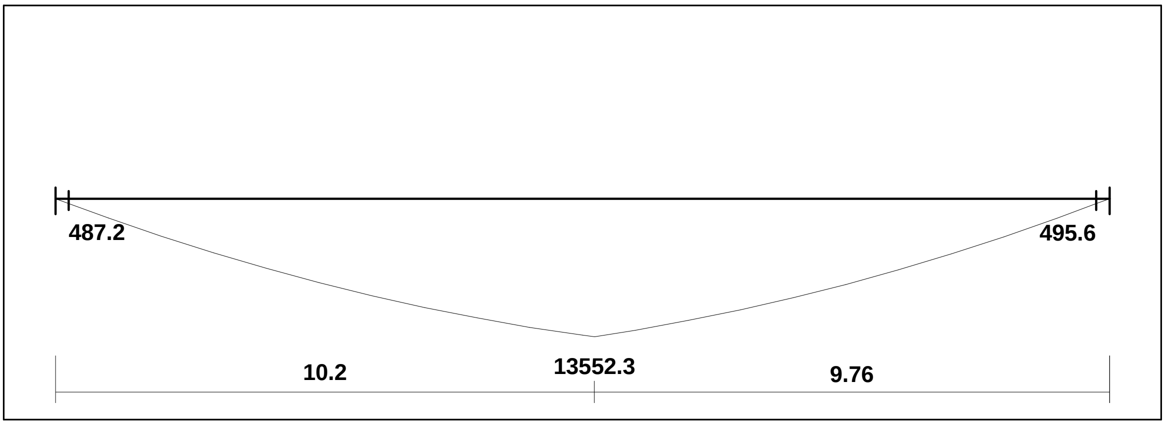


Figure 4. 20: Beam (PTB2, F1) Bending Moment Diagram (Factored).

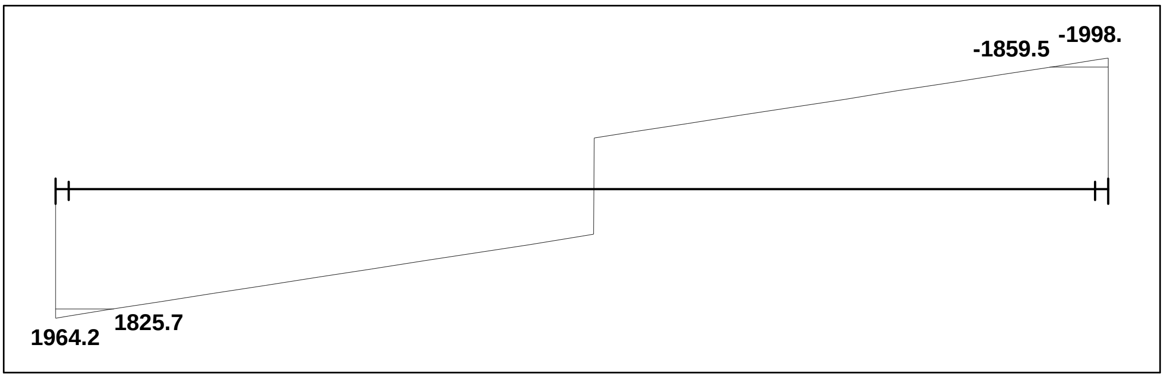


Figure 4. 21: Beam (PTB2, F1) Shear Diagram.

Find Prestressing steel nominal failure stress (f_{ps}) according to ACI, Bonded system:

$$f_{ps} = f_{pu}(1 - \frac{\gamma_p}{\beta_1} \; \rho_p \; \frac{f_{pu}}{f'_c} \; + \; \frac{d * F_y}{d_p * f'_c} \quad \frac{A_s}{b * d} - \frac{A'_s}{b * d}$$

This equation is applies if:

$0.5 f_{pu} \leq f_{pe}$

$f_{pe} = 0.7 f_{pu} - losses = 0.7 * 1860 - 100 = 1202 \; MPa$

$0.5 f_{pu} = 0.5 * 1860 = 930 \; MPa \;\; \leq 1202 \; MPa \Rightarrow okay$

The values of equation parameters:

- $\beta_1 = 0.85 - 0.007 \;\; f'_c - 28 \;\; = \;\; 0.85 - 0.007 \;\; 35 - 25 \;\; = 0.801$
- γ_p from table (4.5)

$$\frac{f_{py}}{f_{pu}} = \frac{0.9 \, f_{pu}}{f_{pu}} = 0.9 \;\; \Longrightarrow \gamma_p = 0.28$$

Table 4. 5: Values of γ_p

f_{py}/f_{pu}	γ_p
≥ 0.80	0.55
≥ 0.85	0.40
≥ 0.90	0.28

- $A_s = A_{s\,min.} = 0.004 A_{ct} = 0.004 * 85 * 150 = 51 \; cm^2$
  use 11$\emptyset$25 $\rightarrow$ $A_s = 53.9 \; cm^2$

- Spacing for 1 layer (S) = 11.65 cm $>$ 2.5 cm $\rightarrow$ $okay$

- $d = 170 - 2 - 1 - \frac{2.5}{2} = 165.75\ cm$
  cover = 2cm and $\emptyset 10\ for\ stirrups$
- $d_p = y_t + a = 85 + 36 = 121 cm$
- $\rho_p = \frac{A_{ps}}{b*d_p} = \frac{95*98.71*10^{-2}}{150*121} = 5.17*10^{-3}$
- No compression reinforcement $(\frac{A'_s}{b*d} = 0)$

So,

$$f_{ps} = 1860(1 - \frac{0.28}{0.801}\ 5.17*10^{-3}\ \frac{1860}{35} + \frac{165.75*420}{121*35}\ \frac{53.9}{150*165.75} - 0 \quad = 1658.31\ MPa$$

$$T_s = A_{st} * f_y = 53.9 * 420 * 10^{-1} = 2263.8\ KN$$

$$T_{ps} = A_{ps} * f_{ps} = 95 * 98.71 * 1658.31 * 10^{-3} = 15550.72\ KN$$

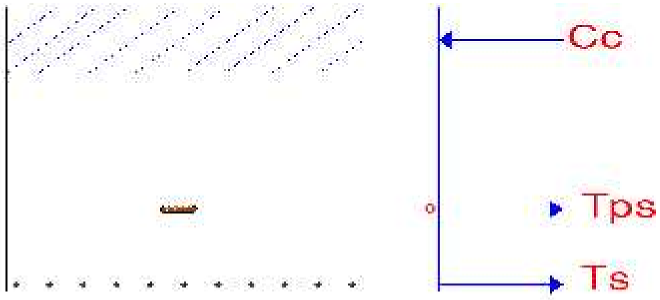


Figure 4. 22: Forces Acting Through PTBeam Section

So,

$$T_s + T_{ps} = C_c$$

$$2263.8 + 15550.72 = 0.85 * 35 * 1.5 * 10^3 * a \quad \Longrightarrow \quad a = 0.3992\ m = 39.92\ cm$$

$$\circlearrowleft M_o = M_n = 2263.8\ \ 1.6575 - 1.21\ \ + 0.85 * 35 * 1.5 * 0.3992 * \ \ 1.21 - \frac{0.3992}{2}\ \ * 10^3 = 19012.62\ KN.m$$

Check for strain:

$$C = \frac{a}{\beta_1} = \frac{39.92}{0.801} = 49.84\ cm$$

$$\varepsilon_t = \frac{(d-c)}{c} * 0.003 = \ \frac{165.75 - 49.84}{49.84}\ * 0.003 = 0.007 > 0.005$$

Tension control $\Rightarrow \emptyset = 0.9$

$\Rightarrow \emptyset M_n = 0.9 * 19012.62 = 17111.36\ KN.m > M_u = 13552.3\ KN.m \quad \Rightarrow$
$\Rightarrow okay$

