Prince Mohammad Bin Fahd University
College of Engineering
Department of Civil Engineering

Structural and Geotechnical Design of Reinforced Concrete Multi-Story Car Park

Senior Design Project (ASSE III) Spring 2018/2019

Students Name & ID:

Ahmed Kamal	201502884
Feras Al Khalil	201500034
Murtadah Al Jutail	201302673
Mohammed Al Ouf	201303081

Advisors:

1. Dr. Tahar Ayadat
2. Engr. Danish

Coordinator:

Dr. Andi Asiz

Acknowledgement

In submitting this report, we would like to express our deepest appreciation to Dr. Andi Asiz the Chair of Civil Engineering Department for his continuous and enthusiastic guidance and encouragement as well as his invaluable advice and patient throughout this research work particularly and other works generally.

We would like to acknowledge and say a big thanks to our final year project advisors (Engr. Danish Ahmed and Dr. Tahar Ayadat), whose contribution in stimulating suggestions and encouragement, helped us to coordinate and design the framed concrete structure of a multi-story car park project.

Last, but not the least, we take this opportunity to express our sincere thanks to all those who provided us the possibility to complete this project, Faculty of Civil Engineering Department, Engineering College, for their co-t encouragement.

Abstract

There is a current shortage in car parks in Prince Mohammad Bin Fahd University (PMU). This leads to continues lateness in attendance as students spend time looking for parking spaces. In addition, some students choose to park their cars on the main road in front of the main campus which leads to a reduction of the number of lanes from three to one. To further propagate the problem, there will be an increase in the population of student after adding new majors to the university as well as opening master degree courses. This will be accompanied by an increase in the number of cars which will lead to a further shortage of parking inside the university. In order to solve this problem, we propose to design a multi-level parking structure to accommodate a large number of vehicles at one place.

In our project, we will examines the design of a framed reinforced concrete structure multi-story car park that has can facilitate 1070 car spaces over five floors (G+4) with a total area of (37800) m^2. The project will focus on the design of the structure elements consisting of a flat slabs system supported by columns of different dimensions, as well as shear walls to support the elevators and stair cases, and the design of a foundation system. Our focus will be on reducing the cost and maximizing the safety of the construction, compared with the other structures of different material. The design will be done according to (MOMRA, ACI and SBC) codes for various structural components.

Table of Contents

List of Figures

List of Tables

1 CHAPTER 1: PROJECT INTRODUCTION

1.1 General

"Car parking areas are part of urban transportation network as a whole: vehicle movement, pedestrian movement and public transport stations. The planning of parking inevitably entails analysis of multiple factors among which: projected types of land use, sites expected to be covered by urban expansion, components of car parking planning policies and resource requirements that can affect the operation/development of parking areas" (KACST, 1987, p.15).

In the past years, there were no posed problems in Saudi Arabia caused by shortage in car parking areas. However, it has become one of the problems facing authorities in the major cities in view of growing population. Thus, car parking has become a principal component in urban transportation. From this point, the idea was generated to design and build a reinforced concrete multi-story car parking in order to serve the urban expansion around all over cities in Saudi Arabia and especially in prince Mohammed Bin Fahd University (PMU) in order to serve the educational community and to minimize the shortage of parking's.

1.2 Project Objective:

The main objective of this project is to design a RCC multi-story car park for the mitigation of traffic challenges around (PMU) using (MOMRA, SBC and ACI) standard codes. Hence, the structure will be designed for ultimate limit state method (ULS). According to (Hørte, Okkenhaug, & Paulshus, 2017), (ULS) must be fulfilled as an established condition in order to comply with engineering demands for strength and stability under design loads. The (ULS) limits the stress that materials experience during the design process, by considering the safety of the structure and its users.

The design will help to minimize the current shortage and future increase in car parking area especially in areas where there is less space for parking inside PMU. In addition, to produce detailed drawings, calculations, and specifications of the structure where we it will provide a safe and easily accessible area for car parking. Moreover, the building is intended to be constructed from reinforced concrete; as it is cheap, readily available, and durable.

