بسم الله الرحمن الرحيم

Palestine Polytechnic University

College of Engineering

Civil Engineering Department

“The Seismic Design of Bethlehem General Hospital”

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Submitted to the College of Engineering

In partial fulfillment of the requirements for the

Bachelor degree in Building Engineering

Hebron, May 2023

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 Bethlehem General Hospital**, submitted by Dana Amjad Radwan and Leena Yahia Qfeeshah for partial fulfillment of the requirements for the bachelor's degree.

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DEDICATION

This project is lovingly dedicated to our parents, 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. Nasr Abboushi who was the best mentor who did not hesitate to help us, without him this project will not be the same. A special thanks to Eng. Mutaz Qafeshah who helped us to write this project to accommodate the scientific research requirements.

Abstract

As all structural engineering projects our project aims to find a suitable structural system for the residential building that we work on to be adequate for human use, our project aims to keep the building standing while facing the seismic and lateral loads.

Most of the buildings in our country are not designed to face seismic loads which is considered as a threat in our building's design and execution methods, in this project, we used ACI 318-19 building code requirements for structural concrete using chapter 18 which includes the earthquake building requirements, we started with work on the analysis of architectural plans in terms of entrances, exits, number of floors of the building, locations of inclusions and others, then finding the suitable columns and shear walls arrange in accordance with the architectural design, decide the loads for the building and start modeling it on Etabs, after that we must check on all the requirements that the ACI 318-14 chapter 18 contains like the maximum story drift and maximum deflection, finally detailing all the structural members of the building achieving the requirements.

As a result, we will have a design that satisfies all the ACI 318-14 requirements and the load provisions of the Jordinain code for buildings loads which will achieve an allowable story drift, from our result the design will keep the building sustainable for human use after an extreme earthquake.

الملخص

كسائر المشاريع الانشائية مشروعنا هذا يهدف الى إيجاد انسب نظام انشائي يناسب المبنى الذي بين أيدينا وهو مبنى مستشفى، والذي يناسب الاستخدام البشري ويحقق متطلبات السلامة في المباني، يهدف مشروعنا هذا الى للحفاظ على المبنى مواجها ضد الموجات الزلزالية والاحمال الافقية مثل الرياح.

في غالب المباني المستخدمة في بلادنا لا يتم تصميمها لمقاومة الموجات الزلزالية وهو ما يمثل كثغرة في أسلوب التصميم المتبع في البلاد، في هذا المشروع قمنا باستخدام الكود الأمريكي ACI 318-14 لتحقيق متطلبات التصميم الامن للمنشآت الخرسانية بحسب الفصل رقم 18 والذي يحتوي على متطلبات التصميم لمقاومة الزلازل، تم البدا بتحليل التصميم المعماري الخاص بالمشفى من حيث المداخل والمخارج وعدد الطوابق والمواقع وباقي التفاصيل المعمارية، ومن ثم البدا بتوزيع الجدران الحاملة والترتيب المناسب للأعمدة الذي يناسب كلا التصميمين المعماري والانشائي، وبحسب الكود الأردني للأحمال وضع الاحمال المناسبة لكل عنصر انشائي.

وكنتيجة نحصل على تصميم يوافق جميع متطلبات التصميمية للزلازل والذي سيحافظ على المبنى مواجها لجميع الاحمال الواقعة عليه.

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Chapter 1: Introduction

Background

Site Description

Bethlehem is a Palestinian city, located in the West Bank in Palestine, about 10 km to the south of Jerusalem, with a population of approximately 30,000 inhabitants. Bethlehem is the center of culture and tourism in Palestine since it is identified by Christian tradition as the birthplace of Jesus. Bethlehem has many churches, most notably the Church of the Nativity, which is registered within the UNESCO World Heritage List.

The Bethlehem Governorate covers an area of 575 square kilometers and includes five major cities, seventy villages, and three Palestinian refugee camps, As for the area of the city of Bethlehem, which is within the boundaries of the structural plan of the city; It is eight thousand acres, and the city is divided into many neighborhoods and commercial markets.

Bethlehem population

According to the Palestinian Central Bureau of Statistics (PCBS), the total population of Bethlehem in 2007 was 25,266; of whom 12,753 are males and 12,513 are females. 5,211 households are living in 6,709 housing units.

Age Groups and Gender:

The General Census of Population and Housing carried out by PCBS in 2007 showed that the distribution of age groups in Bethlehem is as follows: 34.1 percent are less than 15 years, 56 percent are between 15-64 years, 4.9 percent are 65 years and older, and 5 percent are unknown. Data also showed that the sex ratio of males to females in the city is 101.9:100, meaning that males constitute 50.5 percent of the population, and females constitute 49.5 percent of the population.

Health Status

Medical services in Bethlehem city are considered rather well-developed because Bethlehem city is considered the vital center of the governorate. Moreover, the Ministry of Health and non-governmental and private institutions, which work in the health sector, supervise this sector in the city, providing their services through hospitals, clinics, and primary health care centers.

There is also one governmental hospital in Bethlehem city run by the Ministry of Health; Muhammad Sa'ed Kamal Psychiatric Hospital, which contains 280 beds, in addition to two private hospitals, which are:

1- The Holy Family Hospital which contains 47 beds.
2- Caritas Hospital for Children which contains 82 beds.

The Palestinian Ministry of Health also provides the city and the surrounding towns and villages with primary health care services, such as medical examination and treatment, through Bethlehem Health Directorate in the city. In addition, there are many charitable societies, medical institutions, and private health clinics, which, upon their efforts, were able to perform multiple tasks in the areas of health care and social development.

These institutions and societies include The Palestinian Red Crescent Society, the Red Cross International Committee, Ephetah School for hearing and cognitive rehabilitation for deaf children, and the House of Hope (Beit Al Amal) for the blind and the mentally disabled, and others.

Moreover, located in Bethlehem city are several laboratories, radiology centers, dental clinics, physicians and specialized clinics, and pharmacies. It is also worth mentioning that all of these hospitals, clinics, and medical centers serve Bethlehem city and all cities and villages in Bethlehem governorate and the neighboring governorates.

Palestine has been subjected to an average of one major earthquake every century over the past centuries. It is exposed to an average of 100 earthquakes each year, humans due to their weakness or their occurrence during the night may not feel many of these tremors. The areas most affected by the earthquake are the Jordan Valley and the mountainous heights, especially in the regions with intersecting faults.

In the current time with the rise of population and the spread of new and fetal diseases the need of building new hospitals to improve health services in town, and that's why we chose to work on a new hospital design that will serve the town and the whole governorate.

Area earthquake statistics:

According to the "Palestinian Encyclopedia" website, one of the most important earthquakes that struck Palestine was the earthquake of January 1, 1837, its center was the city of Safad, which caused nearly five thousand victims, and led to the destruction of the old city of Safad and the village of Jish, the number of villages in the Tiberias district that were destroyed due to The earthquake was over 17 villages.

There are four seismic foci in Palestine: the Dead Sea focus, which represents the greatest concern, where the earthquake strength reaches 6.3 Richter, and the earthquake is repeated approximately every 100 years, which is particularly affected by the area extending from Jerusalem to Nablus approximately, and the Fara’a al-Karmel area, in which the strength of the earthquake may reach To 7 degrees Richter, an area close to Safed at the intersection of the northern Jordan Valley. Fourth: As for the most violent and difficult focus, the finger of Galilee region is located north of Lake Tiberias, in which an earthquake of up to 5 degrees Richter occurred, which is considered the worst in the history of the region. It is expected that the strength of earthquakes in our region will not exceed 7 degrees, especially if the epicenter of the earthquake is in the Tiberias or the Galilee finger.

As for the southern region of Palestine, the seismic strength in it is considered medium, where the seismic index reaches 0.15 according to the seismic map in Palestine, and since the south region is relatively far from the seismically active foci that were mentioned previously, the impact of the earthquakes on it will be relatively weak, otherwise the calculation of the control The intensity of earthquakes depends on the quality of the soil, the depth of the earthquake and the design of the buildings themselves. Most of the buildings are resistant to weak and medium earthquakes.

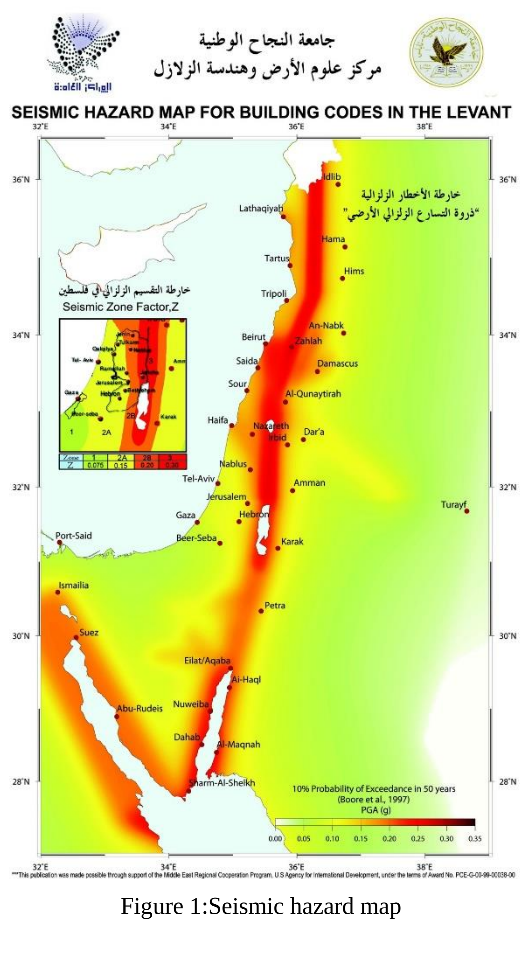


Figure 1:Seismic hazard map

Aims and objectives:

The main aim of our project is to achieve the requirements for the structural seismic design of the building, especially in our country Because of the lack of structural designs that fulfill the requirements of seismic design, and through this goal, the following secondary objectives will be achieved:

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

As for the other goal, it depends on the type of building to be designed, as it is intended to design a hospital in the city of Bethlehem, due to the high population and the spread of new and fetal diseases. The need to build new hospitals to improve health services in the city, and for this reason we chose to work on designing a new hospital, and to provide medical services that serve the city and the whole governorate.

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.

Literature Review

Seismic load is a big concern in structural engineering but still, there is no easy way to answer the question: "What is the requirement of a building structure to resist wind and earthquake forces to get reasonable serviceability at an optimum cost?" Few thumb rules and a huge amount of computer-aided rigorous analyses, are available. It is possible to analyze the structures to appreciate the actual behavior of the structure under ultimate seismic and wind loads which is very likely to the actual behavior and design the building according to its behavior.

There are several studies dealing with the seismic design of buildings and its importance, including:

The theme was developed to mitigate the risks of earthquakes (Gioncu and Mazzolani ،2003): It is an engineering seismology, developed to solve serious earthquake problems, and its purpose is to use the knowledge of seismology for the seismic design of buildings, by proposing seismic procedures, source function, and site characteristics, with the task of solving building weakness problems, and on the basis of which it performs seismic analysis of structures based on loads seismicity and the use of methodologies proposed by engineering seismology, which performs a complex examination of structures, including numerical analysis, structural formation and detail solutions, and an engineering look at the designed structure, in order to achieve the engineering requirements for earthquake resistance that structures are exposed to in order to avoid damages resulting from it.

Design & Analysis of Earthquake Resistant Structure: A Critical Review (Abhishek Kumar Singh, Dr. Rakesh Kumar Pandey Amity University Chhattisgarh, And Raipur 492001 Department of Civil Engineering): the turf of seismic activity Engineering has existed in our nation for over 35 years now. Indian Earthquake Engineers have made momentous hand-outs to the seismic safety of a number of important structures in the country. However, as the recent earthquakes have shown, the performance of normal structures during past Indian earthquakes has been less satisfactory. This is mainly due to the lack of awareness amongst most practicing engineers of the special provisions that need to be followed in earthquake resistant design and thereafter in construction.

In addition, The International Journal of Steel Structures where this journal presents a study aims to suggest a design process of passive control structure, which includes design and placement of the damper for obtaining the seismic capability of the passive control structure as intended by the designer. In addition, dynamic response analysis was conducted on structures designed using the design process suggested in this study to confirm whether the seismic capability intended by the design has been achieved, verifying the validity of the design process.

In the other hand, in our country, engineers do not take up seismic design so, our project aims to make a difference in the way of design in Palestine, when we chose to design our project as a seismic design we were intending to make this type of structural engineering a common way, designing our structures to stand against the seismic and wind loads will save many lives in a case where our country has faced an earthquake reinforced Concrete structures are not perfectly elastic even at lower stress levels. At higher stress levels, it undergoes cracking, etc. The stiffness of Reinforced Concrete elements will decrease appreciably and deformations will increase drastically but it will not collapse immediately if the Reinforced Concrete structures are made sufficiently ductile. It is a bit difficult to understand the behavior of Reinforced Concrete structures under dynamic loads like those subjected to wind, seismic forces, etc. However, buildings should be designed to incorporate ductile detailing to sustain these loads undergoing larger deformations but no collapse

In our project introduction, we will study the architectural design of the building, arrange the suitable columns distribution, and then select the required structural system according to ACI code seismic provisions.

Methodology

- The first step in our project is to study the project from an architect's perspective.
- The second step is to distribute the columns and shear walls satisfactory to the

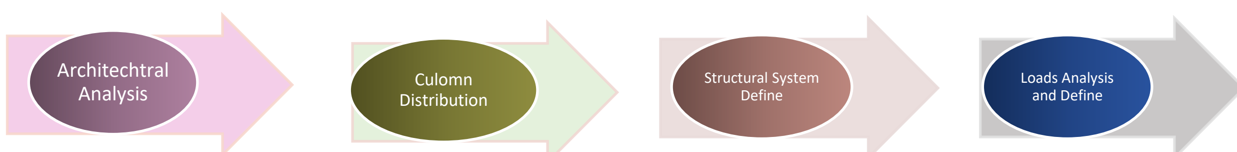


Figure 2: Methodology

  architectural plan making sure no columns will be an obstacle against human use. Then determine the suitable structure system.
- The third step is to calculate the loads starting with possible gravity loads which we include the self-weight of members, cladding, super dead and live loads finishing with the lateral loads (wind and earthquake) specified for the location of the building and soil properties.
- Finally build a model in the etabs program to start designing the building

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, 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: Results and Recommendations.

Time Plan

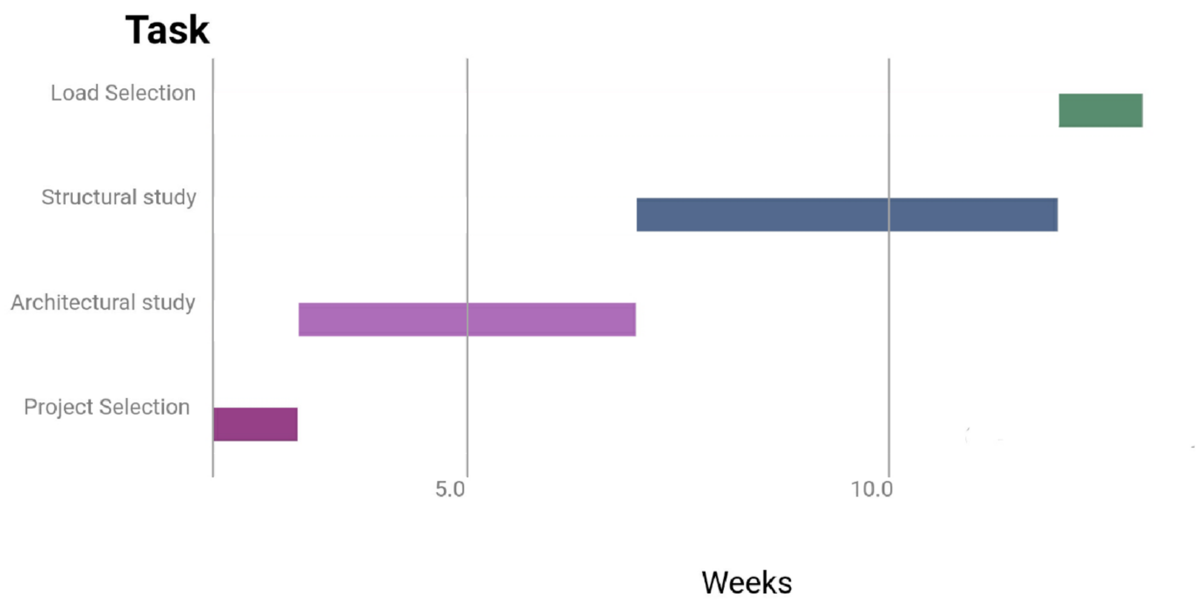


Figure 3:Time Plan

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)

Chapter 2: Architecture 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 hospital with 6 floors, The building is proposed is part of medical city with an area of 19801square 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 Bethlehem. In a midecal 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

2.4.1. Ground Floor

The Ground floor has an Area of 4959m^2, it composed of spaces that is management department which includes registration and accounting department, and medical spaces which includes labs department, dialysis department and emergency department.

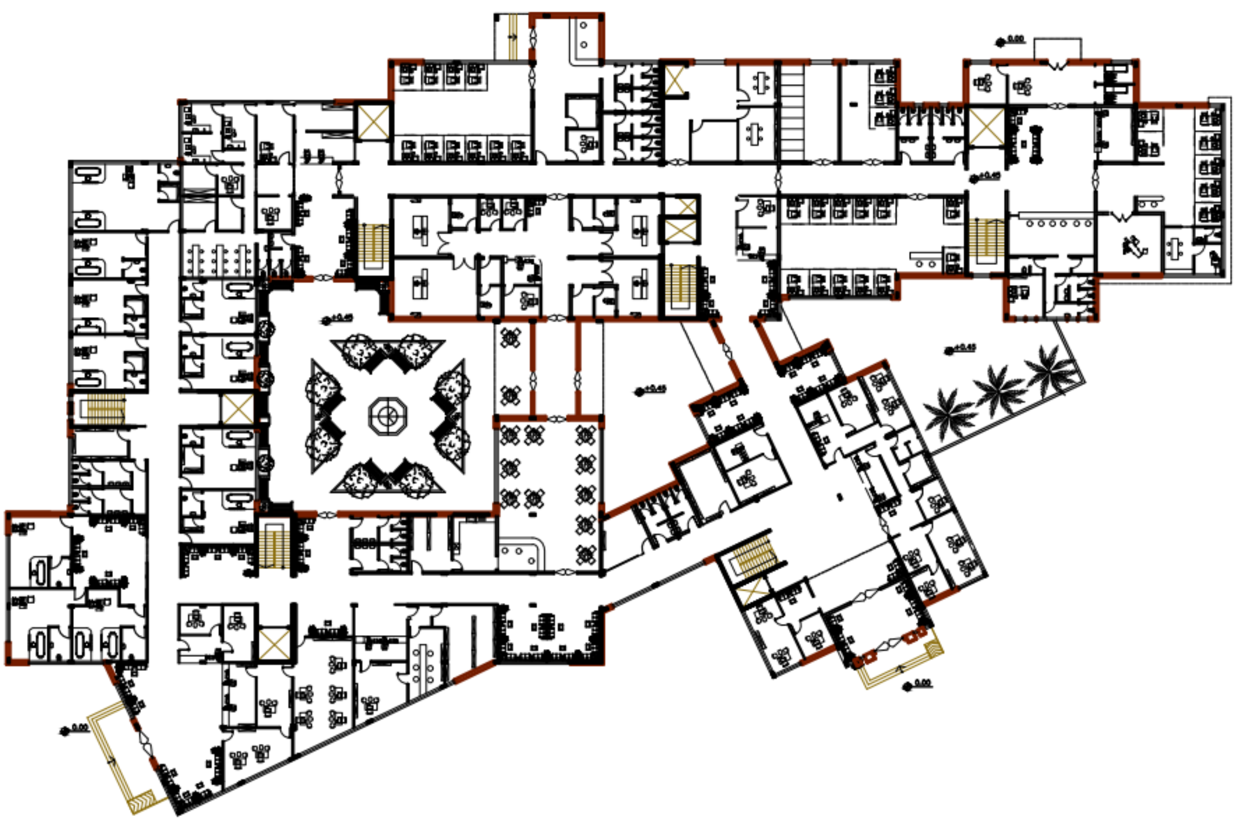


Figure 4:Ground floor plan

2.4.3. First Floor

The first floor has an Area of 4720m2, it contains the administration department, care department and operating department.

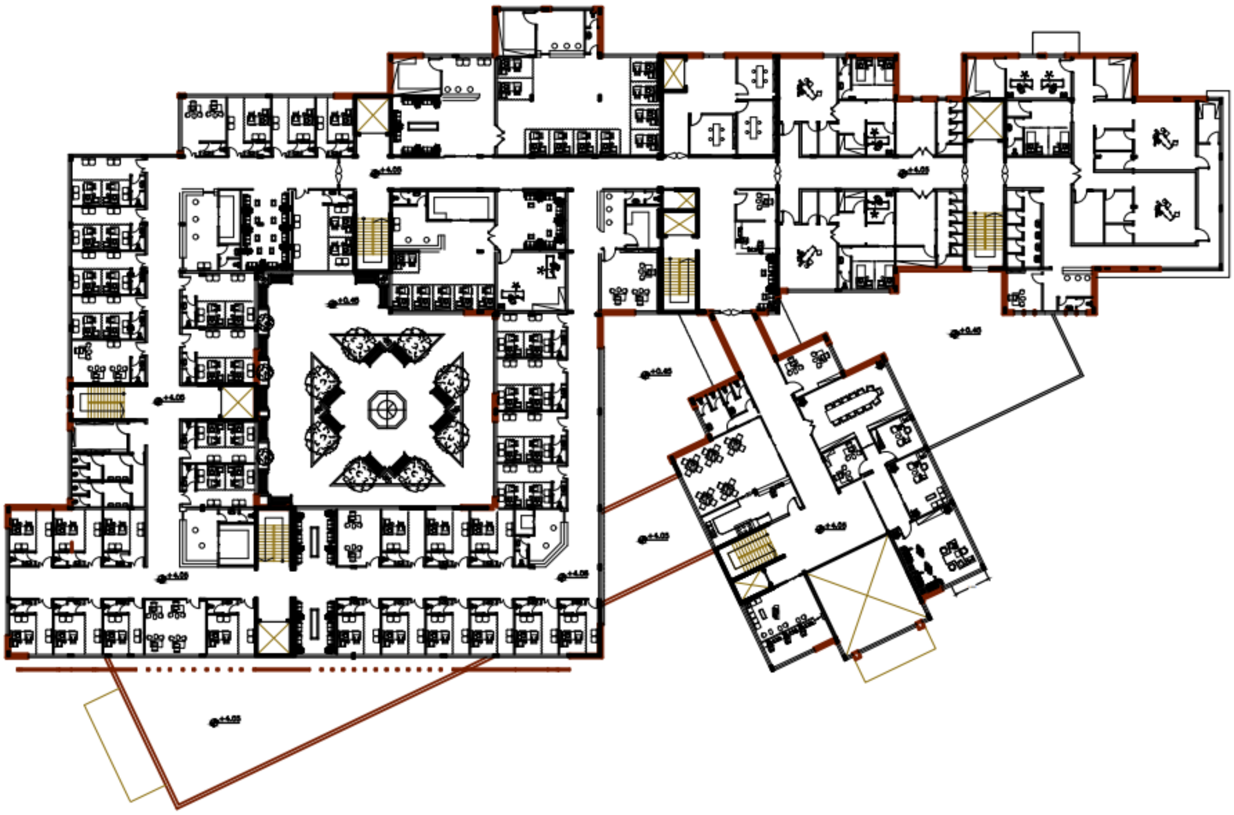


Figure 5:First floor plan

2.4.4. Second Floor

The second floor has an Area of 3913m2, it contains of Nurses duty stations, offices.