Check for cracking moment:

$$M_{cr} = F_e * \left(e + \frac{S_b}{A_g}\right) + f_r * S_b$$

$f_r = 0.62\sqrt{f'_c} = 3.67\ MPa$

$$M_{cr} = 11271.75 * \left(0.36 + \frac{0.7225}{1.84}\right) + 3.67 * 10^3 * 0.7225 = 11135.4\ KN.m$$

$1.2M_{cr} = 13362.5\ KN.m \leq \emptyset M_n = 17111.36\ KN.m \qquad \Rightarrow okay$

4.5.2.4 Design for shear:

$$V_c = \left(0.05\sqrt{f'_c} + 4.8\frac{V_u d_p}{M_u}\right) b_w d$$

To satisfy this equation must:

1. $\frac{V_u d_p}{M_u} \leq 1 \Rightarrow at\ the\ face\ of\ the\ column\ is\ \frac{1895.5*1.21}{495.6} = 4.54 > 1$
   so take $\frac{V_u d_p}{M_u} = 1$
2. $d \geq 0.8h \Rightarrow 0.8 * 170 = 136\ cm$
   so take $d = 136\ cm$

So,

$$V_c = (0.05\sqrt{35} + 4.8 * 1)\ 1500 * 1360 * 10^{-3} = 10395.44\ KN$$

But, not exceed the following equation “The upper limit according to ACI”:

$$V_c = 0.42\sqrt{f'_c} * b_w * d = 5068.9\ KN < 10395.44\ KN$$

So, the control is:

$$V_c = 5068.9\ KN \quad \Rightarrow \quad \emptyset V_c = 0.75 * 5068.9 = 3801.675\ KN$$

$0.5\ \emptyset V_c = 1900.84\ KN$

$0.5\ \emptyset V_c < V_u < \emptyset V_c$ $\quad\Rightarrow$ The minimum shear reinforcement is required

According to ACI, the minimum reinforcement is the smallest of:

1. $\frac{A_v}{s} = \frac{b_w}{3f_y} \Rightarrow \frac{A_v}{s} = \frac{1500}{3*420} * 10^3 = 3571.43\frac{mm^2}{m}$
2. $\frac{A_v}{s} = \frac{A_{ps}*f_{pu}}{80f_y*d}\sqrt{\frac{d}{b_w}} \Rightarrow \frac{A_v}{s} = \frac{95*98.71*1860}{80*420*1360}\sqrt{\frac{1360}{1500}}\ 10^3 = 363.5\frac{mm^2}{m}$

So the minimum is $\frac{A_v}{s} = 363.5\frac{mm^2}{m}$

use Ø10/400 mm $(A_{s\ prov.} = \frac{2*78.54}{0.4} = 392.7\frac{mm^2}{m} > 363.5\frac{mm^2}{m})$

Check for long term deflection from ADAPT program:

100% of dead load and 0% of live load are sustained loads

Total long term deflection = ١٩.46mm $< \frac{l}{240} = \frac{19980}{240} = 83.25\ mm \rightarrow okay$

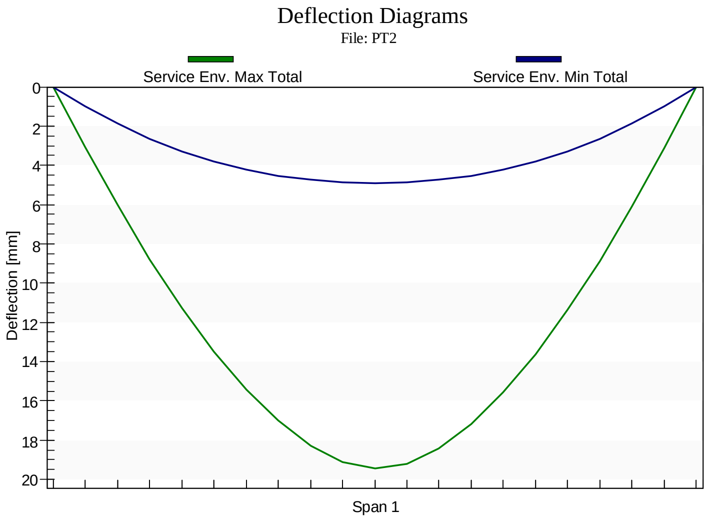


Figure 4. 23: Deflection Diagram for Beam (PTB2, F1).

4.6 Design of Staircase:

4.6.1 Minimum slab thickness:

Minimum slab thickness for deflection is (for simply supported one-way solid slab):

$h_{min} = L/20$

$h_{min} = 5.25/20 = 26.25$ cm

Minimum slab thickness for deflection is (for the slab end are cast with support):

$h_{min} = L/28$

$h_{min} = 5.25/28 = 18.75$ cm

Take h = 25 cm

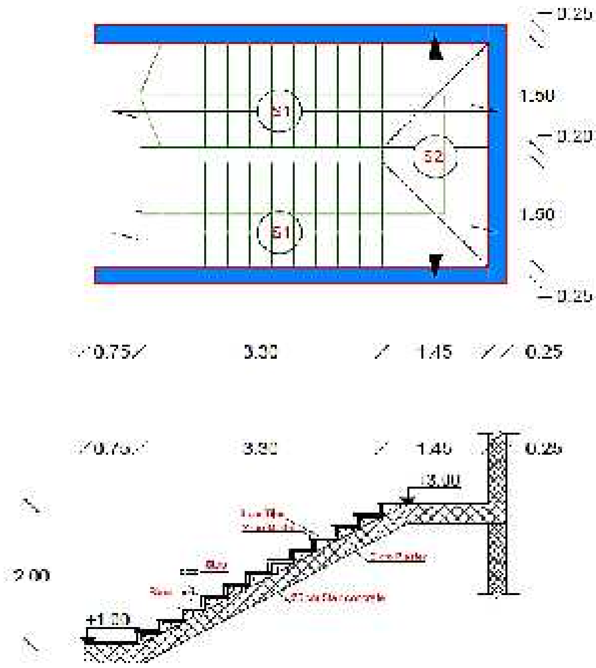


Figure 4. 24: Staircase Dimensions.

4.6.2 Load Calculation:

Load: the applied live loads are based on the plan area (horizontal) projection, while the dead load is based on the sloped length to transfer the dead load into horizontal projection.

The Stair Sloped by $\theta = \tan^{-1}(\text{riser / run})$

$= \tan^{-1}(16.7 / 30) = 29.1^{o}$

- Flight dead load computation:

Table 4. 6: Flight Dead Load Computation

Parts of Flight	Quality Density kN/m^3	Calculation of(W) kN/m
Tiles	**27**	27 * 0.03 * 1 * ((0.35+0.167)/0.3) = 1.34
Mortar	**22**	22 * 0.02 * 1 * ((0.3+0.167)/0.3) = 0.685
Stair Steps	**25**	(25/0.3) * ((0.167 * 0.3)/2)) *1 = 2.09
R.C Solid Slab	**25**	25 * 0.25 * 1 / (cos29.1^{o}) = 7.15
Plaster	**22**	22 * 0.03 * 1 /(cos29.1^{o}) = 0.76
Total Dead Load		**12.03 kN/m**

- Landing dead load computation:

Table 4. 7: Landing Dead Load Computation.