1.3 Scope of Work

The report is composed of eight chapters:

Chapter One: General information and project objective.

Chapter Two: Detailed description of constraints and design codes.

Chapter Three: Structural design including calculations, analysis.

Chapter Four: Steel reinforcement for all elements.

Chapter Five: Geotechnical design of foundation system.

Chapter Six: Software used.

Chapter Seven: Cost Estimation.

Chapter Eight: Conclusion.

1.4 Project Description

The structure will be located directly in front of PMU male gate in AL-Khobar, where it contains a minimum of 1000 spaces over five floors (G+4).

The Parking structure has a total area of (37800) m^2 and has (90×84 m^2) symmetrical area for all floors. In addition, the design of all components such as slab, column and footing will be done with aid of E-TABS, SAFE, and AUTOCAD. The access to each floor has been provided using staircase and lifts. For emergency purpose separate doglegged staircase is provide on both side of the structure.

1.5 Project Layout

The layout was obtained from (Pinterest and Archdaily) web site as an image. The layout has been edited by changing its dimensions as well as the flow of traffic within the car park. In addition, the column positions have been edited while others have been inserted in the layout, as there were no columns in the middle of the layout.

Consequently, a new layout was established due to the modification that has been done upon the layout. The dimensions used where edited according to MOMRA standard. In addition, columns, slabs and shear walls analysis will be done by hand calculation and ETABS software. After that, the foundation of the structure will be designed. Finally, design constrains will be considered in the structural design calculation’s and the cost of the project will be estimated.