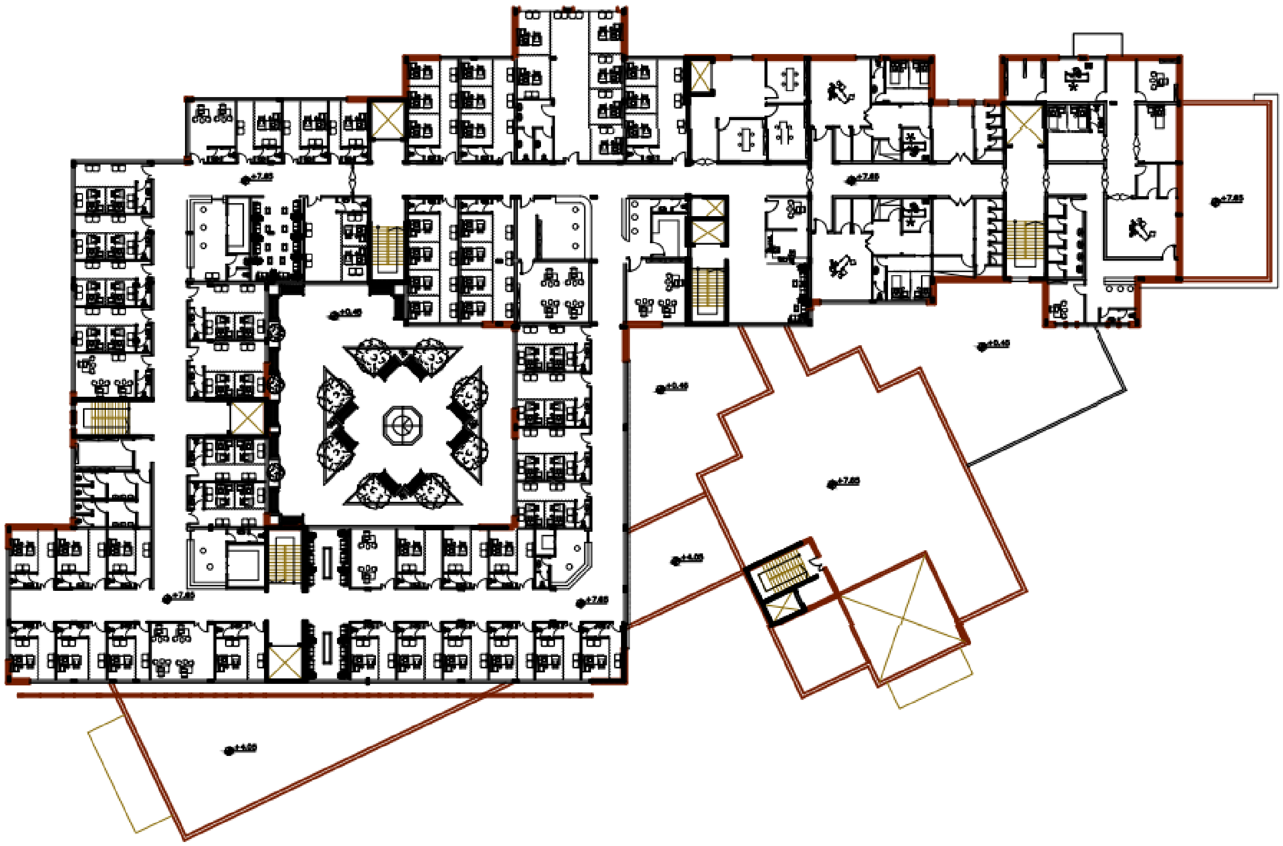


Figure 6:seconed floor plan

2.4.5. Third Floor

The third floor has an Area of 3913m2, it contains of Nurses duty stations, offices.

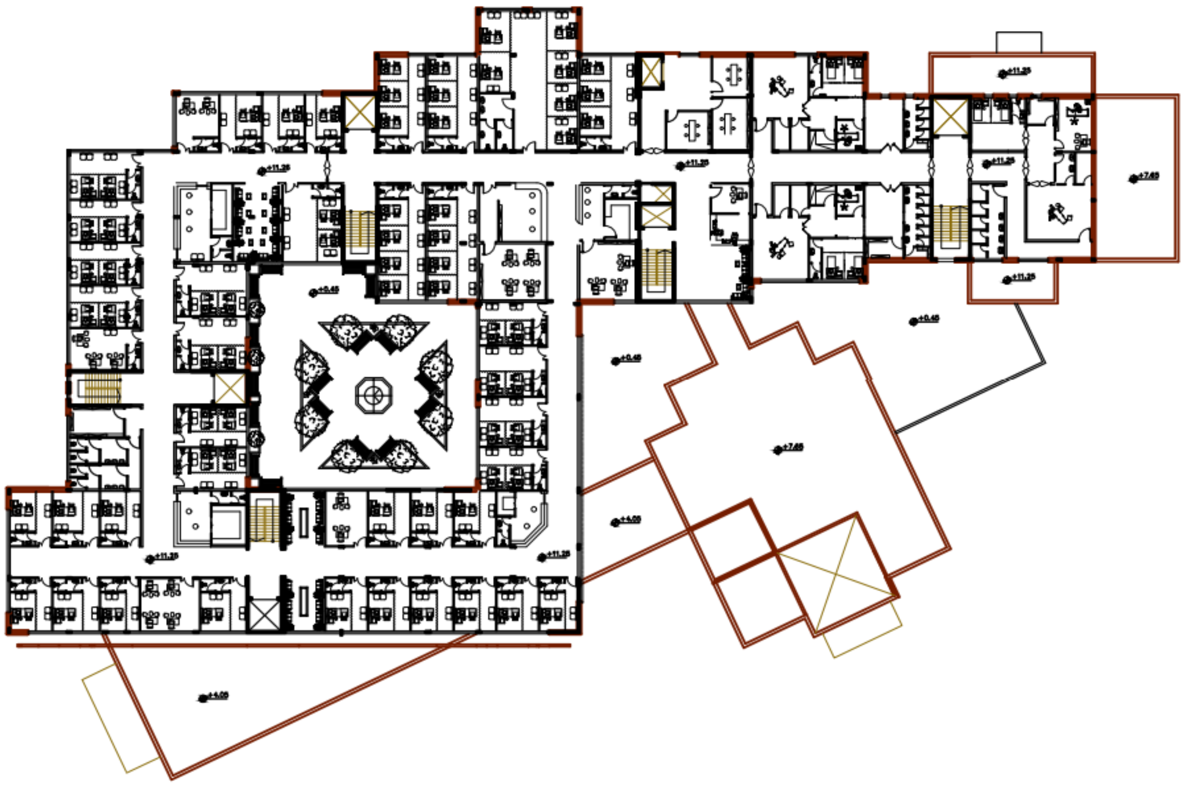


Figure 7:Third floor plan

2.4.6. Fourth Floor

The fourth floor has an Area of 3913m2, it contains of Nurses duty stations, offices.

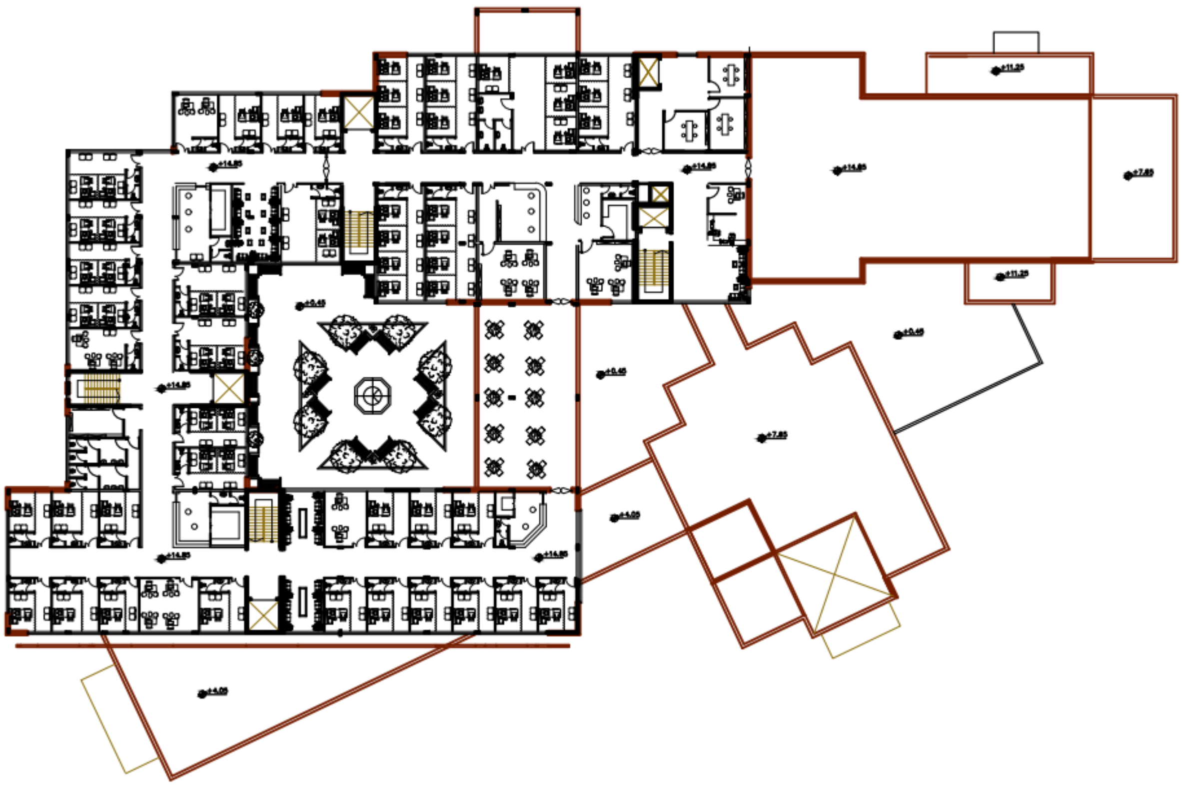


Figure 8: Forth floor plan

2.5. Elevations Descriptions

2.5.1 North Elevation

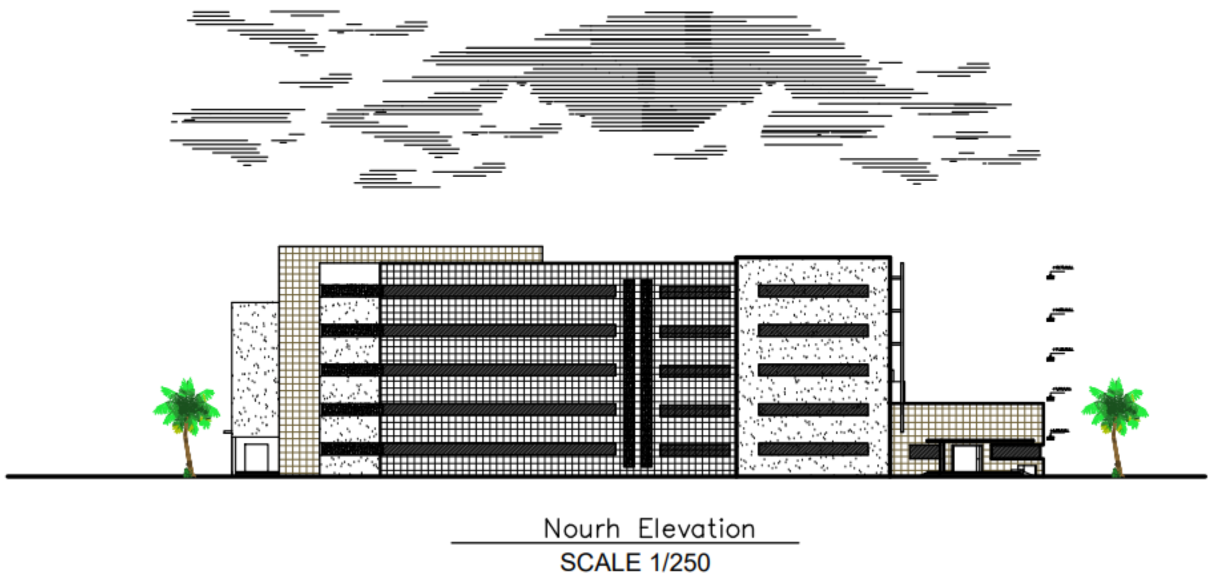


Figure 9:North Elevation

2.5.2 South Elevation

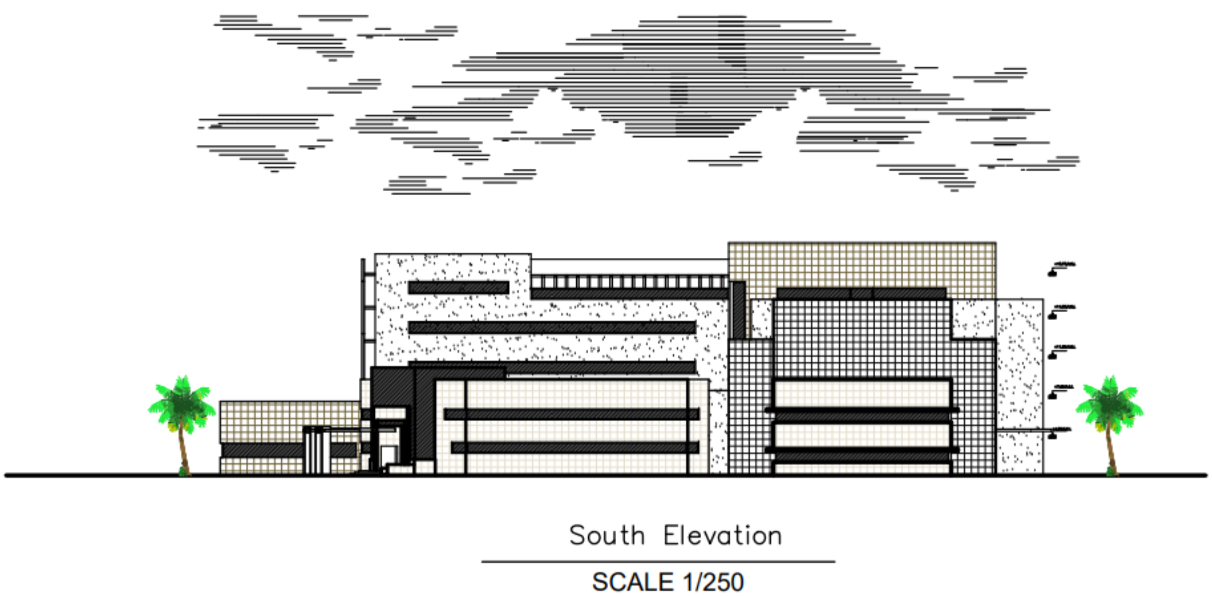


Figure 10:South Elevation

2.5.3 East Elevation

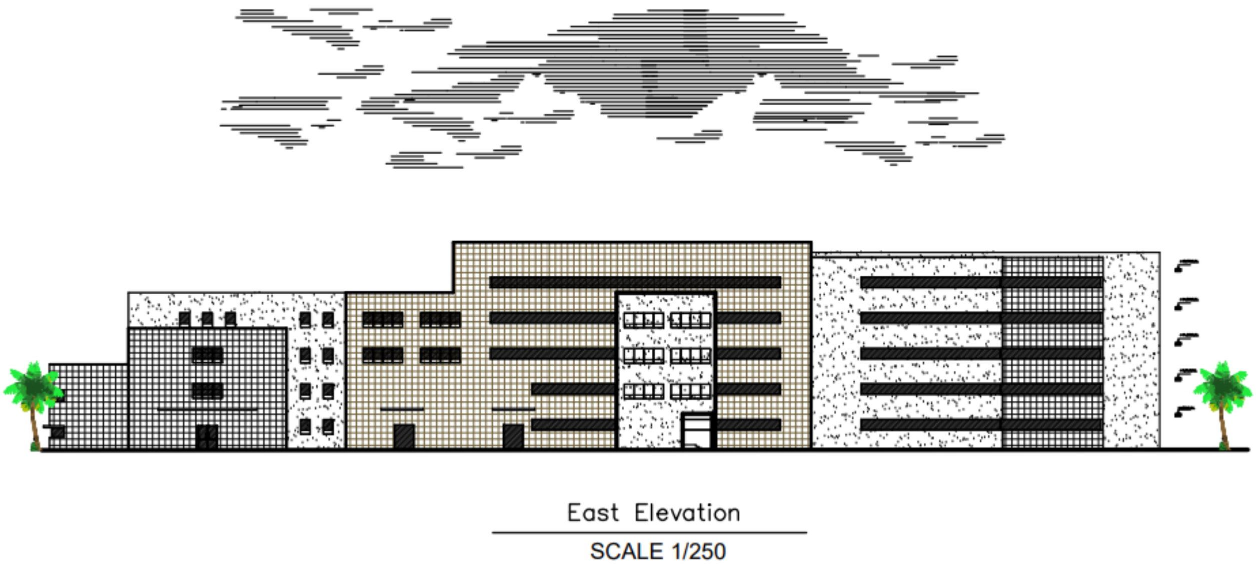


Figure 11:East elevation

2.5.4 West Elevation

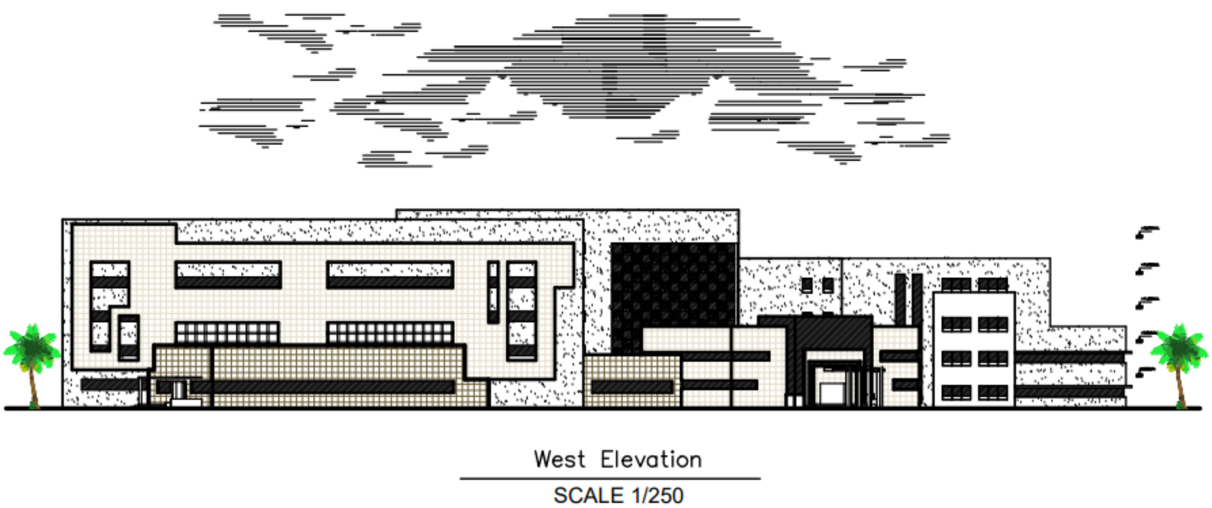
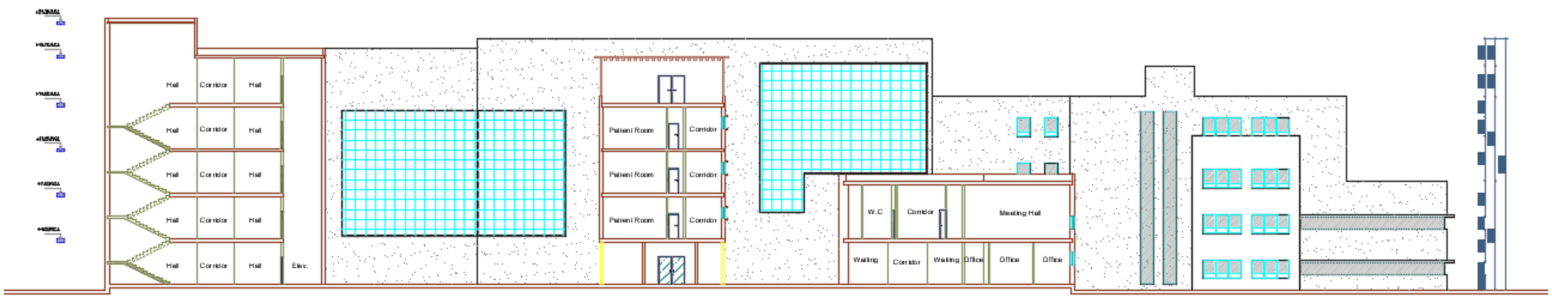


Figure 12: West Elevation

2.6. Sections of the building:

The sections of the building are made in order to show extra and necessary details of the hospital to highlight the movement and the distribution of the spaces and masses along the height of the hospital.



Section A–A

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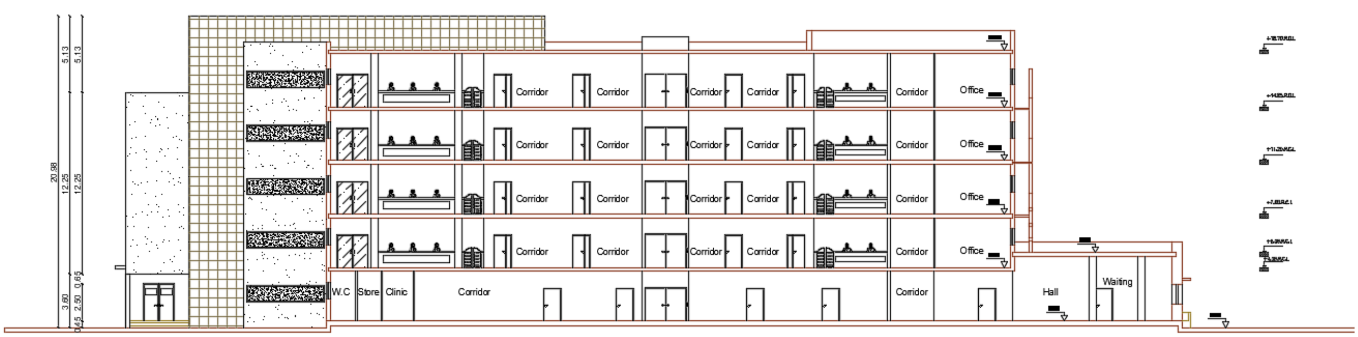
Figure 14: Section A-A



Section B–B

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Figure 13: Section B-B

Chapter3: 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 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, affecting 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 act 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

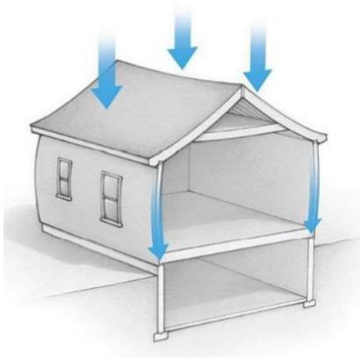

Figure 15: Dead loads

the table shows the calculated dead load

Table 1:Dead Load calculation

Dead Load calculations					
Mterial	density	h	wegiht for 1m strip	unit	for rib
Tiles	22	0.03	0.66	KN/m	0.3432
mortar	22	0.03	0.66	KN/m	0.3432
sand	16	0.07	1.12	KN/m	0.5824
plaster	22	0.03	0.66	KN/m	0.3432
topping	25	0.08	2	KN/m	1.04
hollow block	10	0.25	2.5	KN/m	1.3
partitions			2	KN/m	1.04
total			9.6	KN/m	4.992

3.4.2 Live load

Live loads are those loads produced by the use and occupancy of the building (look figure) 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.

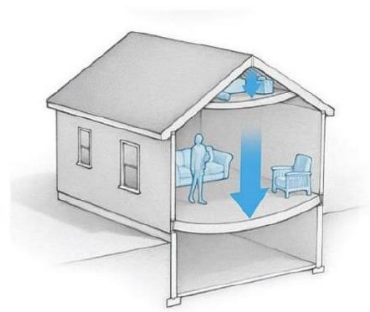

Figure 16: Live loads

The attached table shows the live loads according to Jordanian building loads code

Table 2:Live Loads code table

Type of building	spaces	Live load KN/m2
Educational (Hospital)	**رف المراجل والمحركات والمــراوح وغـــرف المشـروبات والحمــامات والــشرفات والممـــرات وغرف الطعـام وردهـات الاستراحة والبليااردو. (Rooms of Matores , fans , bathroom ,food, room**	**2**
	ممرات والمداخل المعرضة لحركـة المركبــات والعربات المتحركة.	**5**
	المختبرات بما فيها مـن أجهزة، والمطابخ وغرف الغسيل.	**3**
	لممــرات والمـداخل والأدراج و بـــسطات الأدراج الثانويـة.	**3**
	المعـدات قاعــات.	**2**
	غرف الأشـعة والعمليـات والخدمات	**2**
	غرف تبـديل الملابـس وغــرف النـــوم في المستشفيات.	**2**
	لمقصـــورات	**4.5**

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.

