Palestine Polytechnic University

College of Engineering

Civil Engineering Department

"The Seismic Design of Jerico's Five Star Hotel Using Duel System and Intermediate resisting frames"

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Submitted to the College of Engineering

In partial fulfillment of the requirements for the

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The undersigned hereby certify that they have read, examined, and recommended to the Department of Civil Engineering in the College of Engineering at Palestine Polytechnic University the approval of a project entitled: **The Seismic Design of a Five Star Hotel In Jericho Using Duel System and Intermediate resisting frames**, submitted by Marwa Ibrahim Hmoud and Lina Hisham Sbeih for partial fulfillment of the requirements for the bachelor's degree.

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DEDICATION

This project is lovingly dedicated to our parents, Ibrahim and Suhad Hmoud, Hisham Sbeih and Elham Fanasheh who have been our constant source of inspiration.

They have given us the drive and discipline to tackle on task with enthusiast and determination. Without their love and support this project would not have been made possible.

Acknowledgment

We would like to express our deepest gratitude to our university – Palestine Polytechnic University – and Civil Engineering Department for all the knowledge and opportunities that help us to improve ourselves and became a better version of ourselves. We would also to extend our deepest appreciation to our supervisor Dr. Haitham Ayyad who was the best mentor who did not hesitate to help us, without him this project will not be the same.

We cannot begin to express our thanks to Dr. Ghassan Al-Dweik, who gave us the architecture plans of this project and all parts of the project. And answer all our questions that include architectural philosophy. A special thanks to Dr. Ghadi Zakarneh who helped us to write this project to accommodate the scientific research requirements.

Abstract

The aim of our project just like any other structural engineering project, is to implement a structural system to assure the building will be adequate for human use, but for our project is to keep the building standing against extreme seismic loads. Our building is designed using dual system which consists of and intermediate moment resisting frames and special shear walls which are capable of resisting lateral and gravity loads.

The main system which was adapted is dual system which consists of frames and shear walls. The loads are calculated using the ASCE 2016 code which are lateral loads (wind and seismic) and gravity loads (live and Dead and snow). The first step was to distribute the columns in satisfaction with the architectural plan and determine the structural system, second is to decide the load, third is to model the project using Etabs to make sure that the structural system is capable of resisting the lateral loads and having an allowable drift story, make sure the shear walls don't take up more than 75% of the lateral loads, design all the structural members achieving the provisions of ACI 318-14 chapter 18 and finally assure to details every segment (beam, column, or frame) in the range of restrictions allowed.

The outcome of the project in its final form after all the changes will satisfy all the requirements of the ACI 318-14 code and will match the load provisions of ASCE7- 16 code which will eventually end up into story drifts within the allowable arrange. Besides, the building will stay sustainable for human use after extreme earthquake events.

الملخص:

الهدف الرئيسي من هذا المشروع وكأي مشروع انشائي اخر يهدف الى ايجاد نظام انشائي ملائم يضمن استمرارية المبنى باقل التكاليف ولا يشكل عائق امام استخدام الامثل للمبنى . بالإضافة الى ذلك قمنا بالتركيز على مقاومة المبنى للأحمال الرأسية وخاصة احمال الزلازل وذلك ليبقى المبنى قائم وصامد خلال اخطر الموجات الزلزالية التي ممكن ان تحدث في المنطقة , ومن أجل ذلك قمنا باستخدام النظام الانشائي الثنائي والذي يتكون من جدران القص و ونظام الإطارات والقادرة على مقامة الأحمال الرأسية والأفقية .

لقد تم حساب الاحمال باستخدام الكود الأردني للأحمال و الكود الأمريكي ASCE 2016 , ومن ضمن هذه الأحمال التي تم اخذها بعين الاعتبار في التصميم هي الأحمال الأفقية من احمال رياح وزلازل وأحمال رأسية من الوزن الذاتي للمنشأة والأحمال الحية واحمال الثلوج , اول خطوة قمنا بها في المشروع هو توزيع الأعمدة وجدران القص بصورة لا تتعارض مع المخططات المعمارية وبالصورة المثلى لإيجاد افضل نظام انشائي ممكن , بعد ذلك قمنا بحساب جميع الأحمال الممكن حدوثها , ثم قمنا بإدخال نموذج المشروع الى برنامج الايتابس لتأكد من قدرة النظام الإنشائي لمقاومة الأحمال الأفقية والرأسية وأن الإزاحة الأفقية لطوابق ضمن الحدود المسموح بها حسب متطلبات الكود الأمريكي ACI 318 . في النهاية قمنا بتصميم جميع العناصر الانشائية وتحضير المخططات التفصيلية لها .

المخرجات النهائية من هذا المشروع هو الوصول الى تصميم انشائي امن و فعال ومتوافق مع الكود الأمريكي أخذين بعين الاعتبار جميع الأحمال التي يمكن حدوثها .

Table of Contents

List of figures:

List of tables:

Chapter 1: Introduction

1.1 Background

Structural buildings components are specialized structural members specifically designed, engineered and implemented under controlled conditions for a special architectural plan. They are incorporated into the overall building structure by a building engineer to satisfy the safety requirements. The material used can be wood, steel or reinforced concrete. For our project the material is reinforced concrete especially designed for the system which consists of Shear walls and frames designed to resist gravity loads and Lateral loads specifically seismic loads.

1.2 Aims and objectives

This Project was chosen to achieve the following goals:

❖ Correlate the theory that has been gained in the design courses with practical life.

❖ Increase the ability to choose a suitable structural system of elements that meets design requirements.

❖ Get experience in dealing with different problems encountered in the design process.

❖ Practice the structural analysis and design programs as well as theoretical knowledge.

1.3 Problem statement

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

1.4 Literature review

The main topic of our project is to design a structural system that meets the requirements of the architect and assure it will be strong enough to carry the gravity loads and the lateral loads especially seismic load. The importance of our project resins in preparing to the future worst event of earthquakes which results in saving the lives of the resident people in it because obviously the most precious thing we have is the human life. In addition to making the structure sustainable after any earthquake event.

A related literature was executed by a civil engineering undergraduate in Birzeit University, which was titled “seismic analysis: comparison Between Code Based and Performance-Based Analyses”.

The Main aim of this literature was to give a clear and straight-to-the-point contrast between the main three analysis methods; time history. Dynamic response spectrum analysis, Static linear analysis. It demonstrated the results within Palestine geographic region which is the main similarity between this literature and ours. Besides it concluded that the dynamic response spectrum analysis was the most conservative method to approach. This conclusion gave us a step ahead in recommending the best way to analyze the structural system will be designed.

1.5 Methodology

- The first step in our project is to study the project from an architect perspective.
- The second step is to distribute the columns and shear walls satisfactory to the architectural plan making sure no columns will be an obstacle against the human use. Then determine the suitable structure system.
- The third step is to calculate the loads starting with a possible gravity loads which we include the self-weight of members, cladding, super dead and live loads finishing with the lateral loads (wind and earth quake) specified for the location of the building and soil properties.
- The third step is to analyze and design each structural member in the building including beams Columns and shear walls.
- The final step is to detail every member achieving every requirement of the ACI 318-14 code.

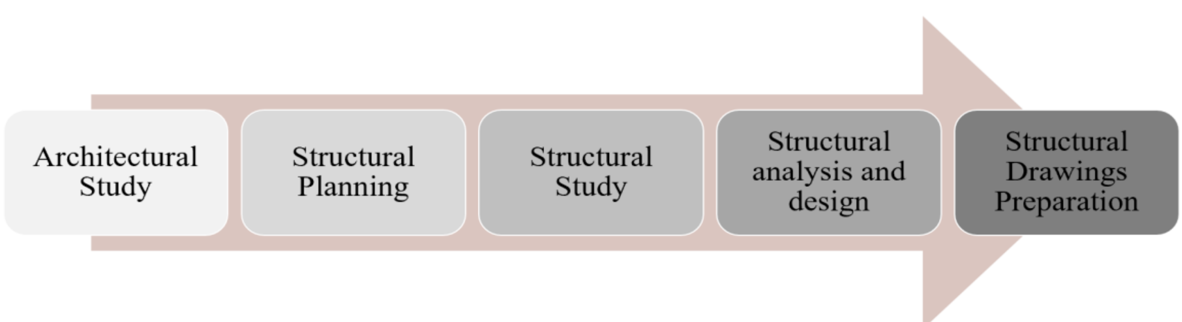

Figure 1-1: steps of implementing the project

1.6 Project scope

This Project contains the following Chapters:

CHAPTER 1: A general introduction: We talked about our objectives, problem and our methodology that we followed through this project.

CHAPTER 2: An architectural description of the project: we talked about the floors plans and the site and the elevations of this project.

CHAPTER 3: A general description of the structural elements: we talked about the structure system in general and our system in particle.

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

CHAPTER 5: Results and Recommendations.

1.7 Time plan

Figure 1-2 shows the project plan and timeline:

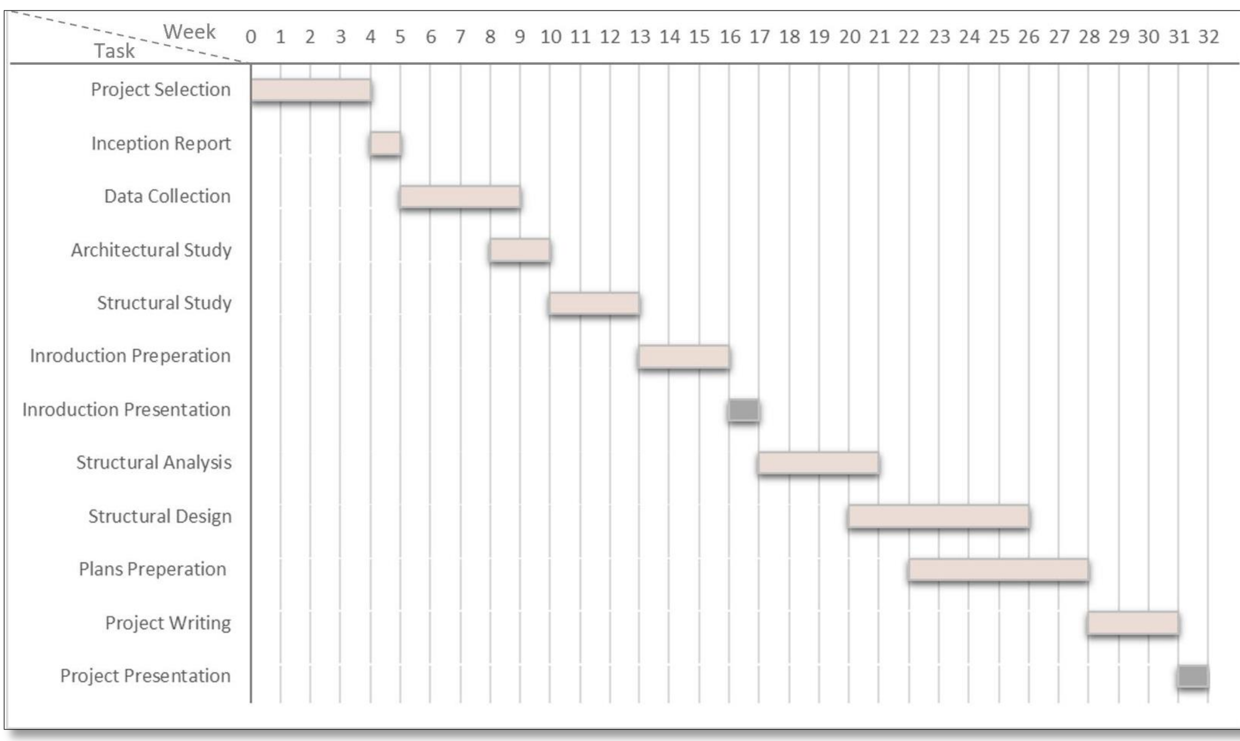

Figure 1-2: Time plan of the project

1.8 Programs used in the project

There are several computer programs used in this project:

1- Microsoft office: this set of programs including (word and excel) were used in various parts of the project such as text writing, formatting, fast calculations and project output

2- AUTOCAD: for detailed drawings of structural members of various types and reinforcement (shear walls, frames, footings)

3- Beamd: calculating the deflection of the ribs and determination of the most suitable thickness of the slab.

4- SPcolumn: design of columns with extruding their section and reinforcement details.

5- Etabs18: design and analysis of structural elements especially for shear walls.

6- Safe16: design of combined and mat footing besides checking the slab deflection on the long term and making sure it is in the ACI-318 limit.

Chapter 2: Architectural Description

2.1 Introduction

The process of bringing any building to light is integrative between several engineering specializations and the design process of each one of these takes time through several stages until the whole building is implemented.

The first step in this specific process starts of the architecture design stage, this stage defines the main shape of the building with pointing out the main function and requirements for which will be shaped out later, then a process of zoning takes place where the distribution of the facilities is determined, follows the deciding of each facility required space and dimensions with taking in consideration the studying of lighting, ventilation, movement, mobility and functional requirements, all of that to make sure the easiness and comfort of the users in the building.

Still, the architectural study must continue to verify that the design is easy to handle and how it will function properly with each other and the movement between these different parts. Yet, it also should make sure is to give a clear view of the project so it will be possible to distribute columns and other structural elements without intersecting with the architectural design, which will eventually lead to a whole efficiency of the building to be built.

2.2. General Identification of the project

The proposed project is a hotel with 9 floors one of them is a gym, the rest is a typical hotel with 25 double rooms and 10 suites. The building is proposed to be built on 850 square meters land. This area is considered sufficient for the construction of such a project, The architect showed her proficiency in design as she was able to use the space to design a building that meets the standards and provides comfort for its residents taking into account the architectural beauty in the overall design.

2.3. General Site description

The proposed project is located in Jericho. In a residential area with good infrastructure of roads, it is easily accessible location with available needed services such as electricity and communication links, besides it is located in near one of the most visited city by tourists which show the sufficient need of this project. The project is built on contoured land, which made the architect choose the function of the building based on the specific characteristics of the site.

2.4. Floors Description

The project contains four types of floors: second basement floor, First basement floor, ground floor, first floor and second floor which is repeated in 5 floors. The total area is 16544.41 m^2. The following is a brief description of each floor.

2.4.1 Second Basement Floor

The basement floor is level -7.20 m below the level of the main street with an area of 1636.94 m^2. It can accommodate 24 parking spot, one generator room with an area of 68.2 m^2, two storage rooms with an area of 50 m^2 both and water tank room with an area of 30 m^2. The entrance and the exit of the basement is from northeast side of the building and the which is clearly shown in the figure 2-3

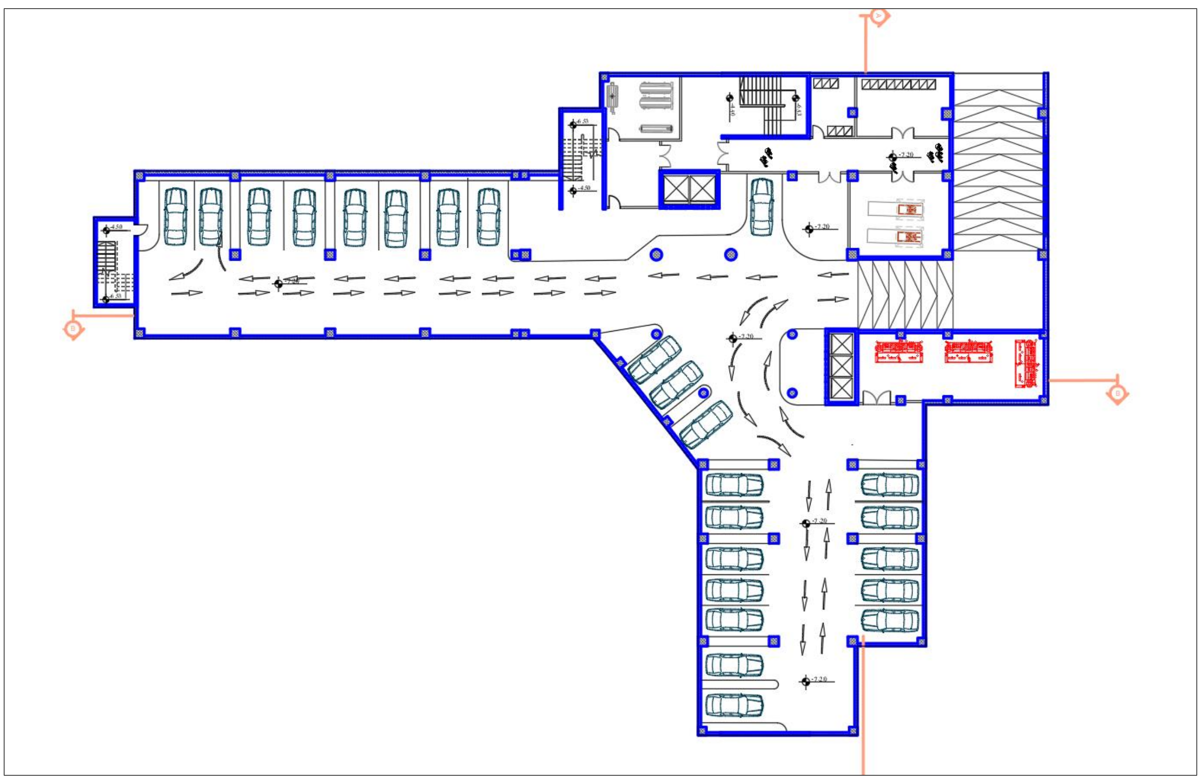

Figure2-1: second basement floor plan

2.4.2 First Basement Floor

The level of second basement floor is -4.50 m below the level of the main street with an area of 1678.95 $m^{2;}$ it mainly consists of the gym of the hotel with an entrance from the north eastern side.

The first basement floor mainly consists of two storage rooms, one laundry room, two cloth washer rooms, restaurant kitchen, a gym of an area of 400 m^2, shower rooms in addition to a secretary with an inside meeting room and an outdoor terrace as shown in figure 2-4.