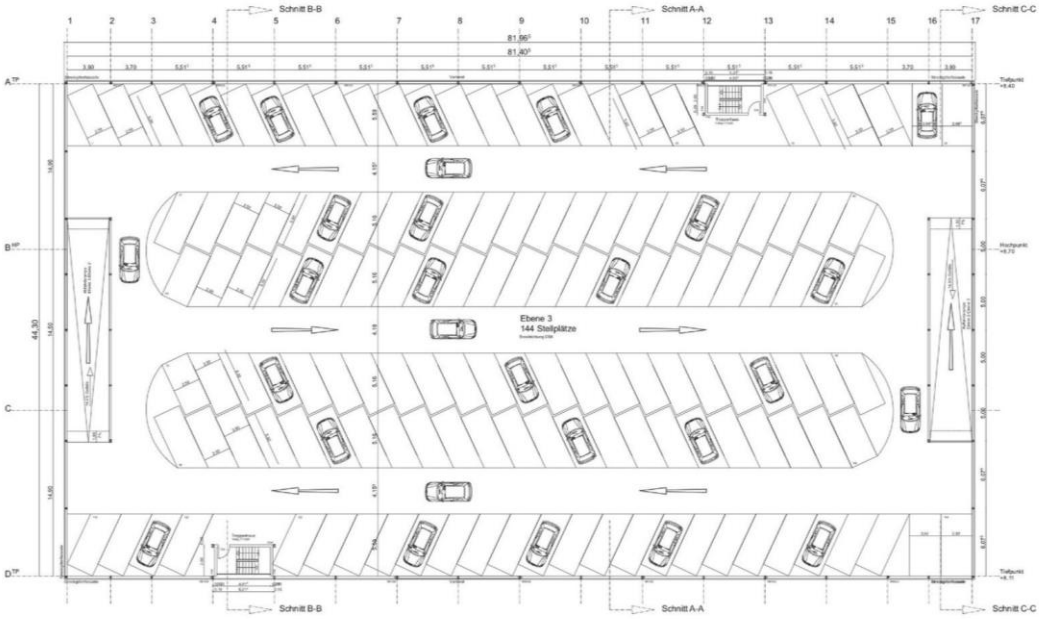


Figure 1.5-1Orignal Layout

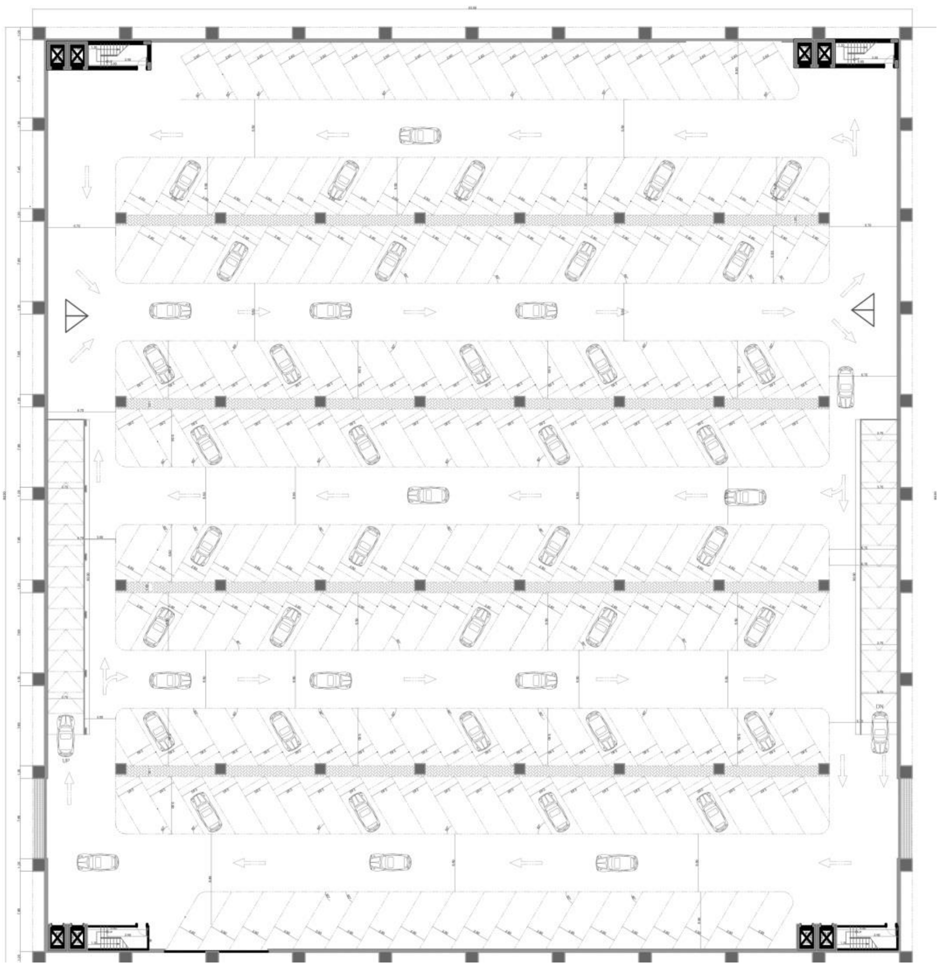