Figure 17:Wind loads

2. **Snow**

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

- Height of the facility above sea level.
- Slope of the roof exposed to snow.

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

Figure 18: 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 the figure.

Figure 19: Earthquake 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

3.5.1 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:

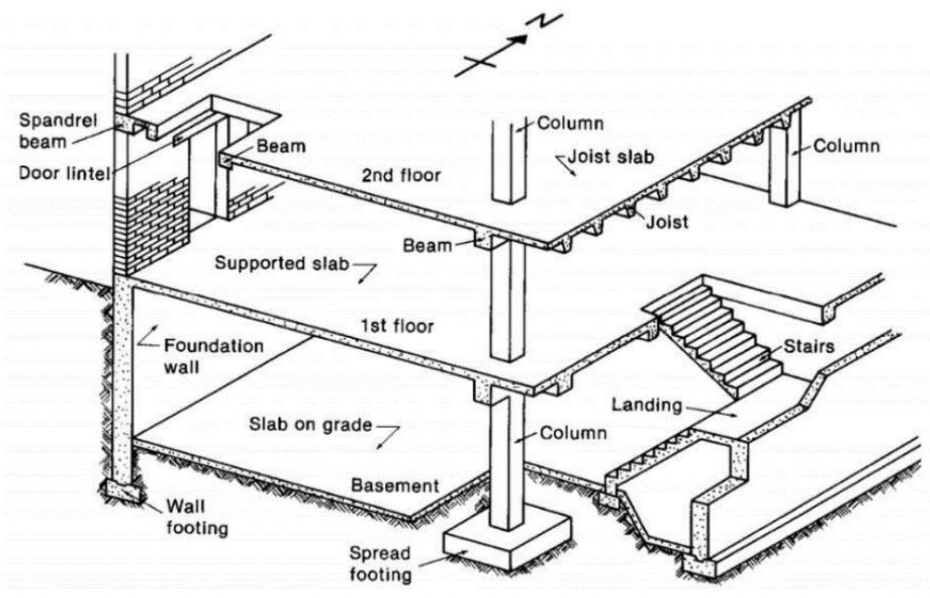

Figure 20: Structural elements

3.5.1.1 Solid slab (two way without beams)

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.

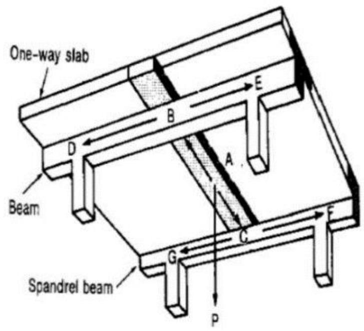

Figure 21: Solis Slab

In our project we used this system in the basement slab, which is a two-way solid slab system with 32cm

thickness which is calculated according to ACI code from the following table:

Table 3:Minimum thickness of slabs code table

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
f_y, MPa†	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
280	ℓ_n/33	ℓ_n/36	ℓ_n/36	ℓ_n/36	ℓ_n/40	ℓ_n/40
420	ℓ_n/30	ℓ_n/33	ℓ_n/33	ℓ_n/33	ℓ_n/36	ℓ_n/36
520	ℓ_n/28	ℓ_n/31	ℓ_n/31	ℓ_n/31	ℓ_n/34	ℓ_n/34

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.
‡Drop panels as defined in 13.2.5.
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

thickness is calculated as follows:

We have in our slab the heights span length is 9m (ln):

$$h = \frac{ln}{33} = \frac{9}{33} = 0.2727$$

32cm is selected

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 describe one-way and two-way ribbed slabs respectively.

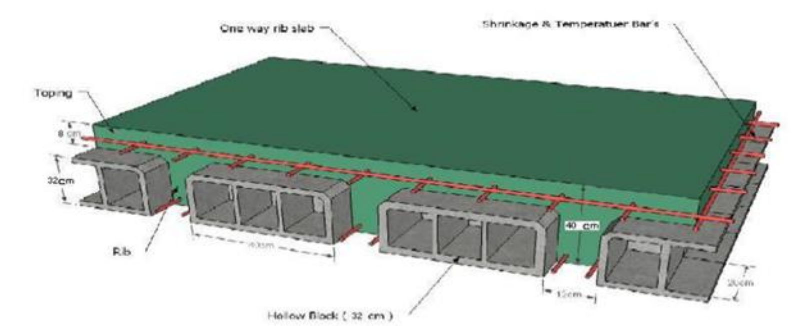

Figure 22:one way slab

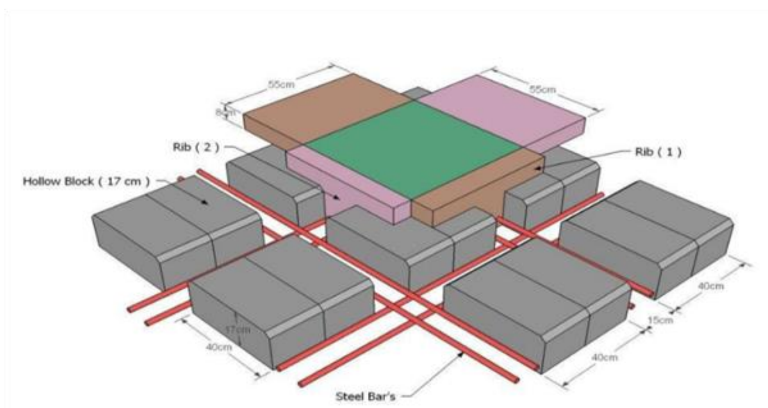

Figure 23:two-way span

In our project, we used ripped slabs from both loading types in order to decrease the cost of the project, as a result of high spans in beams spans of the slab are high, we have to use the two types of ripped slabs in order to associate with the architectural needs the following plan is the ground floor plan which shows the used rippes:

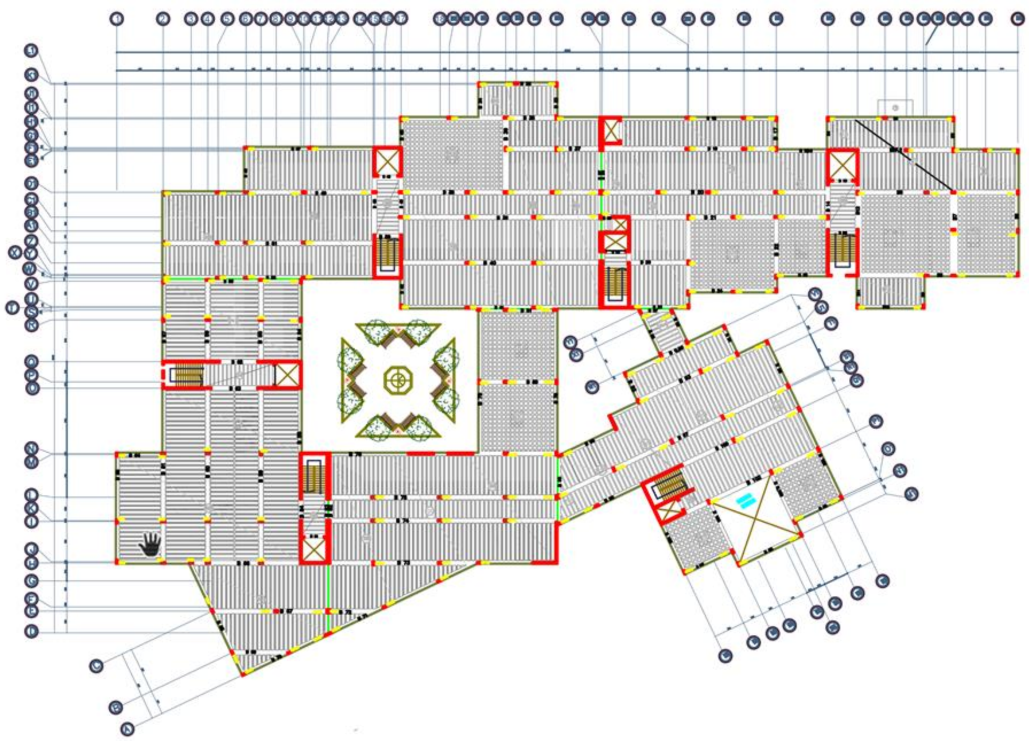

Figure 24: Ground floor key plan

When we needed higher spans without any column in the middle to cover the architectural requirements of these spans as an example:

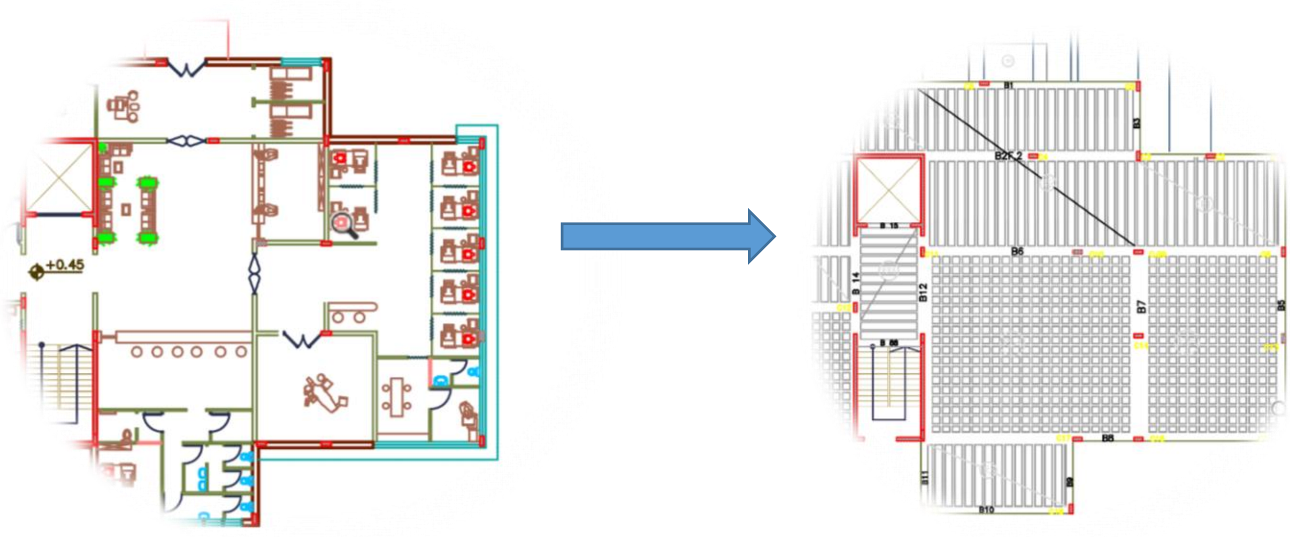

Figure 25:Two-way span use in the project

On the other hand, a small length one-way ripped slab was necessarily needed to solve the problem of the small width paths as an example:

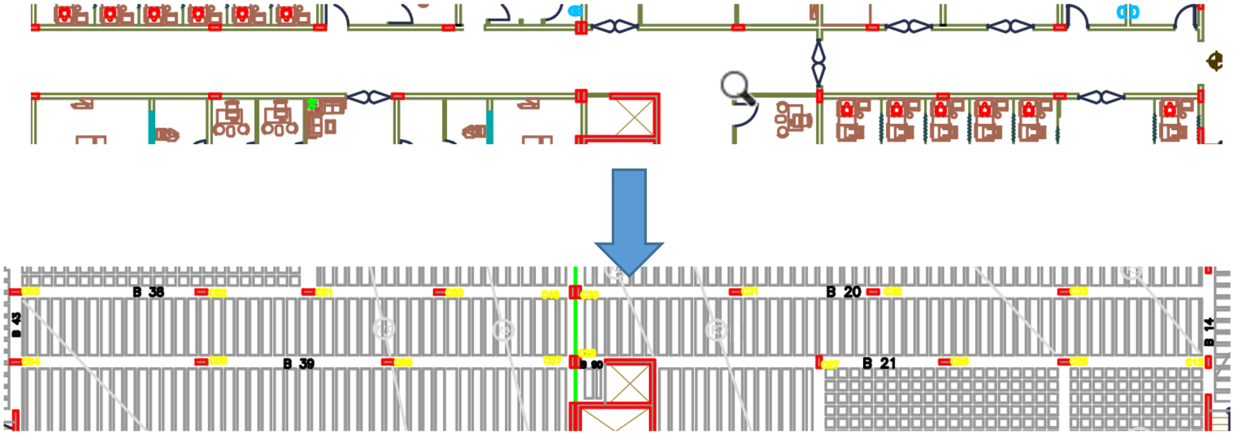

Figure 26:One-way spans use in the project

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

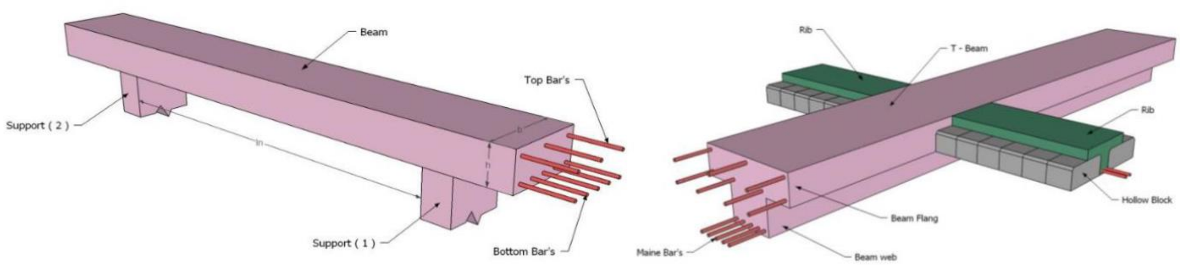

Figure 27: Beams

This type of project requires to have high spans to achieve areas which is empty of columns,

The depth of the floor gives us control to use drop beams with high thickness without affecting the serviceability of the floor or causing a uncomfortable feeling that happens when the ceiling is a falling type

In our project, the minimum span length was around= 3 m and the maximum length is 9m as mentioned before.

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. The figure below shows two types of columns.

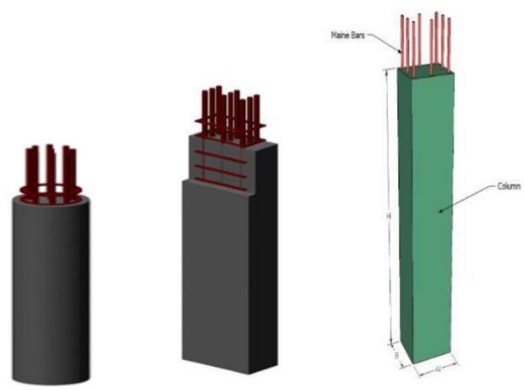

Figure 28:Columns

In column distribution we considered that the columns achieve a statically determined system, the dimensions of columns are initial dimensions the final dimension will be calculated in the next squall of the project, the distribution of columns in the final plans are distributed in a manner that suits the architectural design and the structural system requirements, column are distributed in a way that fits the frame structural system.

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 the figure below.

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

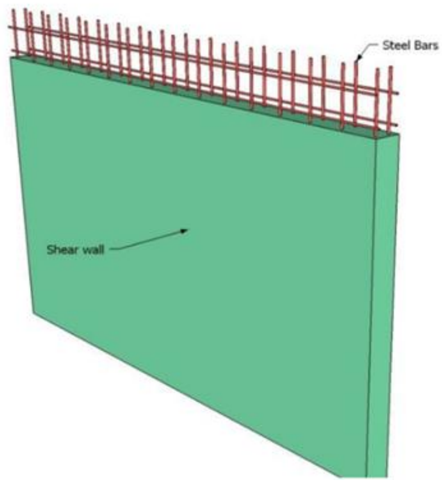

Figure 29:Shear wall

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. It handles the pressure of the sidewalls and provides space for living inside the walls. Basement walls bear the load of the whole structure.

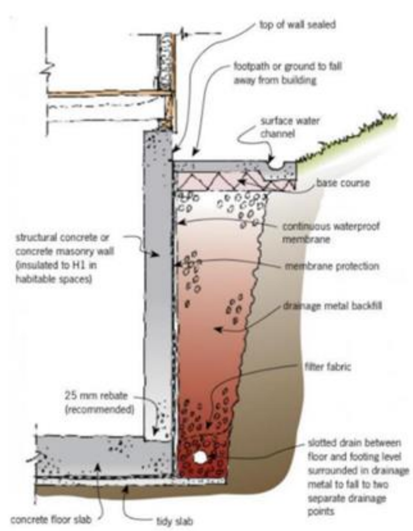

Figure 30:Basement wall

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 many types of foundations that can be used in each project depending on the type of loads and the nature of the soil in the site. A building foundation is shown in the figure

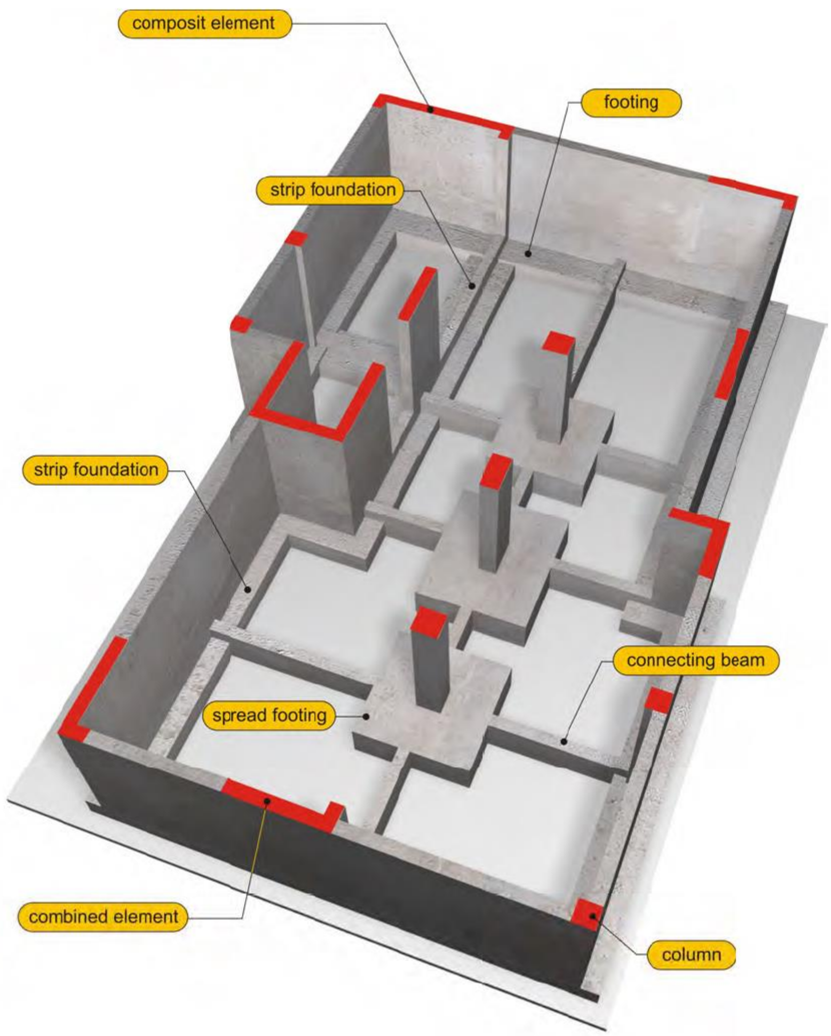

Figure 31:Foundations

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.

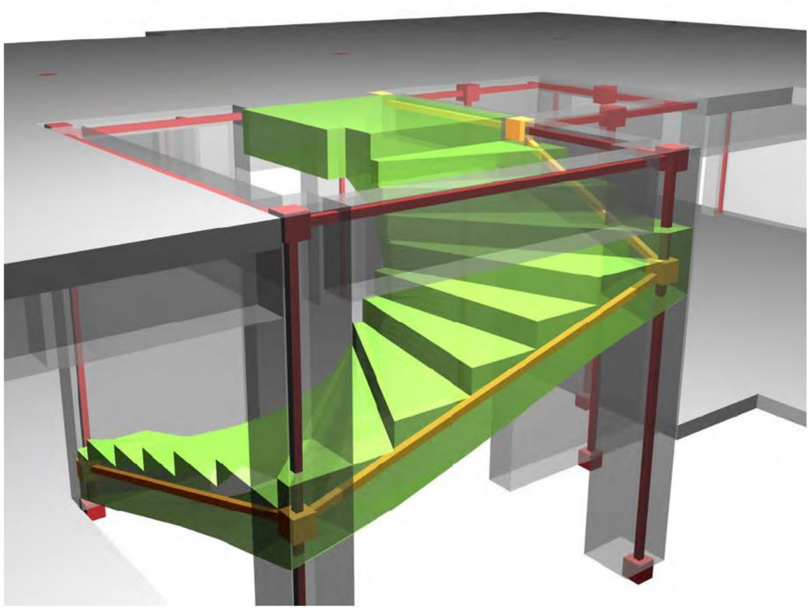

Figure 32: Stairs

3.5.7 Frames

Frame structures are constructions having a blend of columns, beams & slabs to bear the adjacent and gravity loads. These structures are generally used to overcome the large moments emerging owing to the applied loading.

the structural frame must be able to withstand not only the gravity loads but also the loads imposed in a few vital cases during its life span such as the cases of earthquakes.

The figure below shows the deformation of a frame due to seismic action:

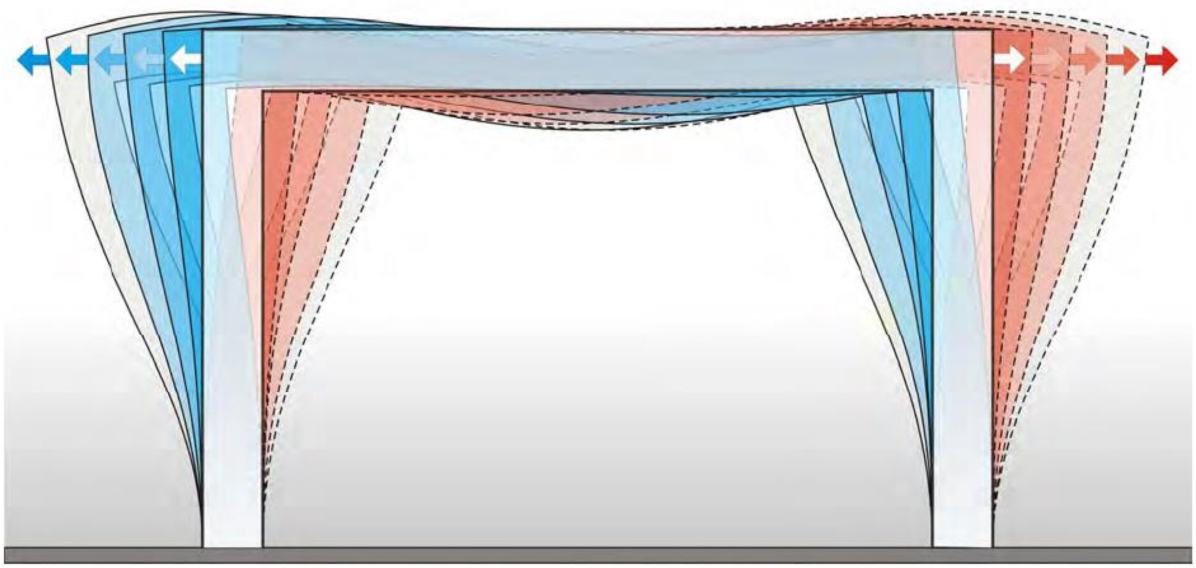

Figure 33:Frame deformation due to seismic loads

Chapter 4: Structural Analysis and Design

Introduction

An adequate load-bearing system is based on a continuous load path throughout the structure.