Parts of Flight	Quality Density kN/m^3	Calculation of(W) kN/m
Tiles	**٢٧**	27 * 0.03 * 1 * = 0.81
Mortar	**22**	22 * 0.02 * 1 * = 0.44
R.C Solid Slab	**25**	25 * 0.25 * 1 = 6.25
Plaster	**22**	22 * 0.03 * 1= 0.66
Total Dead Load		**8.16 kN/m**

- Live Load: LL = 4 kN/m^2.
- Total factored load: w = 1.2 DL + 1.6 LL

For flight w = 1.2 * 12.03 + 1.6 * 4 * 1 = 20.84 kN/m^2.
For landing w = 1.2 * 8.16 + 1.6 * 4 * 1 = 16.2 kN/m^2.

Because the load on landing, where the shear wall surrounding it, is carried into two directions, only half the load will be considered in each direction (16.2 / 2 = 8.1 kN/m).

4.6.3 Design of slab S1:

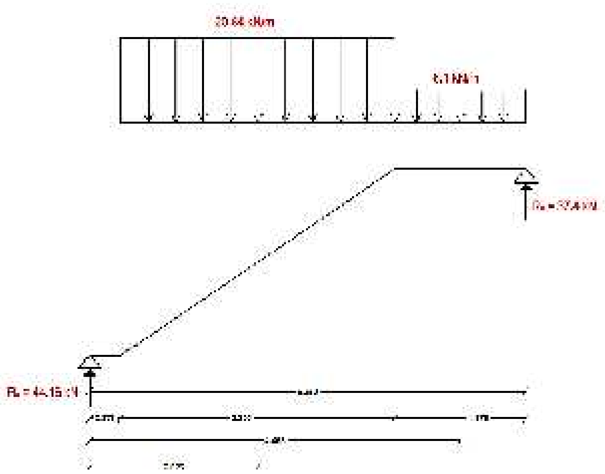


Figure 4. 25: Stair Loading.

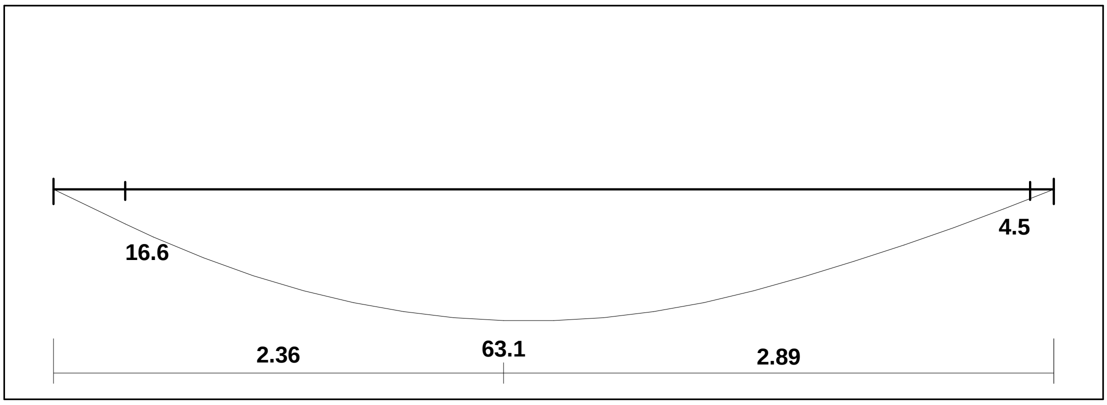


Figure 4. 26: Stair Bending Moment Diagram.

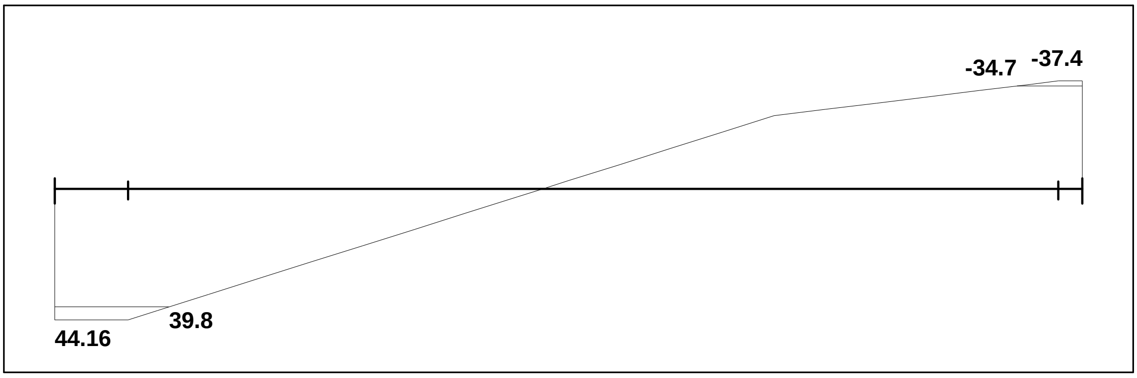


Figure 4. 27: Stair Shear Force Diagram.

4.6.3.1 Check for shear strength:

Assume initial bar diameter □14 $formainreinforccment$, Φ = 0.75 $fors□ear$

Assume bar diameter □14 for main reinforcement

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

$V_c = \frac{1}{6}\sqrt{fc'}\, b_w\, d == \frac{1}{6}\sqrt{24} * 1000 * 223 = 182.1$ kN.

Φ $V_{c\,=}$ 0.75* 182.1 = 136.6 kN

136.6 kN > Vu = 39.8 kN**,** the slab thickness is adequate enough.

4.6.3.2 Design the flight for bending moment:

Mu=51.5 kN.m

$R_n = \frac{M_u}{\phi b d^2} = \frac{63.1\times10^6}{0.9\times1000\times223^2} = 1.41\ Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times1.41}{420}}\right) = 0.0035$$

$A_{s,req} = \rho.b.d = 0.0035 \times 1000 \times 223 = 780.5\ mm^2/m$

$A_{s,min} = 0.0018*1000*250 = 450mm^2/m$

$A_{s,req} = 780.5\ mm^2 > A_{s,min} = 450mm^2/m$

Check for which maximum spacing is control:

1. S = 3h = 3*250 =750 mm

2. $S = 380*(\frac{280}{\frac{2}{3}*420}) - 2.5*20 = 330$ mm **OR** $300 * (\frac{280}{\frac{2}{3}*420}) = 300$ mm control

3. S = 450 mm

- ✓ S = 300 mm is control

Use □14 @ 150 mm, $A_{s,provided} = 1026.7\ mm^2 > A_{s,required} = 780.5\ mm^2$……………… Ok

Check for strain:-

$$a = \frac{A_s.f_y}{0.85b\ f_c'} = \frac{1026.7\times420}{0.85\times1000\times24} = 21.14\ mm$$

$$c = \frac{a}{B_1} = \frac{21.14}{0.85} = 24.87\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{223-24.87}{24.87}\right) = 0.024 > 0.005 \ldots\ldots\ldots\ldots\ldots .\boldsymbol{0k}$$

Temperature and shrinkage reinforcement:

$A_{s,req} = A_{s,min} = 0.0018*1000*250 = 450mm^2$

Use □10 @ 150mm, $A_{s,provided} = 520\ mm^2 > A_{s,required} = 450mm^2$…………………. Ok

4.6.4 Design of slab S2 (landing):

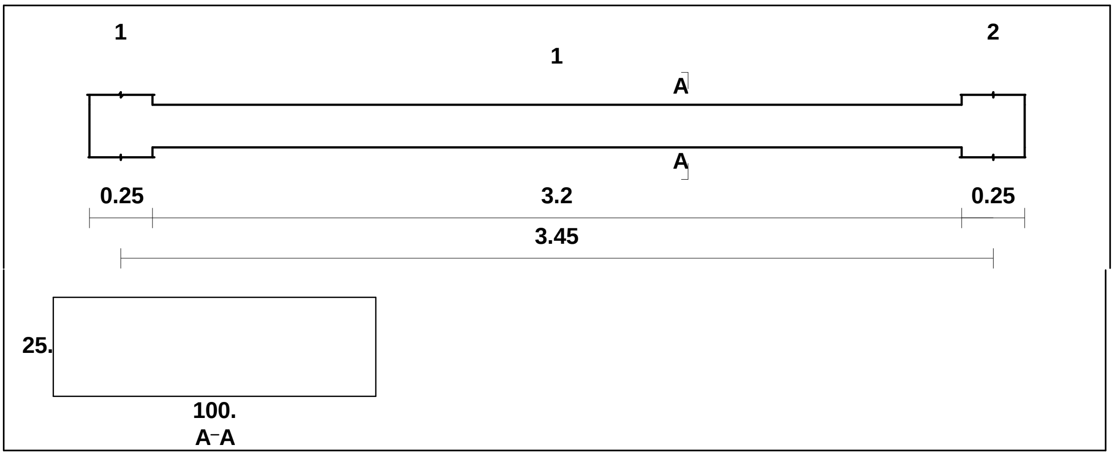


Figure 4. 28: Landing Bending Moment Diagram.