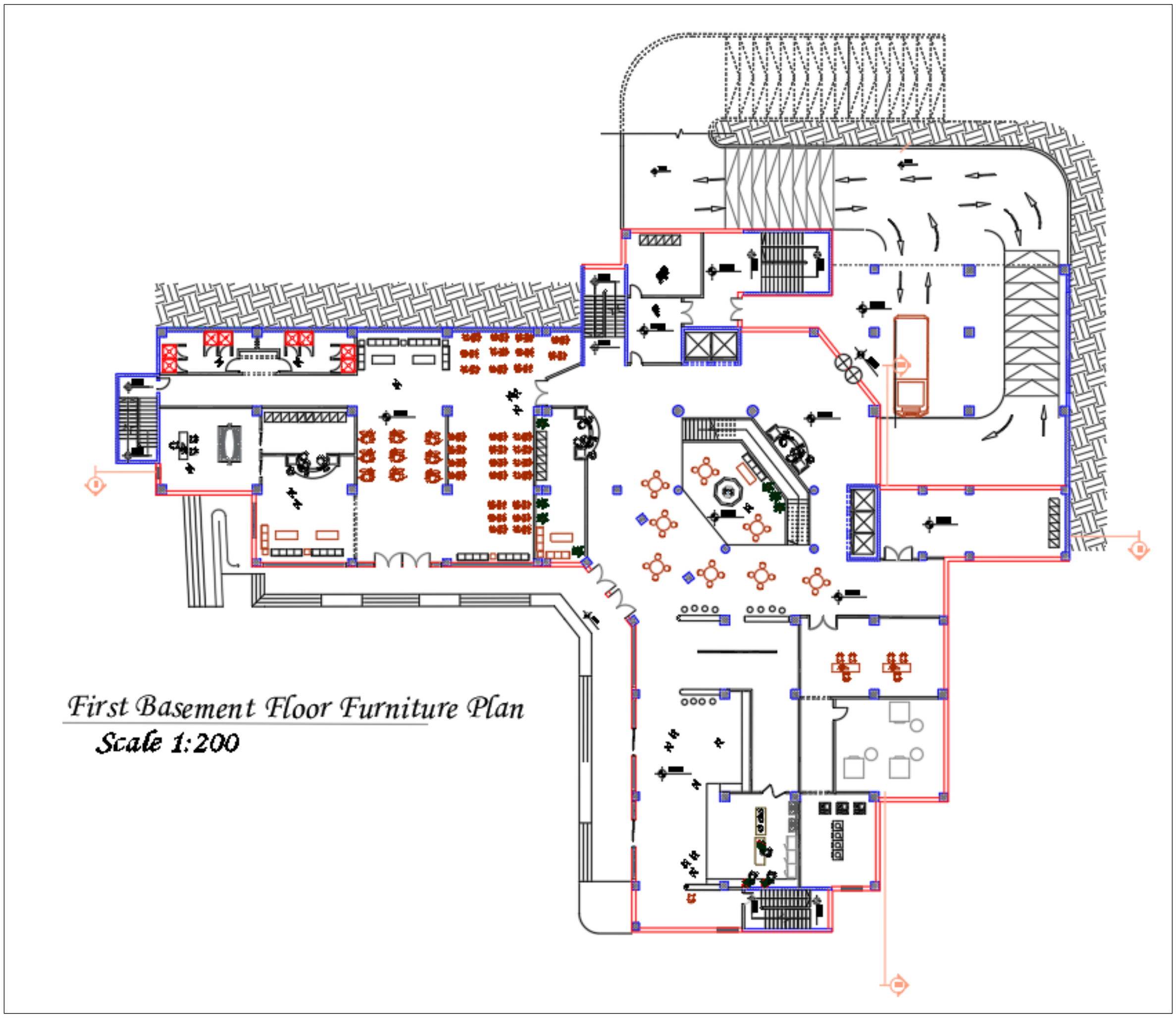

Figure2-2: first basement floor

2.4.3 Ground floor

The ground floor is the floor facing the street with an elevation of 0.00 with a total area of 1438.27 m^2. It mainly consists of the main reception of the hotel. It has two entrances, one is from the north east which is same as street entrance and the second one is from the east side.

It mainly consists of 11 office rooms, 6 bathrooms, 5 elevators, 2 stairs inside of the building and 3 emergency stairs distributed in the east, west and southern side of the hotel.as shown in the figure (2-3).

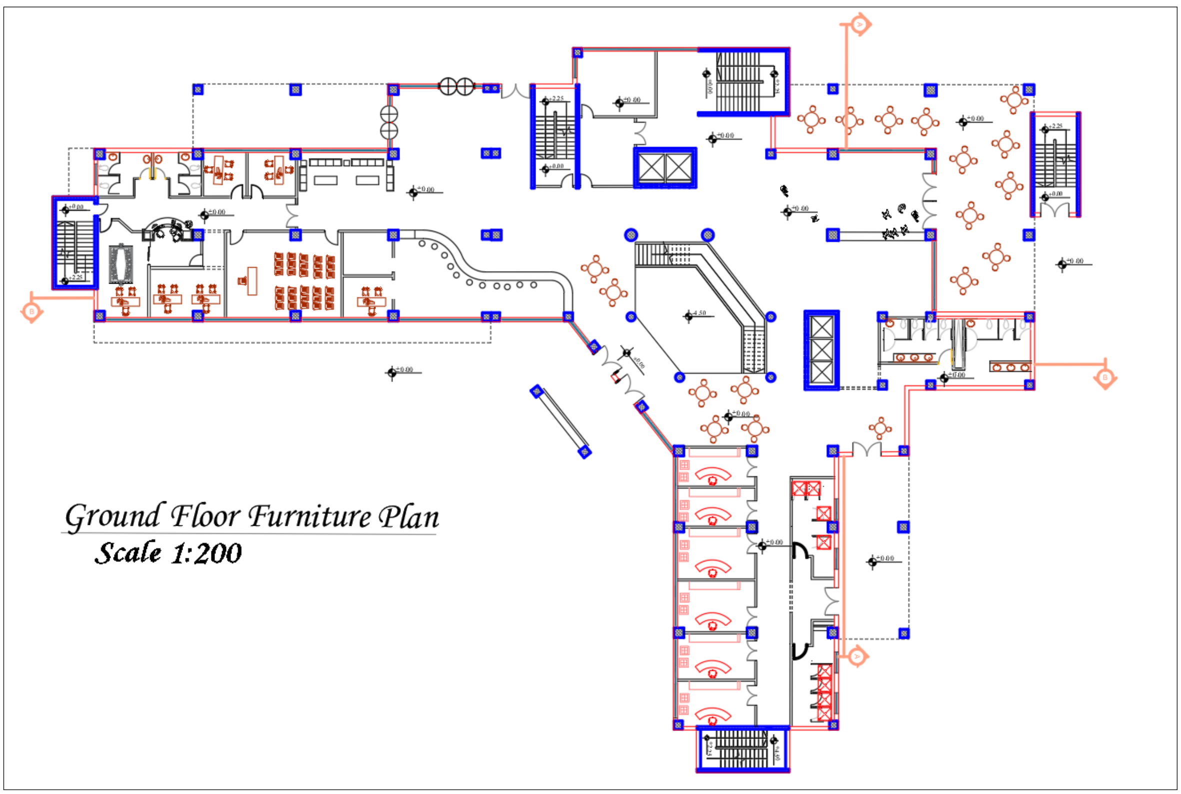

Figure2 -3: ground floor

2.4.3. First Floor

The first floor has a level of 4.5 m and has the main reception of the residents with the total area of 1885.1 m^2. Other than the reception area, it consists of a restaurant with a whole kitchen, two refrigerator rooms, two staff bathroom, one suite, three storage room and 17 apartments of a double bed, a bathroom and a terrace. As shown in the figure.

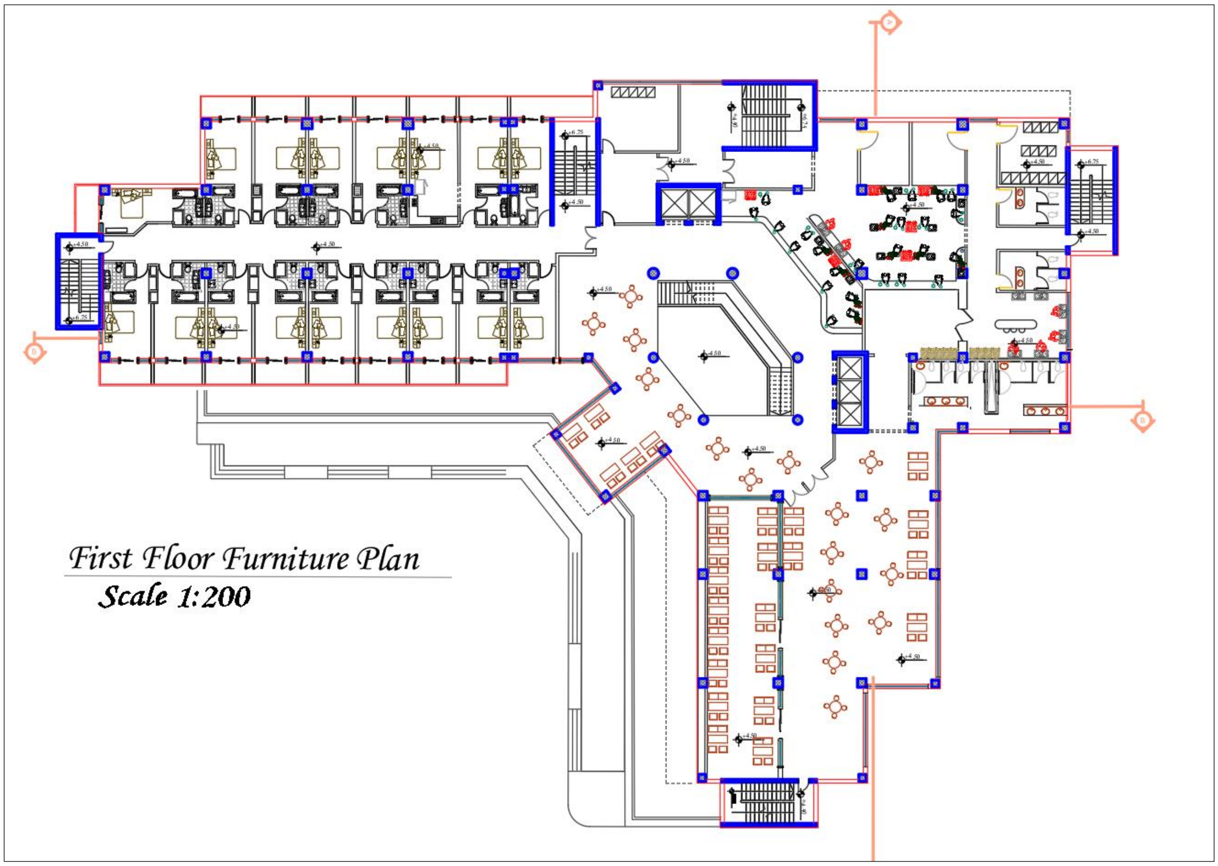

Figure2-4: first floor plan

2.4.3. Second Floor

The second floor is the repeated floor five times, with an area of 1981.03 m^2, which has thirty six double rooms contained a bathroom and a terrace with an area of 25 m^2 each as shown in the figure 2-6.

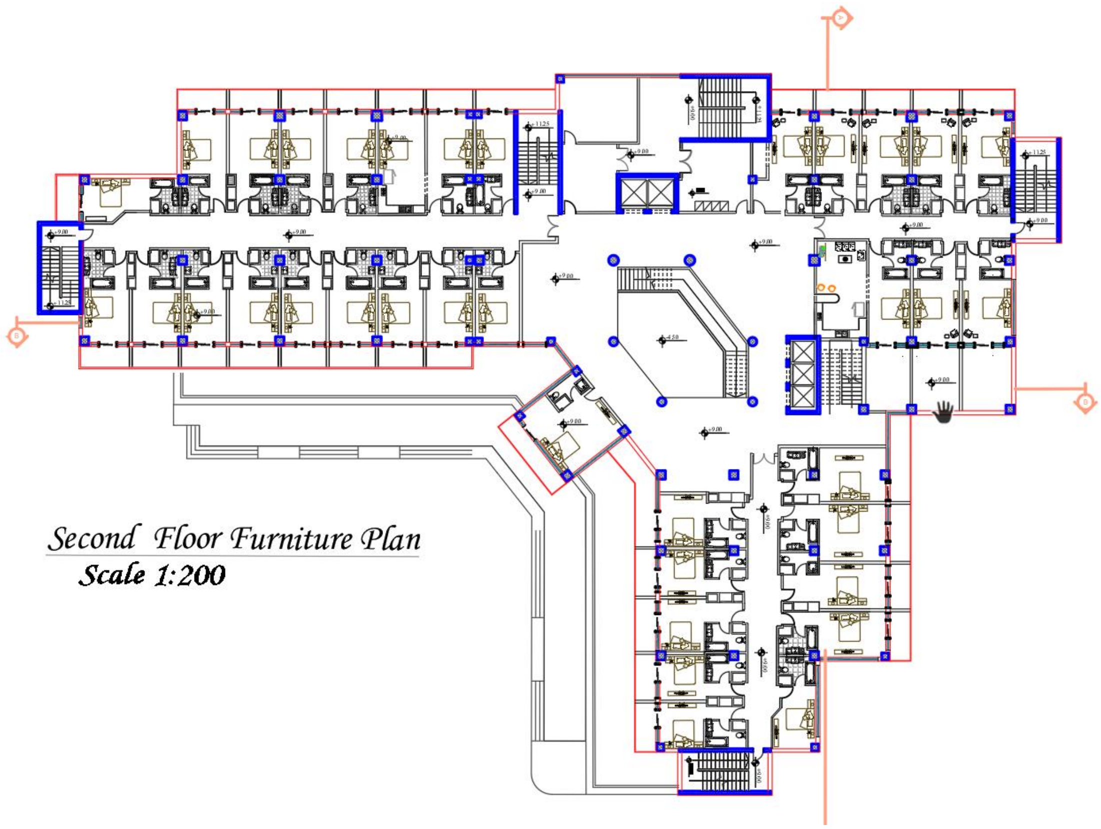

Figure2-5: Second Floor plan

2.5. Movement Areas

The proposed hotel has five elevators and six stairs, three of them are emergency stairs distributed in the western side, eastern side and southern side which makes it fast to empty the building whenever any occurrence of any interruption in the future. Meanwhile, the elevators are located nearly in the middle of the building to make sure it will be efficient to serve as much customers requested. In addition to the location of the stairs which makes it easy to be used by the staff. This eventually leads to a better service of the whole hotel. Not to mention the stair that connects ground floor and the first floor and gives a twist in the appearance of the floor.

2.6 Elevations Descriptions

2.6.1. North-West elevation

The North West elevation is the main elevation which appears from the main street, has the main entrance of the building and shows the integrity of the glass and the stone, the most of the balconies are displayed from this direction as shows in the figure 2-7.

Figure2- 6: North-west elevation

2.6.2 Western elevations

The western elevation shows the second entrance of the building in the lower elevation of the land, besides it shows the emergency stair in the right. Not to mention the integrity of the stone and the glass as shown in the figure 2-8.

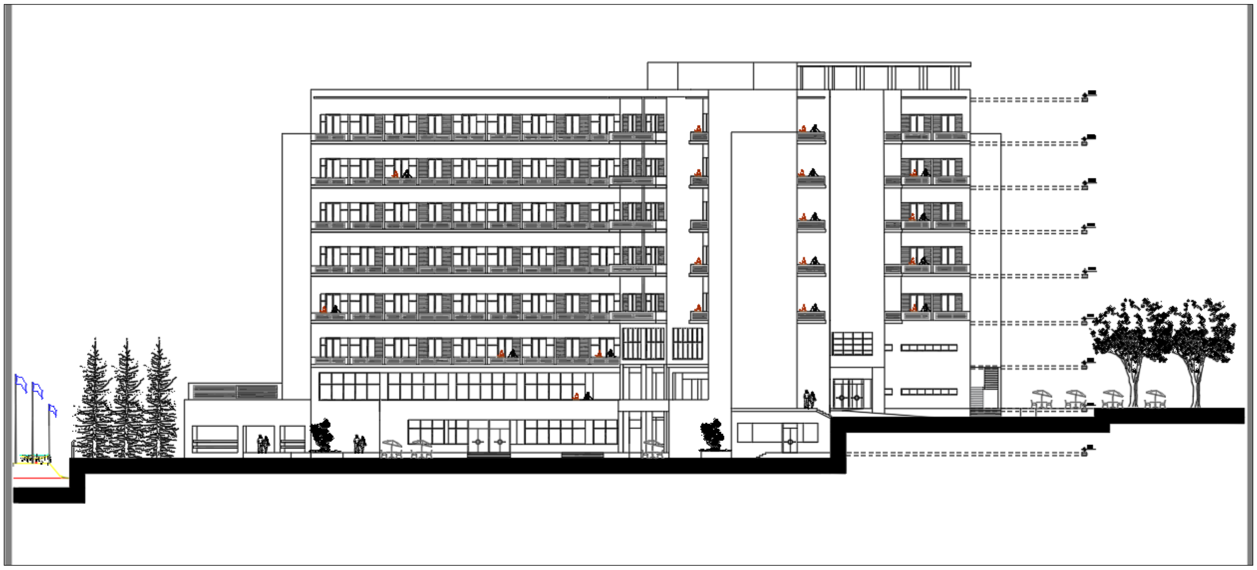

Figure2- 7: Western Elevation

2.6. Sections of the building

The sections of the building are made in order to show extra and necessary details of the hotel to highlight the movement and the distribution of the spaces and masses along the height of the hotel. as shown in the figure (2-8) and the figure (2-9).

Figure2- 8: Section A-A of the building

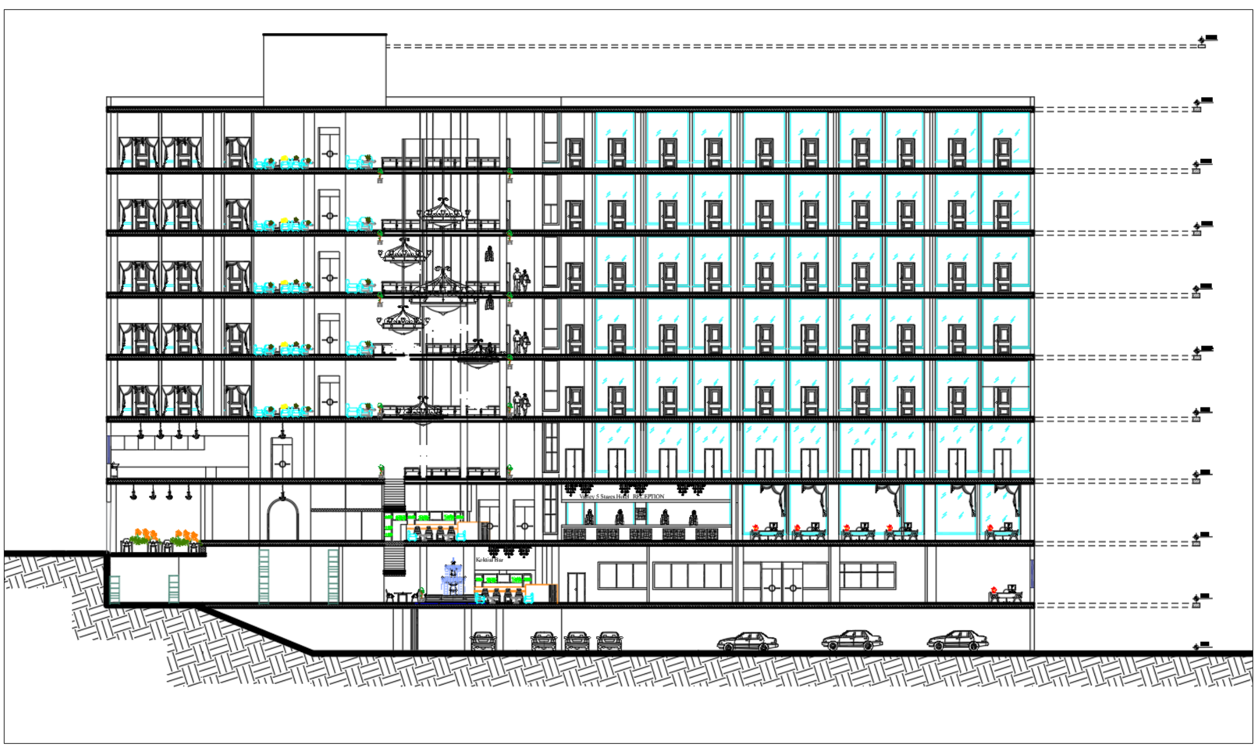

Figure 2-9: section B-B of the hotel

Chapter 3: Structural Description

3.1 Introduction

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

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

3.2 The Aim of the Structural Design

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

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

3.2.2 Durability: The structure should last for a reasonable period of time.

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

3.2.4 Strength: to resist safely the stresses induced by the loads in the various structural members.

3.2.5 Serviceability:

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

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

3.3 Scientific Tests

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

3.4 Loads Acting on the Building

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

3.4.1 Dead loads

Dead loads consist of the weight of all materials of construction incorporated into the building including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes.as shown in the figure 3-1.

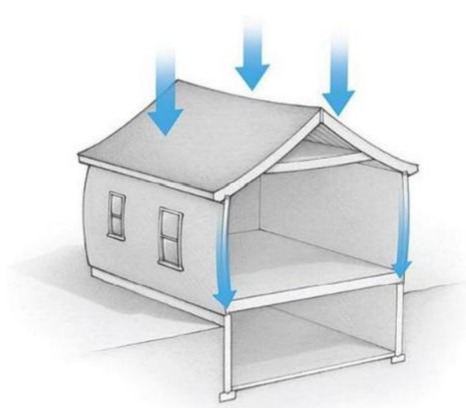

Figure 3-1: dead loads

3.4.2 Live load

Live loads are those loads produced by the use and occupancy of the building (look figure 3-2) or other structure and do not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load, or dead load.