This means that the vertical loads must be carried by the slabs and transferred to the beams;

the beams must transfer these loads to the columns which in their turn must transfer them to the

foundation. Finally, the foundation must carry the loads to the ground.

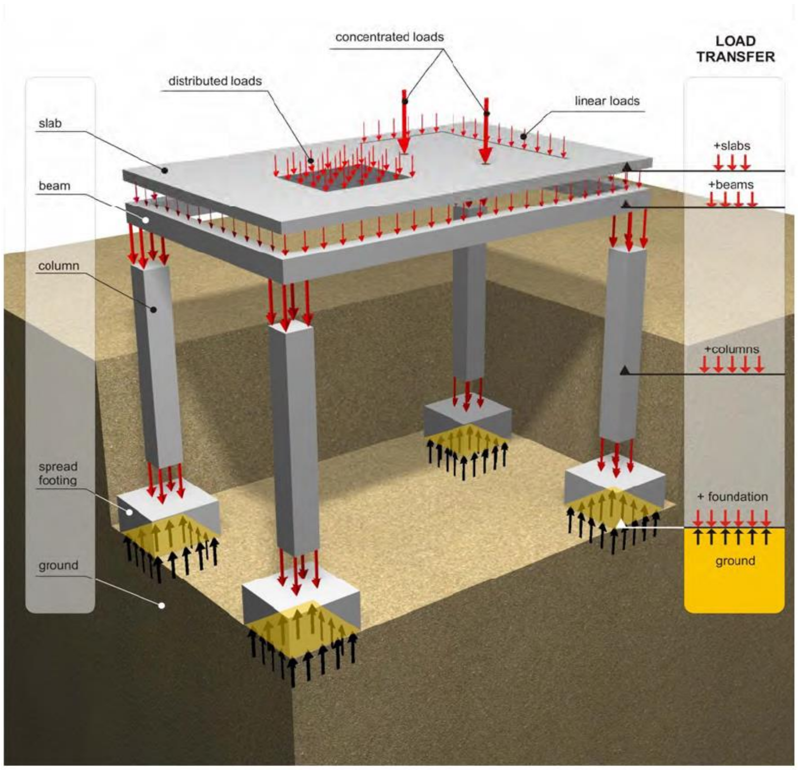

Figure 34: Load transfer system

In this project we have a huge distance so we divided the building to units using expansion joints, for this reason modeling was divided as shown in figure we have the hole building area divided with green thick lines showing the divided areas:

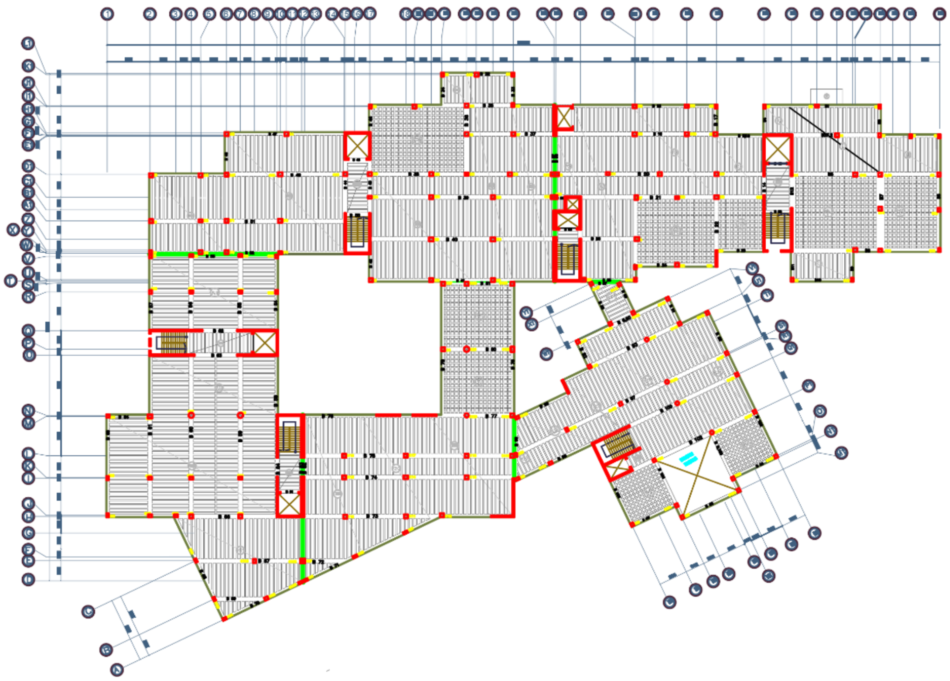

Figure 36: Ground floor Plan appearing expansion joints

As mentioned, modeling building was different we had five models that makes unites combined to form one building, in the following figures Etabs and Safe models are shown

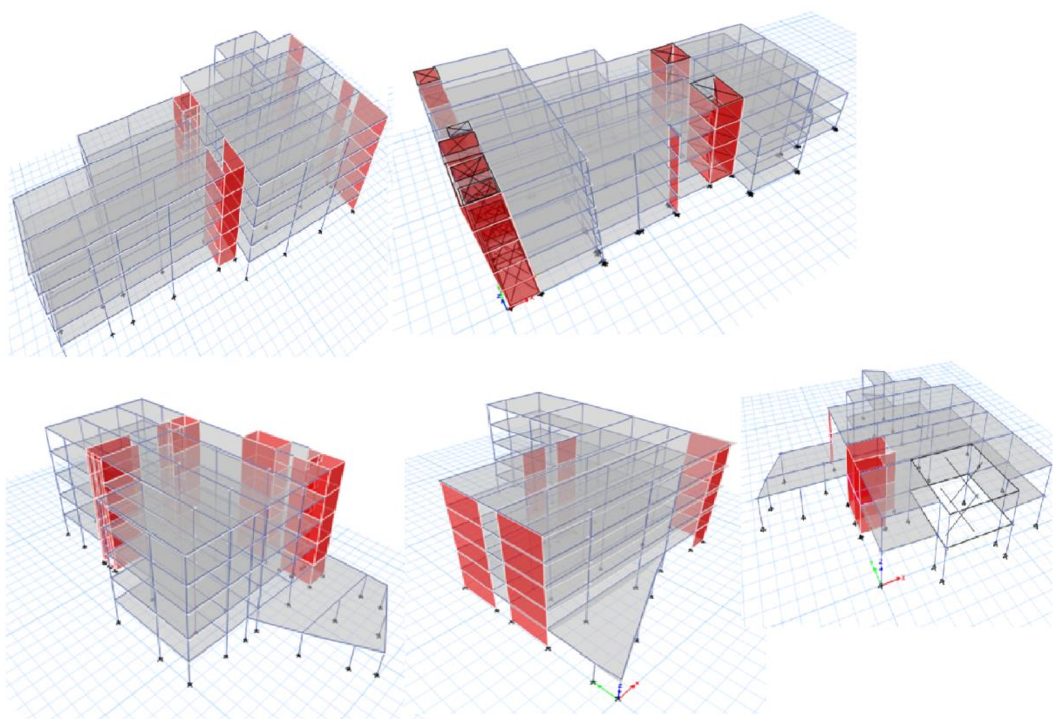

Figure 35: Etabs Models

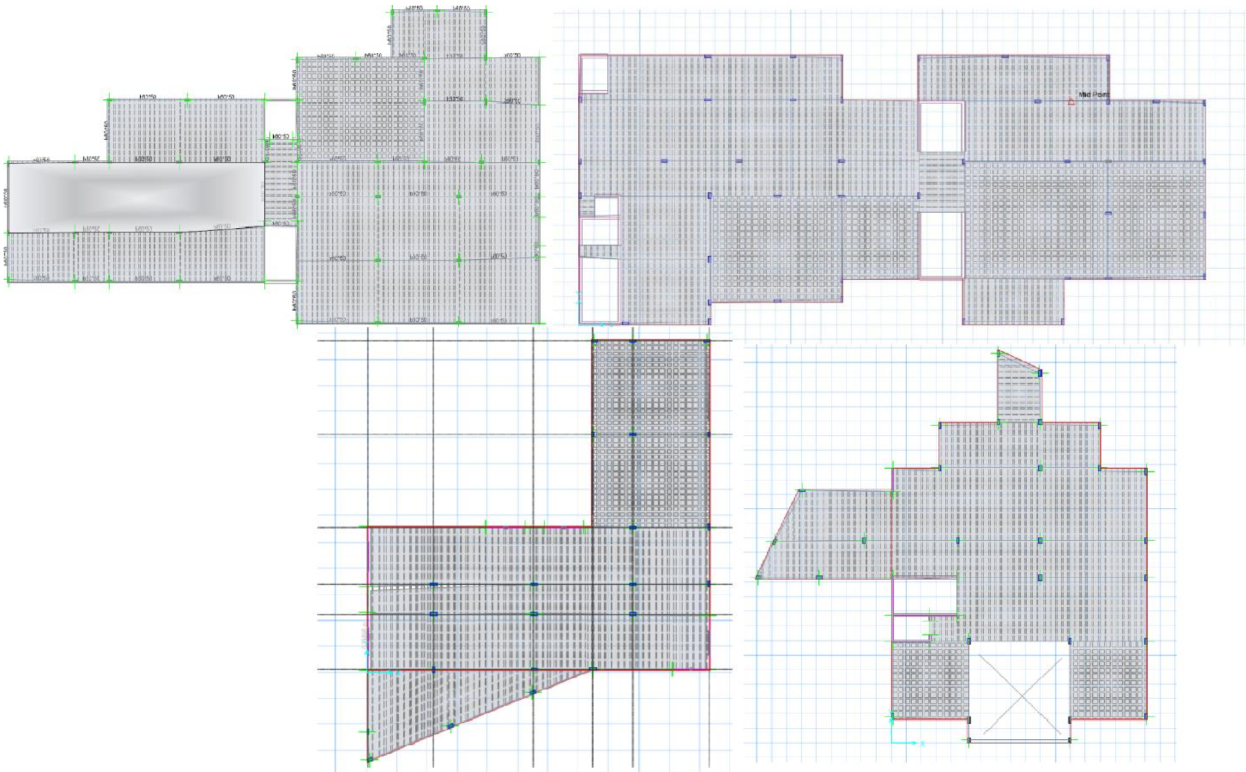

Figure 37:Safe Models

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 TableR5.2.2 Shown below.

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 38:Code Table

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 Shear walls required to resist earthquake effects.

In this section, all members in this section are designed and detailed according to ACI 318-14, the assigned seismic load resisting system is building frame systems, all columns are assigned as a pin-pin supported, special shear walls are designed to resist seismic loads.

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

$\boldsymbol{Wu}$=**1.2** $\boldsymbol{DL}$+**1.6** $\boldsymbol{LL}$

Seismic load

We refer to hazard tables for spectral accelerations seismic event: maximum considered earthquake at Al-Walaja S_s=0.35, S_1=0.09, and has a A site class (hard rock).

From ASCE 7-16 Table 11.4-2 as shown in the figure below F_V=0.8.

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					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
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 39: Fv Code Table

From ASCE 7-16 Table 11.4-1 as shown in the figure below F_a=0.8.

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 40:Fa Code Table

According to ASCE 7-16 (11.4.4) "The $MCER$ spectral response acceleration parameters for short periods (SMS) and 1 s ($SM1$), adjusted for site class effects ".

shall be determined using the following equations:

SMS=F_aS_S=0.8∗0.09=0.072

$SM1$=F_vS_1=0.8∗0.35=0.28

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 :

$$S_{DS}=\frac{2}{3}\,S_{MS}=\frac{2}{3}\times 0.072=0.1867$$

$$S_{D1}=\frac{2}{3}S_{M1}=\frac{2}{3}\times 0.28=0.08$$

The Risk Category of the structure is IV according to table 1604.5 shown below.

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Risk Categories I, III and IV.
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing Group E occupancies with an occupant load greater than 250. • Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000.[a] • Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV. • Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released.[b]
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures. • Buildings and other structures containing quantities of highly toxic materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released.[b] • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

Figure 41:Risk category Code table

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
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	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
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

Figure 42: Sesmic Category Code Table

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

$$T_0 = 0.2\times \frac{S_{D1}}{S_{DS}} = 0.2\times \frac{0.08}{0.1867} = 0.0856 \text{ s}$$

$$T_s = \frac{S_{D1}}{S_{DS}} = 0.180.75 = 0.4284\ s$$

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

Figure 43:Spectral Acceleration curve

According to Table 1.5-2 from ASCE7-16 importance factor using risk category I_e= 1.5

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Figure 44: Importance factor Code table

Structural System Selection

We chose SPECIAL SHEAR WALL system. With Response Modification Coefficient R= 6, and Over strength Factor, Ω_0=2.5 , and Deflection amplification factor C_d= 5.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category B	C	D^d	E^d	F^e
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[l,m]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[l]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[l]	14.2 and 14.2.2.8	2	2½	2	NL	NP	NP	NP	NP

Figure 45: Seismic coefficient Code table

One-Way rib design

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: Minimum Thickness

	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	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

H_{min} for (one end continuous) ribbed one-way slabs = $\ell/21$

= 6180/21=294.285 mm

Take h = 32m

24 cm block + 8 cm topping = 32cm >h_{min}=29.43cm

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.

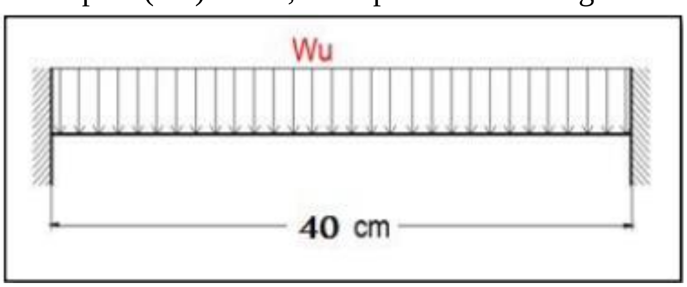

Figure 46: Topping System

Load Calculation for topping:

Table 5: Topping Load calculation

Material	Density	W	
Tiles	22	22×0.03×1	0.66
Sand	16	17×0.07×1	1.12
Mortar	22	22×0.02×1	0.44
RC Topping	25	25×0.08×1	2
Partitions	2	2×1	2
Total Dead Load, KN/m		6.22	

Live Load =2KN/m2 =5×1=5KN/m

Factored Load W_u= 1.2 ×6.22+ 1.6×5 =15.464 KN/ m

W_u= 1.4×6.22=8.708 KN/ m

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

$Mn = 0.42\ \lambda \sqrt{fc'}\ Sm$

$Sm = \frac{bh^2}{6} = \frac{1000*80^2}{6} = 1066666.67 \times 10^{-6}\ mm^3$

$Mn = 0.42 \times 1 \times \sqrt{24} * 1066666.67 \times 10-6 = 2.19\ KN.m$

$\emptyset Mn = 0.55 \times 2.19 = 1.21\ KN.m$

$Mu = \frac{Wul^2}{2} = \frac{15.464 * 0.4^2}{2}$=1. 24 $KN.m$ $\emptyset Mn \gg Mu$

No reinforcement is required by analysis. According to ACI 10.5.4, provide $Asmin$ for slabs a shrinkage and temperature reinforcement.

$\rho_{shrinkage}$ = 0.0018

As = ρ × b × t =0.0018 ×1000×80 = 144 $mm2$ / m

Step (s) is the smallest of:

1. 3h=3×80=240 mm – control

2. 450 mm

3. s = = 380 × ($\frac{280}{fs}$) − 2.5 × c = 380 × ($\frac{280}{\left(\frac{2}{3}\right)\times 420}$) − 2.5 ∗ 20 = 330 mm

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

Take ø 8 @ 200 mm in both direction, s = 200 mm.

Design of One-Way Rib Slab

- Material

    - concrete B300 Fc' = 24 N/mm2

    - Reinforcement Steel fy = 420 N/mm2

- Section

- B=520mm
- Bw=120mm

t= 8mm

h=320mm

d=320-20-10-16/2=282mm

Rib 14 from (c 69-c75), as shown in Ground floor plan:

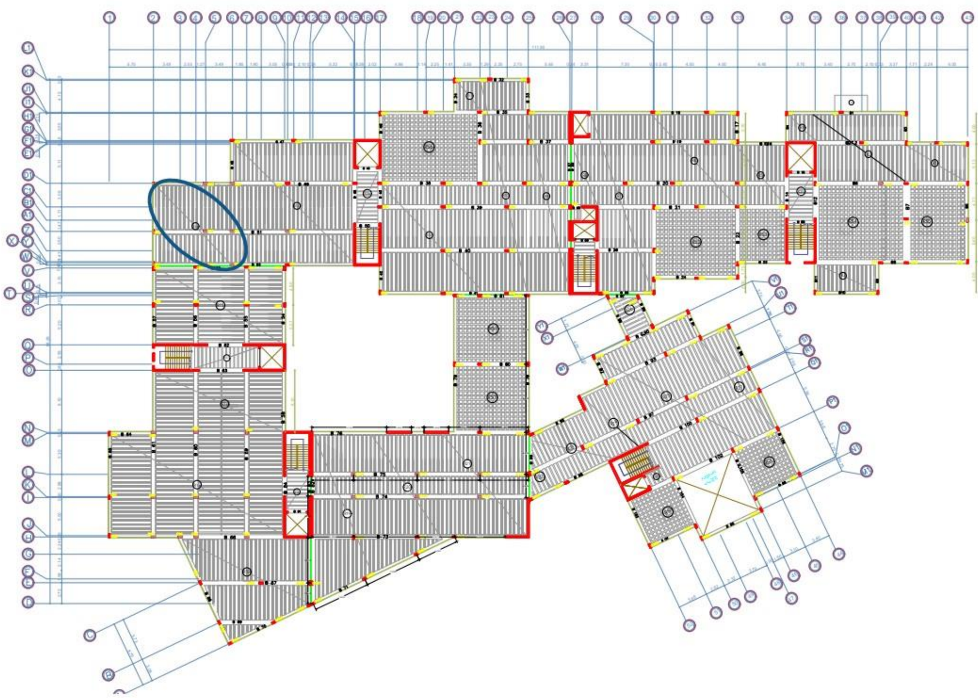

Figure 48:Rib Postion

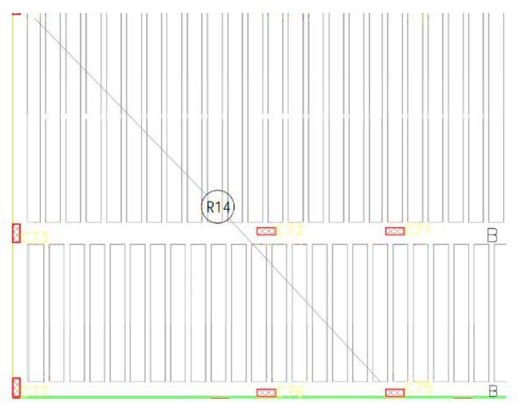

Figure 47:Rib 14

Load Calculation for Rib14:

Dead load:

Table 6: Rib DL calculations

Material	Density	W	
Tiles	22	22×0.03×0.52	0.343
Sand	16	17×0.07×0.52	0.619
Mortar	22	22×0.02×0.52	0.343
RC Rib	25	25×0.24×0.12	0.72
Hollow Block	10	10×0.24×0.4	0.96
Plaster	22	22×0.02×0.52	0.343
Topping	25	25×0.08×0.52	1.04
Partitions	2	2×0.52	1.04
Total Dead Load, KN/m		5.408	

Live Load = 5KN/m2=5×0.52=2.6 KN/m

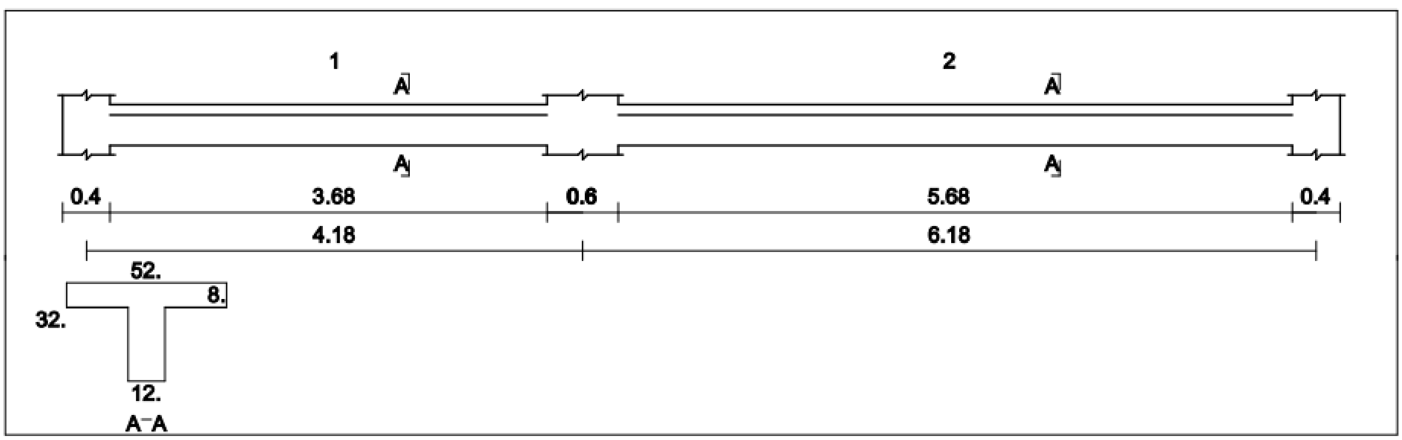

Figure 49: Rib dimensions

Effective Flange Width (be): (ACI-318-14 (8.12.2)) be For T- section is the smallest of the following:

be ≤ L / 4 = 4100/ 4 =102.5 cm

$be \leq bw$ + 16 t = 12 + 16 (8) = 140 cm

be ≤ Center to center spacing between adjacent beams = 52 cm. Control

Flexure Design:

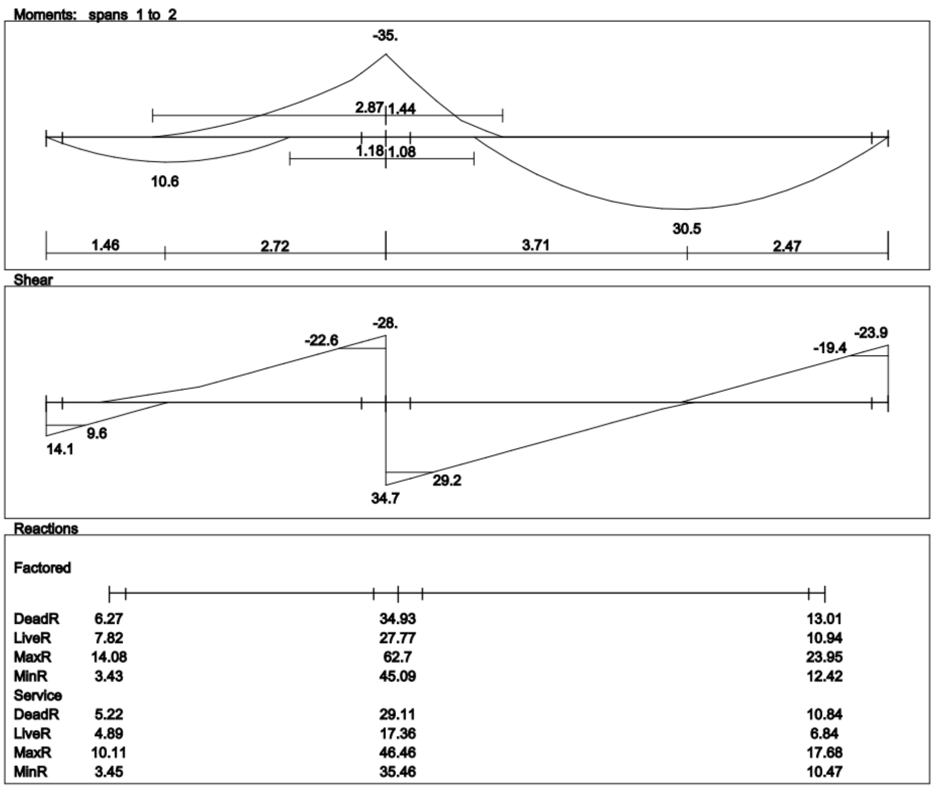

Figure 50:Rib moments

Design of Rib 14 for positive moments:

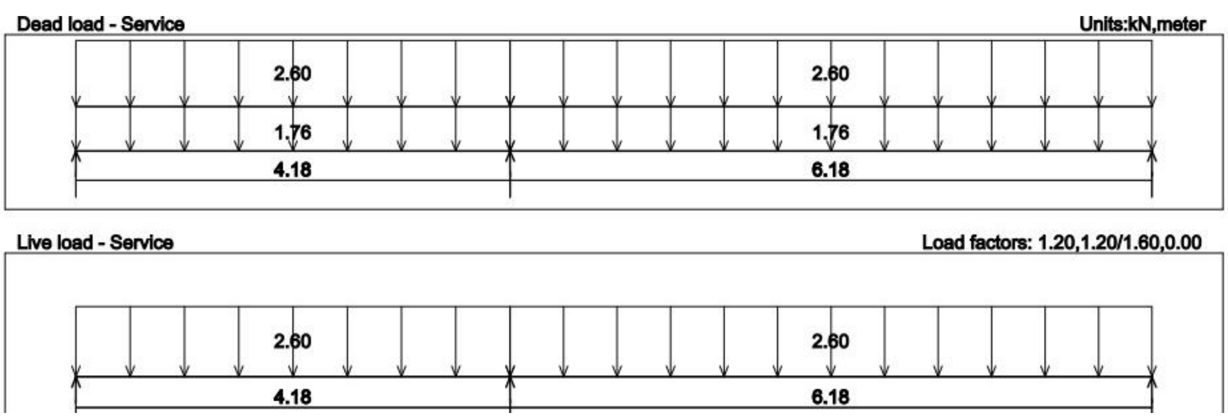

Figure 51:Rib loads

Assume bar diameter Ø 14 for main positive reinforcement

d= 320-20-10- 14/ 2 = 283 mm

- Span 2 (Max Positive Moment= 30.5 KN.m)

Check if a > hf

$\bar{\bar{M}}nf$ = 0.85 × $fc\,'bhf\ (d - \frac{hf}{2})$= 0.85 × 24 ×520 ×80 × $(283 - \frac{80}{2})$ × 10^{-6} = 206.22$KN.\,m$

$\bar{\bar{M}}nf$ = 206.22 $KN.\,m \gg \emptyset$ = $(\frac{30.5}{0.9})$= 33.9$KN.\,m$ → $a < hf$ The section will be designed as rectangular section with b = 520 mm

$Rn = \frac{Mn}{\emptyset bd^2} = \frac{30.5\times10^6}{0.9\times520\times283^2}$ 0.814 MPa

$m = \frac{fy}{0.85fc\prime} = \frac{420}{0.85\times24}$ = 20.6

$\rho = \frac{1}{m}\ (\frac{1-\sqrt{1-2\mathrm{Rnm}}}{fy})$

= $\frac{1}{20.6}\ (\frac{1-\sqrt{1-2\times0.814\times20.6}}{420})$= 0.1978

$As = \rho bd$ = 0. 001978× 520 × 283 = 291.14 mm^2

for $A_{s,min}$

$A_{s,min} = \frac{0.25\sqrt{\mathrm{fc}\prime}}{\mathrm{fy}}\,bwd \quad \geq \frac{1.4}{\mathrm{fy}}\,bwd$

$A_{s,min} = \frac{0.25\sqrt{24}}{420} \times 120 \times 283 = 99.1 mm2 \quad < \frac{1.4}{420} \times 120 \times 283 = 113.2 mm^2$. control

$A_{s,min}$ =1.132cm2 < As ,req=2.9114cm^2 – OK

Use 2Ø14 with

As = 3.079 cm2 > As, = 2.9114cm^2 - OK

Check for strain

$a = \frac{\text{Asfy}}{0.85 fc'b}$

$= \frac{307.9 \times 420}{0.85 \times 24 \times 520}$ =12.19 mm

$c = \frac{\text{a}}{\beta 1} = \frac{12.19}{0.85} = 14.35\ mm$

$\varepsilon t == 0.003(\frac{d-c}{c}) = 0.003(\frac{283-14.35}{14.35}) = 0.0562 > 0.005$ - OK

Design of Rib 14 for negative moments:

Assume bar diameter Ø 14 for main positive reinforcement

d= 320-20-10- 14/ 2 = 283 mm

- (Max Negative Moment= 35 KN.m)

$Rn = \frac{Mn}{\emptyset b d^2} = \frac{35 \times 10^6}{0.9 \times 120 \times 283^2}$ =4.05 MPa

$m = \frac{fy}{0.85fc\prime} = \frac{420}{0.85\times24} = 20.6$

$\rho = \frac{1}{m}\ (\frac{1-\sqrt{1-2\text{Rnm}}}{fy})$

$= \frac{1}{20.6}\ (\frac{1-\sqrt{1-2\times4.05\times20.6}}{420}) = 0.01086$

$As = \rho bd = 0.\ 01086\times 120\times 283 = 368.7\ mm^2$

for $As{,}min$

$As, = \frac{0.25\sqrt{\text{fc}\,\prime}}{\text{fy}}\,bwd \quad \geq \frac{1.4}{\text{fy}}\,bwd$

$As, = \frac{0.25\sqrt{24}}{420}\times120\times283 = 99.1mm2 \quad < \frac{1.4}{420}\times120\times283=113.2mm^2$. control

As,=1.132cm2 < As ,req=3.687cm^2 – *OK*

Use 2Ø16 with $As = 4.042\ cm2 > As, = 3.687cm^2$ - *OK*

Check for strain

$a = \frac{\text{Asfy}}{0.85fc'b}$

$= \frac{402.1\times420}{0.85\times24\times120} = 68.98\ mm$

$\text{c} = \frac{\text{a}}{\beta1} = \frac{68.98}{0.85} = 81.16mm$

$\varepsilon t == 0.003(\frac{d-c}{c}) = 0.003(\frac{283-81.1}{81.1}) = 0.00747 > 0.005$ - *OK*

Shear Design:

Vu at distance d from support = 29.2 KN

Shear strength, 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).

$Vc = 1.1 \times \frac{1}{6} \lambda\sqrt{fc'}bwd = 1.1 \times \frac{1}{6} \sqrt{24} \times 120 \times 283 \times 10^{-3} = 30.5KN$

$\emptyset Vc = 0.75 \times 30.5 = 22.88\ KN$

$Vs, = \frac{1}{16}\sqrt{fc'}bwd = \frac{1}{16}\sqrt{24} \times 120 \times 283 \times 10^{-3} = 10.4\ KN$

$Vs, = \frac{1}{3}\ bwd = \frac{1}{3} \times 120 \times 283 \times 10^{-3} = 11.32\ KN - Control$

$\emptyset(Vc + Vs,min) = 0.75 \times (30.5 + 11.32) = 31.37$ KN

$\emptyset(Vc + Vs,min) > \boldsymbol{Vu} > \emptyset Vc$

Use stirrups U-shape (2legs stirrups) Ø10 with $Av = 2 \times 78.57 = 157.14\ mm2$

$VS,min = \frac{\text{Av,min}}{\mathbf{S}}\ fytd$

$11.32 = \frac{157.14}{\mathbf{S}} \times 420 \times 283 = 1649.97KN$

$\rightarrow S = 1649.97\ mm\ s \leq 600mm \leq \frac{d}{2} = \frac{283}{2} = 141.5m$

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

Comparing reinforcement between Hand calculation with reinforcement BeamD and Safe programs:

Hand calculations:

- **Top reinforcement:**

Use 2Ø16 with $As = 4.042\ cm2 > As, = 3.687cm^2$ - OK

- **Bottom reinforcement:**

Use 2Ø14 with

$As = 3.079\ cm2 > As, = 2.9114cm^2$ - OK

BeamD program:

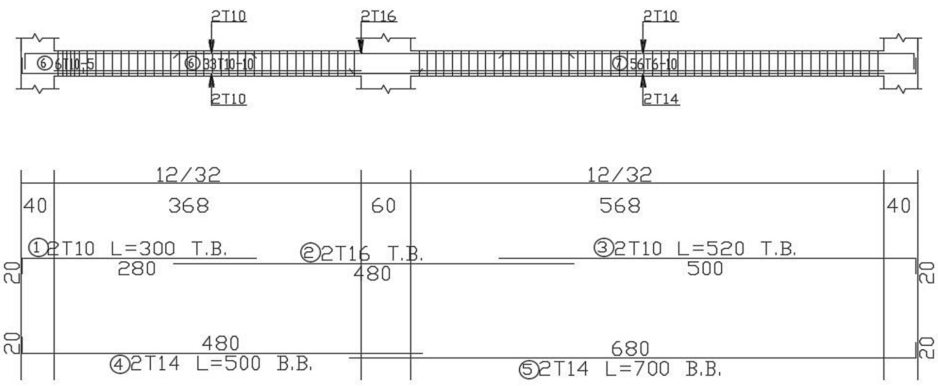

Figure 52: Rib Detail

Design of two-way ribbed slab

rib location

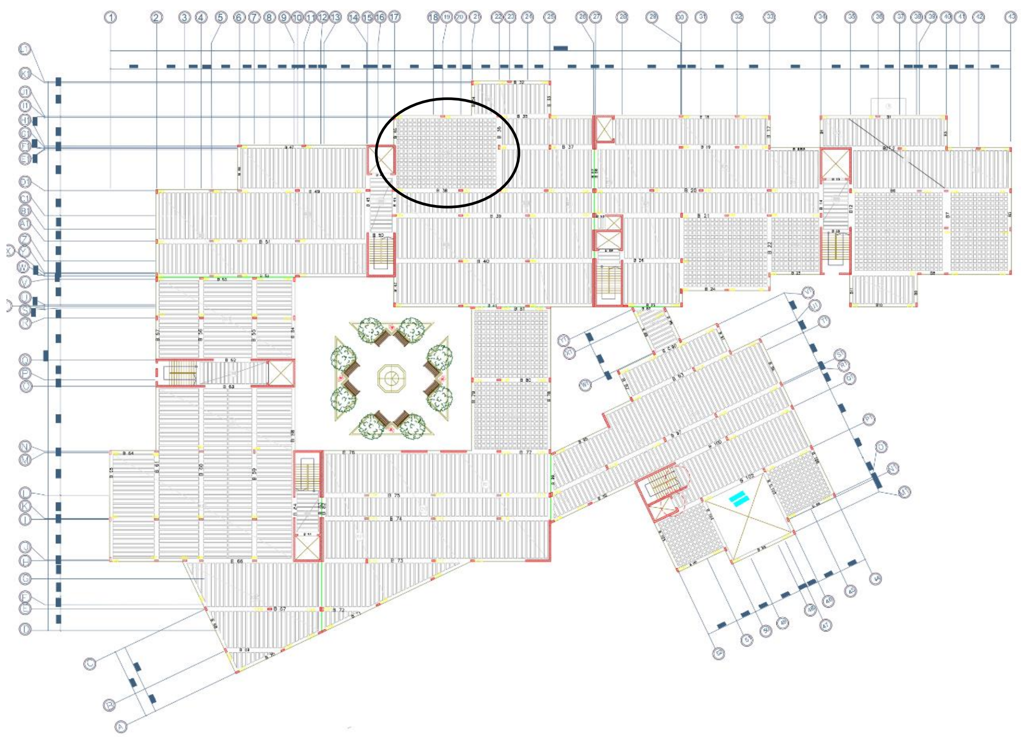

Figure 53:Rib Position

rib dimensions

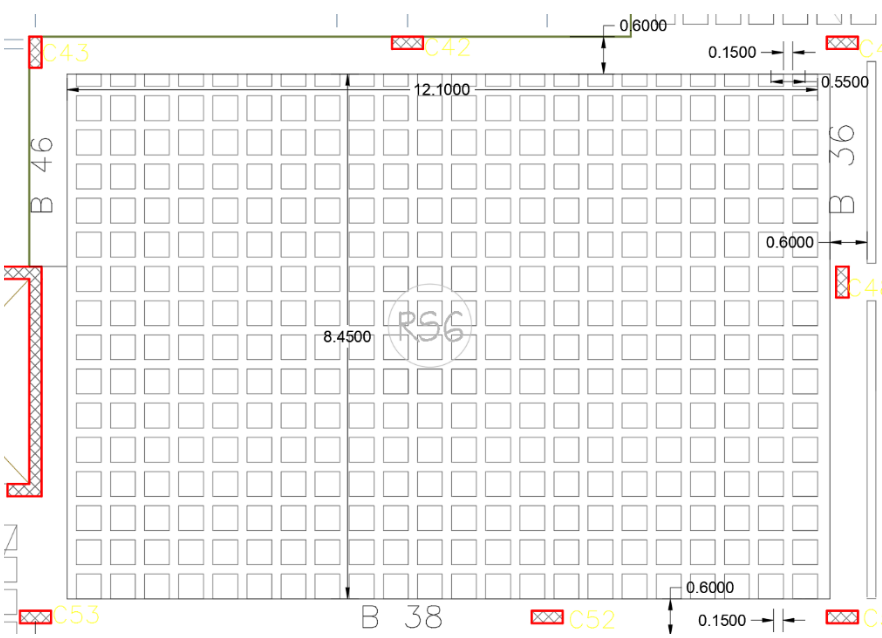

Figure 54:Rib dimensions

Loads Calculation

Table 7:Two-way rib load calculations

Material	Density	W	
Tiles	22	22×0.03×0.55	0.19965
Sand	16	17×0.07×0.55	0.3388
Mortar	22	22×0.02×0.55	0.1331
RC Rib	25	25×0.27×0.15×(.55+.4)	0.961875
Hollow Block	10	10×0.27×0.4	0.81675
Plaster	22	22×0.02×0.55	0.1331
Topping	25	25×0.08×0.52	0.605
Partitions	2	2×0.52	0.605
Total Dead Load, KN/m^2			3.793275
Total Dead Load, KN/m			12.53975

Total dead load= 1.2 × 12.54= 15.048KN/m

Live load= 1.6 × 2 = 3.2 KN/m

Total service loads= 18.24 KN/m

Moments calculations

m = $\frac{L_a}{L_b} = \frac{8.45}{12.1} = 0.698 \approx 0.7$

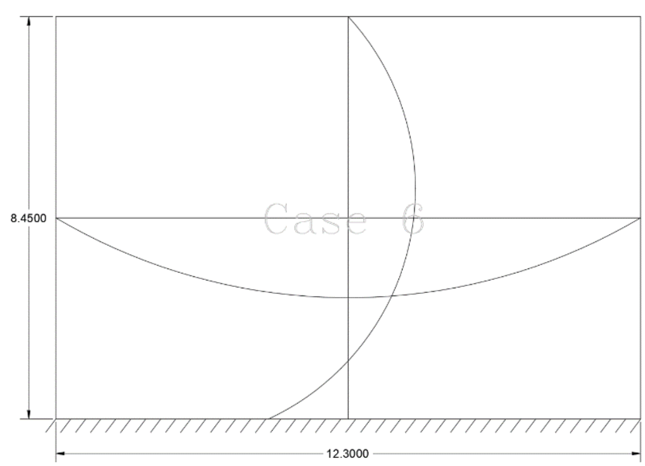

Figure 55:Rib Case

Negative moments:

C_a=0.091

$M_{a,\ neg}$= $C_aWL_a^2b_f$ = 0.091× 18.24×8.45^2×0.55= 65.184kN.m

Positive moments:

For dead loads:

$C_{a,dead}$= 0.051

$M_{a,Pos,\ dead}$= $C_aWL_a^2b_f$= 0.051× 15.048×8.45^2×0.55= 30.14kN.m

$C_{b,dead}$= 0.009

$M_{b,Pos,\ dead}$= $C_bWL_b^2b_f$= 0.009× 15.048×12.1^2×0.55= 10.905kN.m

For live loads:

$C_{a,live}$= 0.06

$M_{a,Pos,\ live}$= $C_aWL_a^2b_f$= 0.06× 3.2×8.45^2×0.55= 7.54kN.m

$C_{b,live}$= 0.013

$M_{b,Pos,\ live}$= $C_bWL_b^2b_f$= 0.013× 3.2 ×12.1^2×0.55= 7.99kN.m

$M_{a,Pos}$= $M_{a,Pos,\ live}$+ $M_{a,Pos,\ dead}$ = 30.14 + 7.54 = 37.68 kN.m

$M_{b,Pos}$= $M_{b,Pos,\ live}$+ $M_{b,Pos,\ dead}$ = 10.905 + 7.99 = 18.895kN.m

In Short Direction:

Design for negative moments: M_u= 65.184kN.m

Assume bar diameter ø 14 for main reinforcement

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

R_n= $\frac{\mathrm{M_u}}{\emptyset \mathrm{bd}^2}=\frac{65.184\times 10^6}{0.9\times 150\times 313^2}=4.928 Mpa$

m= $\frac{f_y}{0.85 f_c'}=\frac{420}{0.85\times 24}=20.588$

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{420}}\right)=\frac{1}{20.588}\left(1-\sqrt{1-\frac{2\times 20.588\times 4.928}{420}}\right)=0.0136571$

$A_{s,req}$ = ρbd = 0.0136571×150×313 = 641.2mm^2

Check for $A_{s,min}$

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

$A_{s,min}$ = $0.25\frac{\sqrt{f_c{}'}}{f_y}b_w d$= $0.25\frac{\sqrt{24}}{420}.150.313$ = 136.908 mm^2 - control

$A_{s,min}$ = $\frac{1.4}{f_y}b_w d$= $\frac{1.4}{420}.150.313$ =156.5 mm^2

A_s =641.2 mm^2 > $A_{s,\,min}$=136.908 mm^2 - OK

Use 2 ø 22 with A_s =760.26 mm^2

Step= $\frac{150-20\times2-10\times2-22\times2}{1} = 46\ mm$

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85b\ f_c'} = \frac{760.26 \times 420}{0.85 \times 150 \times 24} = 104.35\ mm$$

$$c = \frac{a}{\beta_1} = \frac{104.35}{0.85} = 122.76\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{313-122.76}{122.76}\right) = 0.00465 > 0.005$$

Design for positive moments: M_u= 36.45 kN.m

Assume bar diameter ø 14 for main reinforcement

d =h- cover $-\frac{d_b}{2} = 320 - 20 - 10 - \frac{14}{2} = 283\ mm$

R_n= $\frac{\mathrm{M_u}}{\mathrm{\emptyset bd^2}} = \frac{36.45\times10^6}{0.9\times550\times283^2} = 0.92Mpa$

m= $\frac{f_y}{0.85f_c'} = \frac{420}{0.85\times24} = 20.588$

ρ= $\frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{420}}\right) = \frac{1}{20.588}\left(1-\sqrt{1-\frac{2\times20.588\times0.92}{420}}\right) = 0.00227$

$A_{s,req}$ = ρbd = 0.00227×550×283 = 353.32mm^2

Check for $A_{s,min}$

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

$A_{s,min} = 0.25\frac{\sqrt{f_c{}'}}{f_y} b_w d = 0.25\frac{\sqrt{24}}{420} . 150.283 = 123.78$ mm^2 - control

$A_{s,min} = \frac{1.4}{f_y} b_w d = \frac{1.4}{420} . 150.283 = 141.5$ mm^2

$A_s = 353.32\ m^2 > A_{s,\,min} = 123.78\ mm^2$ - OK

Use 2 ø 16 with $A_s = 402.12$ mm^2

Step= $\frac{150-20\times2-10\times2-16\times2}{1} = 58\ mm$

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85b\ f_c'} = \frac{353.32 \times 420}{0.85 \times 550 \times 24} = 13.22\ mm$$

$$c = \frac{a}{\beta_1} = \frac{13.22}{0.85} = 15.55\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{283-13.22}{13.22}\right) = 0.0515 > 0.005$$

In Long Direction:

Design for positive moments: $M_u = 17.44$ kN.m

Assume bar diameter ø 14 for main reinforcement

d =h- cover $-\frac{d_b}{2} = 320 - 20 - 10 - \frac{14}{2} = 283\ mm$

$R_n = \frac{M_u}{\emptyset b d^2} = \frac{17.44\times10^6}{0.9\times550\times283^2} = 0.44 Mpa$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85\times24} = 20.588$

$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{420}}\right) = \frac{1}{20.588}\left(1-\sqrt{1-\frac{2\times20.588\times0.92}{420}}\right) = 0.00107$

$A_{s,req} = \rho bd = 0.00107\times550\times283 = 166.54\ mm^2$

Check for $A_{s,min}$

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

$A_{s,min} = 0.25\frac{\sqrt{f_c'}}{f_y} b_w d = 0.25\frac{\sqrt{24}}{420}.150.283 = 123.78\ mm^2$ - control

$A_{s,min} = \frac{1.4}{f_y} b_w d = \frac{1.4}{420}.150.283 = 141.5\ mm^2$

$A_s = 166.54\ m^2 > A_{s,min} = 123.78\ mm^2$ - OK

Use 2 ø 12 with $A_s = 226.19\ mm^2$

$Step = \frac{150-20\times2-10\times2-12\times2}{1} = 66\ mm$

Check for strain:

$$a = \frac{A_{s.f_y}}{0.85 b\ f_c'} = \frac{226.19\times420}{0.85\times550\times24} = 8.46\ mm$$

$$c = \frac{a}{\beta_1} = \frac{8.46}{0.85} = 9.966\ mm$$

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{283-9.96}{9.96}\right) = 0.0822 > 0.005$$

Shear Design

Maximum shear coefficient will be in the Short direction for the slab with boundary conditions as in case 6. W_a= 0.91

The total load on the panel being = 12.1 $\times$ 6.45$\times$ 16.636 $= 1700.94\ KN$

The load per rib at face of the long beam = 0.91 $\times 1700.94 \times 0.55 \div (2 \times 12.1) = 35.18\ KN$

The shear critical section is at d distance from the beam face:

$$V_{u,d} = V_{u,face} - w_u b_f d = 35.18 - 16.635 \times 0.55 \times 0.283 = 32.596\ KN$$

The shear strength of one rib in the slab is

$$V_c = 1.1 \frac{1}{6}\sqrt{f_c{}'}\, b_w d = 1.1 \times \frac{1}{6} \times \sqrt{24} \times 150 \times 283 \times 10^{-3} = 38.12\ KN$$

ϕ=0.75 – for shear

$$\phi\ V_c = 0.75 \times 38.12 = 28.59\ KN$$

$$V_{u,d} = 32.596\ KN > \phi\ V_c = 28.59\ KN - \text{Section needs Shear design}$$

$$V_{s,min} = \frac{1}{16}\sqrt{f_c{}'}\, b_w d \geq \frac{1}{3} b_w d$$

$$V_{s,min} = \frac{1}{16}\sqrt{f_c{}'}\, b_w d = \frac{1}{16} \times \sqrt{24} \times 150 \times 283 \times 10^{-3} = 12.997\ KN$$

$$V_{s,min} = \frac{1}{3} b_w d = \frac{1}{3} \times 150 \times 283 \times 10^{-3} = 14.15\ KN - control$$

$$\phi\ (V_c + V_{s,min}) = 42.74\ KN > V_{u,d} = 32.596\ KN > \phi\ V_c = 28.59\ KN$$

Provide minimum reinforcement

Use 2ø10 for stirrups $A_{v,2\ ø10}$ =157.07 mm^2

$$\frac{A_{v,min}}{s} = \frac{1}{3} \times \frac{b_w}{S} = \frac{1}{3} \times \frac{150}{420} = 0.119 \rightarrow \frac{A_{v,min}}{s} = \frac{157.07}{s} = 0.119 \rightarrow S = 1319.3\ mm$$

$$S_{max} \leq \frac{d}{2} \qquad and \quad \leq 600\ mm, \qquad S_{max} \leq \frac{283}{2} = 141.5\ mm\ < 600\ mm$$