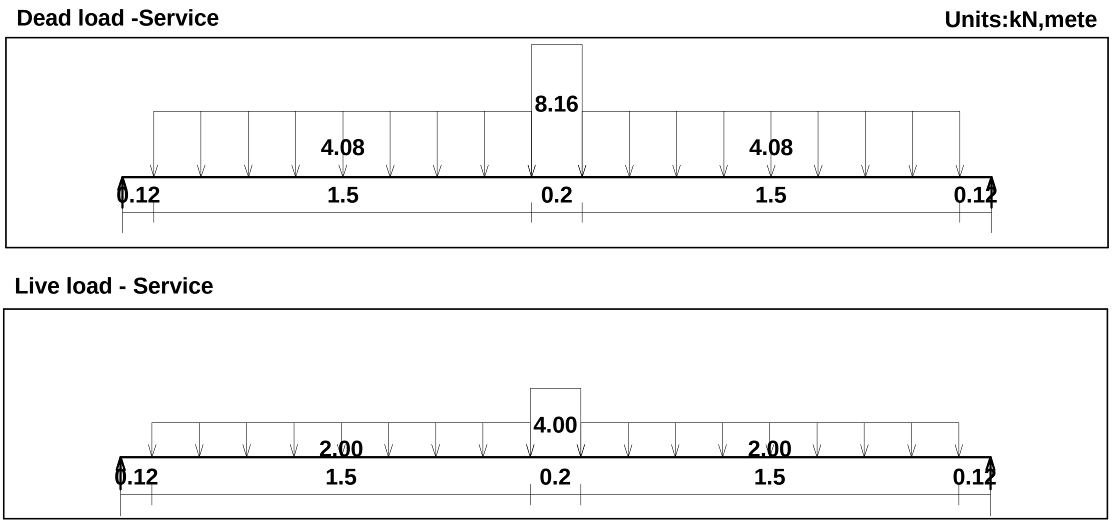


Figure 4. 29: Landing Loading System.

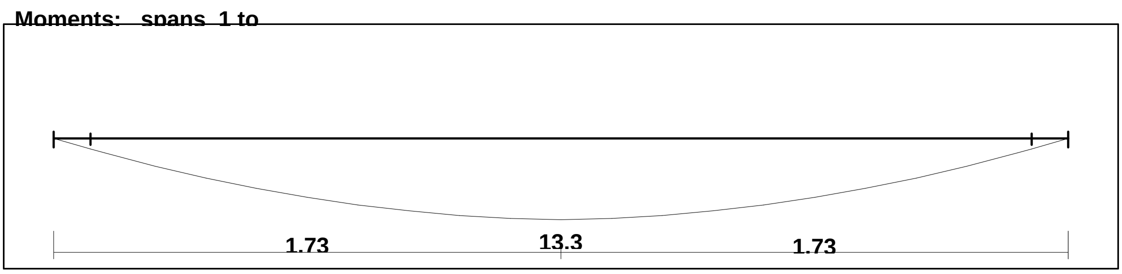


Figure 4. 30: Landing Bending Moment Diagram.

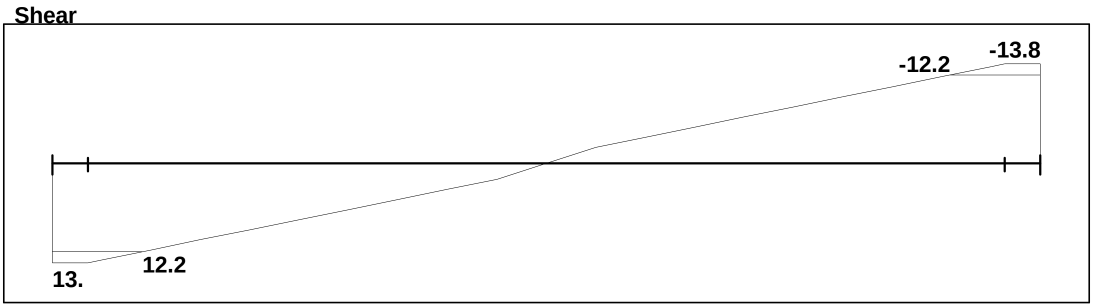


Figure 4. 31: Landing Shear Force Diagram.

4.6.4.1 Check for shear strength:

Assume bar diameter ø 14 for main reinforcement

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

$V_c = \frac{1}{6}\sqrt{fc'}\,b_w\,d == \frac{1}{6}\sqrt{24} * 1000 * 223 = 182.1$ kN

$\Phi * V_c = 0.75 * 182.1 = 136.6$kN

136.6kN > Vu = 12.2 kN**,** the slab thickness is adequate enough.

4.6.4.2 Design the landing for bending moment:

Mu=13.3 kN.m

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{13.3\times10^6}{0.9\times1000\times223^2} = 0.3\ Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.3}{420}}\right) = 0.00072$

$A_{s,req}$ = ρ.b.d = 0.00072 ×1000×223 = 160.5 mm^2/m

$A_{s,min}$= 0.0018*1000*250 = 450mm^2/m

$A_{s,req}$ = 160.5 mm^2< $A_{s,min}$=450mm^2/m, use $A_{s,min}$=450mm^2/m.

Check for which maximum spacing is control:

1. S = 3h = 3*250 =750 mm

2. S = 380*($\frac{280}{\frac{2}{3}*420}$) – 2.5*20 = 330 mm **OR** 300 * ($\frac{280}{\frac{2}{3}*420}$) =300 mm control

3. S = 450 mm

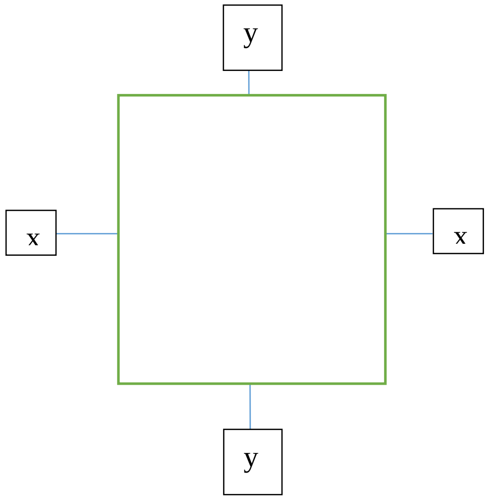
- ✓ S = 300 mm is control, use □10 @ 150 mm, $A_{s,provided}$= 520 mm^2 > $A_{s,required}$= 450 mm^2……………… Ok

Temperature and shrinkage reinforcement:

$A_{s,req}$= $A_{s,min}$ =0.0018*1000*250 = 450mm^2

Use 10 @ 150mm, $A_{s,provided}$= 520 mm^2 > $A_{s,required}$= 450mm^2 ………………….Ok

4.7 Design of Column:



Select column 92 from Group (H):

Assume column dimension of 45cm in x-direction

and 55cm in y-direction.