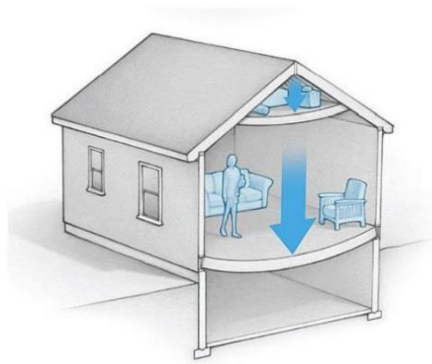

Figure 3-2: live loads

3.4.3 Environmental Loads

It is the third type of load that must be taken into account in the design , and these loads are:

1. Wind Loads

They are horizontal forces that affect the building and their effect appears in tall buildings. They are the forces that the wind affects buildings, installations, or parts of, and they are positive if they are caused by pressure and negative if they are caused by tension, and are measured in kilotons per square meter (KN / m2). Wind loads are determined depending on the height of the building above the ground, and the location in terms of surrounding buildings, whether high or low. As shown in the figure 3-3.

Figure 3-3: wind loads

2. Snow

Snow loads (as figure 3-4) can be evaluated based on the following principles:

• Height of the facility above sea level.

• Slope of the roof exposed to snow.

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

*check

Figure 3-4: snow loads

3. Seismic Loads

One of the most important environmental loads that affect the building, which are horizontal and vertical forces that generate torque, and can be resisted by using shear walls designed with thicknesses and sufficient reinforcement to ensure the safety of the building when it is exposed to such loads that must be observed in the design process to reduce Risks and maintenance of the building's performance of its function during earthquakes as shown in figure 3-5.

Figure 3-5: seismic loads

4. Shrinkage and expansion loads

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

3.5 Structural Elements of the Building

All buildings usually consist of a set of structural elements that work together to maintain the continuity of the building and its suitability for human use, The most important of these slabs, beams, columns, and load-bearing walls, etc. are being defined.as shown in figure 3-6.

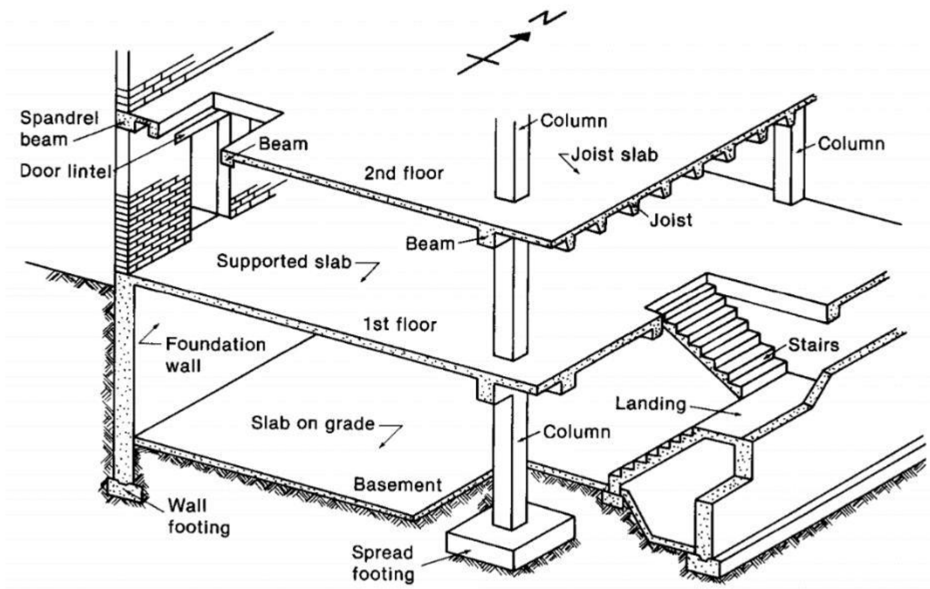

Figure 3-6: structural elements of a building

3.5.1 Slabs

Structural elements are capable of delivering vertical forces due to the loads affecting the building's load-bearing structural elements such as beams, columns, and walls, without distortions.

There are many different Structural systems of reinforced concrete slabs, including the following:

3.5.1.1 Solid slab (one or two way)

Solid Slabs are fully customizable concrete slabs of varying width, length, and thickness. They can be used in a variety of applications such as bridges, piers, and building floors. It is known that solid slabs should be supported by drop beams as shown in figure 3-7.

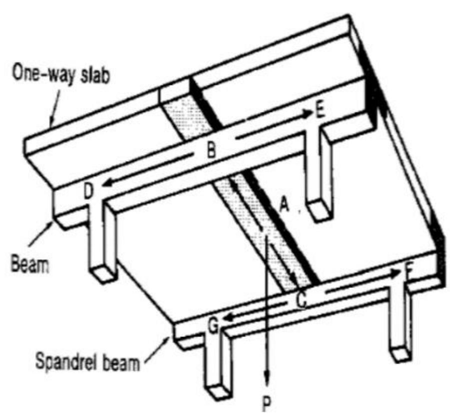

Figure 3-7: drop beams in solid slab

3.5.1.2 Ribbed slab (one or two way)

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

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

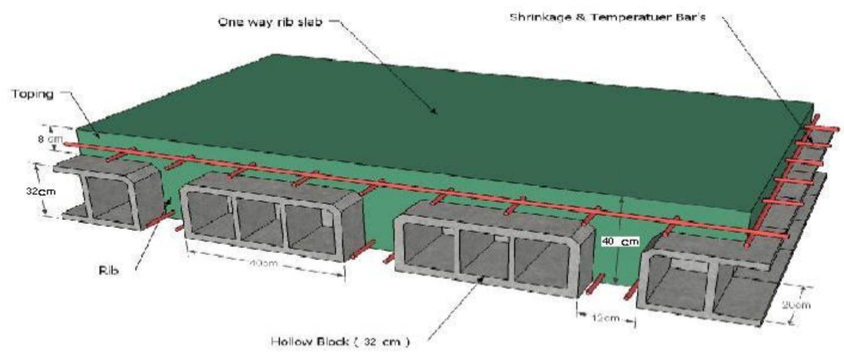

Figure 3-8: a typical section in one way ribbed slab

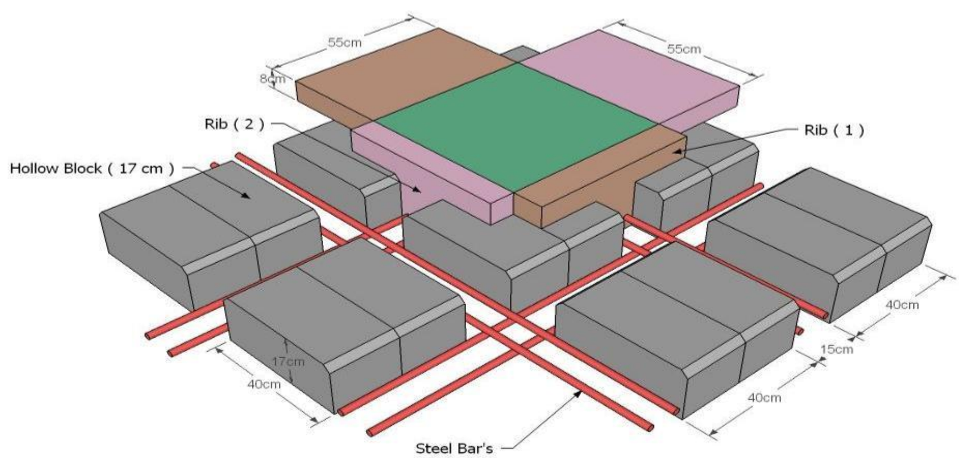

Figure 3-9: a typical section in two way ribbed slab

3.5.2 Beams

They are basic structural elements in transferring loads from slabs to the columns, and they are of two types, hidden inside the slab and Dropped Beams that emerge from the slab from the bottom as shown in figure 3-10.

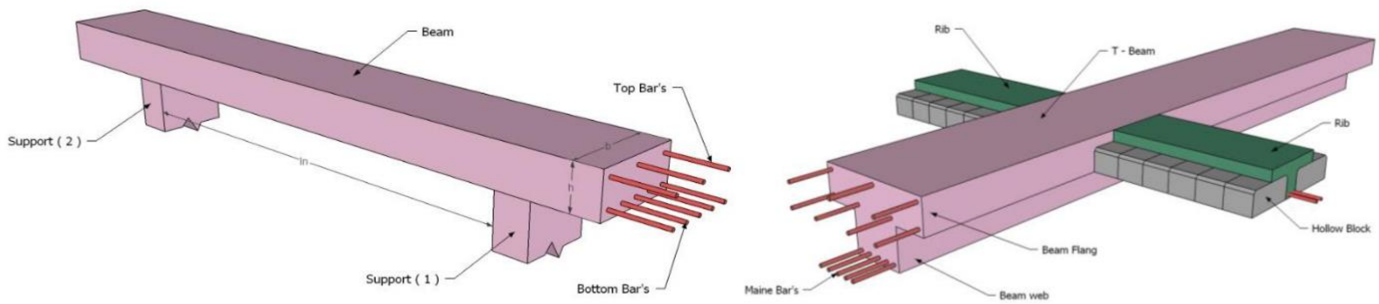

Figure 3-10: Difference between hidden beams and drop beams

3.5.3 Columns

Columns are the main member in transporting loads from slabs and beams to foundations, and as such, they are a necessary structural component for conveying loads and building stability. Therefore, they must be designed to be able to carry and distribute the loads on them. Figure 3-11 shows two types of columns.

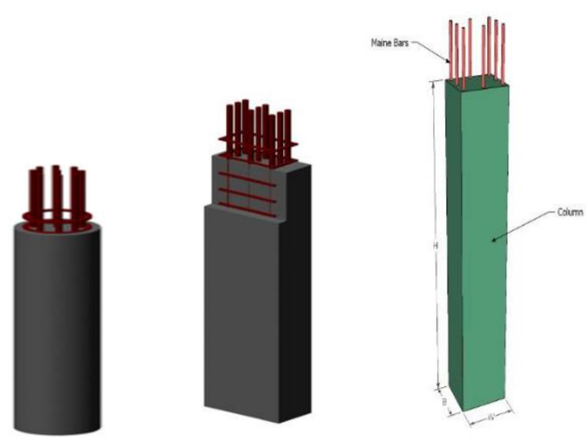

Figure 3-11: two types of columns; circular section and rectangular section

3.5.4 Shear walls

They are structural load-bearing elements that resist vertical and horizontal forces located on them and are mainly used to resist horizontal loads such as wind and earthquake forces. As shown in figure 3-12.

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

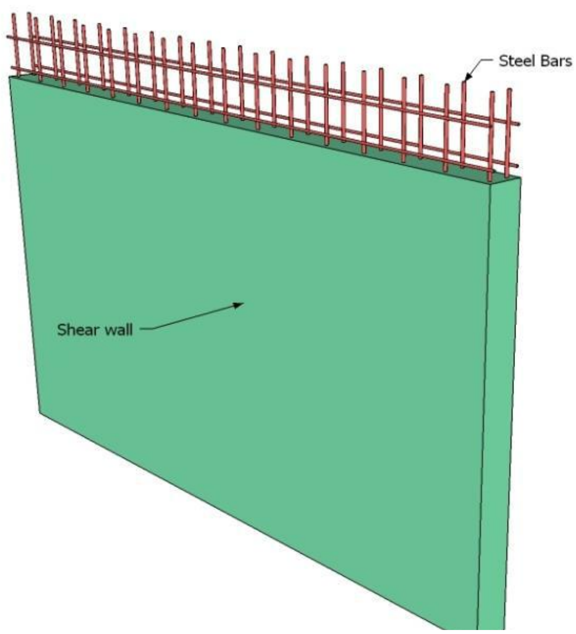

Figure 3-12: a typical shear wall

3.5.5 Basement walls

A basement wall is a wall that is used on the floor and ceiling to provide support to the side walls as well as to the structure as shown in figure 3-13. It handles the pressure of the sidewalls and provides space for living inside the walls. Basement walls bear the load of the whole structure.

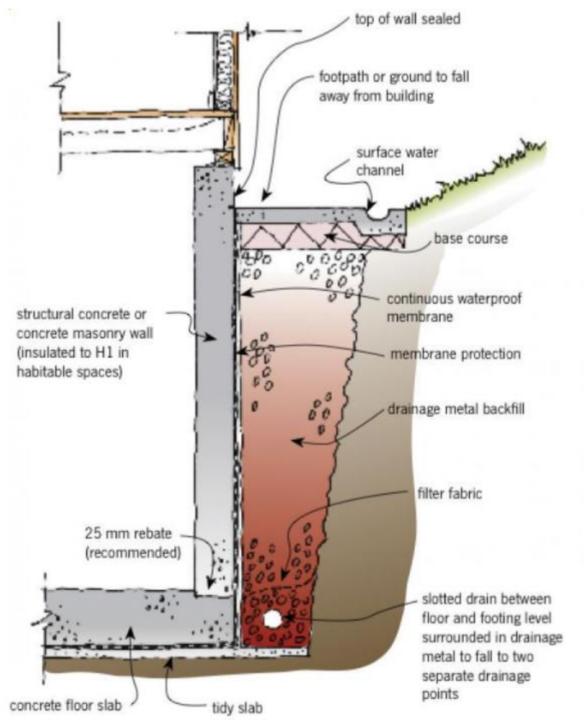

Figure 3-13: a typical basement wall

3.5.6 Foundations

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

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

There a many types of foundations that can be used in each project it depends on the type of loads and the nature of the soil in the site. An isolated footing is shown in the figure 3-13.

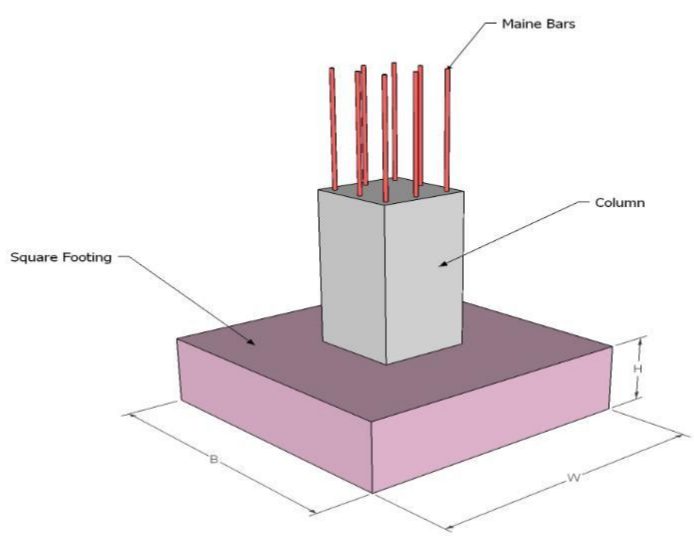

Figure 3-14: Isolated Footing

3.5.6 Stairs

Stairs must be provided in almost all buildings. It consists of rises, runs, and landings. The total steps and landings are called a staircase as shown in figure 3-15, There are different types of stairs, which depend mainly on the type and function of the building and the architectural requirements.

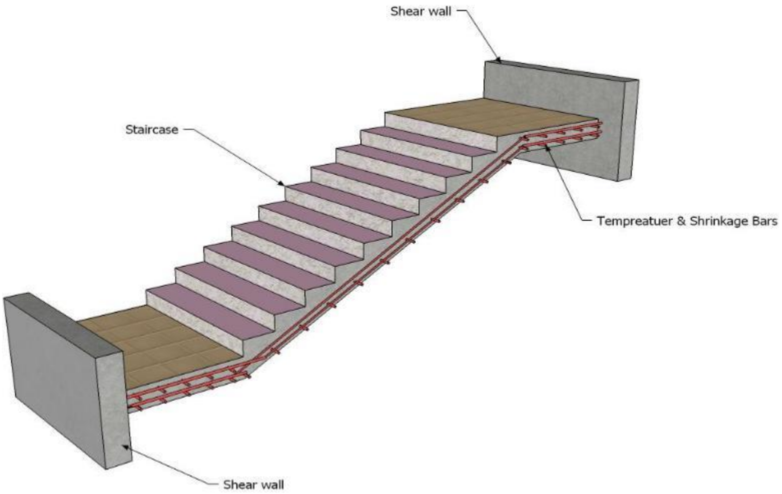

Figure 3-15: components of a typical staircase

3.5.7 Frames

Frame structures are the constructions having a blend of column, beam & slab to bear the adjacent and gravity loads. These structures are generally used to overcome the large moments emerging owing to the applied loading. As in figure 3-16

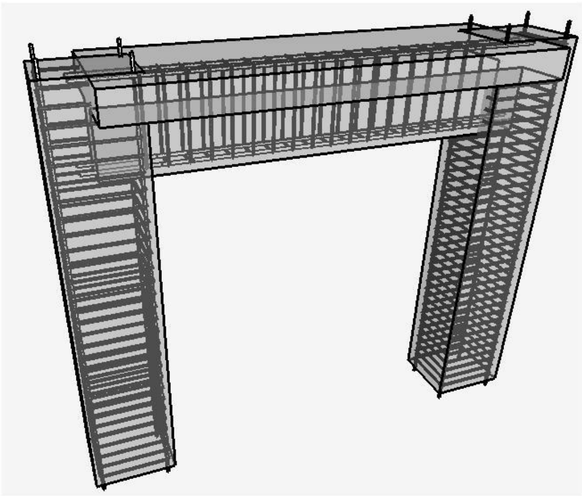

Figure 3-16: a typical frame

Chapter 4: Structural Analysis and Design

4.1. Introduction

Many structures are built of reinforced concrete: bridges, buildings, retaining walls, tunnels, and others.

Reinforced concrete is logical union of two materials: plain concrete, which possesses high compressive strength but little tensile strength, and steel bars embedded in the concrete, which can provide the needed strength in tension.

Plain concrete is made by mixing cement, fine aggregate, coarse aggregate, water, and frequently admixtures.

Understanding of reinforced concrete behavior is still far from complete, building codes and specifications that give design procedures are continually changing to reflect latest knowledge.

4.2. Scope:

Chapter 18 contains the minimum requirements that must be satisfied for cast-in-place and precast concrete structures subject to design earthquake forces prescribed in a legally adopted general building code, such as the 2012 IBC.

Since the design earthquake forces are considered less than those corresponding to linear response at the anticipated earthquake intensity, the integrity of the structure in the inelastic range of response should be maintained provided the applicable detailing requirements of Chapter 18 are satisfied.

Section 18.1.1 requires that all structures be assigned to a Seismic Design Category (SDC) in accordance with the legally adopted general building code. In areas without such a code, the SDC is determined by the local authority having jurisdiction. SDCs in the 2014 ACI Code are adopted directly from ASCE/SEI 7-10 and are a function of the seismic risk level at the site, soil type, and occupancy or use of the structure.

Before the 2008 Code, low, intermediate, and high seismic risk designations were used to define detailing requirements. A comparison of SDCs and seismic risk designations used in various codes, standards, and resource documents is given in Table 5.2.2.

Table R5.2.2—Correlation between seismic-related terminology in model codes

Code, standard, or resource document and edition	Level of seismic risk or assigned seismic performance or design categories as defined in the Code		
ACI 318-08, ACI 318-11, ACI 318-14; IBC of 2000, 2003, 2006, 2009, 2012; NFPA 5000 of 2003, 2006, 2009, 2012; ASCE 7-98, 7-02, 7-05, 7-10; NEHRP 1997, 2000, 2003, 2009	SDC[1] A, B	SDC C	SDC D, E, F
ACI 318-05 and previous editions	Low seismic risk	Moderate/ intermediate seismic risk	High seismic risk
BOCA National Building Code 1993, 1996, 1999; Standard Building Code 1994, 1997, 1999; ASCE 7-93, 7-95; NEHRP 1991, 1994	SPC[2] A, B	SPC C	SPC D, E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0, 1	Seismic Zone 2	Seismic Zone 3, 4

[1]SDC = seismic design category as defined in code, standard, or resource document.
[2]SPC = seismic performance category as defined in code, standard, or resource document.