Use 2ø10 @ 135cm c/c for all the distance

Note that the shear force at any distance > ϕ V_c

Beam Design

Figure shows the location of Beam 59in reference to the Ground floor

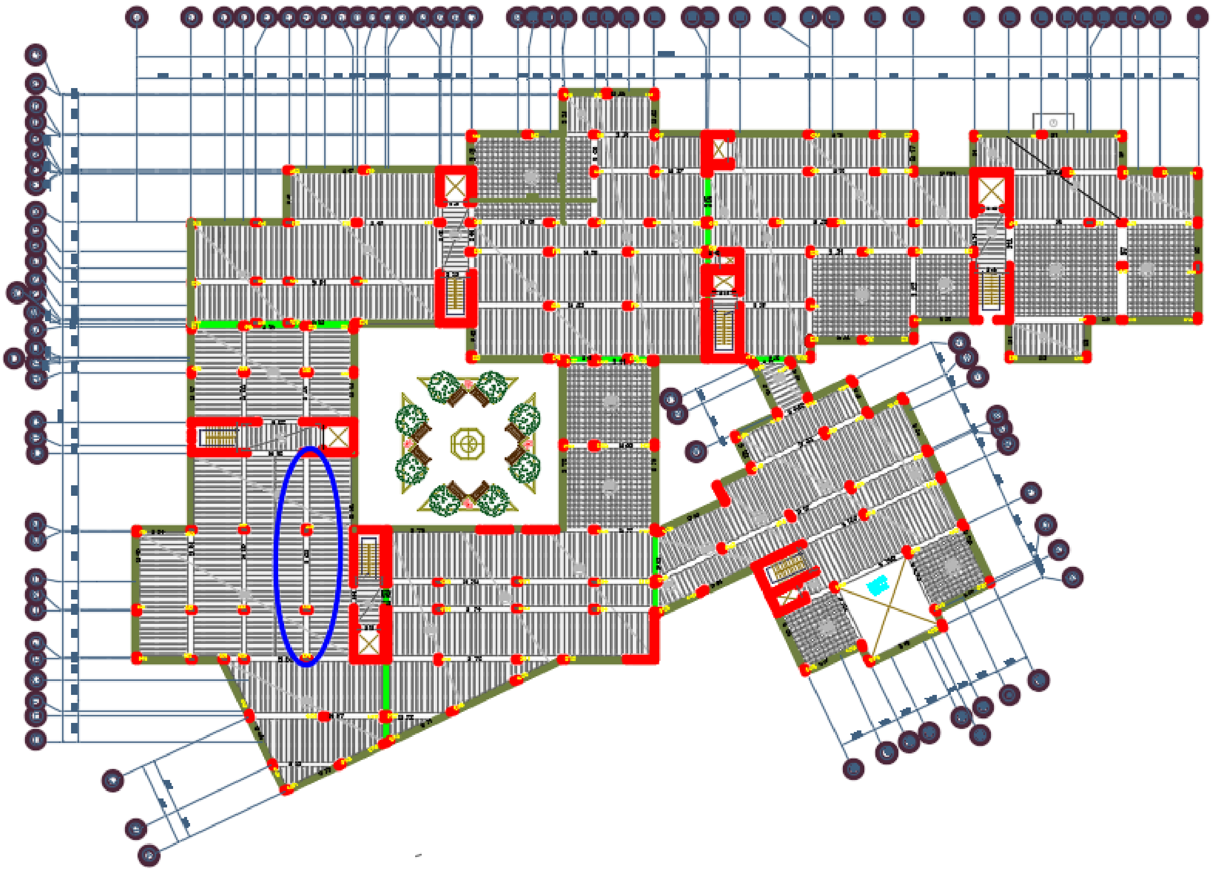

Figure 56:Beam Position

Material

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

Section

- B = 600 mm
- h= 700 mm
- d=700-40-10-16/2= 642 mm

Load Calculation for Beam59

Self-weight of beam =25×0.6×0.7=10.5 KN/m

w_{DL}, $from\ rib$16+rib17 = 85.98 /0.52 = 165.35KN/m

w_{LL}, $from\ rib$16+rib17 = 26.28 /0.52 = 50.54 KN/m

. Flexure Design for beam 59 as part of intermediate moment resisting frame:

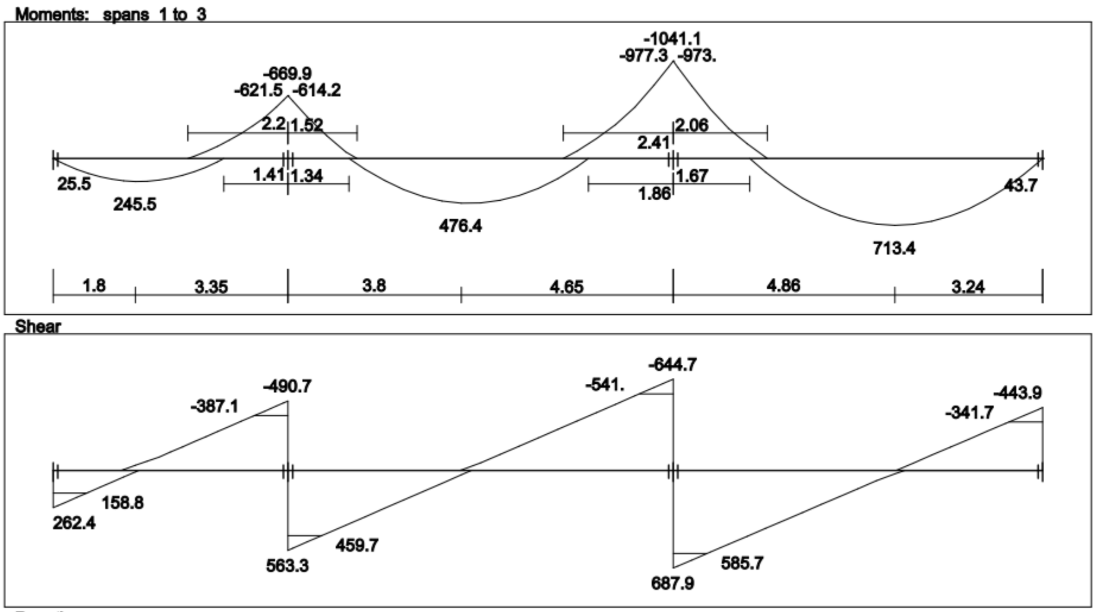

Figure 57:Beam moments

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

$c = \frac{3}{7}d = \frac{3}{7} \times 642 = 275.143\ m$

$c = \frac{a}{\beta}$ = a=0.85×275.143=233.87mm

$\emptyset Mn,ax = \emptyset 0.85 \times fc' \times a \times b \times (d - \frac{a}{2})$

$= 0.8 \times 0.85 \times 24 \times 233.87 \times 600 \times (642 - 233.87/2) \times 10^{-6} = 1202.43\ KN.m$

$Mu = 977.3KN.m < \emptyset M_{n,max}\ 1202.43\ KN.m$

Design the section as singly reinforced concrete section.

Design of Beam 59 for positive moments:

Assume bar diameter Ø 16 for main positive reinforcement

d= 7000-40-10- 16 /2 = 642 mm

- Span 3 (Max Positive Moment= 713.4 KN.m)

$Rn = \frac{Mn}{\emptyset bd^2} = \frac{713.4\times10^6}{0.9\times600\times642^2}$ =3.2 MPa

$m = \frac{fy}{0.85fc\prime} = \frac{420}{0.85\times24}$ = 20.6

$\rho = \frac{1}{m}\ (\frac{1-\sqrt{1-2\mathrm{Rnm}}}{fy})$

$= \ \frac{1}{20.6}\ (\frac{1-\sqrt{1-2\times3.2\times20.6}}{420})$= 0.0083345

$As = \rho bd$ = 0.0083345× 600 × 642 = 1455.91 mm^2

for As,min

$As,min = \frac{0.25\sqrt{\mathrm{fc}\prime}}{\mathrm{fy}}\, bwd \quad \geq \frac{1.4}{\mathrm{fy}}\, bwd$

$As,min = \frac{0.25\sqrt{24}}{420}$ ×600×642= $1123.266mm2 \quad < \frac{1.4}{420}$ ×600×642=12484mm^2 . *control*

As,min =1.248cm2 < As ,req=1.455cm^2 – OK

Use 8Ø16 with

As = 1.6088 $cm2$ > As,req = 1.455cm^2 - OK

Check for strain

$a = \frac{Asfy}{0.85fc'b}$

$= \frac{1608.8 \times 420}{0.85 \times 24 \times 600} = 55.20\ mm$

$c = \frac{a}{\beta 1} = \frac{55.20}{0.85} = 64.95\ mm$

$\varepsilon t == 0.003(\frac{d-c}{c}) = 0.003(\frac{642-64.95}{64.95}) = 0.02665 > 0.005$ - OK

Check for bar placement:

$s_b = \frac{600-40\times2-10\times2-8\times16}{7} = 63.43mm > 25mm$

Design of Beam 59 for negative moments:

Assume bar diameter Ø 16 for main positive reinforcement

d= 7000-40-10- 16 /2 = 642 mm

• Span 3 (Max Positive Moment= 977.3 KN.m)

$Rn = \frac{Mn}{\emptyset bd^2} = \frac{977.3\times10^6}{0.9\times600\times642^2} = 4.39\ MPa$

$m = \frac{fy}{0.85fc\prime} = \frac{420}{0.85\times24} = 20.6$

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

$= \frac{1}{20.6} \left(\frac{1-\sqrt{1-2\times 4.39\times 20.6}}{420}\right) = 0.011914$

$As = \rho bd = 0.01191\times 600 \times 642 = 4589.47\ mm^2$

for As,min

$As,n = \frac{0.25\sqrt{\mathrm{fc}\,\prime}}{\mathrm{fy}}\, bwd \quad \geq \frac{1.4}{\mathrm{fy}}\, bwd$

$As,\mathrm{m}in = \frac{0.25\sqrt{24}}{420} \times 600\times 642 = 1123.266mm2 \quad < \frac{1.4}{420} \times 600\times 642 = 12484 mm^2$. *control*

$As,\mathrm{m}in = 1.248 cm^2 < As$,req=4.589cm^2 – *OK*

Use 23Ø16 with

$As = 4.6253\ cm^2 > As,\mathrm{re}q = 4.525 cm^2$ - *OK*

Check for strain:

$a = \frac{\mathrm{Asfy}}{0.85 fc'b}$

$= \frac{4625.3\times 420}{0.85\times 24\times 600} = 158.71\ mm$

$\mathrm{c} = \frac{\mathrm{a}}{\beta 1} = \frac{158.71}{0.85}\, 186.72\ mm$

$\varepsilon t == 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{642-186.72}{186.72}\right) = 0.00731 > 0.005$ - *OK*

$sb = \frac{600-40\times2-10\times2-23\times16}{22}$ =13.3$mm < 25mm$ →Two layers

Shear Design of beam 59 :

$Vu\ max$ =585.7kN

d=700-40-10-16-(25/2)=621.5mm

$Vn = \frac{Vumax}{\emptyset} = \frac{585.7}{0.75} =$ 780.93kN

$Vc = \frac{1}{6}\ \lambda\sqrt{fc'}bwd = \frac{1}{6}\ \sqrt{24} \times 600 \times 621.5 \times 10^{-3} = 304.5kN$

$\emptyset Vc = 0.75 \times 30.5 = 228.4\ kN$

$Vs = Vn\text{-}Vc$ =780.93 - 304.5= 476.433kN

$Vs,\ max = \frac{2}{3}\ \sqrt{fc}\ bwd = \frac{2}{3}\ \times \sqrt{24} \times 600 \times 621.5 \times 10^{-3} = 1217.886\ kN$

$Vs,\ max$ =1217.886 $kN > Vs =$ 476.433kN → *The section is enough*

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

$\frac{Av}{s} = \frac{Vs}{fyd} \rightarrow s = \frac{Avfyd}{Vs}$

$s = \frac{4\times0.140\times420\times621.5}{476.433} = 306.82mm$

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*70 = 140 cm.

from the face of the support is the smallest of the following:

smax = d/4=155.37mm (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.

Column Design

Columns loads are taken from safe and attire then designed using spcolumn, results are shown below

Material Properties

f'c = 24 MPa

fy = 420 MPa

Ec = 23025.2 MPa

Es = 199955 MPa

Ultimate strain = 0.003 mm/mm

Beta1 = 0.85

Section

Rectangular: Width = 550 mm, Depth = 500 mm

Gross section area, Ag = 275000 mm^2

Ix = 5.72917e+009 mm^4 Iy = 6.93229e+009 mm^4

rx = 144.338 mm ry = 158.771 mm

Loads

Dead load= 2155.10 KN

Live load= 520.00 KN

Reinforcement

Bar selection: Minimum area of steel

Asmin = 0.01 * Ag = 2750 mm^2, Asmax = 0.08 * Ag = 22000 mm^2

Confinement: Tied; #8 ties with #20 bars, #10 with larger bars.

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular

Pattern: All Sides Equal (Cover to transverse reinforcement)

Total steel area: As = 3048 mm^2 at rho = 1.11%

Minimum clear spacing = 111 mm

12 #18 Cover = 40 mm

Results summary

```
Factored Loads and Moments with Corresponding Capacities:
=========================================================
  Design/Required ratio PhiMn/Mu >= 1.00
  NOTE: Each loading combination includes the following cases:
        First line  - at column top
        Second line - at column bottom
       Load         Pu        Mux        Muy     PhiMnx     PhiMny PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.  Combo        kN        kNm        kNm        kNm        kNm                mm       mm
  --- ------ ---------- ---------- ---------- ---------- ---------- -------- -------- -------- -------- ------
    1  1 U1     3418.12     136.39     134.75     137.86     136.20    1.011      605      651  0.00023  0.650
    2           3418.12     136.39     134.75     137.86     136.20    1.011      605      651  0.00023  0.650
```

Figure 58:spcolumn results

Interaction diagram:

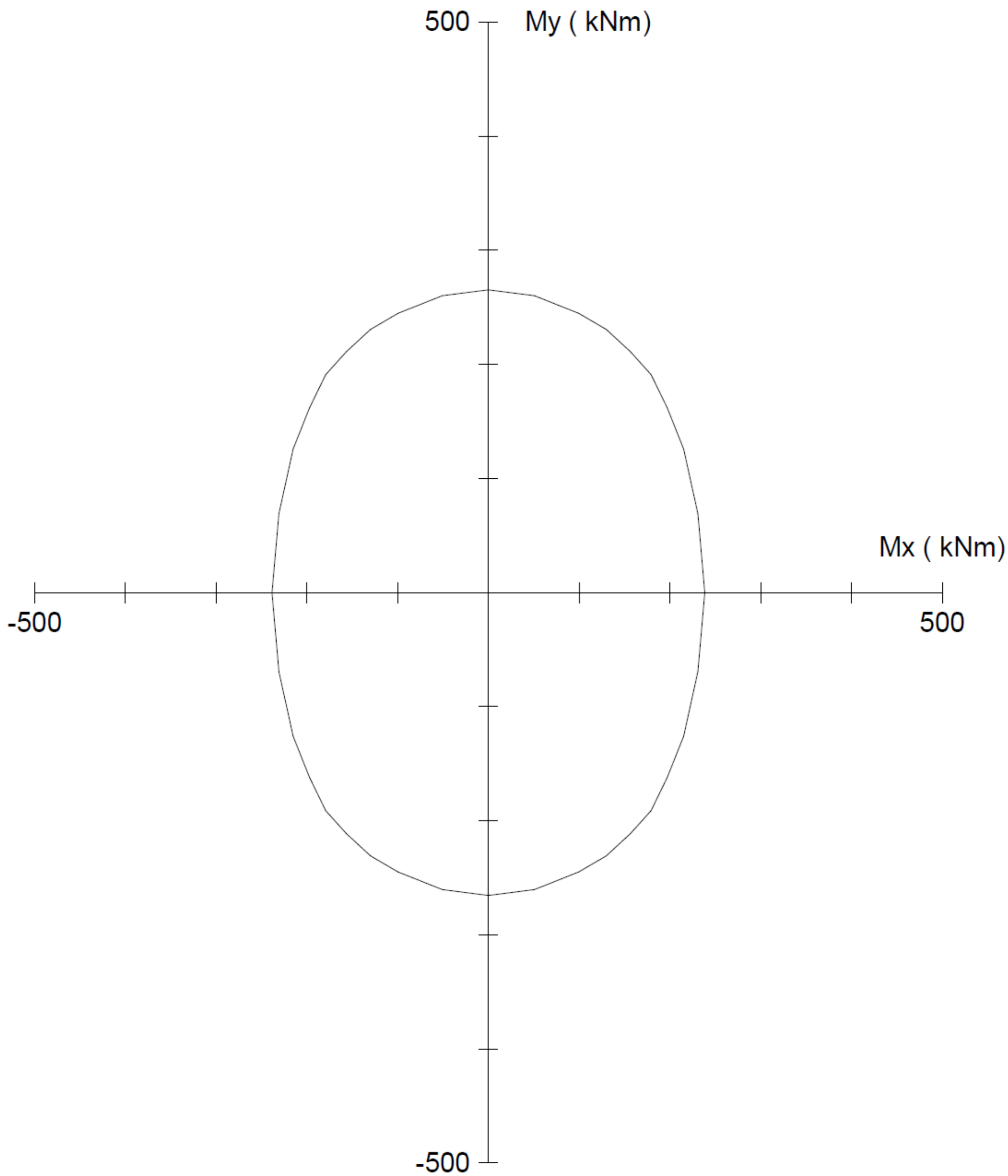

Figure 59: Column interaction diagram

Shear Wall Design

In the beginning of shear walls seismic analysis, building units are modeled and analyzed statically to compute building irregularities, the figure below shows units numbers:

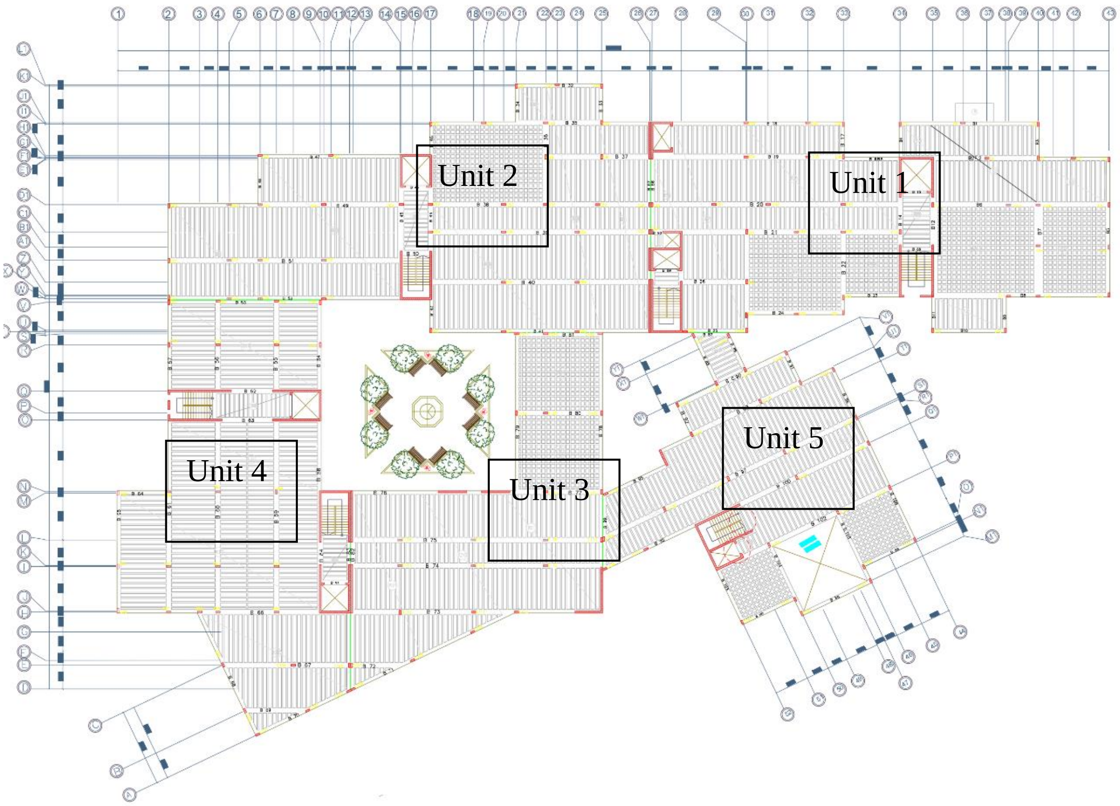

Figure 60:Units Plan

Unit 5 is a two-story building, which has a short shear walls, the seismic effect doesn't appear in short wall so there is no need to analyze building in static or dynamic analysis, shear walls in unit 5 are designed only resisting gravity loads.

For the rest of units, the calculated irregularities are shown in tables below:

Table 8: Unite 1 Irregularities

unit 1

Torsional Irergullarity

Story	Output Case	Ratio	
Story5	EQX+5	1.073	0.719581
Story4	EQX+5	1.047	0.685131
Story3	EQX+5	1.046	0.683823
Story2	EQX+5	1.044	0.68121
Story1	EQX+5	1.041	0.677301

Story	Output Case	Ratio	
Story5	EQX-5	1.026	0.657923
Story4	EQX-5	1.007	0.633781
Story3	EQX-5	1.005	0.631266
Story2	EQX-5	1.003	0.628756
Story1	EQX-5	1	0.625

Story	Output Case	Ratio	
Story5	EQY+5	1.178	0.867303
Story4	EQY+5	1.435	1.287016
Story3	EQY+5	1.438	1.292403
Story2	EQY+5	1.486	1.380123
Story1	EQY+5	1.483	1.374556

Story	Output Case	Ratio	
Story5	EQY-5	1.076	0.72361
Story4	EQY-5	1.202	0.903003
Story3	EQY-5	1.198	0.897003
Story2	EQY-5	1.224	0.93636
Story1	EQY-5	1.206	0.909023

Stifness Irergullarity

Story	Output Case	Stiff X	ki/ki+1	kh/(ki+1+
		kN/m		ki+2+ki+3
Story5	EQX+5	301455.5	3.668737	#VALUE!
Story4	EQX+5	1105961	1.616324	#VALUE!
Story3	EQX+5	1787591	1.436362	2.410913
Story2	EQX+5	2567628	1.829712	2.580772
Story1	EQX+5	4698021		

Story	Output Case	Stiff Y	ki/ki+1
		kN/m	
Story5	EQY+5	961191.2	2.387804
Story4	EQY+5	2295136	1.569942
Story3	EQY+5	3603230	1.346862
Story2	EQY+5	4853053	1.876266
Story1	EQY+5	9105621	

Mass Irergullarity

Story	Diaphragm	Mass X	Mass Y	mi/mi+1	mi/mi-1
		kg	kg		
Story5	D3	260098.5	3.790993		
Story4	D3	986031.5	1.149899	0.263783	3.296805
Story3	D3	1133837	1.141947	0.869641	1.006964
Story2	D3	1294781	1.000257	0.875697	1.141654
Story1	D3	1295114		0.999743	#DIV/0!