4.7.1 Load Calculation:

- Service Load:
  Dead Load =2300 kN.
  Live Load =760 kN.
- Factored Load:

  P_U = 1.2 ×2300+ 1.6×760 =3976 kN.

4.7.2 Check Slenderness Parameter:

$$\frac{klu}{r} < 34 - 12\frac{M1}{M2} \leq 40$$

Lu: Actual unsupported (Unbraced) length.

K: effective length factor. According to ACI 318-02 (10.10.6.3) the effective length factor k, shall be permitted to be taken as 1.0 for bracing system.

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

Lu = 3.0 - 0.52= 2.48 m

$(\frac{M1}{M1})$ = 1.0 braced frame with M_{min}.

K=1.0 - for columns in nonsway frames.

$$\frac{klu}{r} < 34 - 12 = 22 \leq 40$$

$$\frac{klu}{rx} = \frac{1.0 * 2.48}{0.3 * 0.45} = 18.37 \prec 22$$ - Short column for bending about x-axis.

$$\frac{klu}{ry} = \frac{1.0 * 2.48}{0.3 * 0.55} = 15.03 \prec 22$$ - Short column for bending about y-axis.

4.7.3 Design of the Short Column for longitudinal bars:

$$¤Pn = Pu = ¤ * ¤ '*(0.85 * fc'*(Ag - Ast) + Ast * fy)$$

$$3976 * 10 = 0.65 * 0.8 * (0.85 * 24 * (45 * 55 - Ast) + Ast * 420)$$

$$Ast = 65\ cm^2$$

$$\circ = \frac{Ast}{Ag} = \frac{65}{45 * 55} = 0.0262 \succ 0.01 \qquad ok$$

$$use\ 16 ¤ 25 \longrightarrow Ast = 78.56 \succ 65\ (ok)$$

4.7.4 Design of column Stirrups:

The spacing of ties shall not exceed the smallest of:

$$spacing \le 16 \times d_b = 16 \times 2.0 = 32\ cm$$
$$spacing \le 48 \times d_s = 48 \times 1.0 = 48\ cm$$
$$spacing \le least\ \dim = 35\ cm$$

Use ¤10 @ 20 *cm*

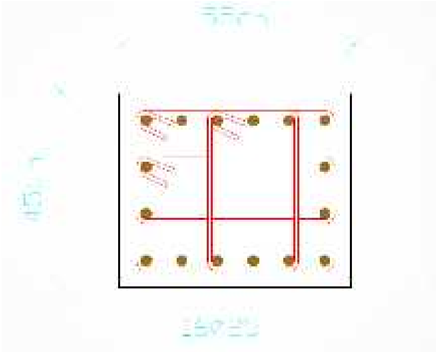


Figure 4. 32:Column Reinforcement Details.

4.8 Design of Shear Wall:

4.8.1 Shear wall general information:

- ⇒ Shear Wall Thickness h = 25 cm
- ⇒ Shear Wall Width Lw = 5.3 m
- ⇒ Shear Wall Height Hw = 4.0 m

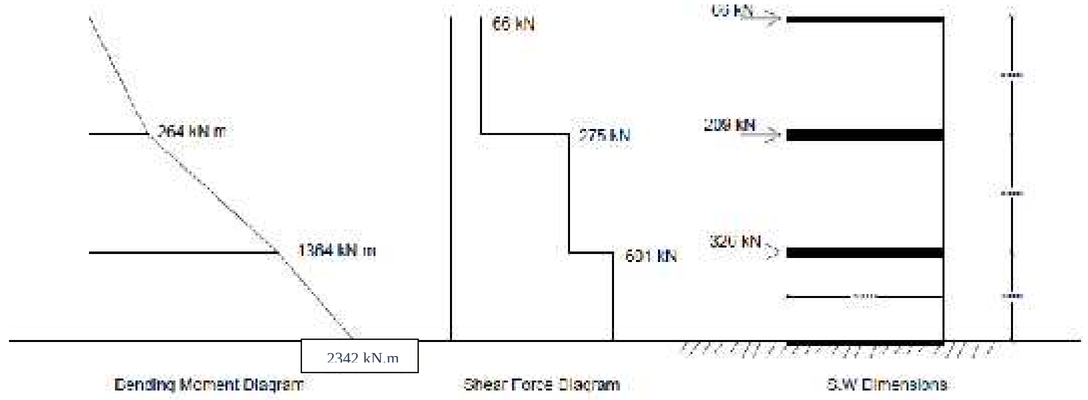


Figure 4. 33: Shear Wall Dimensions and Loads.

4.8.2 Check for shear strength:

4.8.2.1 Maximum shear strength permitted:

$\emptyset V_{nmax} = \emptyset \frac{5}{6}\sqrt{f_c'}hd$, d = 0.8$L_w$ = 0.8*5.3 = 4.24m.

$= 0.75 * 0.83 * \sqrt{24} * 250 * 4.24 = 3232.6\ kN >$ $V_{u,max}$ = 601kN.

4.8.2.2 Shear strength provided by concrete Vc:

Critical Section for shear is the smaller of:

$$\frac{lw}{2} = \frac{5.3}{2} = 2.65m - control$$

$$\frac{hw}{2} = \frac{11}{2} = 5.5m$$

Story height = 3m.

V_c is the smallest of :

$1 - V_c = \frac{1}{6}\sqrt{f_c'}hd = \frac{1}{6}\sqrt{24} * 250 * 4.24 = 865.5KN$ …….. Control

$$2 - V_c = 0.27\sqrt{f_c'}hd + \frac{N_u d}{4l_w} = 0.27\sqrt{24} * 250 * 4.24 + 0 = 1402.1KN$$

$$3-V_c = 0.05\ \sqrt{f_c'} + \frac{l_w\left(0.1\sqrt{f_c'}+0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u}-\frac{l_w}{2}}\ hd$$

$$\Longrightarrow M_u = 1364 + 326 * (3 - 2.65) = 1478.1 kN.m$$

$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{1478.1}{601} - \frac{5.3}{2} = -0.19$ <1, so the previous equation does not available to apply for shear calculations.

- Determine required horizontal shear reinforcement:

$Vu = 601 > \frac{1}{2} Vc = \frac{1}{2} * 0.75 * 865.5 = 324.6 kN$ …..reinforcement is required.

Shear reinforcement must be provided in accordance with 11.9.9.

Assume Ø10 for shear reinforcement.

$Vu \leq ØVn = Ø(Vc + Vs)$

$Vs = \frac{Vu}{Ø} - Vc = \frac{601}{0.75} - 865.5 = -64.2$, *use minimum ratio of* ρt = 0.0025

Minimum shear reinforcementis required,

Maximum spacing is the least of:

$$\frac{Lw}{5} = \frac{5300}{5} = 1060\ mm.$$

$$3h = 3 * 250 = 750\ mm.$$

$$450\ mm - control.$$

Select Ø10, two layers:

$$\rho t = \frac{Avh}{hS_2} = \frac{2 * 78.5}{200 * S_2} = 0.0025$$

So that S_2 = 314 mm < 450, Take Ø10@200mm in both sides.

- Determine vertical shear reinforcement:

$$\frac{hw}{Lw} = \frac{11.00}{5.3} = 2.075$$

$$\rho l = [0.0025 + 0.5\left(2.5 - \frac{h_w}{Lw}\right)(\rho t - 0.0025)] \geq 0.0025$$

$$= [0.0025 + 0.5(2.5 - 2.075)(0.0025 - 0.0025)] \geq 0.0025$$

So, ρl = 0.0025.