Figure 4-1 table R5.2.2

The provisions in Chapter 18 relate detailing requirements to the type of structural framing and the SDC. As noted previously, the provisions in this chapter were revised and renumbered to present seismic requirements in order of increasing SDC.

Traditionally, seismic risk levels have been classified as low, moderate, and high.

Table R5.2.2contains a summary of the seismic risk levels, seismic performance categories (SPC), and seismic design categories (SDC) specified in the IBC, the three prior model building codes now called legacy codes, as well as other resource documents.

all structures must satisfy the applicable provisions of Chapter 18, except for those assigned to SDC A or exempted by the legally adopted building code. The design and detailing requirements of Chapters 1 through 19 and Chapter 22 are considered to provide adequate toughness for these structures subjected to low level earthquake intensities. The designer should be aware that the general requirements of the code include several provisions specifically intended to improve toughness, in order to increase resistance of concrete structures to earthquake and other catastrophic or abnormal loads. For example, when a beam is part of the seismic force-resisting system of a structure, a portion of the positive moment reinforcement must be anchored at supports to develop its yield strength* (see 12.11.2).

In essence, design and detailing requirements should be compatible with the level of energy dissipation or toughness assumed in the computation of the design earthquake forces. To facilitate this compatibility, the code uses throughout Chapter18 the terms “ordinary,” “intermediate,” and “special” in the description of different types of structural systems. The degree of required detailing (and, thus, the degree of required toughness), which is directly related to the SDC, increases from ordinary to intermediate to special types of structural systems.

The legally adopted building code (or the authority having jurisdiction in areas without a legally adopted building code) prescribes the type of seismic-force-resisting system that can be utilized as a function of SDC. There are essentially no restrictions on the type of seismic-force-resisting system that can be used for structures assigned to SDC A or B; as noted previously, only the requirements of 18.1.2 must be satisfied in addition to those in Chapters 1 to 19 and 22 for structures assigned to SDC B.

The seismic-force-resisting systems that typically can be utilized in structures assigned to SDC C are ordinary cast-in-place structural walls, intermediate precast walls, intermediate moment frames, or any combination thereof.

Ordinary structural walls need not satisfy any provisions of Chapter 18 (18.2.1.7(b)). Walls proportioned by the general requirements of the code are considered to have sufficient toughness at anticipated drift levels. Intermediate precast walls must satisfy 18.4 in addition to the general requirements of the code.

This section (18.2.1.7) includes certain reinforcing details, in addition to those contained in Chapters 1 through 19 and 22, that are applicable to reinforced concrete moment frames (beam-column or slab-column framing systems) required to resist earthquake effects.

These so called “intermediate” reinforcement details will serve to accommodate an appropriate level of inelastic behavior if the frame is subjected to an earthquake of such magnitude as to require it to perform inelastically. The type of framing system provided for earthquake resistance in a structure assigned to SDC C will dictate whether any special reinforcement details need to be incorporated in the structure. If the seismic force-resisting system consists of moment frames, the details of 18.4 for Intermediate Moment Frames must be provided, and 21.1.1.5 shall also apply. Note that even if a load combination including wind load effects governs design versus a load combination including earthquake force effects, the intermediate reinforcement details must still be provided to ensure a limited level of toughness in the moment resisting frames.

Whether or not the specified earthquake forces govern design, the frames are the only defense against the effects of an earthquake. For a combination frame-shear wall structural system, inclusion of the intermediate details will depend on how the earthquake loads are “assigned” to the shear walls and the frames. If the total earthquake forces are assigned to the shear walls, the intermediate detailing of 18.4 is not required for the frames. If frame-shear wall interaction is considered in the analysis, with some of the earthquake forces to be resisted by the frames, then the intermediate details of 18.4 are required to toughen up the frame portion of the dual framing system.

Model codes have traditionally considered a dual system to be one in which at least 25% of the design lateral forces are capable of being resisted by the moment frames. If structural walls resist total gravity and lateral load effects, no intermediate details are required for the frames; the general requirements of the code apply.

It is important to note that The general code permit the use of intermediate moment frames as part of dual systems for buildings assigned to SDC D,E or F.(R18.2), based on this section this building was determined to select intermediate moment frames to resist seismic loads.

4.3. Design method and requirements.

The design strength provided by a member is calculated in accordance with the requirements and assumptions of ACI_code (318_14).

Strength design method:-

In ultimate strength design method, the service loads are increased by factors to obtain the load at which failure is considered to be occurring.

This load called factored load or factored service load. The structure or structural element is then proportioned such that the strength is reached when factored load is acting. The computation of this strength takes into account the nonlinear stress-strain behavior of concrete.

The strength design method is expressed by the following,

Strength provided ≥ strength required to carry factored loads.

- ✓ **Code**
  ACI 318-14
- ✓ **Material**
  Concrete: B300
  ${f_c}'$= 30 MPa (for circular sections)
  ${f_c}'$= 30*0.8= 24MPa (for rectangular section)
  Reinforcement steel-:
  The specified yield strength of the reinforcement (f_y= 420 N/mm² (MPa)).
- ✓ **Load combinations**
  $\boldsymbol{W_u = 1.2\ DL + 1.6\ LL}$ [1]
  $\boldsymbol{W_u = 1.4\ DL}$ [2]

4.4. Seismic load

We refer to a hazard maps for spectral accelerations seismic event : maximum considered earthquake at Jericho $S_s = 0.9$, $S_1 = 0.18$, and has a C site class .

From ASCE 7-16 Table 11.4-2 as shown in the figure below $F_V = 1.5$.

[1] ACI318-14, 5.3.1.b

[2] ACI318-14, 5.3.1.a

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

Figure 4-2 : Table 11.4-2

From ASCE 7-16 Table 11.4-1 as shown in the figure below $F_a = 1.2$.

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Figure 4-3 Table 11.4-1

According to ASCE 7-16 (11.4.4) " The MCE_R spectral response acceleration parameters for short periods (S_{MS}) and 1 s (S_{M1}) , adjusted for site class effects ".

shall be determined using the following equations :

$S_{MS} = F_a S_S = 1.2 * 0.9 = 1.08$

$$S_{M1} = F_v S_1 = 1.5 * 0.18 = 0.27$$

According to ASCE 7-16 (11.4.5) the Design earthquake spectral response acceleration parameters at short periods , S_{DS} and at 1-s periods S_{D1} , shall be determined using the following equations :

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} * 1.08 = 0.72$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} * 0.27 = 0.18$$

The Risk Category of the structure is III and according to table 11.6-1 and 11.6-2 as shown in the figure below , the Seismic Design Category is D .

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

Figure 4-4 : Table 11.6-1 and Table 11.6-2

For Design Spectral Acceleration curve according to ASCE 7-16 (11.4.6) shall be developed as follows :

$$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2 * \frac{0.18}{0.75} = 0.048 \text{ s}$$

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.18}{0.75} = 0.24 \, s$$

The following is the Constructed Design Spectral Acceleration curve using etabs .

Figure 4-5 : Design Spectral Acceleration curve

- Structural System Selection

We chose a DUAL SYSTEM with INTERMIDIATE MOMENT FRAMES and SPECIAL SHEAR WALL . With Response Modification Coefficient R= 6.5 , and Over strength Factor , $\Omega_0 = 2.5$, and Deflection amplification factor C_d= 5 .

4.5. Check of Minimum Thickness of Structural Member.

Minimum Thickness of Non pre-stressed Beam or One-Way Slabs Unless Deflections are calculated. **(ACI 318M-11).**

Table 4-1 minimum thickness for one way slabs[3]

	Minimum thickness ,h			
	Simply supported	One end continuous	Both ends continuous	cantilever
member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	ℓ/20	ℓ /24	ℓ /28	ℓ /10
Beams or ribbed one-way slabs	ℓ /16	ℓ /18.5	ℓ /21	ℓ /8

$H_{min}$for(one end continuous)=L/18.5=670/18.5=36.2cm
H_{min} for(both end continuous)=L/21=679/24=28.29 cm
H_{min} for(cantilever)=L/8=240/8=30cm
H_{min} for(Simply supported)=L/16=335/16=20.93cm
Take h = 35 cm
27 cm block + 8 cm topping = 35cm

[3] Table 9.5(a) - ACI318-11m.

4.5.1 Design of Topping.

❖ Statically System For Topping :-

Consider the topping as strip of (1m) width, and span of mold length with both end fixed in the ribs.

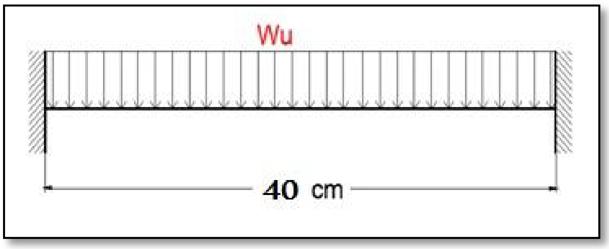

Figure 4-6: a topping strip static system

4.5.2 Load Calculation for topping

Table 4-2: load calculations for topping

Load calculations				
layer N.O	Layer name	Thickness	Density	W(KN/m)
1	Tiles	0.03	23	0.69
2	mortar	0.03	22	0.66
3	sand	0.07	17	1.19
4	toping R.C	0.08	25	2
8	partitions		3.53	3.53
			D.L	8.07

Live Load =4KN/m2 =4*1=4KN/m

❖ Factored Load

W_u= 1.2 ×8.07+ 1.6×4 =16.08 KN/m

W_u= 1.4*8.07=11.298 KN/m

Check the strength condition for plain concrete, $\phi Mn \geq Mu$, where ø = 0.55

$Mn = 0.42\, \lambda \sqrt{fc'}\, Sm$ [4]

$M_{n=} 0.42 * 1 * \sqrt{24} * 1066666.67 * 10^{-6} = 2.19\, KN.m$

$\emptyset M_n = 0.55 * 2.19 = 1.21\, KN.m$

$$M_u = \frac{W_u l^2}{12} = \frac{16.08 * 0.4^2}{12} = 0.2144\, KN.m$$

$\emptyset M_n \gg M_u$ No reinforcement is required by analysis. According to ACI 10.5.4, provide As_{min} for slabs a shrinkage and temperature reinforcement.

$\rho_{shrinkage}$ = 0.0018[5]

$A_s = \rho \times b \times h = 0.0018 \times 1000 \times 80 = 144\ mm^2$/m

Step (s) is the smallest of:

1. 3h=3*80=240 mm - control
2. 450 mm
3. $s = 380 * \left(\frac{280}{f_s}\right) - 2.5c^6 = 380 * \left(\frac{280}{\frac{2}{3} * 420}\right) - 2.5 * 20 = 330\, mm$
4. $s \leq 300 * \left(\frac{280}{\frac{2}{3} * 420}\right) = 300$mm

Take ø 8 @ 200 mm in both direction, S = 200 mm <S_{max} = 240 mm … O

4.5.3 Design of One Way Rib Slab

- **Material**
  - concrete B300 Fc' = 24 N/mm2
  - Reinforcement Steel fy = 420 N/mm2
- **Section**
  - B = 520mm
  - Bw= 120 mm
  - h= 350 mm
  - t= 80 mm

[4] ACI318-14, equation 14.5.2.1a.
[5] ACI318-14, 7.12.2.1
[6] ACI318-14, 24.3.2

- d=350-20-10-16/2= 312 mm

❖ **Statically System and Dimensions**

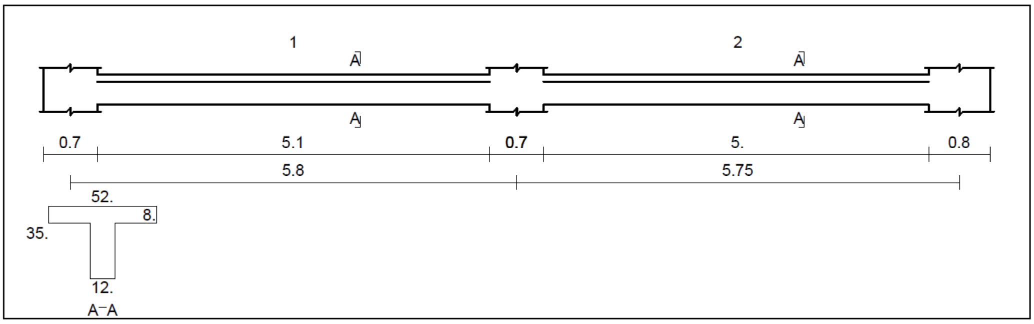

Figure 4-7: rib geometry and dimensions

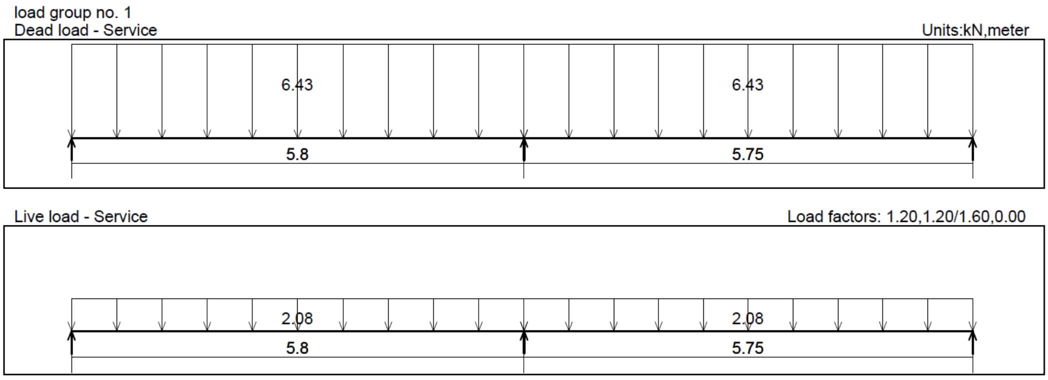

Figure 4-8: load placement on rib

4.5.4 Load Calculation for Rib1

Table 4-3: load calculations for rib

Load calculations				
layer N.O	Layer name	Thickness	Density	W(KN/m)
1	Tiles	0.03	23	0.359
2	mortar	0.03	22	0.343
3	sand	0.07	17	0.619
4	toping R.C	0.08	25	1.04
5	Block	0.27	10	1.08
6	ribs	0.27	25	0.81
7	Plaster	0.03	22	0.343
8	partitions		3.53	1.836
			D.L	6.43

Live Load = 4KN/m2=4*0.52=2.08 KN/m.

4.5.5 Effective Flange Width (b_e):-(ACI-318-14 (8.12.2))

b_e For T- section is the smallest of the following:-
$b_e \leq$ L / 4 = 400/ 4 =100cm
$b_e \leq b_w$ + 16 t = 12 + 16 (8) = 140 cm
$b_e \leq$ Center to center spacing between adjacent beams = 52 cm. ***Control***

4.5.6 Flexure Design

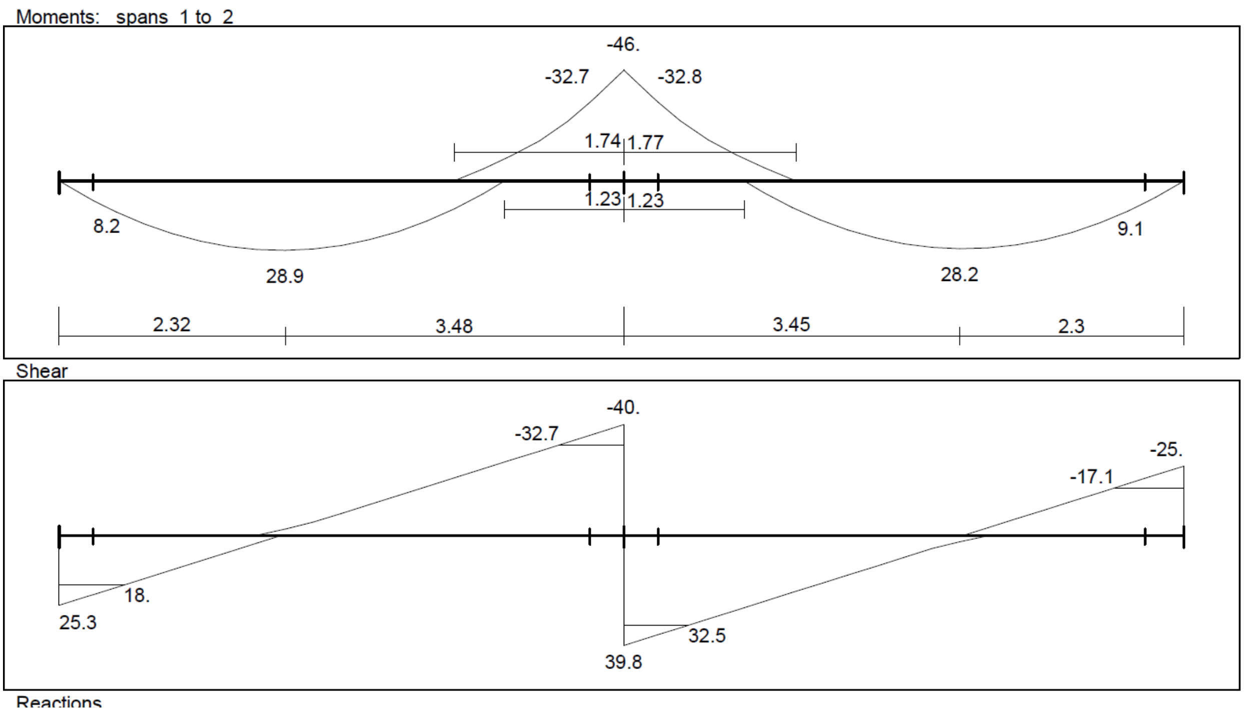

Figure4-9: moment and shear values for rib

Reactions

Factored			
DeadR	16.83	55.7	16.59
LiveR	8.44	24.02	8.38
MaxR	25.27	79.73	24.97
MinR	15.65	67.64	15.37
Service			
DeadR	14.03	46.42	13.82
LiveR	5.27	15.02	5.24
MaxR	19.3	61.43	19.06
MinR	13.29	53.88	13.06

Figure 4-10: factored and service reactions

4.5.6.1 Design of Rib 1 for positive moments

Assume bar diameter Ø 14 for main positive reinforcement

d= 350-20-10- $\frac{14}{2}$ = 313 mm

- Span 1 (Max Positive Moment= 28.9 KN.m)

Check if a > h_f

$$\bar{\bar{M}}_{nf} = 0.85 * f_c' b h_f \left(d - \frac{h_f}{2}\right) = 0.85 * 24 * 520 * 80 * \left(313 - \frac{80}{2}\right) * 10^{-6}$$
$$= 231.69 KN.m$$

$$\bar{\bar{M}}_{nf} = 231.69\ KN.m \gg \frac{M_u}{\emptyset} = \frac{28.9}{0.9} = 32.11\ KN.m$$

$$\rightarrow a < h_f$$

The section will be designed as rectangular section with b = 520 mm

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{28.9 * 10^6}{0.9 * 520 * 313^2} = 0.63\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 0.63 * 20.59}{420}}\right) = 0.001525$$

$$A_s = \rho b d = 0.001525 * 520 * 313 = 248.16\ \ mm^2$$

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} 120 * 313 = 109.53\ mm^2$$

$A_{s,min} = \frac{1.4}{420} 120 * 313 = 125.2\ mm^2$ –control

Use 2Ø14 with $A_s = 3.079\ cm^2 > \ A_{s,req} = 2.4816 cm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{307.88 * 420}{0.85 * 24 * 520} = 12.19\ mm$$

$$c = \frac{a}{\beta_1} = \frac{12.19}{0.85} = 14.34\,mm$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{313-14.34}{14.34}\right) = 0.0625 > 0.005 - OK$$

4.5.6.2 Design of Rib 1 for negative moments

- ✓ (Max moment = 32.8 KN.m)