Table 9: Unite 2 Irregularities

Unit 2

Torsional Irergullarity

Story	Output Case	Ratio	
Story5	EQX+5	1.089	0.81675
Story4	EQX+5	1.085	0.81375
Story3	EQX+5	1.08	0.81
Story2	EQX+5	1.073	0.80475
Story1	EQX+5	1.06	0.795

Story	Output Case	Ratio	
Story5	EQX-5	1.005	0.75375
Story4	EQX-5	1.003	0.75225
Story3	EQX-5	1.001	0.75075
Story2	EQX-5	1.002	0.7515
Story1	EQX-5	1.006	0.7545

Story	Output Case	Ratio	
Story5	EQY-5	1.735	1.30125
Story4	EQY-5	1.743	1.30725
Story3	EQY-5	1.752	1.314
Story2	EQY-5	1.767	1.32525
Story1	EQY-5	1.799	1.34925

Story	Output Case	Ratio	
Story5	EQY+5	1.355	1.01625
Story4	EQY+5	1.355	1.01625
Story3	EQY+5	1.357	1.01775
Story2	EQY+5	1.361	1.02075
Story1	EQY+5	1.384	1.038

Stifness Irergullarity

Story	Output Case	Stiff X kN/m	ki/ki+1	kh/(ki+1+ ki+2+ki+3
Story5	EQX+5	362517.8	1.536973	#VALUE!
Story4	EQX+5	557180	1.236628	#VALUE!
Story3	EQX+5	689024.2	1.280283	1.645056
Story2	EQX+5	882146.1	1.926045	2.394887
Story1	EQX+5	1699053		

Story	Output Case	Stiff X kN/m	ki/ki+1
Story5	EQX-5	379316.4	1.536271
Story4	EQX-5	582732.7	1.236143
Story3	EQX-5	720341.2	1.279962
Story2	EQX-5	922009.6	1.887741
Story1	EQX-5	1740516	

Story	Output Case	Stiff Y kN/m	ki/ki+1	kh/(ki+1+ ki+2+ki+3
Story5	EQY-5	605125.8	1.7734	#VALUE!
Story4	EQY-5	1073130	1.419847	#VALUE!
Story3	EQY-5	1523680	1.453158	2.074509
Story2	EQY-5	2214148	1.977967	2.73096
Story1	EQY-5	4379512		

Story	Output Case	Stiff Y kN/m	ki/ki+1
Story5	EQY+5	718662.9	1.778714
Story4	EQY+5	1278296	1.424956
Story3	EQY+5	1821515	1.460047
Story2	EQY+5	2659497	2.001893
Story1	EQY+5	5324027	

Mass Irergullarity

Story	Diaphragm	Mass X kg	Mass Y kg	mi/mi+1	mi/mi-1
Story5	D1	1192255	1192255	1.016916	
Story4	D1	1212423	1212423	1	0.983366

Table 10: Unite 3 Irregularities

Unit 3

Torsional Irergullarity

Story	Output Case	Ratio	
Story5	EQX+5	1.174	0.8805
Story4	EQX+5	1.176	0.882
Story3	EQX+5	1.177	0.88275
Story2	EQX+5	1.179	0.88425
Story1	EQX+5	1.241	0.93075

Story	Output Case	Ratio
Story5	EQX-5	1.083
Story4	EQX-5	1.089
Story3	EQX-5	1.095
Story2	EQX-5	1.1
Story1	EQX-5	1.134

Story	Output Case	Ratio	
Story5	EQY-5	1.332	0.999
Story4	EQY-5	1.327	0.99525
Story3	EQY-5	1.324	0.993
Story2	EQY-5	1.324	0.993
Story1	EQY-5	1.338	1.0035

Story	Output Case	Ratio
Story5	EQY+5	1.466
Story4	EQY+5	1.462
Story3	EQY+5	1.459
Story2	EQY+5	1.46
Story1	EQY+5	1.474

Stifness Irergullarity

Story	Output Case	Stiff X kN/m	ki/ki+1	kh/(ki+1+ki +2+ki+3/2)
Story5	EQX+5	362517.807	1.536973	#VALUE!
Story4	EQX+5	557180.02	1.236628	#VALUE!
Story3	EQX+5	689024.152	1.280283	1.6450563
Story2	EQX+5	882146.066	1.926045	2.3948873
Story1	EQX+5	1699053.01		

Story	Output Case	Stiff X kN/m
Story5	EQX-5	379316.39
Story4	EQX-5	582732.68
Story3	EQX-5	720341.173
Story2	EQX-5	922009.567
Story1	EQX-5	1740515.58

Story	Output Case	Stiff Y kN/m	ki/ki+1	kh/(ki+1+ki +2+ki+3/2)
Story5	EQY-5	610529.577	1.718889	#VALUE!
Story4	EQY-5	1049432.4	1.380597	#VALUE!
Story3	EQY-5	1448843.19	1.411538	1.9735203
Story2	EQY-5	2045096.68	1.914698	2.5855741
Story1	EQY-5	3915741.89		

Story	Output Case	Stiff Y kN/m
Story5	EQY+5	590426.169
Story4	EQY+5	1016759.3
Story3	EQY+5	1406184.32
Story2	EQY+5	1986661.22
Story1	EQY+5	3797509.17

Mass Irergullarity

Story	Diaphragm	Mass X kg	Mass Y kg	mi/mi+1	mi/mi-1
Story5	D1	678882.07	678882.1	1.0130809	
Story4	D1	687762.43	687762.4	1	0.987088

Table 11: Unite 4 Irregularities

Unit 4

Torsional Irergullarity

Story	Output Case	Ratio	
Story5	EQX+5	2.168	1.626
Story4	EQX+5	2.182	1.6365
Story3	EQX+5	2.188	1.641
Story2	EQX+5	2.196	1.647
Story1	EQX+5	2.217	1.66275

Story	Output Case	Ratio	
Story5	EQX-5	2.179	1.63425
Story4	EQX-5	2.178	1.6335
Story3	EQX-5	2.179	1.63425
Story2	EQX-5	2.182	1.6365
Story1	EQX-5	2.191	1.64325

Story	Output Case	Ratio	
Story5	EQY-5	1.628	1.221
Story4	EQY-5	1.634	1.2255
Story3	EQY-5	1.641	1.23075
Story2	EQY-5	1.652	1.239
Story1	EQY-5	3.907	2.93025

Story	Output Case	Ratio	
Story5	EQY+5	1.562	1.1715
Story4	EQY+5	1.569	1.17675
Story3	EQY+5	1.576	1.182
Story2	EQY+5	1.587	1.19025
Story1	EQY+5	3.904	2.928

Stifness Irergullarity

Story	Output Case	Stiff X kN/m	ki/ki+1	kh/(ki+1+ ki+2+ki+3
Story5	EQX+5	1152020	1.642227	#VALUE!
Story4	EQX+5	1891879	1.418159	#VALUE!
Story3	EQX+5	2682985	0.667509	0.938163
Story2	EQX+5	1790917	1.755162	1.481366
Story1	EQX+5	3143350		

Story	Output Case	Stiff X kN/m	ki/ki+1
Story5	EQX-5	1051774	1.767054
Story4	EQX-5	1858541	1.410436
Story3	EQX-5	2621353	0.817791
Story2	EQX-5	2143720	1.740606
Story1	EQX-5	3731371	

Story	Output Case	Stiff Y kN/m	ki/ki+1	kh/(ki+1+ ki+2+ki+3
Story5	EQY-5	548995.6	1.420128	#VALUE!
Story4	EQY-5	779643.9	1.172618	#VALUE!
Story3	EQY-5	914224.4	1.288385	1.575495
Story2	EQY-5	1177873	2.411189	2.96692
Story1	EQY-5	2840076		

Story	Output Case	Stiff Y kN/m	ki/ki+1
Story5	EQY+5	638061	1.365494
Story4	EQY+5	871268.6	1.171776
Story3	EQY+5	1020931	1.284602
Story2	EQY+5	1311490	2.444205
Story1	EQY+5	3205551	

Mass Irergullarity

Story	Diaphragm	Mass X kg	Mass Y kg	mi/mi+1	mi/mi-1
Story5	D1	772262.5	772262.5	1.016013	
Story4	D1	784629	784629	0.999975	0.984239

Building irregularities shows that the static analysis cannot be used to any unit of building, dynamic analysis shall be done to unites and designed based on the results, taking unit 2 as example, all the results bellow are Etabs 18 Dynamic analysis results, As calculated and mentioned before the seismic loads are assigned to all units using etabs.

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

Figure 61:Code Table

- Modal Response Spectrum Analysis

12.9.1.1 Number of Modes. An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of 100% of the structure's mass. For this purpose, it shall be permitted to represent all modes with periods less than 0.05 s in a single rigid body mode that has a period of 0.05 s.

Alternatively, the analysis shall be permitted to include a minimum number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each orthogonal horizontal direction of response considered in the model.

In all units, modes number is 15 for less, the minimum modal mass participations has reached to 98% of the actual mass, which is accepted.

- Scaling Design Values of Combined Response

12.9.1.4 Scaling Design Values of Combined Response. A base shear (V) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 12.8. 12.9.1.4.1 Scaling of Forces. Where the calculated fundamental period exceeds CuTa in a given direction, CuTa shall be used in lieu of T in that direction. Where the combined response for the modal base shear (Vt) is less than 100% of the calculated base shear (V) using the equivalent lateral force procedure, the forces hall be multiplied by V/Vt where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8,

Vt = the base shear from the required modal combination.

- Modification of Response for Design

12.9.2.5.1 Determination of Maximum Elastic and Inelastic Base Shear. For each ground motion analyzed, a maximum elastic base shear, designated as VEX and VEY in the X and Y directions, respectively, shall be determined. The mathematical model used for computing the maximum elastic base shear shall not include accidental torsion. For each ground motion analyzed, a maximum inelastic base shear, designated as VIX and VIY in the X and Y directions, respectively, shall be determined as follows:

$$V_{IX} = \frac{V_{EX} I_e}{R_X} \qquad (12.9\text{-}1)$$

$$V_{IY} = \frac{V_{EY} I_e}{R_Y} \qquad (12.9\text{-}2)$$

application in unit 2:

Table 12:Scale Factor

Static / dynamic			
Output Case	FX kN	Output Case	FY kN
EQX	-1103.12	EQY	-1814.63
RSAX	989.0329	RSAY	951.4218
FX=	-1.11535	FY=	-1.90729

Unit 2 dynamic analysis results (Etabs Report):

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

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [ASCE Table 12.8-2] $C_t = 0.02\text{ft}$

Coefficient, x [ASCE Table 12.8-2] $x = 0.75$

Structure Height Above Base, h_n $h_n = 59.06\ \text{ft}$

Long-Period Transition Period, T_L [ASCE 11.4.5] $T_L = 4\ \text{sec}$

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1] $R = 6$

System Overstrength Factor, Ω_0 [ASCE Table 12.2-1] $\Omega_0 = 2.5$

Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	$C_d = 5$
Importance Factor, I [ASCE Table 1.5-2]	$I = 1.5$

Ss and S1 Source = 0.75

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.2]	$S_s = 0.35g$
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.2]	$S_1 = 0.09g$
Site Class [ASCE Table 20.3-1] = A - Hard Rock	
Site Coefficient, F_a [ASCE Table 11.4-1]	$F_a = 0.8$
Site Coefficient, F_v [ASCE Table 11.4-2]	$F_v = 0.8$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.4, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.28g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.4, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.072g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.186667g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.048g$

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2]	$C_S = \frac{S_{DS}}{(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-3] $C_{S,max} = \frac{S_{D1}}{T(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-5] $C_{S,min} = \max(0.044 S_{DS} I, 0.01) = 0.01232$

[ASCE 12.8.1.1, Eq. 12.8-6] $C_{S,min} = 0.5 \frac{S_1}{(\frac{R}{I})} \text{for} S_1 = 0.6g$

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

Calculated Base Shear

Table 13: Base shear in x-direction

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	0.687	0.017475	63124.7567	1103.1164

Applied Story Forces

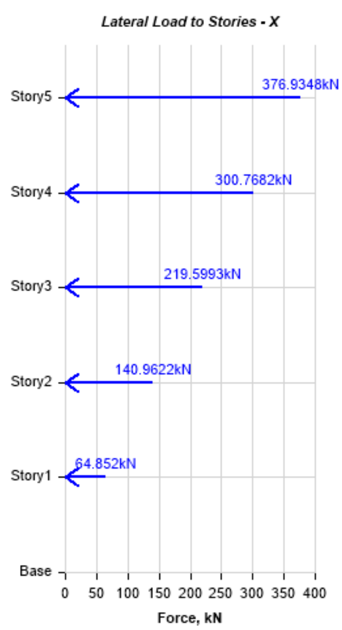

Figure 62: Story Forces

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [ASCE Table 12.8-2]	$C_t = 0.02\text{ft}$
Coefficient, x [ASCE Table 12.8-2]	$x = 0.75$
Structure Height Above Base, h_n	$h_n = 59.06\ \text{ft}$
Long-Period Transition Period, T_L [ASCE 11.4.5]	$T_L = 4\ \text{sec}$

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	$R = 6$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	$\Omega_0 = 2.5$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	$C_d = 5$
Importance Factor, I [ASCE Table 1.5-2]	$I = 1.5$

Ss and S1 Source = 0.75

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.2]	$S_s = 0.35\text{g}$
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.2]	$S_1 = 0.09\text{g}$

Site Class [ASCE Table 20.3-1] = A -
Hard Rock

Site Coefficient, F_a [ASCE Table 11.4-1]	$F_a = 0.8$
Site Coefficient, F_v [ASCE Table 11.4-2]	$F_v = 0.8$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.4, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.28g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.4, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.072g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.186667g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.048g$

Equivalent Lateral Forces

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

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

[ASCE 12.8.1.1, Eq. 12.8-3]

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

[ASCE 12.8.1.1, Eq. 12.8-5]

$$C_{S,min} = \max(0.044 S_{DS} I, 0.01) = 0.01232$$

[ASCE 12.8.1.1, Eq. 12.8-6]

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

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

Calculated Base Shear

Table 14: Base shear Y-dir

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
Y	0.417	0.028747	63124.7567	1814.6332

Applied Story Forces

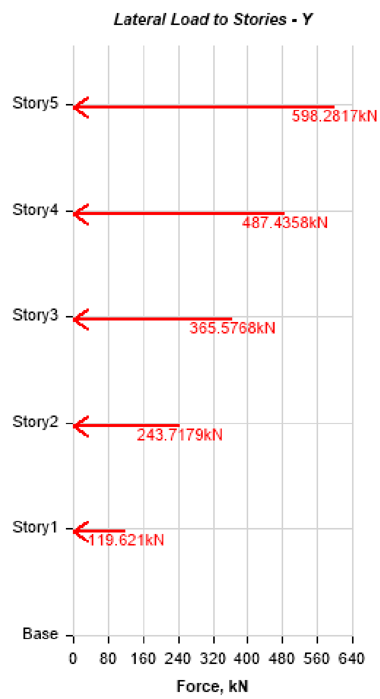

Figure 63:Story Forces Y-dir

Stairs Design

Dimensions:

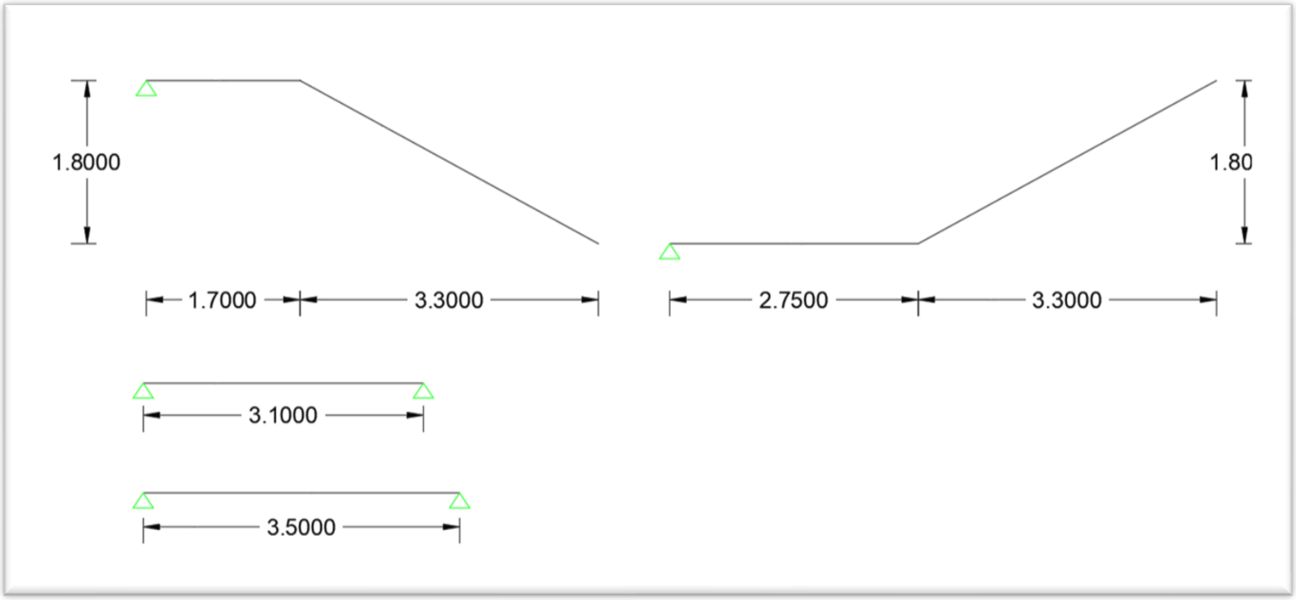

Figure 64:Stairs Dimensions

Materials properties:

$f_{c'}$=24 MPa

f_{y}=420 MPa

H=$\frac{6.05}{28} = 0.216$

Take H=25cm

Load calculations:

- Live load = 3 KN/m

$$\theta = \tan^{-1}\frac{rise}{run} = \frac{150}{300} = 26.56°$$

- Flight dead load

Table 16:Flight DL

Material	Density	W	
Tiles	22	$22 \times (\frac{0.15 \times 0.35}{0.3}) \times 0.03 \times 1$	0.1155
Mortar	22	$22 \times (\frac{0.15 \times 0.3}{0.3}) \times 0.02 \times 1$	0.066
Stair Steps	25	$\frac{25}{0.3} \times (\frac{0.15 \times 0.3}{2}) \times 1$	1.875
RC solid slab	25	$\frac{25 \times 0.25}{\cos 26.56}$	6.99
Plaster	22	$\frac{22 \times 0.03}{\cos 26.56}$	0.74
Total Dead Load, KN/m			9.7865

- Landing dead load:

Table 15:Landing DL

Material	Density	W	
Tiles	22	$22 \times 0.03 \times 1$	0.66
Mortar	22	$22 \times 0.02 \times 1$	0.44
RC solid slab	25	$25 \times 0.25 \times 1$	6.25
Plaster	22	$22 \times 0.03 \times 1$	0.66
Total Dead Load, KN/m			8.01

- Total factored load:

For flight:

$$w = 1.2D.L + 1.6\,L.L = (1.2*9.7865) + (1.6*3) = 15.9438\ KN/m$$

For landing:

$$w = 1.2D.L + 1.6\,L.L = (1.2*8.01) + (1.6*3) = 13.812 KN/m$$

Structural analysis:

- Landing

1- First Landing = 3.5m

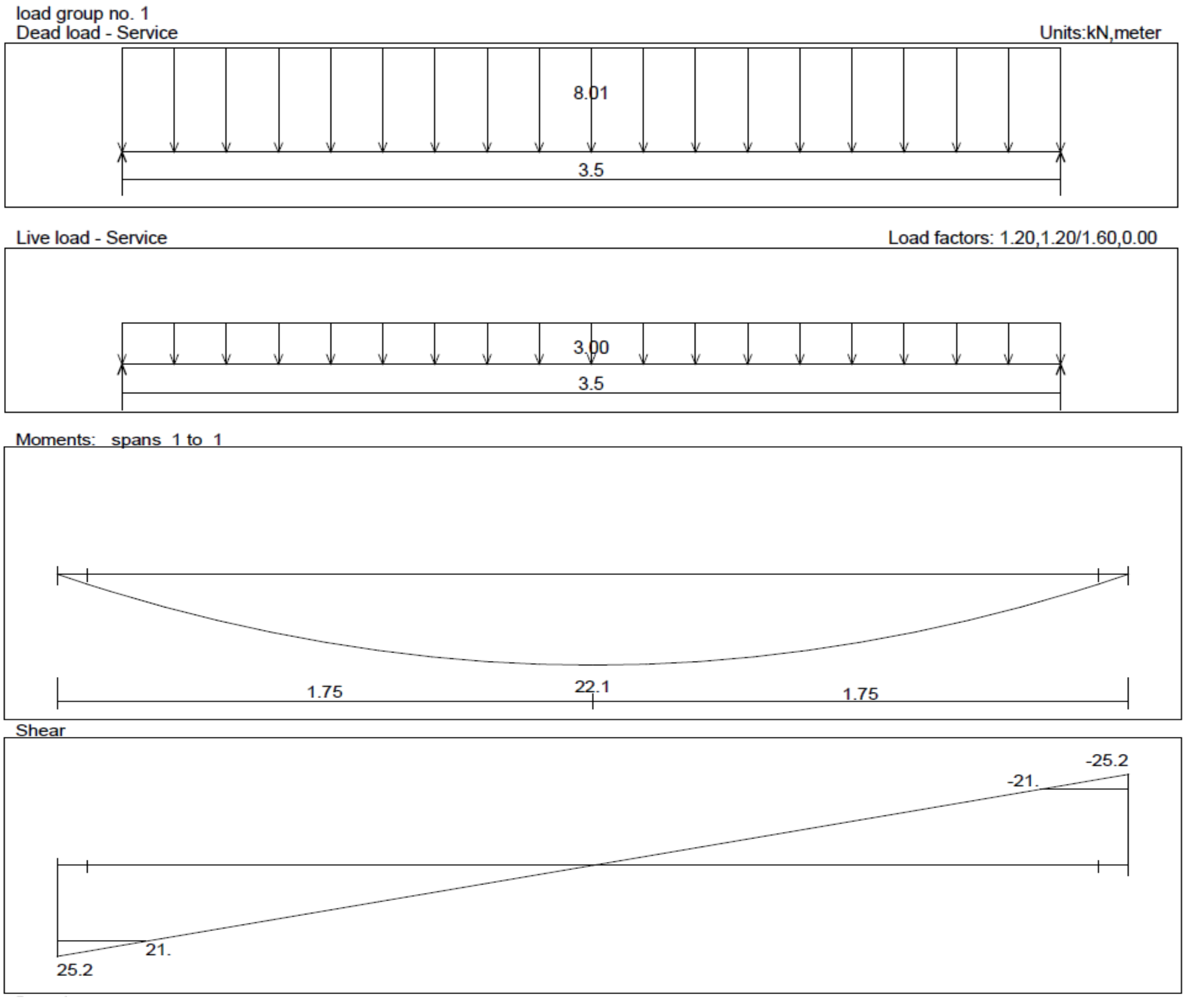

Figure 66: Landing 1 moments

Second landing=3.1

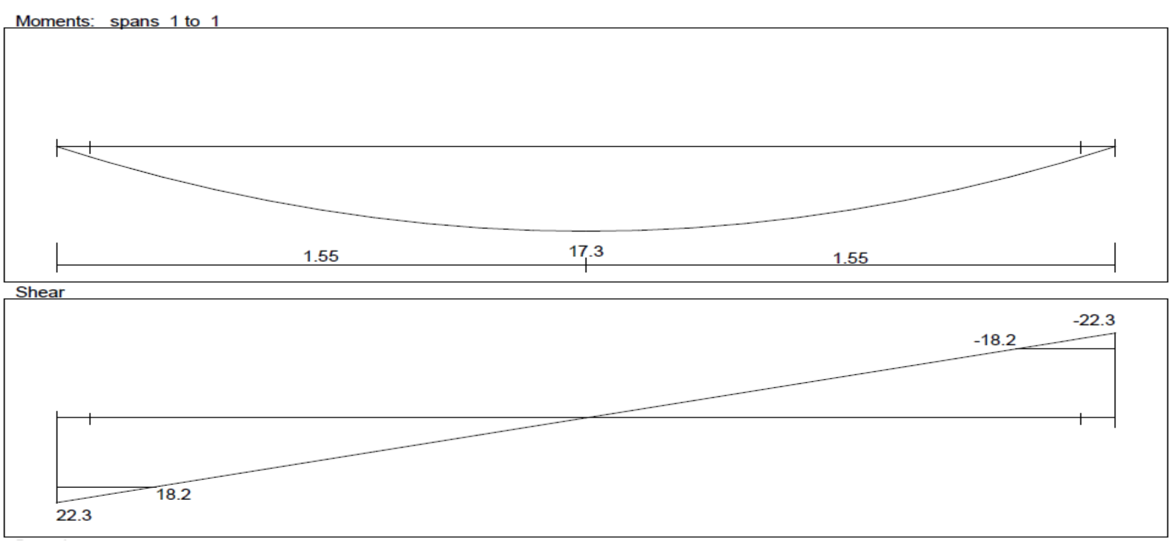

Figure 65: Landing 2 moments

- Flight

1- first flight

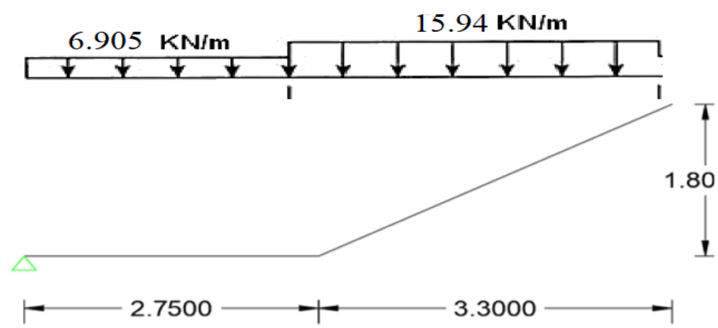

Figure 67: Flight 1 System

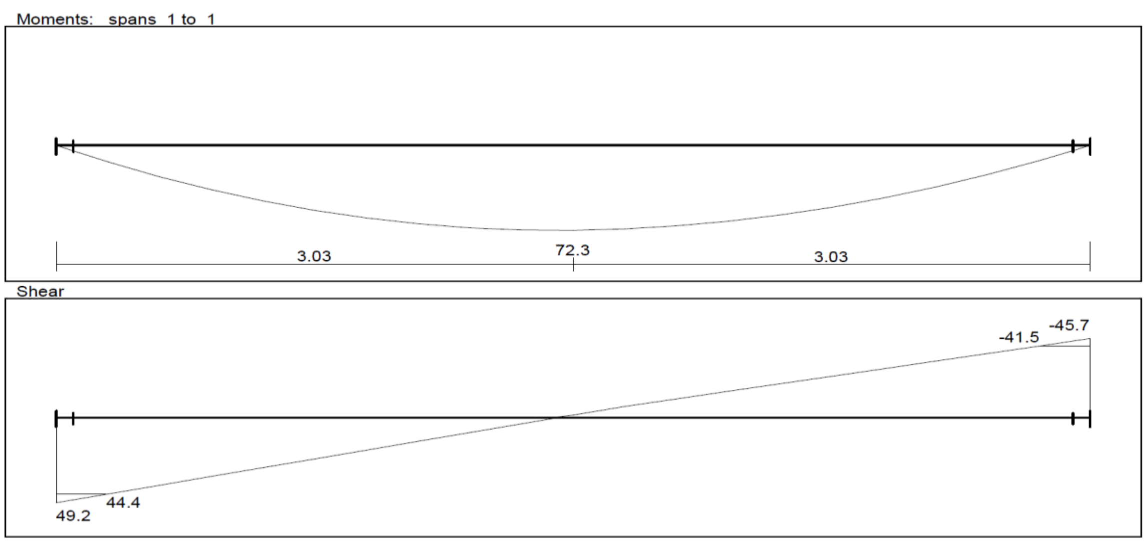

Figure 68:Flight 1 moments

2- second flight

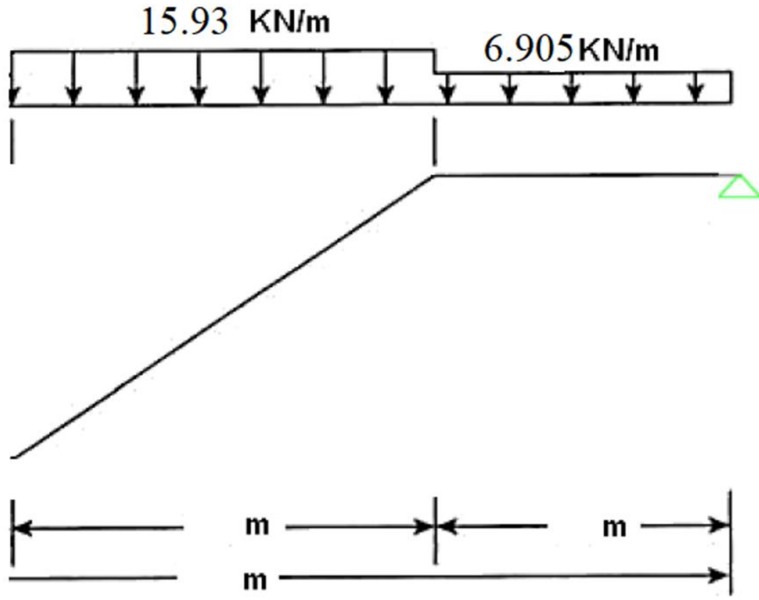

Figure 69: Flight 2 system

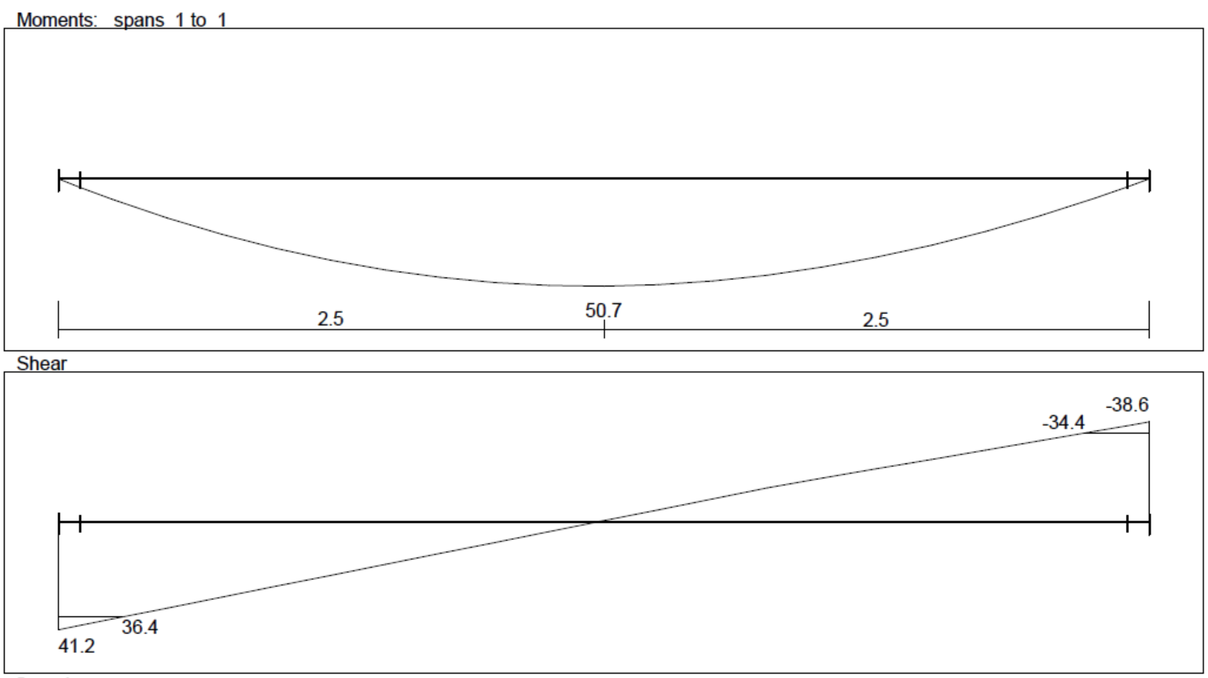

Figure 70:Flight 2 moments

- check for shear strength:

Assume bar diameter ø 14 for main reinforcement

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

Assume beam width 25cm

Max shear V_u=38.6KN

$V_c = \frac{1}{6}\sqrt{fc'}b_w\ d == \ \frac{1}{6}\sqrt{24} * 1000 * 223 * 10^{-3} = 182.1\ KN$

$\Phi V_{c\,=}$ 0.75* 182.1 = 136.6KN

$V_{u,max}$=38.6 $KN < \frac{1}{2}\,\Phi V_c$=68.7 KN

The thickness of the slab is adequate enough

- Reinforcement design:

- For landing:

(Max positive moment= 22.1 Kn.m)

Assume bar diameter ø 14 for main reinforcement

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

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{22.1\times10^6}{0.9\times1000\times223^2} = 0.49Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times0.49}{420}}\right) = 0.00118$

$A_{s,req}$ = ρbd = 0.00118×1000×223 = 263.37mm^2

Check for $A_{s,min}$

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

A_s =263.37 mm^2 < $A_{s,\,min}$=450 mm^2

Use ϕ12

$n = \frac{A_s}{A_{s\emptyset 12}} = \frac{450}{113.09} = 3.9, \qquad s = \frac{1}{n} = \frac{1}{3.9} = 0.2513$

Take ϕ12@250mm OR 4 ϕ12/m with A_s=452.36mm^2/m

Steps is the smallest of:

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

S = 250 mm < S_{max} = 300mm – OK

- For Flight:

(Max positive moment= 22.1 Kn.m)

Assume bar diameter ø 14 for main reinforcement

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

$R_n = \frac{M_u}{\emptyset bd^2} = \frac{72.3\times10^6}{0.9\times1000\times223^2} = 1.615Mpa$

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

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{420}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2\times20.6\times1.6154}{420}}\right) = 0.00401198$

$A_{s,req} = \rho bd = 0.00401198\times1000\times223 = 894.6715mm^2$

Check for $A_{s,min}$

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

$A_s = 894.67\ mm^2 > A_{s,\,min} = 450\ mm^2$

Use ϕ14

$n = \frac{A_s}{A_{s\emptyset 14}} = \frac{894.67}{153.93} = 5.8, \qquad s = \frac{1}{n} = \frac{1}{5.8} = 0.1724$

Take ϕ14@150mm OR 6 ϕ14/m with A_s=923.58mm^2/m

Steps is the smallest of:

1. 3h = 3*250=750mm

2. 450 mm

3. $s = 380\ (\frac{280}{fs}) - 2.5c_c = 380\ (\frac{280}{280}) - 2.5*20 = 330mm$

4. $s \leq 300(\frac{280}{fs}) = 300 * (\frac{280}{280}) = 300mm - control$

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

- Temperature and shrinkage reinforcement.

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

$n = \frac{A_s}{A_{s\emptyset12}} = \frac{450}{113.09} = 3.9,\qquad s = \frac{1}{n} = \frac{1}{3.9} = 0.2513$

Take ϕ12@250mm OR 4 ϕ12/m with A_s=452.36mm^2/m

Design Footing

Isolated footing design:

Loads that act on footing F1 are: -

PD =152 kN , PL = 29 kN

The following parameters are used in design:

- γ concrete = 25 kN/m3

- γ soil = 18 kN/m3

- σallow = 200kN/m2

- clear cover =7.5cm

- the diminution of column is 50*30

- service surcharge = 5 KN/m2

- soil depth = 0.40 cm

- fc' = 24 MP , fy = 420 MPa

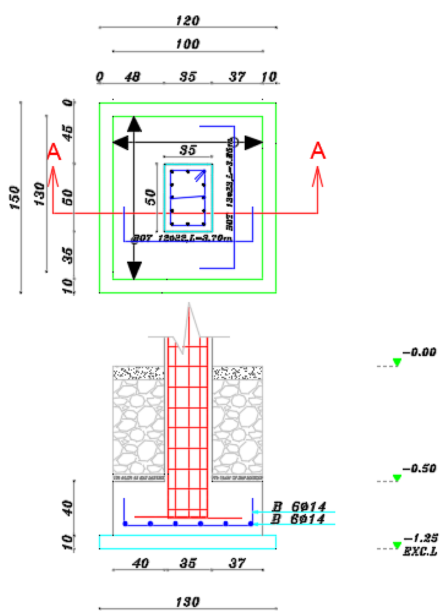

Figure 71: footing 1

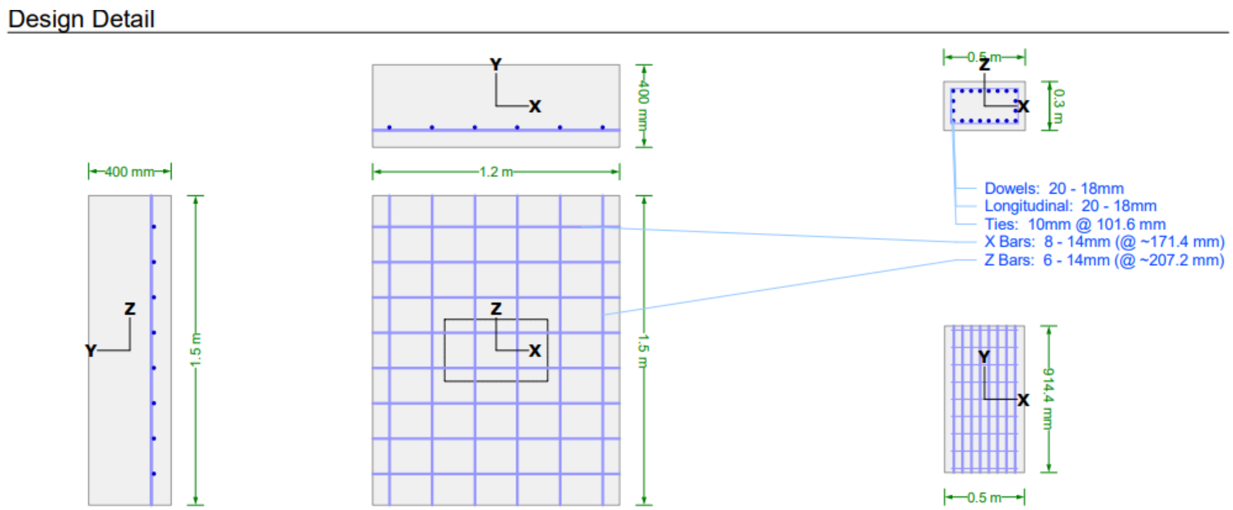

Figure 72:Footing detail

Footing X-Direction Capacity:

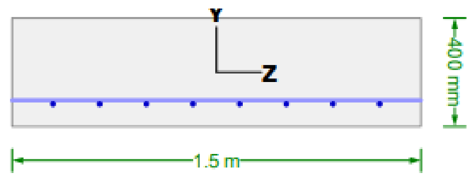

Figure 73:footing X-dir. detail

General Section (ACI 318-11 10.2.7):

$$\emptyset Mn,x = \emptyset \times fy' \times As\,(d - \frac{a}{2})$$

$$\emptyset Mn,x = 0.9 \times 420 \times 1239 \times (318 - \frac{17}{2}) = 144.9\ kN.m$$

$$\emptyset Vc = 2\sqrt{fc'}bwd = 0.75 \times \sqrt{24} \times 1500 \times 318 = 291.1\text{Kn}$$

$$As,\min = \frac{0.0018 \times 60000}{\text{fy}} Ag = \frac{0.0018 \times 60000}{420} \times 600000 = 1064 mm^2$$

$$a = \frac{\text{Asfy}}{0.85 fc'b}$$

$$= \frac{1239 \times 420}{0.85 \times 24 \times 1500} = 17\ mm$$

$$c=\frac{a}{\beta 1}=\frac{17}{0.85}= \ 20\ mm$$

$$\varepsilon t == 0.003(\frac{d-c}{c}) = 0.003(\frac{318-20}{20}) \ = 0.0447$$

Development Length:

ψ t = 1.0

- ψe 1.0 = (bar not epoxy coated)

ψs =0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s/2 = 167 mm / 2 = 83.5 mm

cover d + b / 2 = 75 mm + 13.97 mm / 2 = 81.99 mm

cb =81.99 mm (lesser of half spacing, ctr to surface)

Ktr 0.0 = (no transverse reinforcement)

$$=\frac{\text{cb+Ktr}}{\text{db}}=\frac{81.99+0}{13.97} = 5.868$$

$$\text{Ld}=\frac{3}{40\ \lambda\ fy2.5}\frac{\psi s\ \psi e\ \psi\ t}{\sqrt{fc\prime}}=\frac{3}{40\ \lambda 420*2.5}\frac{1*1*0.8}{\sqrt{24}}=34.62cm$$

Footing Z-Direction Capacity:

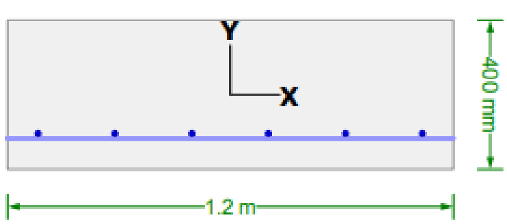

Figure 74:footing Z-dir detail

General Section (ACI 318-11 10.2.7):

$\emptyset Mn,x = \emptyset \times fy' \times As(d - \frac{a}{2})$

$\emptyset Mn,x = 0.9 \times 420 \times 929 \times (304 - \frac{15.94}{2})$ =104 $kN.m$

$\emptyset Vc = 2\sqrt{fc'}bwd$=0.75×√24×1200×304=222.6Kn

$As,\min = \frac{0.0018\times 60000}{\text{fy}} Ag = \frac{0.0018\times 60000}{420} \times 600000 = 1064 mm^2$

$a = \frac{\text{Asfy}}{0.85 fc' b}$

$= \frac{929\times 420}{0.85\times 24\times 1200} = 15.94\ mm$

$c = \frac{a}{\beta 1} = \frac{15.94}{0.85} = 18.75\ mm$

$\varepsilon t == 0.003(\frac{d-c}{c}) = 0.003(\frac{304-18.750}{18.750}) = 0.0456$

Development Length:

ψ t = 1.0

- ψe 1.0 = (bar not epoxy coated)

ψs =0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s/2 = 167 mm / 2 = 83.5 mm

cover d + b / 2 = 75 mm + 13.97 mm / 2 = 81.99 mm

cb =81.99 mm (lesser of half spacing, ctr to surface)

Ktr 0.0 = (no transverse reinforcement)

$= \frac{\text{cb+Ktr}}{\text{db}} = \frac{81.99+0}{13.97} = 5.868$

$$Ld=\frac{3}{40\ \lambda\ fy2.5}\frac{\psi s\ \psi e\ \psi\ t}{\sqrt{fc\prime}}=\frac{3}{40\ \lambda 420*2.5}\frac{1*1*0.8}{\sqrt{24}}=34.62cm$$

Footing Punching Shear Capacity:

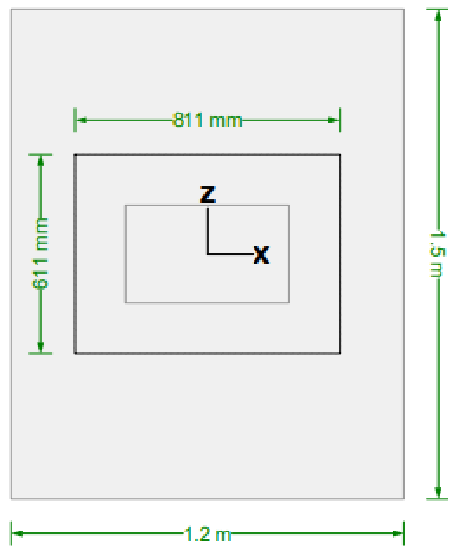

Figure 75: footing punching shear

Punching Shear (ACI 318-11 11.11.1.2, 11.11.2.1):

bo =2844 mm (perimeter of critical section)

β =1.6667 (column width ratio)

αs= 40.0 (interior column)

$Vc= \left(1+\frac{4}{\beta}\right)\lambda\sqrt{fc'}bod=\left(1+\frac{4}{1.6667}\right)\lambda\sqrt{24}*2488*311$ =1583kN

$Vc = \left(2+\frac{\alpha sd}{bo}\right)\lambda\sqrt{fc'}bod=\left(2+\frac{40*311}{2844}\right)\sqrt{24}*2488*311=2294kN$

$Vc = 4\lambda\sqrt{fc'}bod = 4*\sqrt{24}*2488*311$=1439kN→control

$\emptyset Vc$=0.75*1439=1080Kn

ØVn=ØVc/(bo*d)=1080/(311*2844)=1.22MPa

Pedestal Shear Capacity:

Shear - X (ACI 318-11 11.2.1.2, 11.4.7.2, 11.1.1)

$\emptyset Vc = \emptyset 2\left(1+\frac{Nu}{2000Ag}\right)\lambda\sqrt{fc'}bwd = 0.75*2\left(1+\frac{0}{2000*150000}\right)\lambda\sqrt{24}*300*443 = 81.09kN$

$\emptyset Vs = \emptyset\ \frac{Avfyd}{s} = 0.75\frac{154.8*420*443}{101.6} = 212.7\text{kN}$

$\emptyset Vn = \emptyset Vs + \emptyset Vc = 81.09 + 212.7 = 293.7\text{kN}$

Shear - Z (ACI 318-11 11.2.1.2, 11.4.7.2, 11.1.1)

$\emptyset Vc = \emptyset 2\left(1+\frac{Nu}{2000Ag}\right)\lambda\sqrt{fc'}bwd = 0.75*2\left(1+\frac{0}{2000*150000}\right)\lambda\sqrt{24}*300*243 = 74.13kN$

$\emptyset Vs = \emptyset\ \frac{Avfyd}{s} = 0.75*\frac{154.8*420*243}{101.6} = 116.7\text{kN}$

$\emptyset Vn = \emptyset Vs + \emptyset Vc = 74.13 + 116.7 = 190.8\text{Kn}$

Pedestal Axial + Flexural Capacity:

Axial (ACI 318-11 10.3.6.2):

ϕPnmax =0.80 ϕ (0.85 F'c(Ag -As)+fy As) = = 0.80* 0.650* (0.85 *24(150000 - 5032mm^2)+ 420 *5032 mm^2 = 2637 kN

ρg = As / Ag = 5032 mm^2 / 150000 mm^2 = 0.0335

Reinforcement Limits

Min Steel Check (ACI 318-11 Ch 10.5.4, 7.12.2.1)

As =1239 mm^2 ≥ Asmin 1064 mm^2

Min Steel Check (ACI 318-11 Ch 10.5.4, 7.12.2.1)

A_s 929 mm² = ≥ A_{smin} 851 mm²

Min Strain Check (ACI 318-11 Ch 10.3.5)

ε_t 0.0447 = ≥ ε_{tmin} 0.0040 =

Min Strain Check (ACI 318-11 Ch 10.3.5)

ε_t 0.0456 = ≥ ε_{tmin} 0.0040

Z-Flexure (+X side)

$M_{z2} = R_{z2}\, d_{z2}$ = 73.8 kN* 175 mm = 12.92 kN·m

ϕM_n= 144.9 kN·m ≥ M_u = 12.92 kN·m OK

Z-Flexure (-X side)

$M_{z1} = R_{z1}\, d_{z1}$ = 73.8 kN *175 mm = 12.92 kN·m

ϕM_n 144.9 kN·m = ≥ M_u 12.92 kN·m OK

Z-Flexure (-X side)

$M_{x1} = R_{x1}\, d_{x1}$ = 101.2 kN 300 mm = 30.36 kN·m

ϕM_n =104 kN·m ≥ M_u =30.36 kN·m OK

X-Flexure (+Z side)

$M_{x2} = R_{x2}\, d_{x2}$ = 101.2 * 300 mm = 30.36 kN·m

ϕM_n= 104 kN·m ≥ M_u= 30.36 kN·m OK

Footing Shear

Shear (+X side)

$V_{x2} = R_{x4}$ = 6.74 kN = 6.74 kN

ϕV_n= 291.1 kN $\geq$ V_u= 6.74 kN

Shear (-X side)

$V_{x1} = R_{x3}$ = 6.74 kN = 6.74 kN

ϕV_n= 291.1 kN $\geq$ V_u= 6.74 Kn

Shear (+Z side)

$V_{z2} = R_{z4}$ = 49.93 kN = 49.93 kN

ϕV_n = 222.6 kN $\geq$ V_u =49.93 kN

Shear (-Z side)

$V_{z1} = R_{z3}$ = 49.93 kN = 49.93 kN

ϕV_n= 222.6 kN $\geq$ V_u= 49.93 kN

Chapter 5: Conclusion and Recommendation

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).

Recommendation

Based on the personal experience in analyzing, designing and detailing The Seismic Design of Bethlehem General Hospital, 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

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