Maximum spacing is the least of:

$$\frac{Lw}{5} = \frac{5300}{5} = 1060\ mm.$$

$3h = 3 * 250 = 750\ mm.$

$450\ mm - control.$

Select Ø10 @ 200 mm.

4.8.3 Design of shear wall for flexure:

Uniformly distributed flexure reinforcement method was followed.

The uniformly distributed vertical reinforcement Ø10 @ 200 mm

$$A_{st} = \frac{5300}{200} * 2 * 78.5 = 4160.5\ mm^2$$

$$w = \frac{A_{st}}{L_w h}\frac{f_y}{f_c'} = \frac{4160.5}{5300 * 250}\frac{420}{24} = 0.055$$

$$\alpha = \frac{P_u}{l_w h f_c'} = 0$$

$$\frac{C}{l_w} = \frac{w + \alpha}{2w + 0.85\beta_1} = \frac{0.055 + 0}{2 * 0.055 + 0.85 * 0.85} = 0.066$$

$$\emptyset M_n = \emptyset\ \ 0.5 A_{st} f_y l_w (1 + \frac{P_u}{A_{st} f_y})(1 - \frac{c}{l_w})$$

$$= 0.9\ \ 0.5 * 4.160.5 * 420 * 5.300(1 + 0)(1 - 0.066)\ \ = 3892.5\ kN.\text{m} > 2342\ kN.m$$

Vertical reinforcement provided is enough.

4.9 Design of Basement wall and its Footing:

4.9.1 Basement wall design:

4.9.1.1 General information:

Height of the basement wall = 3.9 m.

Thickness of the basement wall is 30 cm.

Soil internal friction angle Φ = 30,density = 18 kN/m^3, and 12 kN/m^2 surcharge.

Loads acting on basement wall:

$q_1 = Earthpressuresoil$

$q_1 = \gamma_H * h * k_0$

$k_0 = 1 - sin\Phi = 1 - sin\,30 = 0.5$

$q_1 = 18 * 3.9 * 0.5 = 35.1$ kN/m^2

$q_2 = Surchargepressure = q_{\text{surcharge vertical}} * k_0$

$q_2 = 12 * 0.5 = 6$ kN/m^2.

❖ Designed the basement wall as pin-pin.

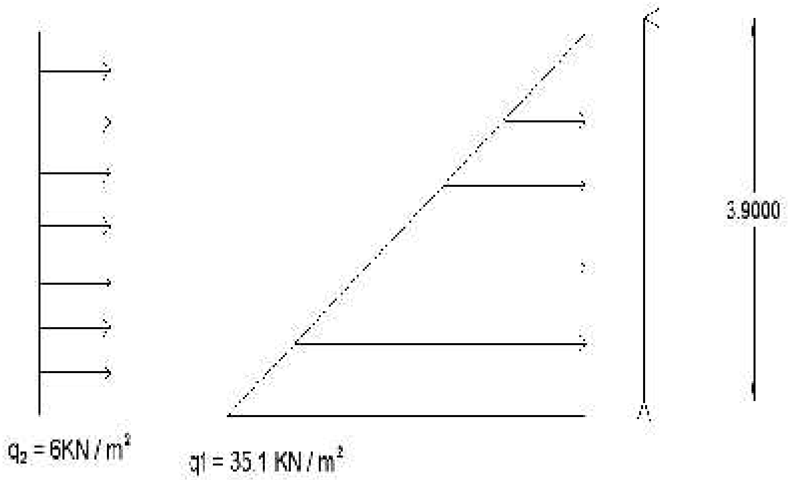


Figure 4. 34: Loads effects on Basement Wall.

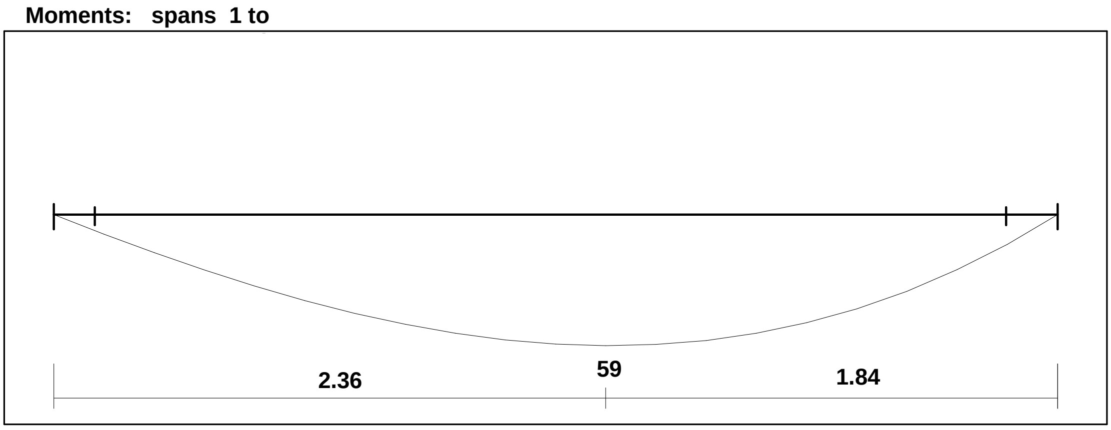


Figure 4. 35: Bending Moment Diagram of Basement Wall.

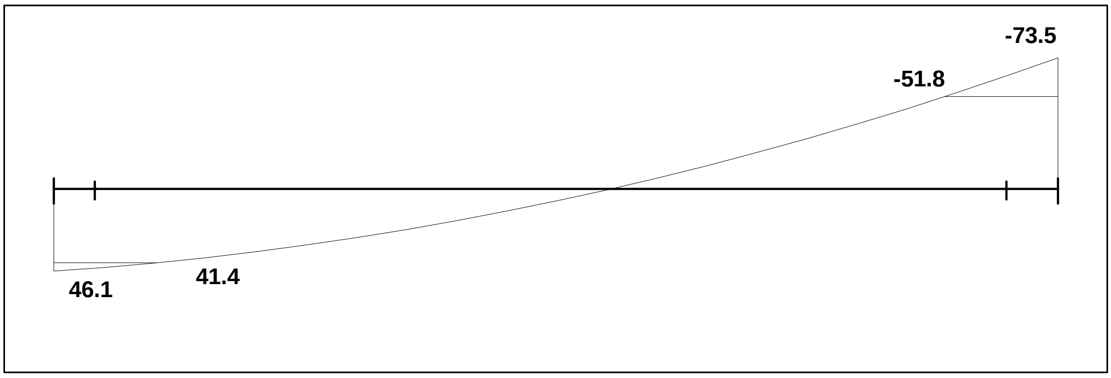


Figure 4. 36: Shear Force Diagram of Basement Wall.

4.9.1.2 Check basement wall for shear design:

From atirprogram $V_u = 51.8\ kN$, assume Ø16 for main reinforcement,

$$d = 300 - 20 - \frac{16}{2} = 272\ mm$$

$\emptyset V_c = 0.75 * 0.17 * \sqrt{24} * 1000 * 272 * 10^{-3} = 169.9\ Kn > V_u = 51.8\ KN \gg \gg okay$

4.9.1.3 Design for vertical reinforcement:

- ✓ Tension side:

$$M_u = 59\ kN.m$$

$$R_n = \frac{M_u}{0.9bd^2} = \frac{51.8 * 10^6}{0.9 * 1000 * 272^2} = 0.886$$

$$m = \frac{f_y}{0.85f'_c} = 20.6\ MPa$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = 0.0021 \quad \ggg\gg A_s = \rho bd = 5.73\ cm^2/m$$

$$A_{s,min} = \frac{0.25 * \sqrt{f'_c}}{f_y} bd = 7.93\ \frac{cm^2}{m} \quad OR \quad \frac{1.4}{f_y} bd = 9.1\ cm^2/m$$

$A_{s,min} = 9.1\ cm^2/m$ - is control

So, use $\emptyset 14/15cm$

- ✓ Compression side:

Use $A_{sv,min} = 0.0012bh = 0.0012 * 100 * 30 = 3.6\ cm^2/m$

So, use $\emptyset 10/20cm$

4.9.1.4 Design for horizontal reinforcement:

Use $A_{sh,min} = 0.002bh = 0.002 * 100 * 30 = 6\ cm^2/m$

For one side is $\frac{6}{2} = 3cm^2/m$

So, use $\emptyset 10/20cm$ for each side.