Assume bar diameter Ø 14 for main positive reinforcement

d= 350-20-10- $\frac{14}{2}$ = 313 mm

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{32.8 * 10^6}{0.9 * 120 * 313^2} = 3.1\ \ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 3.1 * 20.59}{420}}\right) = 0.00805$$

$$A_s = \rho b d = 0.00805 * 120 * 313 = 302.36\,mm^2$$

Check for $A_{s,min}$

$$A_{s,min} = 0.25\frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25\frac{\sqrt{24}}{420} 120 * 313 = 109.52\,mm^2$$

$A_{s,min} = \frac{1.4}{420} 120 * 313 = 125.2\,mm^2$ –control

Use 2Ø14 with $A_s = 307.88\,mm^2 > \ A_{s,req} = 302.36\,mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{307.88 * 420}{0.85 * 24 * 120} = 52.82\,mm$$

$$c = \frac{a}{\beta_1} = \frac{52.82}{0.85} = 62.14\,mm$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{312-62.14}{62.14}\right) = 0.01211 > 0.005 - OK$$

4.5.7 Shear Design

Vu at distance d from support = 32.7 KN

Shear strength Vc , 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).**

$$V_c = 1.1 \times \frac{1}{6}\lambda\sqrt{f_c'}b_w d = 1.1 \times \frac{1}{6}\sqrt{24} * 120 * 313 * 10^{-3} = 33.73KN$$

$$\emptyset V_c = 0.75 * 33.73 = 25.3\ KN$$

$$V_{s,min} = \frac{1}{16}\sqrt{f_c'}b_w d = \frac{1}{16}\sqrt{24} * 120 * 313 * 10^{-3} = 11.50\ KN$$

$$V_{s,min} = \frac{1}{3}b_w d = \frac{1}{3} * 120 * 313 * 10^{-3} = 12.52\ KN - Control$$

$$\emptyset\left(Vc + V_{s,min}\right) = 0.75 * (33.73 + 12.52) = 34.69\text{ KN}$$

$$\emptyset\left(Vc + V_{s,min}\right) > \boldsymbol{V_u} > \emptyset V_c$$

Use stirrups U-shape (2legs stirrups) **Ø10** with $A_v = 2 * 78.57 = 157.14\ mm^2$

$$V_{S,min} = \frac{A_{v,min}}{S}f_{yt}d = \frac{157.14}{S} * 420 * 313 = 12.52\ KN\ \rightarrow S = 1649.97\ mm$$

$$s \le 600mm \le \frac{d}{2} = \frac{313}{2} = 156.5mm$$

Take U-shape (2legs stirrups) Ø10 @150mm along 1.5 m from each side of the face of support

4.6. Frame Design

4.6.1. Notes on Design of intermediate moment resisting Frame

The provisions for beams and columns in intermediate moment frames are presented in Table---, respectively, which can be found in 18.4.2 and 18.4.3, respectively. The shear provisions are also included in those tables. Hoops instead of stirrups are required at both ends of beams for a distance not less than 2h from the faces of the supports. The likelihood of spalling and loss of shell concrete in some regions of the frame are high. Both observed behavior under actual earthquakes and experimental research have shown that the transverse reinforcement will open at the ends and lose the ability to confine the concrete unless it is bent around the longitudinal reinforcement and its ends project into the core of the element. Similar provisions are given in 18.4.3 for columns. For columns in intermediate moment frames, the code require that the design shear strength Vn must not be less than the smaller (a) and (b) (18.4.3.1):

a- The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending and the shear from factored gravity loads. Column flexural strength must be calculated for the factored axial force consistent with the direction of the lateral force considered, resulting in the highest flexural strength.

b- The maximum shear obtained from design load combinations that include E, with E increased by Ωo. In 2008 code the increase factor was 2. This factor was replaced with the over strength factor Ωo in 2001 Code. For intermediate moment frame Ωo = 3.0 (ASCE 7-10). The new provision in 2011 was intended to reduce the risk of column shear failure in intermediate moment frames. Provisions in 18.4.3.3 address detailing requirements for columns supporting reactions from discontinuous stiff members such as walls. In such cases, discontinuous walls or other stiff members impose, among other things, large axial forces on supporting columns during an earthquake. Thus, transverse reinforcement as defined in 18.4.3.3 for columns in intermediate moment frames is required over the entire length of the column if the portion of the factored axial compressive force due to earthquake effects exceeds Ag›/10.

The limit of Ag›/10 is increased to Ag›/4 where design forces have been magnified by the overstrength factor Wo required by ASCE/SEI 7-10. The transverse reinforcement over the height of the column and over the lengths above and below the column is to improve column toughness when subjected to anticipated earthquake loads.

4.6.2. Frame 1 calculations

4.6.2.1 Materials check:

No Limits on materials in intermediate moment resisting frames, yet table 19.2.1.1 limits the general application of normal weight concrete’s compressive strength to be 17 MPa.

Fc’ used in frames are: 24 MPa, 28MPa which are greater than 17 MPa, so it is permitted.

4.6.2.2 **Flexural member calculation of frame 1 :**

Figure 4-7 shows the location of Frame 1 in reference to the ninth floor .

Figure 4-11: location of frame 1

❖ Dimensions

Figure shows a section in Frame 1 with indication of the geometry and indication of beam 1 location

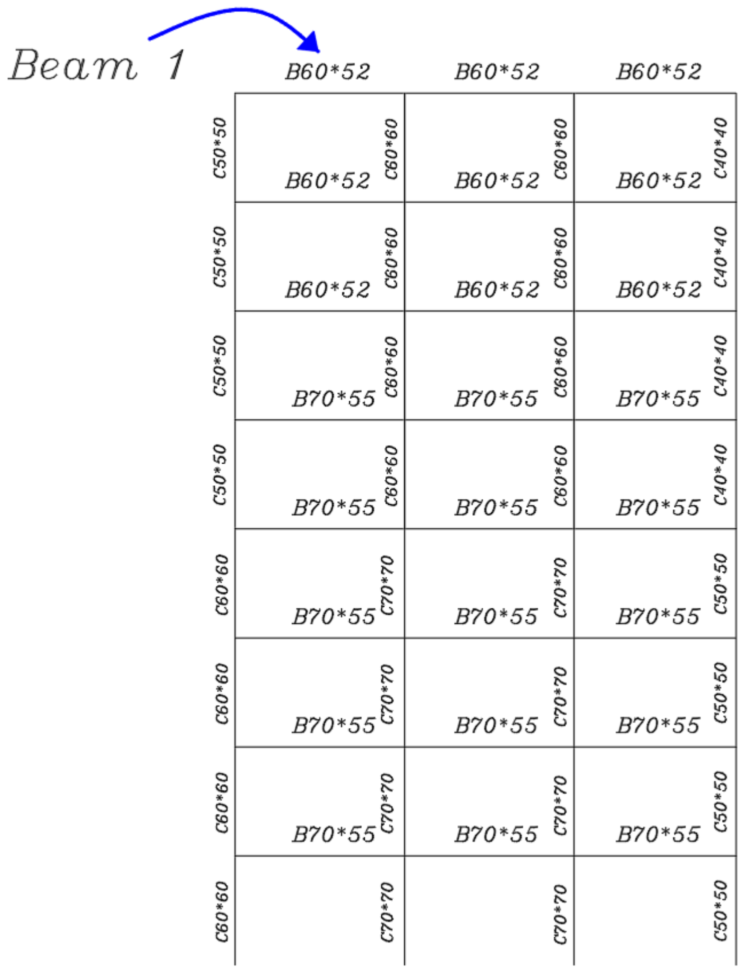

Figure 4-12: Geometry of beam 1

❖ Material

- concrete B300 Fc' = 24 N/mm2
- Reinforcement Steel fy = 420 N/mm2

❖ Section

- B = 600 mm
- h= 550 mm
- d=550-40-10-16/2= 492 mm

4.6.3 Load Calculation for Beam1

Self-weight of beam =25*0.8*0.55=11 KN/m

$$w_{DL, from\, rib1} = \frac{47.76}{0.52} = 91.85\ KN/m$$

$$w_{LL, from\, rib1} = \frac{15.45}{0.52} = 29.72\ KN/m$$

Seismic loads are obtained from dynamic analysis ; response spectra analysis based on the seismic factors of Jericho.

4.6.4 Flexure Design for beam 1 as part of intermediate moment resisting frame

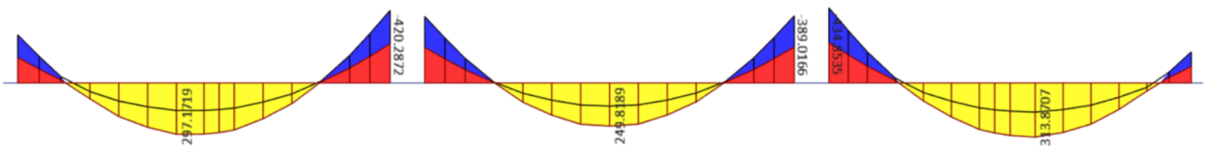

Figure 4-13: moment values for beam1

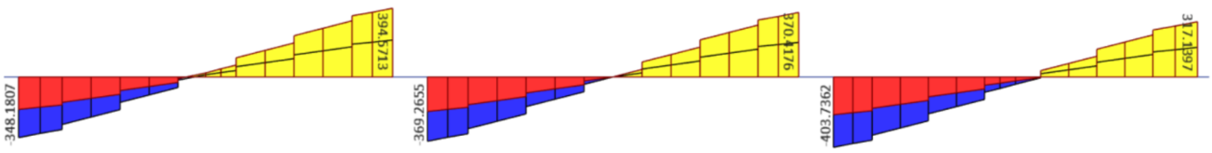

Figure 4-14: shear moment values for beam1

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

$$c = \frac{3}{7} d = \frac{3}{7} 492 = 210.85\ mm$$

$$a = \beta_1 c = 0.85 * 210.85 = 179.22\ mm$$

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

$$= 0.8 * 0.85 * 24 * 179.59 * 600 * \left(492 - \frac{179.59}{2}\right) * 10^{-6}$$

$$= 707.29\ KN.m$$

$$M_u = 311.4 KN.m < \emptyset M_{n,max} = 707.29\ KN.m$$

Design the section as singly reinforced concrete section.

4.6.5 Design of Beam 1 for positive moments

Assume bar diameter Ø 16 for main positive reinforcement

d= 550-40-10- $\frac{16}{2}$ = 492 mm

- Span 1 (Max Positive Moment= 297.17 KN.m)

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{297.17 * 10^6}{0.9 * 600 * 492^2} = 2.273\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 2.273 * 20.59}{420}}\right) = 0.00575$$

$A_s = \rho b d = 0.00575 * 600 * 492 = 1697.4\ mm^2$ -control

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} * 600 * 492 = 860.82 mm^2$$

$$A_{s,min} = \frac{1.4}{420} * 800 * 487.5 = 984\ mm^2$$

$$As > As\ min$$

Use 9Ø16with $A_s = 1809\ mm^2 > \ A_{s,req} = 1697.4 mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{1697.4 * 420}{0.85 * 24 * 600} = 58.24 mm$$

$$c = \frac{a}{\beta_1} = \frac{58.24}{0.85} = 68.52\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{492-68.52}{68.52}\right) = 0.0185 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40*2 - 10*2 - 9*16}{8} = 44.5mm > 25mm > 1.5db = 1.5*16 = 24mm$$

- Span 2 (Max Positive Moment= 249.82 KN.m)

Assume bar diameter Ø 16 for main positive reinforcement

d= 550-40-10- $\frac{16}{2}$ = 492 mm

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{249.82*10^6}{0.9*600*492^2} = 1.911\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2*1.911*20.59}{420}}\right) = 0.00478$$

$$A_s = \rho bd = 0.00478*600*492 = 1412.9\ mm^2$$

Check for $A_{s,min}$

$$A_{s,min} = 0.25\frac{\sqrt{f_c'}}{f_y}b_w d \geq \frac{1.4}{f_y}b_w d$$

$$A_{s,min} = 0.25\frac{\sqrt{24}}{420}*600*492 = 860.82mm^2$$

$A_{s,min} = \frac{1.4}{420}*600*492 = 984\ mm^2$ –control

Use 8Ø16 with $A_s = 1608\ mm^2 > \ A_{s,req} = 1412.9cm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85f_c' b} = \frac{1608*420}{0.85*24*600} = 55.18\ mm$$

$$c = \frac{a}{\beta_1} = \frac{55.18}{0.85} = 64.9mm$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{492-64.9}{64.9}\right) = 0.0197 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600-40*2-10*2-8*16}{7} = 53.143mm > 25mm > 1.5db = 1.5*16$$
$$= 24mm$$

- Span 3 (Max Positive Moment= 313.87 KN.m)

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{313.87*10^6}{0.9*600*492^2} = 2.401\ MPa$$

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

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1-\sqrt{1-\frac{2*2.401*20.59}{420}}\right) = 0.0061$$

$$A_s = \rho bd = 0.0061*600*492 = 1800.78\ mm^2$$

Check for $A_{s,min}$

$$A_{s,min} = 0.25\frac{\sqrt{f_c'}}{f_y}b_w d \geq \frac{1.4}{f_y}b_w d$$

$$A_{s,min} = 0.25\frac{\sqrt{24}}{420}*600*492 = 860.82\ mm^2$$

$A_{s,min} = \frac{1.4}{420}*600*492 = 984\ mm^2$ –control

Use 9Ø16 with $A_s = 1809\ mm^2 > A_{s,req} = 1800.78mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85f_c'b} = \frac{1809*420}{0.85*24*600} = 62.07mm$$

$$c = \frac{a}{\beta_1} = \frac{62.07}{0.85} = 73.027\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{492-73.027}{73.027}\right) = 0.0172 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40 * 2 - 10 * 2 - 9 * 16}{8} = 44.5mm > 25mm > 1.5db = 1.5 * 16 = 24mm$$

4.6.6 Design of Beam 1 in intermediate moment resisting frame for negative moments

- Span 1 (Max Negative Moment= 420.1 KN.m)

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{420.1 * 10^6}{0.9 * 600 * 492^2} = 3.21\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 3.21 * 20.59}{420}}\right) = 0.00837$$

$A_s = \rho b d = 0.00837 * 600 * 492 = 2472.01\ mm^2$–control

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} * 600 * 492 = 860.82\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} * 600 * 492 = 984\ mm^2$$

Use 13Ø16 with $A_s = 2613\ mm^2 > \ A_{s,req} = 2472.01 mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2613 * 420}{0.85 * 24 * 600} = 89.66\ mm$$

$$c = \frac{a}{\beta_1} = \frac{89.6617}{0.85} = 105.48\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{492 - 105.48}{105.48}\right) = 0.0109 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40 * 2 - 10 * 2 - 13 * 16}{12} = 25.1\ mm > 25mm > 1.5db = 1.5 * 16 = 24mm$$

- Span 2 (Max Negative Moment= 389.02 KN.m)

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{389.02 * 10^6}{0.9 * 600 * 492^2} = 2.976\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 2.976 * 20.59}{420}}\right) = 0.00769$$

$A_s = \rho b d = 0.00769 * 600 * 492 = 2271.75\ mm^2$–control

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} * 600 * 492 = 860.82\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} * 600 * 492 = 984\ mm^2$$

Use 12Ø16 with $A_s = 2400\ mm^2 > \ A_{s,req} = 2271.75 mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2400 * 420}{0.85 * 24 * 600} = 82.35\ mm$$

$$c = \frac{a}{\beta_1} = \frac{82.35}{0.85} = 96.89\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{492 - 96.89}{96.89}\right) = 0.0122 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40 * 2 - 10 * 2 - 12 * 16}{11} = 28\ mm > 25mm > 1.5db = 1.5 * 16 = 24mm$$

- Span 3 (Max Negative Moment= 434.85 KN.m)

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{434.85 * 10^6}{0.9 * 600 * 492^2} = 3.3227\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 3.32 * 20.59}{420}}\right) = 0.00868$$

$A_s = \rho b d = 0.00868 * 600 * 492 = 2564.79\ mm^2$–control

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} * 600 * 492 = 860.82\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} * 600 * 492 = 984\ mm^2$$

Use 13Ø16 with $A_s = 2613\ mm^2 > \ A_{s,req} = 2472.01 mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2613 * 420}{0.85 * 24 * 600} = 89.66\ mm$$

$$c = \frac{a}{\beta_1} = \frac{89.6617}{0.85} = 105.48\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{492 - 105.48}{105.48}\right) = 0.0109 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40 * 2 - 10 * 2 - 13 * 16}{12} = 25.1\ mm > 25mm > 1.5db = 1.5 * 16 = 24mm$$

- Left edge moment =-279.3494 kN.m

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{279.3494 * 10^6}{0.9 * 600 * 492^2} = 2.137\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 2.137 * 20.59}{420}}\right) = 0.00538$$

$A_s = \rho b d = 0.00538 * 600 * 492 = 1590.269\ mm^2$–control

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{24}}{420} * 600 * 492 = 860.82\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} * 600 * 492 = 984\ mm^2$$

Use 8Ø16 with $A_s = 1601\ mm^2 > \ A_{s,req} = 1590.269 mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{1601 * 420}{0.85 * 24 * 600} = 54.936\ mm$$

$$c = \frac{a}{\beta_1} = \frac{54.936}{0.85} = 64.63\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{492 - 64.63}{64.63}\right) = 0.0198 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40 * 2 - 10 * 2 - 8 * 16}{7} = 53.14\ mm > 25mm > 1.5db = 1.5 * 16 = 24mm$$

- Right edge moment =-434.85 kN.m

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{434.85 * 10^6}{0.9 * 600 * 492^2} = 3.33\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 3.33 * 20.59}{420}}\right) = 0.0087$$

$A_s = \rho bd = 0.0087 * 600 * 492 = 2568.23\ mm^2$–control

Check for $A_{s,min}$

$$A_{s,min} = 0.25\frac{\sqrt{f_c'}}{f_y}b_w d \geq \frac{1.4}{f_y}b_w d$$

$$A_{s,min} = 0.25\frac{\sqrt{24}}{420} * 600 * 492 = 860.82\ mm^2$$

$$A_{s,min} = \frac{1.4}{420} * 600 * 492 = 984\ mm^2$$

Use 13Ø16 with $A_s = 2600\ mm^2 > \ A_{s,req} = 2568.23\ mm^2$- OK

Check for strain

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2600 * 420}{0.85 * 24 * 600} = 89.2\ mm$$

$$c = \frac{a}{\beta_1} = \frac{89.2}{0.85} = 104.95\ mm$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{492 - 104.95}{104.95}\right) = 0.0111 > 0.005 - OK$$

Check for bar placement:

$$s_b = \frac{600 - 40 * 2 - 10 * 2 - 13 * 16}{12} = 25.1\ mm > 25mm > 1.5db = 1.5 * 16 = 24mm$$

The most appropriate detailing for beam 1 is figure:

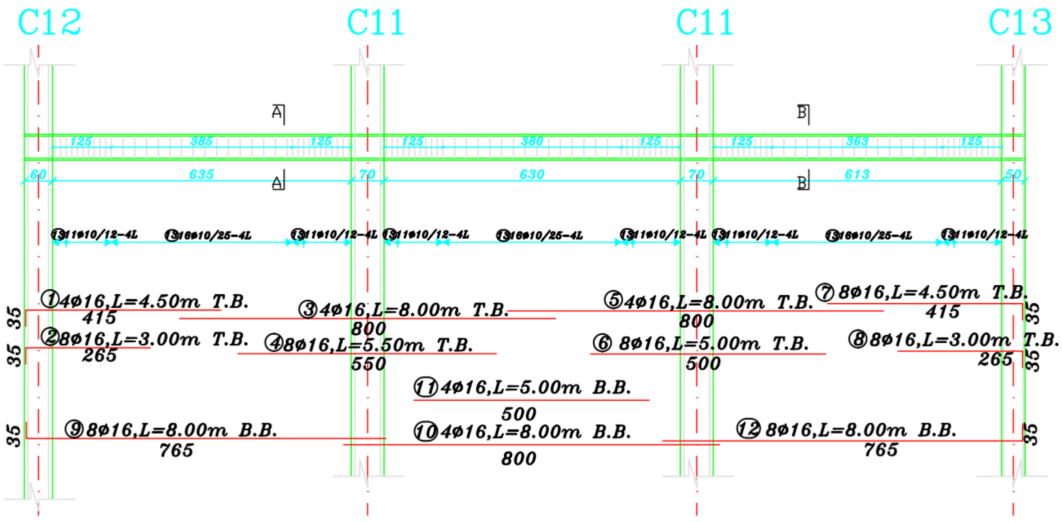

Figure 4-15 the Reinforcement of Beam 1

4.6.7 Shear Design of beam 1 (first span)

Design for shear forces corresponding to end moments that are calculated by assuming the stress in the tensile flexural reinforcement equal to Fy and a strength reduction factor, f = 1.0 (nominal flexural strength), plus shear forces due to factored tributary gravity loads.

The following equation can be used to compute Mn:

$$M_n = AsFy(d - \frac{a}{2})$$

$$where\ a = \frac{AsFy}{0.85fc'b}$$

For example, for sidesway to the right, the interior joint must be subjected to the negative moment Mn which is determined as follows:

For 12Ø16,As=12×201=2412 mm

$$where\ a = \frac{2412*420}{0.85*24*60} = 82.76$$

$$M_n = AsFy(d - \frac{a}{2}) = 2412 * 420 * (492 - \frac{82.7}{2}) * 10^{-6} = 456.493 \text{ KN.m}$$

Similarly, for the near joint, the positive moment Mn based on the 8Ø16 bottom bars is equal to 313.645 KN.m. The probable flexural strengths for sidesway to the left can be obtained in a similar fashion.

Wu =1.2D+1.0L

$$w_{DL,from\ rib29} = \frac{35.11}{0.52} = 67.52\ KN/m$$

Wu= 1.2*67.52+4=85.02

Reactions=

$$= \frac{85.02 * 7}{2} = 297.58\ KN/m$$

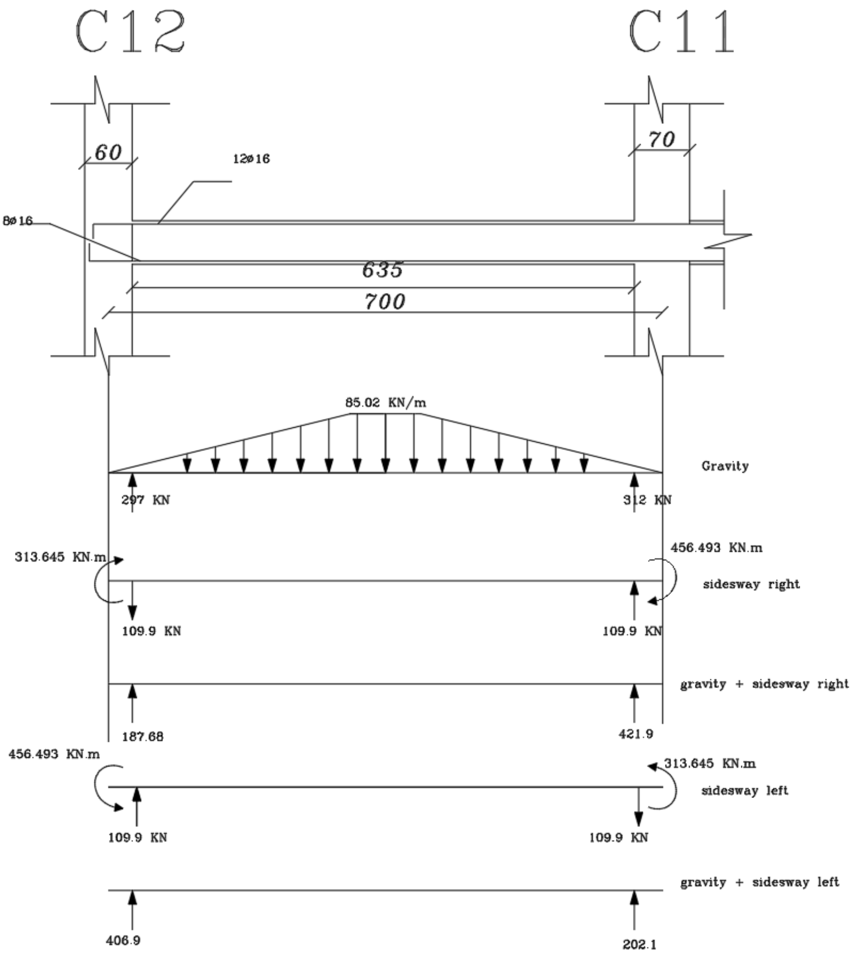

Figure 4-16 shear forces at exterior span

Figure 29-28 shows the exterior beam span and the shear forces due to the gravity loads. Also shown are the nominal flexural strengths Mn at the joint faces for sidesway to the right and to the left and the corresponding shear forces due to these moments.

$$V_c = \frac{1}{6}\lambda\sqrt{f_c'}b_w d = \frac{1}{6} * 1 * \sqrt{24} * 600 * 492 * 10^{-3} = 241.023\ KN$$

Check for section dimensions:

$$V_S = \frac{V_u}{\emptyset} - V_C = \frac{421.9}{0.75} - 241.023 = 321.51\ KN$$

where the strength reduction factor φ is 0.75.

$$V_{s,max} = \frac{2}{3}\sqrt{f_c'}b_w d = \frac{2}{3}\sqrt{24} * 600 * 492 * 10^{-3} = 964.12\ KN$$

Use stirrups (4legs stirrups) **Ø10** with $A_v = 4 * 78.57 = 314.29\ mm^2$

$$\frac{A_V}{s} = \frac{V_s}{f_{yt}d} \Longrightarrow s = \frac{A_v f_{yt} d}{V_s} = \frac{4 * 0.11 * 420 * 492}{321.51} = 282.79\ mm$$

Note that 4 legs are required for lateral support of the longitudinal bars. 18.4.2.4 Maximum allowable hoop spacing (smax) within a distance of 2h = 2*55 = 110 cm. from the face of the support is the smallest of the following:

smax

= d/4=123mm (governs)

= 8*(diameter of smallest longitudinal bar) = 8*16 = 128mm

= 24*the diameter of the hoop bar= 24*10=240 mm

= 300 mm

. Therefore, hoops must be spaced at 12 cm. on center with the first one located at 5 cm. from the face of the support. ten hoops are to be placed at this spacing. Where hoops are no longer required, stirrups with seismic hooks at both ends may be used.

Take 2U-shape (4legs stirrups) Ø10 @125mm

4.7. Column Design

4.7.1. Dimension and longitudinal Reinforcement

According to combination of (Envelope) max Pu= 547 KN

Ag*f'c /10 = (50 * 50) *24/10 = 6000 kips < Pu, Thus, the provisions of 18.4.3.Apply.

Determine required longitudinal reinforcement. Based on the Envelope load combination, a 50 * 50 cm. column with 18Ø18 (ρg = 1.70%) is adequate for the column supporting the first floor level. Note that 0.01 < ρg ≤ 0.08 O.K.

* transverse reinforcement requirements 18.4.3.3

4.7.2 Confinement reinforcement

Transverse reinforcement for confinement is required over a distance l_o from the column ends where

Lo ≥

Max cross sectional dimension od column = 500 mm

. 1/6 (clear height) = (4.5)/6 = 750 mm. (governs)

450 mm

Maximum allowable spacing of rectangular hoops assuming Ø10 hoops

S max =

= 8*(diameter of smallest longitudinal bar) = 8*18 = 144mm

= 24*the diameter of the hoop bar= 24*10=240 mm

= 300 mm

= one half of the smallest Max cross sectional dimension of column = 500 * 0.5= 250mm

4.7.3 Transverse reinforcement for shear

As in the design of shear reinforcement for beams, the design shear for columns is 18.4.3.1 based not on the factored shear forces obtained from a lateral load analysis but rather on the nominal flexural strengths provided in the columns. The column design shear forces shall be determined from the consideration of the maximum forces that can be developed at the faces of the joints, with the probable flexural strengths calculated for the factored axial compressive forces resulting in the largest moments acting at the joint faces. The largest probable flexural strength that may develop in the column can conservatively be assumed to correspond to the balanced point of the column interaction diagram.

With the strength reduction factors equal to 1.0 and fy = 1* 420 = 420 Mpa, the moment corresponding to balanced failure is 835 KN.m. Thus, Vu = (2 * 835)/4.5 = 371.1 KN.

4.7.4. Intermediate resisting frame Joint design :

According to ACI 318-14 (15.2.2):

“ If gravity load , wind ,earthquake , or other lateral forces cause transfer of moment at beam-column or slab-column joints , the shear resulting from moment transfer shall be considered in the design of the joint .”

Check on joints at frame 14– the circled one

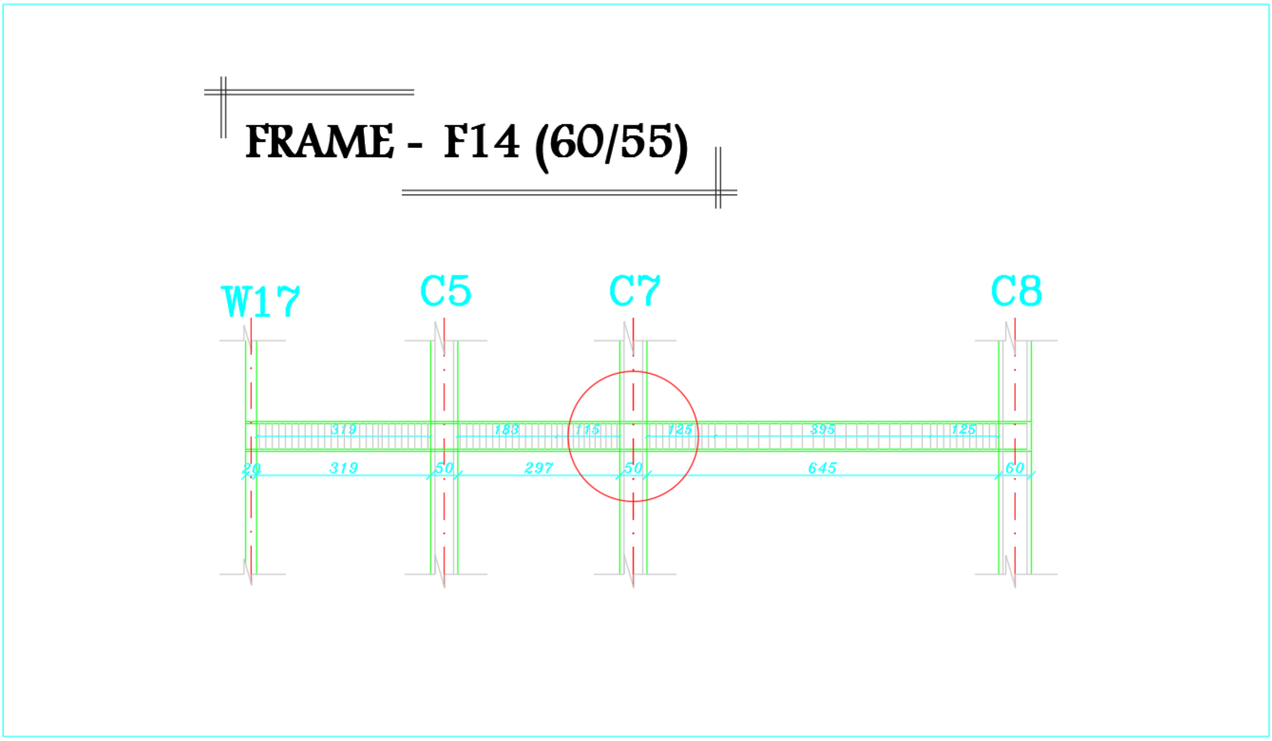

Figure 4-17 Joint location

The joint is to be type 1 (intermediate frame)

$f_c' = 24\ Mpa$

$f_y = 420\ Mpa$

ɣ=20

The height of the floor equal 4.5m and the clear height equal 3.95m And the reinforced of the frame joint as shown at the figure below :

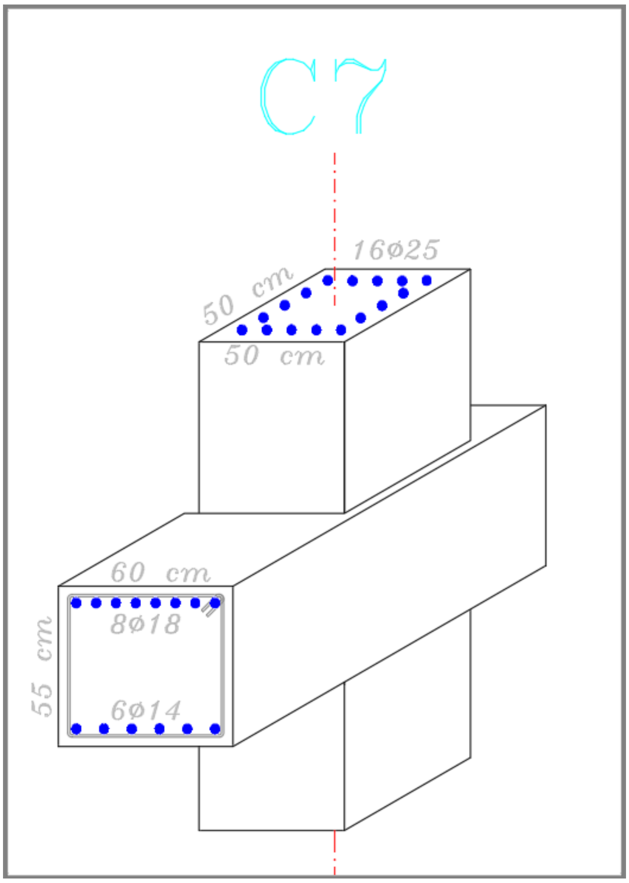

Figure 4-18 Joint Reinforcement

T=C

$As * Fy = 0.85 * f_c' * b * a$

8*254.5*420=0.85*24*600*a

a=69.82 mm

$\beta = 0.85 - 0.007(28 - 24) = 0.85$

$C = \frac{69.82}{0.85} = 82.14\ mm$

d=600-40-10-18/2=491mm

$\epsilon_s = \frac{d - c}{c} * 0.003 = \frac{491 - 82.14}{82.14} * 0.003 = 0.0149 > 0.005 - OK$

$M_{npr} = As * Fy * \left(d - \frac{a}{2}\right) = 8 * 254.5 * 420 * \left(491 - \frac{69.82}{2}\right) * 10^{-6} = 389.77\ KN.m$

$$V_{col} = \frac{M_{npr}}{\left(\frac{h_1 + h_2}{2}\right)} = \frac{389.77}{\left(\frac{4.5 + 3.95}{2}\right)} = 92.25\ KN$$

$$T = 8 * 254.5 * 420 * 10^{-3} = 854.58\ KN$$

$$JOINT\ V_U = 854.58 - 92.25 = 762.33\ KN$$

❖ Effective joint width . bj

i. $(b\ beam\ +\ b\ col\ )/2\ =\ (600 + 500)/2 = 550\ mm$

ii. $\left(beam\ + \frac{mh}{2}\right)$

$$\frac{mh}{2} = \frac{0.5 * 550}{2} = 137.5\ mm$$

Extension of the beam beyond the beam edge = (600-500)/2=50 mm – control

$$\left(beam\ + \frac{mh}{2}\right) = 600 + 2 * 50 = 700\ mm$$

iii. $b_{col} = 500mm$ - **control**

$$V_n = \gamma\sqrt{f_c'} * b_j * h = \frac{20}{12} * \sqrt{24} * 500 * 550 * 10^{-3} = 2041.24\ KN$$

$$\emptyset V_n = 0.75 * 2041.24 = 1530.93\ KN > JOINT\ V_U = 761.32\ KN - ok$$

❖ b beam ≥ 3/4 b col

600 mm ≥ 3/4*500 = 375 mm

∴ **the joint is confined**

4.8. shear wall Design

Analysis and design were done using ETABS program in which the seismic loads were taken into account. The following is a sample calculation for one of the walls, S.W8.

The following data that used in design :

- Shear Wall thickness = h = 25 cm
- Shear Wall length Lw = 6.5 m
- Building height Hw=43.8 m
- Critical section shear : Lw<hw → d =0.8*Lw =5.20 m
- $f_c' = 28\ Mpa\ , f_y = 420\ Mpa$

4.8.1. Design of Horizontal Reinforcement

Calculation of Shear Strength Provided by concrete Vc:

- Shear Strength of Concrete is the smallest of :

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

= $\frac{1}{6}\sqrt{28} \times 250 \times 5200 \times 10^{-3}$ $= \mathbf{1146.49\ kN}$ ≪ Controlled

2- Vc = $0.27\sqrt{f_c'}hd + \frac{Nu \times d}{4Lw}$

= $0.27 * \sqrt{28} * 250 * 5200 + 0 = 1857.32\ kN$

3- $Vc = \left[\frac{\sqrt{fc'}}{2} + \frac{Lw\left(0.1\sqrt{fc'} + \frac{2Nu}{Lw.h}\right)}{\frac{Mu1}{Vu} - \frac{Lw}{2}}\right] \times hd$

Where :

- Mu1=857.02 kN.m

- $\frac{Mu1}{Vu} - \frac{Lw}{2} = \frac{857.02}{342.92} - \frac{5.2}{2} = -0.817 < 0$ → This equation is not applicable .

∴ $Vc = 1146.49$ kN → ØVc $>$ Vumax[7] = 342.92kN →Horizontal Reinforcement is the minimum .

$$\left(\frac{\text{Avh}}{\text{s}}\right)\text{min} = 0.002 * \text{h} = 0.0025 * 250 = \mathbf{0.625}$$

→ Avh : For 2 layers of Horizontal Reinforcement

Select Ø10 :

Avh = 2 *79 = 158 mm^2

$$\frac{Avh}{s} = 0.5 \rightarrow Sreq = \frac{158}{0.625} = 252.8\ mm$$

Smax= Lw/5 = 6500/5 = 2166.67 mm

= 3h = 3*250 =750 mm

$= 45cm$ ≪ Controlled.

∴**Select Ø10 @ 250 mm at each side .**

4.8.2. Design of Vertical Reinforcement

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

$$\frac{hw}{lw} = \frac{43.8}{6.5} = 6.738 > 2.50$$

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

∴ $\frac{\text{Avv}}{\text{Sver}} = 0.625$

Smax = Lw/3 =6500/3 = 2166.66 mm

= 3h = 3*250 =750 mm

$= 450\ mm \ll$ Controlled.