4.9.2 <u>Design of basement wall footing (strip footing):</u>

- The service loads form ETABS program:
  - Dead load “including self-weight”= 300 kN/m.
  - Live load = 90 kN/m.
- Soil density = 18 KN/m3
- Allowable soil Pressure = 500 KN/m2

4.9.2.1 Determination of q_{net}:

Assume footing thickness is 40 cm,

$q_{net} = 500 - (25 \times 0.4) - (18 \times 1.1) = 470.2\ KN/m^2$

$$A = \frac{total\ service\ loads}{q_{net}} = \frac{390}{470.2} = 0.83\ m^2$$

$$q_{net} = \frac{(1.2 \times 300) + (1.6 \times 90)}{1 \times 1} = 504\ KN/\ m^2$$

4.9.2.2 Check for one way shear:

$d = 400 - 75 - 9 = 316\ mm.$

$$\emptyset V_c = 0.75 \times 0.17 \times \sqrt{24} \times 1000 \times 316 \times 10^{-3} = 197.4kN$$

$$V_u = 504 \times \left(\frac{1 - 0.3}{2} - 0.316\right) = 17.136 < 197.136\ kN \ \ggg\gg\ okay$$

4.9.2.3 Design for main reinforcement:

$M_u = 504 \times 1 \times 0.35^2 * 0.5 = 30.87\ KN.m$

$R_n = 0.343\ MPa$

$m = 20.6$

$\rho = 0.000825 \rightarrow A_s = 2.61\ cm^2/m$

$A_{S\,min} = 0.0018\ bh = 5.7\ cm^2/m$

$A_{S\,min} = 5.7\ cm^2/m$ —control

So, use Ø 12/15 cm.

For shrinkage and temperature reinforcement, use $A_{S\,min} = 5.7\ cm^2/m$

So, use Ø 12/15 cm.

4.10 Design of Isolated Footing: (Footing Group 6)

Material:
Concrete B300 $Fc' = 24\ kN/m^2$
Reinforcement Steel $Fy = 420\ kN/m^2$

4.10.1 Load Calculations:

Dead Load = 1643 kN“included own weight”

Live Load = 496 kN

Total services load = 1643 + 496 = 2139 kN

Total Factored load = 1.2*1643 + 1.6*496 = 2765.2 kN

Column Dimensions (a*b) = 40*50 cm

Soil density = 18 Kg/cm3

Allowable Bearing Capacity = 500 KN/m2

Assume h = 60cm

$q_{net-allow} = 500 - 25 * 0.6 - 18 * 0.9 = 468.8\ KN/m^2$

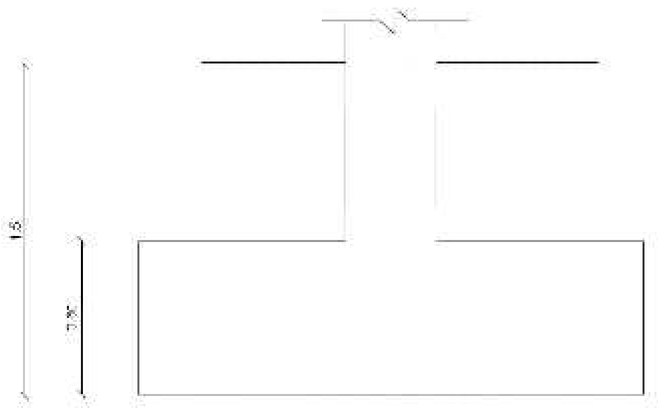


Figure 4. 37: Footing description.

4.10.2 Area of Footing:

$$A = \frac{P_{total\,service}}{q_{net-allow}} = \frac{2139}{468.8} = 4.563\ m^2$$

Assume Square Footing

b required = 2.14 m

Select b = 2.25 m

Bearing Pressure:

$$q_{u,net} = \frac{P_{total\,factored}}{A} = \frac{2765.2}{2.25 * 2.25} = 543.21\ KN/m^2$$

4.10.3 Design of Footing:

4.10.3.1 Design footing for one way shear:

Critical Section at distance (d) from face of column.

Assume h = 60cm, bar diameter Ø 12 for main reinforcement and 7.5 cm Cover.

d = 600 – 75 – 12 = 513 mm

$$Vu = q_u * \frac{B-a}{2} - d \quad * L$$

$$Vu = 543.21 * \frac{2.25-0.4}{2} - 0.513 \quad * 2.25$$

$$= 579.98\ KN$$

$$\emptyset V_c = 0.75 * 0.17 * \sqrt{f'_c} * b_w * d$$

$$\emptyset V_c = 0.75 * 0.17 * \sqrt{24} * 2250 * 513 * 10^{-3}$$
$$= 720.97\ KN$$

$\emptyset V_c > V_u$ »»»»»»» *okay*

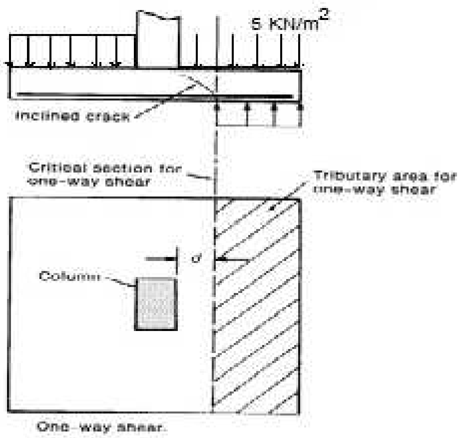


Figure 4. 38: One Way Shear of Footing.

4.10.3.2 Design Footing for two way shear:

$$V_u = P_u - FR_b$$

$$FR_b = q_{u,net} * area\ of\ critical\ section$$

$$V_u = 2765.2 - 543.21 * \quad 0.5 + 0.513 \quad * \quad 0.4 + 0.513 \quad = 2262.8\ KN$$

The punching shear strength is the smallest value of the following equations:-

$$\phi.V_c = \phi.\frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\, b_o d$$

$$\phi.V_c = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o / d} + 2\right)\sqrt{f_c'}\, b_o d$$

$$\phi.V_c = \phi.\frac{1}{3}\sqrt{f_c'}\, b_o d$$

Where:-

$$\beta_c = \frac{Column\ Length\ (a)}{Column\ Width\ (b)} \quad »» \quad \frac{50}{40} = 1.25$$

b_o = Perimeter of critical section taken at (d/2) from the loaded area

$$b_o = 2 * \quad 51.3 + 50 \quad + 2 * \quad 51.3 + 40 \quad = 385.2\ cm$$

α_s = 40 for interior column

$$\phi.V_C = \phi.\frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\, b_o d = 3146.25\ kN$$

$$\phi.V_C = \phi.\frac{1}{12}\left(\frac{\alpha_s}{b_o / d} + 2\right)\sqrt{f_c'}\, b_o d = 4433.24\ \text{kN}$$

$$\phi.V_C = \phi.\frac{1}{3}\sqrt{f_c'}\, b_o d = 2420.2\ kN$$

$\emptyset V_c = 2420.2\ KN > V_u = 2262.8\ kN$ »»»» *okay*

4.10.3.3 Design of Bending Moment:

Critical section at the face of column.