Select Ø12 :

Avv = 113.097

$$\frac{\text{Avv}}{\text{s}} = 0.625 \rightarrow \text{Sreq} = \frac{113.097}{0.625} = 180.955 \text{ mm}$$

∴**Select Ø12 @ 185 mm at each side .**

4.8.3. Design of Bending Moment

Moment diagram were obtained from ETABS

→ Max Mu =857.02 kN.m

→ Part of Moment that resisted through Avv :

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

Where :

- Asv=2*113 *$\frac{6500}{250}$ = 5876 mm^2
- $\frac{Z}{Lw} = \frac{1}{2+\frac{0.85*\beta1*fc'*Lw*h}{Asv*fy}} = \frac{1}{2+\frac{0.85*0.85*28*6500*250}{5876*420}} = 0.065$

∴ Muv = 0.9 [0.5*5876*420*6500 (1-$\frac{0.065}{2}$)] = 6983.07 kN.m

Muv = 6983.07 kN.m > Mu =857.02 kN.m

So, Boundary Element is not required. #

4.9. Basement wall Design

4.9.1. System and load :

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

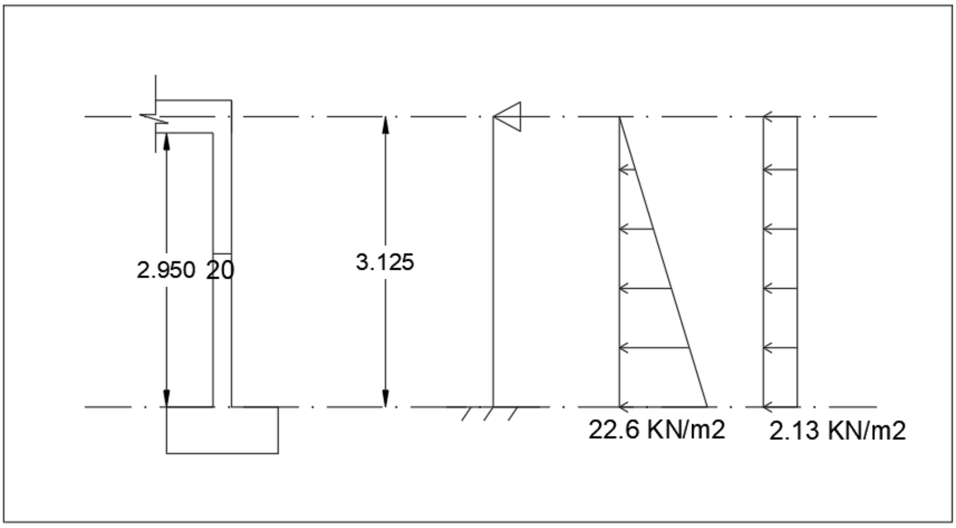

Figure 4-19 Basement wall structure system

The different lateral pressures on a 1m length of the wall are calculated as follows:

$k_o = 1 - \sin 35 \ = 0.426$

Due to soil pressure at rest : qu1 = $k_o.\gamma.h$ = 0.426*17*3.125=22.6 kN/m^2

Due to surcharge : qu2 = 5*0.426 =2.13 kN/m^2

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

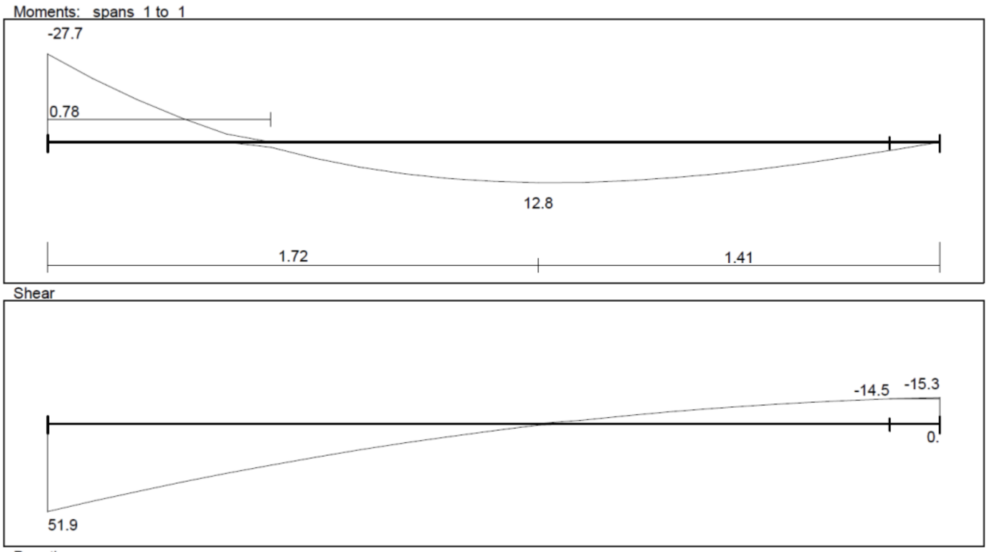

Figure 4-20 Moment diagram of basement wall

4.9.2. Shear Design of Basement wall

Max value shear force is , Vu= 51.9 kN

d=200-50-20/2 = 140 mm

Ø * Vc = 0.75 * $\frac{1}{6}$ * $\sqrt{28}$ * 1000 * 140 = 92.06 kN > Vu

∴ take h=20cm.

4.9.3. Flexure Design of Basement wall

- (Max Negative Moment= 27.7 KN.m)

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{27.7 * 10^6}{0.9 * 1000 * 140^2} = 1.57\ MPa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 28} = 17.65$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 1.57 * 17.65}{420}}\right) = 0.00387$$

$$A_s = \rho b d = 0.00387 * 1000 * 140 = 541.84\ mm^2/m$$

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{28}}{420} * 1000 * 140 = 440.96\ mm^2/m$$

$A_{s,min} = \frac{1.4}{420} * 1000 * 140 = 466.67 mm^2/m$ –control

$$A_{s,min} = 0.0012 * bh = 0.0012 * 1000 * 200 = 240\ mm^2/m$$

$A_s = 541.84\ mm^2/m > A_{s,min} = 466.67 mm^2/m$ –OK

$$n = \frac{541.84}{154} = 3.52\ , \quad take\ 4\phi 14 \quad or\ \phi 14/250 mm$$

- (Max positive Moment= 12.8 KN.m)

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{12.8 * 10^6}{0.9 * 1000 * 140^2} = 0.725\ MPa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 28} = 17.65$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 0.725 * 17.65}{420}}\right) = 0.00175$$

$A_s = \rho bd = 0.00175 * 1000 * 140 = 245.46\ mm^2/m$

Check for $A_{s,min}$

$$A_{s,min} = 0.25 \frac{\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4}{f_y} b_w d$$

$$A_{s,min} = 0.25 \frac{\sqrt{28}}{420} * 1000 * 140 = 440.96\ \ mm^2/m$$

$A_{s,min} = \frac{1.4}{420} * 1000 * 140 = 466.67\ mm^2/m$ –control

$A_{s,min} = 0.0012 * bh = 0.0012 * 1000 * 200 = 240\ mm^2/m$

$$A_s = 245.46\ \frac{mm^2}{m} < A_{s,min} = 466.67\ mm^2/m$$

$$n = \frac{466.67}{153.9} = 3.03\ , \quad take\ 4\phi 14 \quad or\ \phi 14/250mm$$

The minimum horizontal As according to the ACI Code , Section 14.3 is

As=0.002bh=0.002*200*1000=400 mm^2/m

$n = \frac{400}{113} = 3.54\ , \quad take\ 4\phi 12 \quad or\ \frac{\phi 12}{250mm}\ on\ each\ side$.

4.10. Strip footing Design

4.10.1 . Design Criteria

$f_c' = 28\ Mpa$
$f_y = 420\ Mpa$
$q_{all} = 350\ KN/m^2$
$thickness\ of\ wall = 20\ cm$
$\gamma\ soil = 18\ KN/m^3$
$\gamma\ concrete = 25\ KN/m^3$
$base\ level = 0.6\ m$

Surcharge = 5 KN/m^2

4.10.2. Loads

D.L. = 270.726 KN/m
L.L.= 7.86 KN/m
M.D.=2.47 KN.m
M.L.=0.12 KN.m
Pu= 443.65 KN
Mu=3.84 KN.m

4.10.3. The size of the footing

Assume the thickness of the strip footing is 25cm

$q_{a,net} = 250 - 5 - (25 * 0.25) - (0.6 * 18) = 327.95\ KN/m2$

$$q = \frac{P}{A} + \frac{6M}{Bl^2} = \frac{270.73 + 7.86}{1 * l} + \frac{6 * (2.47 + 0.12)}{1 * l^2} = 327.95\ KN/m2$$

$l = 0.789m \rightarrow \boldsymbol{take\ l = 0.8\ m}$

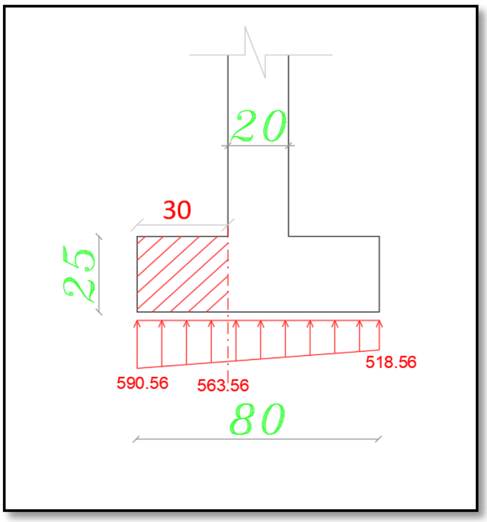

Figure 3-21 : Critical section of shear force

- Depth of footing (One-way shear check)

$$q_{max} = \frac{P_u}{l} + \frac{6M_u}{l^2} = \frac{443.65}{0.8} + \frac{6*3.84}{0.8^2} = 590.56\ KN/m^2$$

$$q_{min} = \frac{P_u}{l} - \frac{6M_u}{l^2} = \frac{443.65}{0.8} - \frac{6 * 3.84}{0.8^2} = 518.56\ KN/m^2$$

$d_{avg} = 250\ - 75 - 20 = 155\ mm$

Vu at distance d from the face of the support :

$$V_u = 590.56 * \left(\frac{0.8}{2} - \frac{0.2}{2} - 0.155\right) = 85.632\ \text{KN}$$

$$\emptyset V_c = \emptyset \frac{1}{6}\sqrt{f_c'} * b_w d = 0.75 * \frac{1}{6} * \sqrt{28} * 1000 * 155 * 10^{-3} = 102.52KN$$

The footing is OK for one-way shear

- Design for flexure

Take steel bars of ф20

Critical section for design moment is :

$$l = \frac{0.8}{2} - \frac{0.2}{2} = 0.3\ m$$

$q_{u,at\ the\ face} = 563.56\ KN/m^2$

$$M_u = 563.56 * \frac{0.3^2}{2} + \frac{1}{2}(590.56 - 563.56) * 0.3 * \frac{2}{3} * 0.3 = 26.17\ KN.m$$

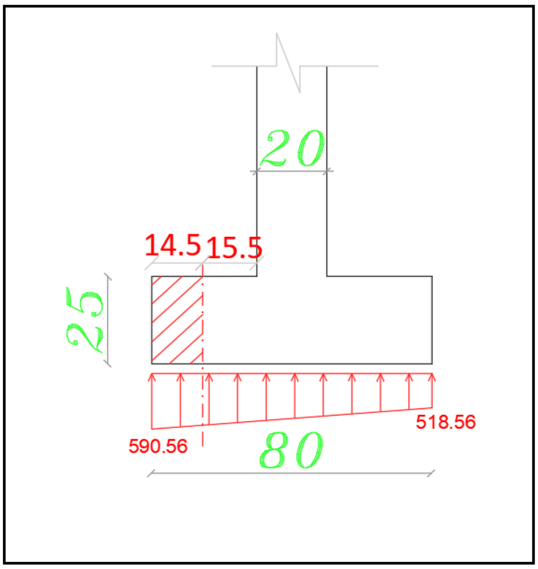

Figure 4-22 Bending moment Critical Section

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{26.17 * 10^6}{0.9 * 1000 * 155^2} = 1.21\ Mpa$$
$$m = \frac{f_y}{0.85 * f_c'} = \frac{420}{0.85 * 28} = 17.64$$
$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{17.64}\left(1 - \sqrt{1 - \frac{2 * 17.64 * 1.21}{420}}\right) = 0.00296$$
$$A_s = \rho b d = 0.00296 * 1000 * 155 = 458.64\ mm^2/m$$
$$A_{s,min} = 0.0018 b h = 0.0018 * 1000 * 250 = 450\ mm^2/m$$
$$A_S = 458.64 mm^2 > A_{s,min} = 450\ mm^2 - OK$$
$$n = \frac{A_s}{A_{S\emptyset 12}} = \frac{458.64}{113.1} = 4.06$$
$$s = \frac{1}{n} = \frac{1}{4.06} = 0.247m$$
Take ф12@200 mm

Smax the smallest of :

1. 3h=3*250 =750 mm
2. 450 mm – control

- Select the minimum reinforcement (temperature)

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

$$n = \frac{A_s}{A_{S\emptyset 12}} = \frac{450}{113.1} = 3.979$$
$$s = \frac{1}{n} = \frac{1}{3.979} = 0.251m$$
Take ф12@250 mm

Smax the smallest of :

1. 5h=5*250 = 1250 mm
2. 450 mm – control

4.11. Isolated footing

4.11.1. Isolated footing design :

Loads that act on footing F1 are :

- PD =5008.86 kN , PL = 761.96 kN , Mx sur =5.7 KN/m2 , My sur = 3.66 KN/m2
- Pu = 7495.89 kN , Mx = 7.97 KN/m2 , My = 4.75 KN/m2

The following parameters are used in design :

- γ_{concrete} = 25 kN/m^3
- γ_{soil} = 18 kN/m^3

- σ_{allow} = 350 kN/m^2
- clear cover =7.5cm
- the diminution of column is 70*70 cm
- service surcharge = 5 KN/m2
- soil depth = 0.60 cm
- $f_c' = 28\ MPA \quad , \quad f_y = 420\ MPA$

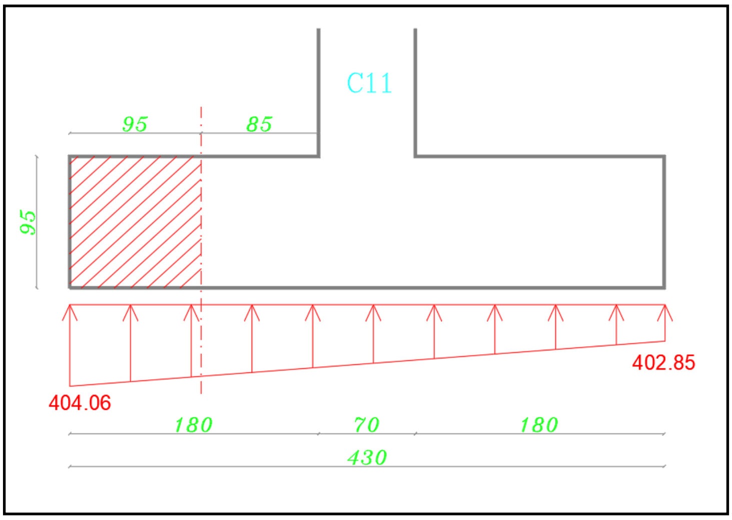

Figure 4-23:Critical Section of one way shear

4.11.2. Determination of footing dimension

Assume the thickness of the isolated footing =0.95 m

$$q_{all,net} = 350 - 5 - (0.6 * 18) - (0.95 * 25) = 310.45\ KN/m^2$$

Assume the soil pressure distribution at the base of the footing is trapezoidal (no tension zone) .

Assume the width of the footing at the x-axis b= 3.30 m , so the width at y- direction is :

$$q_{max} = \frac{P}{Bl} + \frac{6M}{Bl^2} = \frac{5008.86+761.96}{4.3*l} + \frac{6*5.7}{4.3*l^2} = 310.45 \rightarrow l = 4.3m$$

The width at x- dir. Equal

$$q_{max} = \frac{P}{Bl} + \frac{6M}{Bl^2} = \frac{5008.86 + 761.96}{4.3 * l} + \frac{6 * 3.66}{4.3 * l^2} = 310.45 \ \rightarrow l = 4.3m$$

$$\therefore \boldsymbol{the\ footing\ dimentions\ is\ 4.3 * 4.3\ m}$$

4.11.3. The factored soil pressure

$$q_{max,X} = \frac{P_u}{Bl} + \frac{6M_{u,Y}}{Bl^2} = \frac{7459.89}{4.3^2} + \frac{6 * 4.75}{4.3^3} = 403.81\ \text{KN}/m^2$$

$$q_{min,X} = \frac{P_u}{Bl} - \frac{6M_{u,Y}}{Bl^2} = \frac{7459.89}{4.3^2} - \frac{6 * 4.75}{4.3^3} = 403.09\ \text{KN}/m^2$$

$$q_{max,Y} = \frac{P_u}{Bl} + \frac{6M_{u,Y}}{Bl^2} = \frac{7459.89}{4.3^2} + \frac{6 * 7.97}{4.3^3} = 404.057\ \text{KN}/m^2$$

$$q_{min,Y} = \frac{P_u}{Bl} - \frac{6M_{u,Y}}{Bl^2} = \frac{7459.89}{4.3^2} - \frac{6*7.97}{4.3^3} = 402.85\ \text{KN}/m^2$$

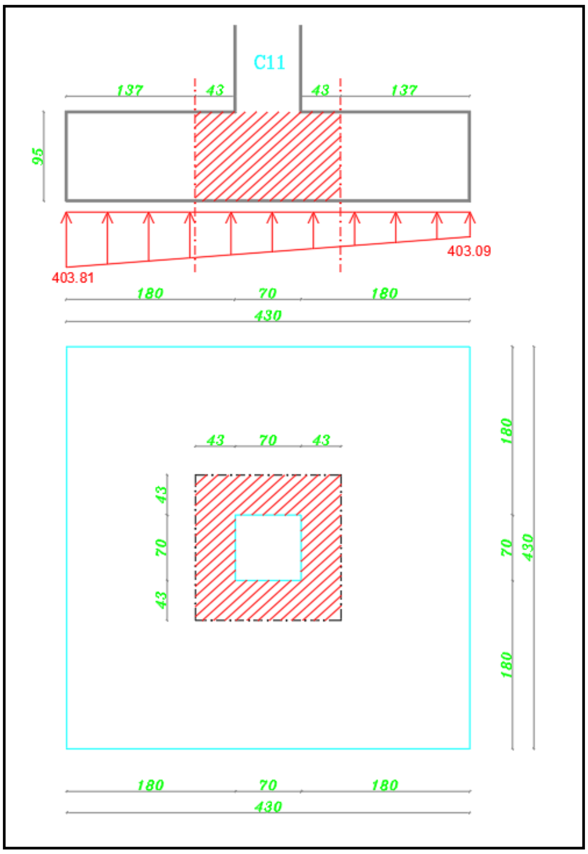

Figure 4-24:Critical Section of two way shear

4.11.4.One-way shear check

$$d_{avg} = 950 - 75 - 25 = 850mm$$

$$V_{u,X} = 403.81 * 4.3 * \left(\frac{4.3}{2} - \frac{0.7}{2} - 0.85\right) = 1649.57\ KN$$

$$V_{u,Y} = 404.06 * 4.3 * \left(\frac{4.3}{2} - \frac{0.7}{2} - 0.85\right) = 1650.55 KN$$

$$\emptyset V_c = \emptyset \frac{1}{6}\sqrt{f_c'} b_w d$$

$$= 0.75 * \frac{1}{6} * \sqrt{28} * 4300 * 0.85 * 10^{-3} = 2417.55\ KN$$

$$\emptyset V_c = 2417.55 > V_u = 1650.55\ KN \quad \therefore \textit{the thickness is enought}$$

4.11.5 Two-way shear check

$$\beta = \frac{700}{700} = 1$$

$$b_o = 4 * (0.7 + 0.85) = 6.2\ m$$

$$\propto_s = 40 - \textit{interior column}$$

$$q_{av} = \frac{403.81 + 403.09}{2} = 403.46\ KN/m^2$$

$$V_u = (403.46 * 4.3 * 4.3) - (403.46 * (0.7 + 4.3)^2) = 6490.588\ KN$$

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

$$\textit{where}: \frac{1}{6}\left(1 + \frac{2}{\beta}\right) = \frac{1}{6}\left(1 + \frac{2}{1}\right) = 0.5$$

$$V_c = \frac{1}{12}\left(2 + \frac{\propto_s d}{b_o}\right)\sqrt{f_c'} b_o d\ , \quad \textit{where}: \frac{1}{12}\left(2 + \frac{\propto_s d}{b_o}\right) = \frac{1}{12}\left(2 + \frac{40 * 0.85}{6.2}\right) = 0.624$$

$$V_c = \frac{1}{3}\sqrt{f_c'} b_o d\ , \quad \textit{where}: \frac{1}{3} = 0.333 - \textit{control}$$

$$\emptyset V_c = \emptyset \frac{1}{3}\sqrt{f_c'} b_o d = 0.75 * \frac{1}{3}\sqrt{28} * 6.2 * 850 = 6971.55\ KN$$

$$\emptyset V_c = 6971.55\ KN > V_u = 6490.588\ KN \quad \therefore \textit{The thickness if enought}$$

4.11.6 Design for flexure

- X- direction

d = 950-75-(25/2)=862.5 mm
width of critical section = (4.3/2)-(0.7/2)= 1.8 m
q at 1.8 m = 403.51 KN/m2

$$M_u = 403.51 * 4.3 * \frac{1.8^2}{2} + \frac{1}{2}(403.81 - 403.51) * 1.8 * \frac{2}{3} * 1.8 * 4.3 = 2812.27\ KN.m$$

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{2812.27 * 10^6}{0.9 * 4300 * 862.5^2} = 0.977\ MPa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 28} = 17.65$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 0.977 * 17.65}{420}}\right) = 0.0023$$

$$A_s = \rho bd = 0.0023 * 4300 * 862.5 = 8810.62\ mm^2 - contol$$

Check for $A_{s,min}$

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

$$= 0.0018 * 4300 * 950 = 7353\ mm^2$$

∴ use 18 ф 25

$s = \frac{4300-75*2-18*25}{17} = 217.65\ mm -$ ok

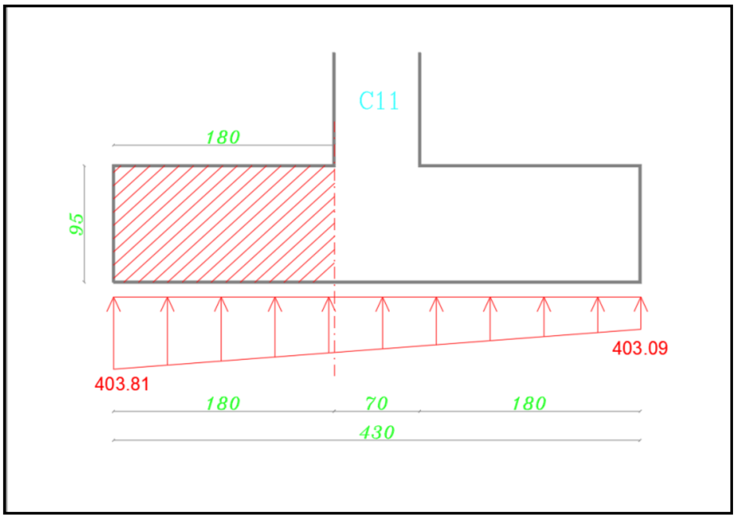

Figure 4-25: Bending Moment Critical Section

s_{max} *is the smallest of*

1. 3h = 3*950=2850 mm
2. 450 mm – control

❖ Y- direction

d = 950-75-25-(25/2)=837.5 mm
width of critical section = (4.3/2)-(0.7/2)= 1.8 m
q at 1.8 m = 403.55 KN/m2

$$M_u = 403.55 * 4.3 * \frac{1.8^2}{2} + \frac{1}{2}(404.05 - 403.55) * 1.8 * \frac{2}{3} * 1.8 * 4.3 = 2813.4\ KN.m$$

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{2812.4 * 10^6}{0.9 * 4300 * 837.5^2} = 1.036\ MPa$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2 * 1.036 * 17.65}{420}}\right) = 0.0025$$

$$A_s = \rho bd = 0.0025 * 4300 * 837.5 = 9089.413\ mm^2 - contol$$

Check for $A_{s,min}$

$A_{s,min} = 0.0018bh = 0.0018 * 4300 * 950 = 7353\ mm^2$

∴ use 19 ϕ 25

$s = \frac{4300-75*2-19*25}{18} = 204.17\ mm$ – ok

s_{max} *is the smallest of*

1. 3h = 3*950=2850 mm
2. 450 mm – control

4.12. Stairs Design

The stairs at the middle of the hotel:

4.12.1. The structure system :

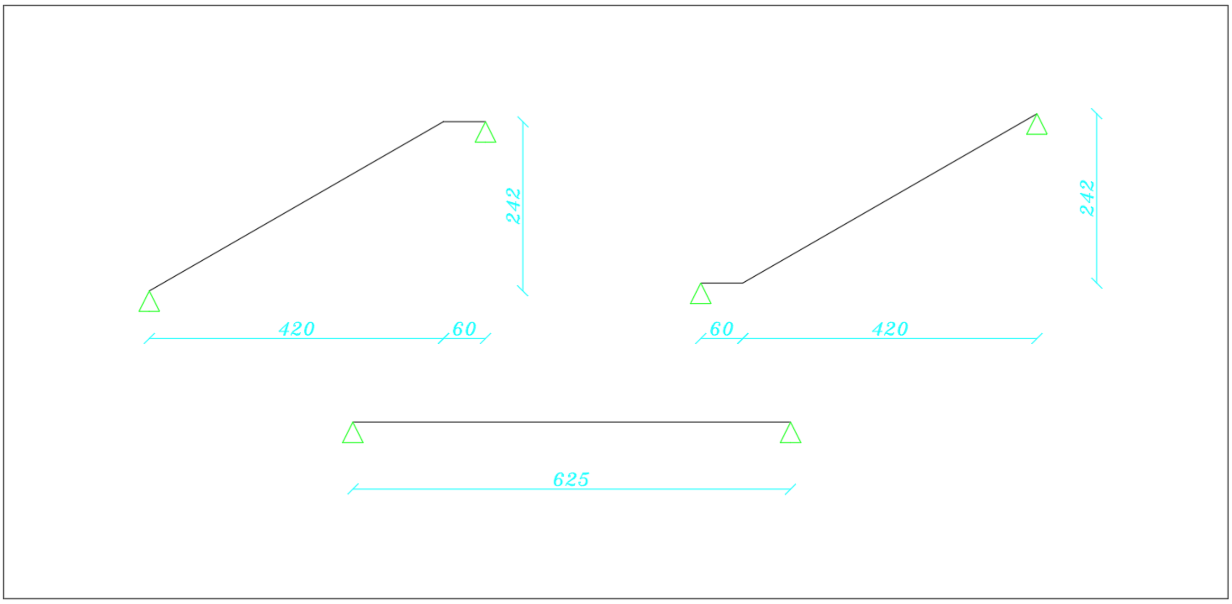

Figure 4-26 Stairs structure system

❖ Materials properties :

f_c'=24 MPa

f_y=420 MPa

The thickness of stairs using beam d =25 cm

4.12.2. Load calculations:

➢ Live load = 4 KN/m

$$\theta = \tan^{-1}\left(\frac{rise}{run}\right) = \tan^{-1}\left(\frac{2.42}{4.2}\right) = 29.95^{\circ}$$

- Dead load for flight :

Table 4-4 Dead load for flight

Material	Density	W (KN/m)
Tiles	23	$23 \times \left(\frac{0.173+0.3}{0.3}\right)$*0.03=1.088
Mortar	22	$22 \times \left(\frac{0.173+0.25}{0.3}\right)$*0.02=0.62
Stair steps	25	$\frac{25}{0.3} * \left(\frac{0.173*0.25}{2}\right) = 1.8$
Reinforced Concrete solid slab	25	$\frac{25*0.25}{\cos 29.95} = 7.21$
Plaster	22	$\frac{22*0.03}{\cos 29.98} = 0.762$
Total Dead load , KN/m		11.48

- Dead load for landing :

Table 4-5 Dead load for landing

Material	Density	W (KN/m)
Tiles	23	0.69
Mortar	22	0.44
Reinforced Concrete solid slab	25	6.25
Plaster	22	0.66
Total Dead load , KN/m		8.04

- Total factored load :

For flight :

$$w = 1.2D.L + 1.6\,L.L = (1.2 * 11.48) + (1.6 * 4) = 20.176\,KN/m$$

For landing :

$$w = 1.2D.L + 1.6\,L.L = (1.2 * 8.04) + (1.6 * 4) = 16.048KN/m$$

4.12.3. The structure analysis :

- For landing

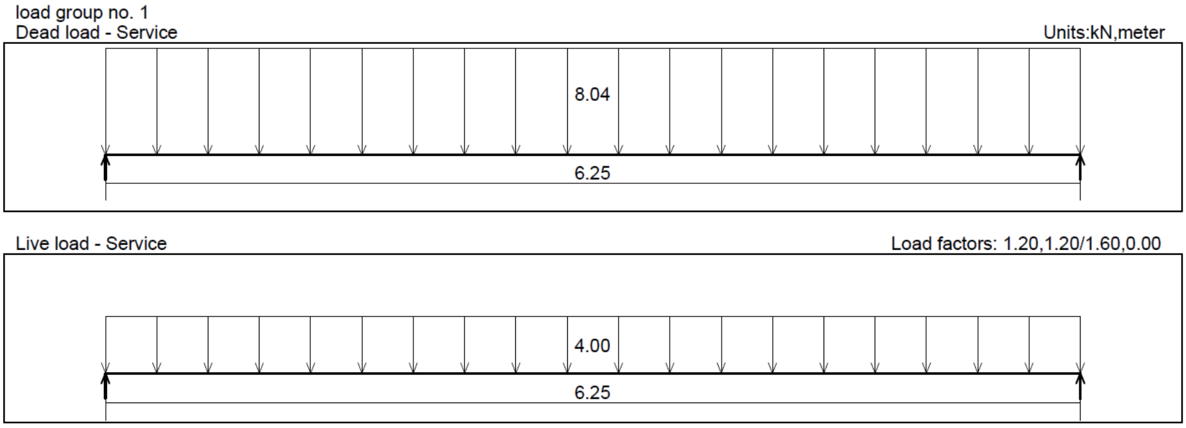

Figure 4-27 Service load for landing

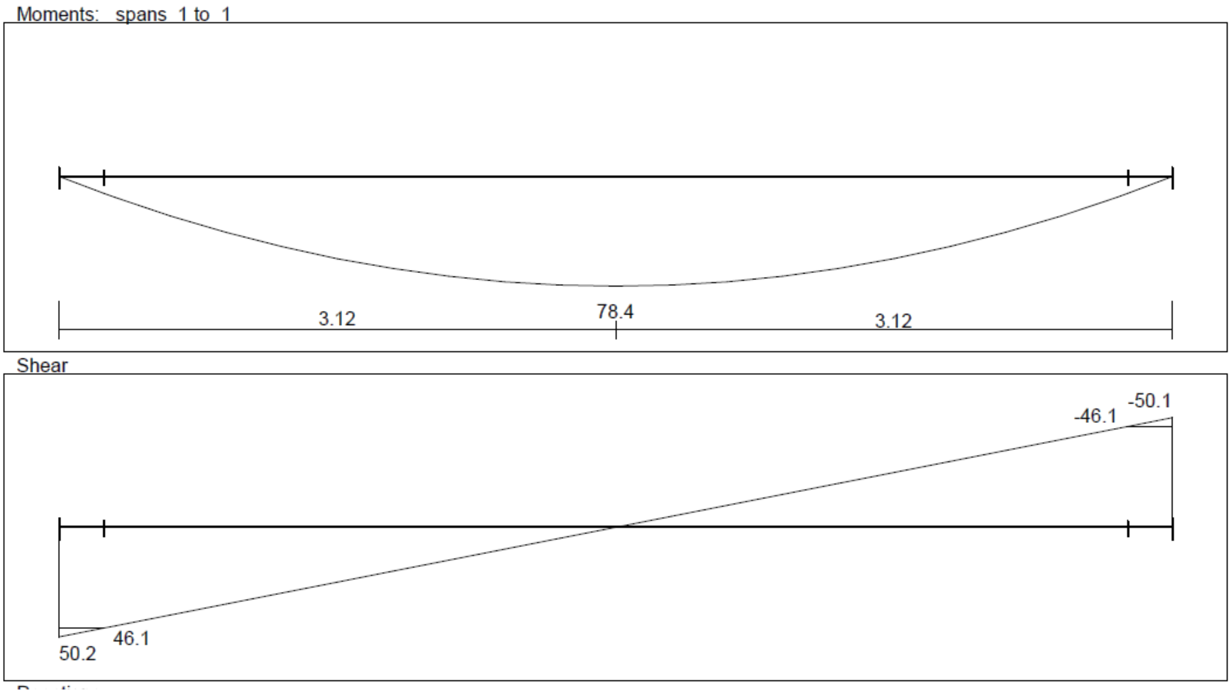

Figure 4-28 Shear and Bending moment diagram for landing

- For flight

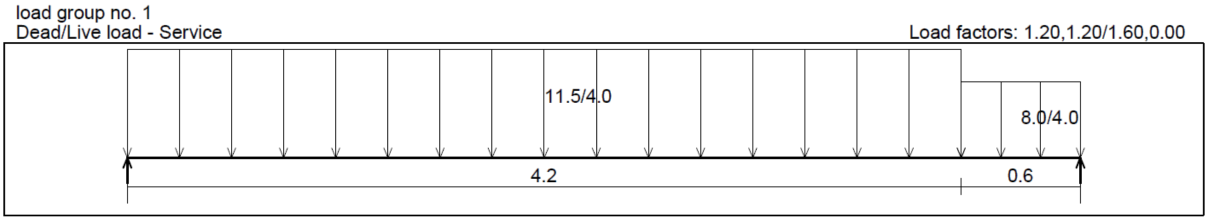

Figure 4-29 Service Load for flight

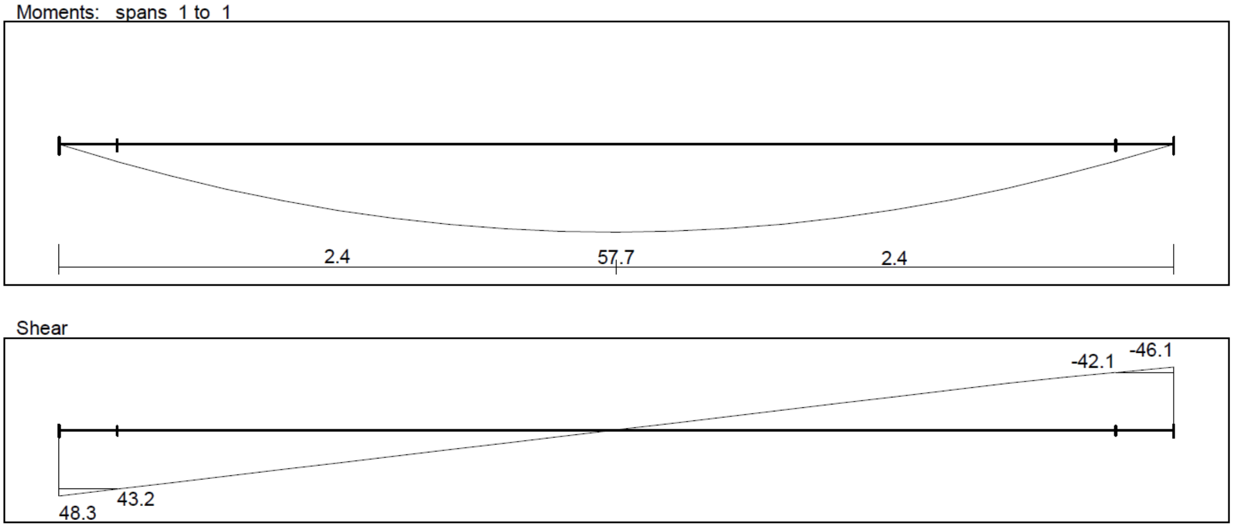

Figure 4-30 Shear and Bending moment for flight

❖ Check for shear strength :

Assume db. ϕ14

d= 250-20-(14/2)=223mm

Assume beam width 50 cm

Max shear V_u=46.1 KN

$$V_c = \frac{1}{6}\lambda\sqrt{f_c'}b_w d = \frac{1}{6} * 1 * \sqrt{24} * 1000 * 273 * 10^{-3} = 222.90\ KN$$

$$\emptyset(Vc) = 0.75 * 222.90 = 167.18\ KN$$

$\boldsymbol{V_u} < \frac{\mathbf{1}}{\mathbf{2}}\emptyset(Vc) = 83.58\ KN$

The thickness of the slab is adequate enough .

- ❖ Reinforcement design :
- ❖ For landing :

(Max positive Moment= 78.4 KN.m)

$$R_n = \frac{M_u}{\emptyset bd^2} = \frac{78.4 * 10^6}{0.9 * 1000 * 273^2} = 1.688\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 1.688 * 20.59}{420}}\right) = 0.0042$$

$$A_s = \rho bd = 0.0042 * 1000 * 273 = 1146.79\ mm^2$$

Check for $A_{s,min}$

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

$$A_s = 1146.79\ mm^2 > A_{s,min} = 450\ mm^2$$

Use ϕ14

$$n = \frac{A_s}{A_{s\emptyset 16}} = \frac{1146.79}{201.1} = 5.70, \quad s = \frac{1}{n} = 0.175\ m$$

Take ϕ16@175mm

Steps is the smallest of :

1. 3h = 3*250=750mm
2. 450 mm
3. $s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = 380\left(\frac{280}{280}\right) - 2.5 * 20 = 330mm$
4. $s \leq 300\left(\frac{280}{f_s}\right) = 300 * \frac{280}{280} = 300mm - control$

S=175 mm < S_{max}=300mm - OK

- ❖ For Flight :

(Max positive Moment= 57.7 KN.m)

$$R_n = \frac{M_u}{\emptyset b d^2} = \frac{57.7 * 10^6}{0.9 * 1000 * 273^2} = 0.86\ MPa$$

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

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2R_n m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 * 0.86 * 20.59}{420}}\right) = 0.00209$$

$$A_s = \rho b d = 0.00209 * 1000 * 273 = 571.31\ mm^2$$

$$A_s = 571.31\ mm^2 > \ A_{s,min} = 450\ mm^2$$

Use ϕ14

$$n = \frac{A_s}{A_{s\emptyset 14}} = \frac{571.31}{153.9} = 3.71\ , \qquad s = \frac{1}{n} = 0.269\ m$$

Take ϕ14@250mm

Steps is the smallest of :

5. 3h = 3*250=750mm
6. 450 mm
7. $s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = 380\left(\frac{280}{280}\right) - 2.5 * 20 = 330mm$
8. $s \le 300\left(\frac{280}{f_s}\right) = 300 * \frac{280}{280} = 300mm - control$

S=250 mm < S_{max}=300mm - OK

➢ Temperature and shrinkage reinforcement .

$$A_s = 0.0018 * b * h = 0.0018 * 1000 * 250 = 450\ mm^2$$

$$n = \frac{A_s}{A_{s\emptyset 12}} = \frac{450}{113} = 3.98\ , \qquad s = \frac{1}{n} = 0.251\ m$$

Take ϕ12@250mm

Chapter 5 : Conclusion and Recommendation

5.1.Conclusion:

In the end of the final stages of our project we were able to understand the behavior of the building in the action of seismic loads and we were able to manage how to determine the percentage of seismic loads that will be resisted by the various structure components.

More importantly, we were able to implement the provisions of chapter 18 in ACI318-14 to translate the dual system (intermediate moment frames and special shear walls) into a constructible project which can be implemented easily in our country (Palestine).

5.2.Recommendation :

Based on the personal experience in analyzing, designing and detailing jericho's five-star hotel, we recommend to use the very exact seismic factors for Site of the building in the analysis stage, besides to use the dynamic analysis response spectra method because it is more conservative in designing structures in term of seismic design

5.3.References

[1] building code requirements for for structural concrete (ACI318-14), USA: American Concrete Instutiute,2014.

[2] كود البناء الاردني، كود الاحمال والقوى،عمان،الاردن: مجلس البناء الوطني الأردني،2006م

[3] ACI 318-11 Building Code Requirements for Structural Concrete and Commentary &PCA Notes On 318-11, USA: American Concrete Instutiute,2014.

[4] ACI DETAILING MANUAL MNL-66(20), USA: American Concrete Instutiute,2020.

[5] STANDARD ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures,2016.