$$M_u = 543.21 * \frac{2.25}{2} * \left(\frac{2.25-0.4}{2}\right)^2 = 522.88\ KN.m$$

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{522.88\times10^6}{0.9\times2250\times513^2} = 0.98\ MPa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.6$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.R_n}{420}}\right) = \frac{1}{20.6}\left(1-\sqrt{1-\frac{2\times20.6\times0.98}{420}}\right) = 0.0024$$

$A_{s,req}$ = ρ.b.d = 0.0024×2250×513 = 27.61 cm^2

$A_{s,min}$ = 0.0018*2250*600 = 24.3 cm^2

$A_{s,req}$ **is control**

Check for Spacing:

S = 3h = 3*60 = 180cm

S = 380*($\frac{280}{\frac{2}{3}*420}$) – 2.5*75 = 192.5 cm

S = 45 cm **……… is control**

Use 24ø12 in Both Direction

4.10.3.4 Development length of steel reinforcement in footing:

- Tension development length in footing:

$$Ld_{T\,req} = \frac{9}{10} * \frac{F_y}{\lambda \sqrt{f_c}} * \frac{\psi_e \psi_s \psi_t}{\frac{ktr+cb}{db}} * db \geq 300\text{mm}$$

$$Ktr = 0 \quad Nostripes$$

$$cb = 75 + \frac{12}{2} = 81\ mm$$

$$Or\ cb = \frac{87.5}{2} = 43.75\ mis\ control$$

$$\frac{ktr+cb}{db} = \frac{0+43.75}{12} = 3.65 > 2.5$$

$$\frac{ktr+cb}{db} = 2.5$$

$$D = \frac{9}{10} * \frac{420}{1*\sqrt{24}} * \frac{1*1*0.8}{2.5} * 12 = 296.3\ mm < 300\text{mm}$$

So, $Ld_{T\,req} = 300\ mm$

$$Ld_{T\,available} \frac{2250 - 500}{2} - 75 = 800\ mm$$

$Ld_{T\,available} = 800\ mm > ld_{req} = 300\ mm$ »»»» $\boldsymbol{okay}$

- Compression development length in footing:

$$Ld_{c\,req} = \frac{0.24 * Fy * dB}{\sqrt{f'_c}} \geq 0.043 * \mathrm{f_y} * \mathrm{d}_B \ \geq 200\mathrm{mm}$$

$$Ld_{c\,req} = \frac{0.24 * 420 * 12}{\sqrt{24}} = 246.9\ mm \geq 0.043 * 420 * 12 = 216.72\mathrm{mm} \geq 200\mathrm{mm}$$

$Ld_{c\,req} = 246.9mm$

$Ld_{c\,ava} = 600 - 75 - 12 - 12 = 501mm > Ld_{c\,req}$ »»»» $okay$

- Lap splice of dowels in column:

$L_{sc} = 0.071 * f_y * d_B = 357.8\ mm > 300mm$

Select $L_{sc} = 40\ cm$

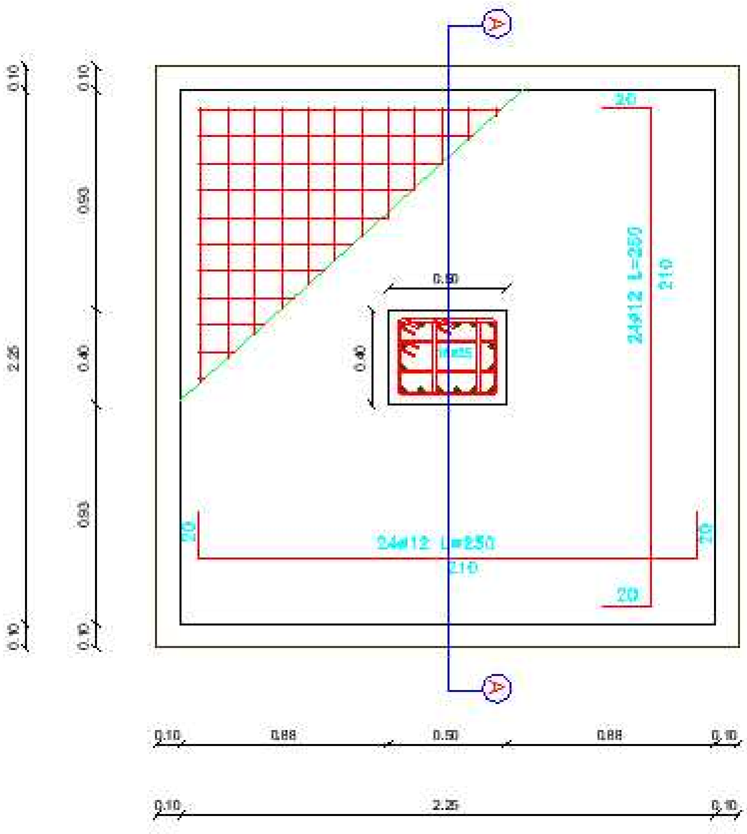


Figure 4. 39: Plan with Reinforcement of the Footing.

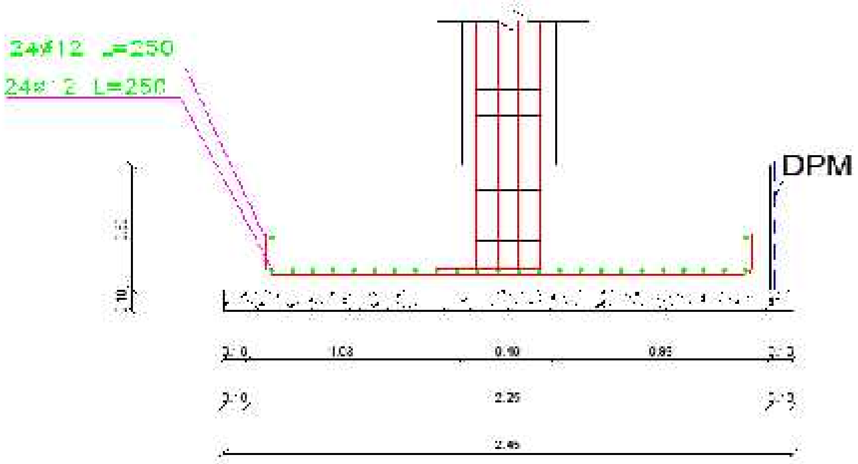


Figure 4. 40: Section in the Footing.

Chapter Five

Results, Recommendations and References

5

5.1Results:

1. Each student or structural designer should be able to design manually, so hecan get the experience and knowledge in using the computer software.
2. One of the factors that must be taken 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. 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.
5. The useful software programs were used:
    - AutoCAD 2014, to draw the detail of drawings for structural drawings.
    - ATIR, Etabs, Safe, Sp column, Adapt to analysis and design the structural members.

5.2 Recommendations:

This project has an important role in widening and enhancing the understanding to the nature of the structural project including all the details, analysis, and designs. It is very helpful-through this experience-to introduce a group of recommendations. 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 is essential at this stage to have information about the project site, the soil, the soil strength capacity at the site from the geotechnical report, after that the bearing walls and the columns is going to be set up alongside the architectural team in a compatible manner. The civil engineer tries at this stage to plant as much as possible the reinforced concrete walls, which should be use after that in resisting the earthquake loads and other lateral loads.

5.3 References:

1. American concrete institute, Building code requirements for structural concrete ACI 318M, 2011.
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4. Post-tensioning institute, Post-tensioning Manual, Sixth Edition, First Printing, 2006.
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