Figure 1.5-2 Modified Layout

1.6 Grid Line

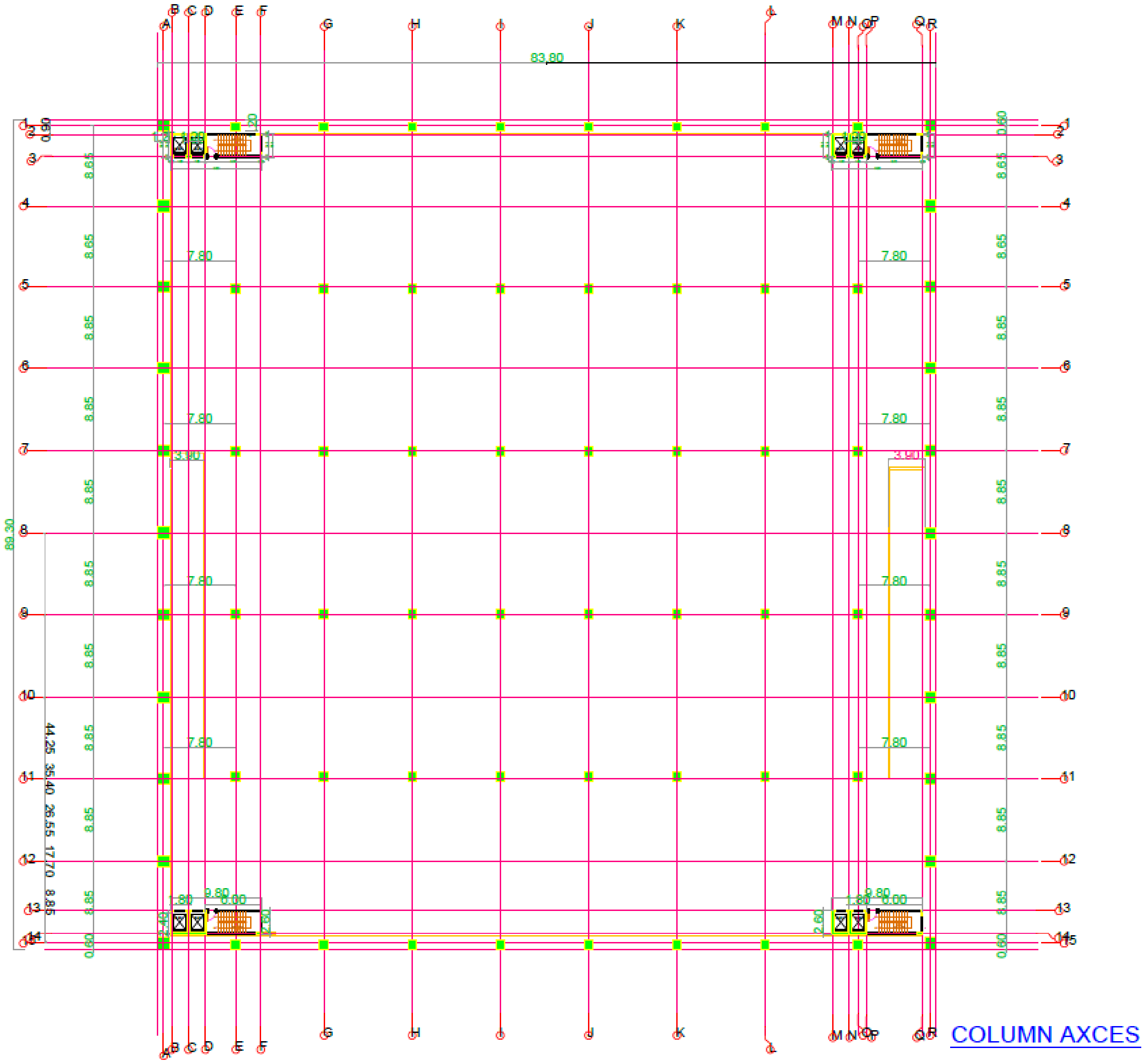


Figure 1.6-1 Grid Line

Column Names:

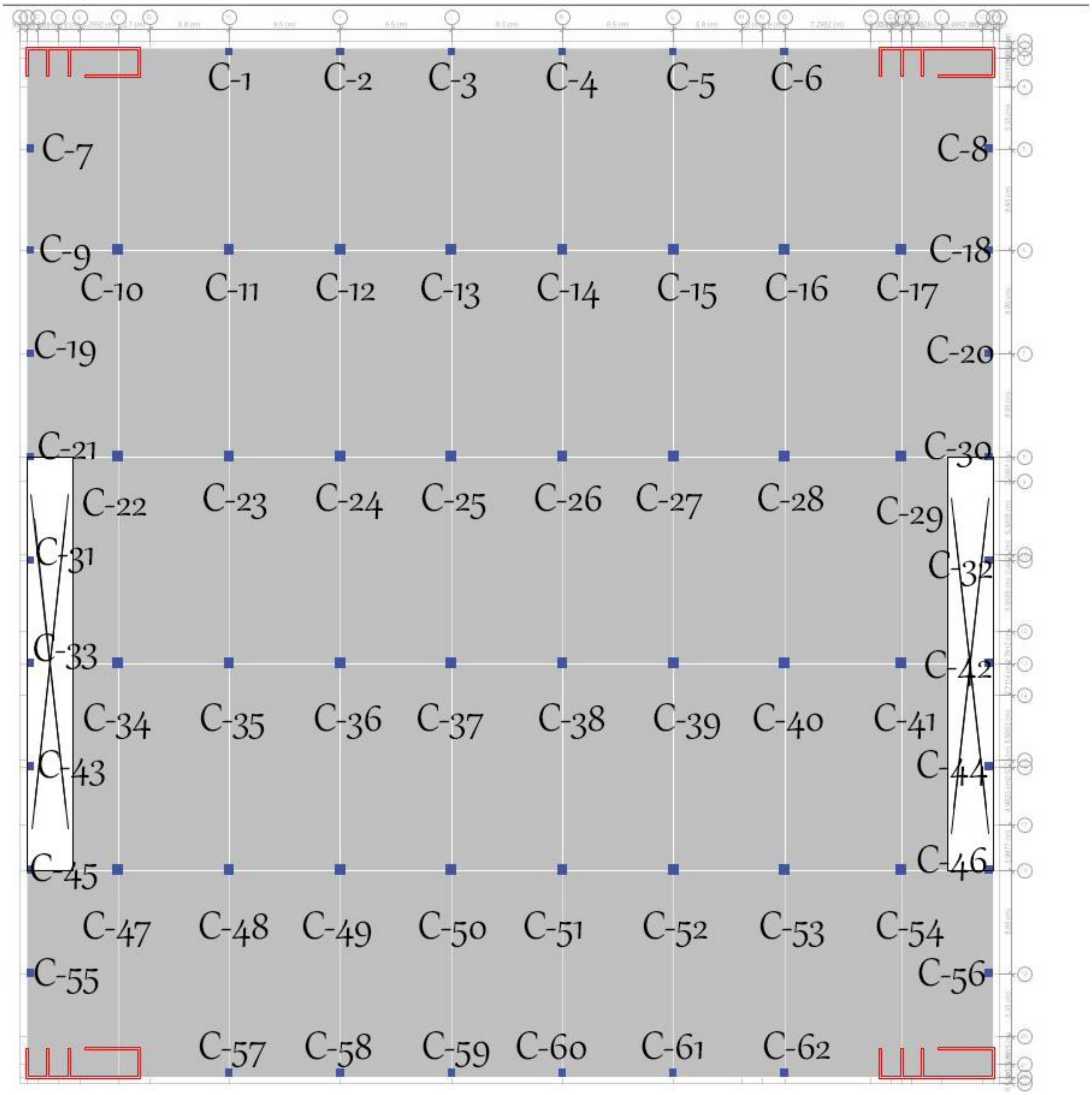


Figure 1.6-2: Column Names

2 CHAPTER 2: CONSTRAINTS AND DESIGN CODES

2.1 Constraints

In any project there are a number of factors and conditions that constraint the project in certain aspects. These force the project to be design in a certain way in order to accommodate them and cause an increase in the total cost.

The following table doesn't incorporate every aspect of design, but reflect the fact that your design required you to consider multiple constraints

Table 2.1-1 Constraints

Constraints		Solution
Geotechnical	Water table at 1 m	Dewatering
		Permanent drainage system
	Sabkha Soil	Excavation and Replacement
Material	Sulfates	Type 5 sulfate resistance Portland cement
	Chlorides	Coating the foundation
Environmental	Contaminated Water	Transporting the contaminated water to the right disposal place
	Contaminated Soil	Dispose the contaminated soil into hazardous landfill
Structural	Size of Structure	Min. 1000 car bays
		Maximum 6 stories
Safety	Road Crossing	Pedestrian walkway

2.1.1 Geotechnical Constraints:

The soil profile consists of 3 layers. These are six meters of loose to medium dense sand with/without silt, 22 meters of very fine to fine low plasticity sandy clay (CL), and finally a refusal layer which is most likely bedrock. The water table is at one meter from the ground level, since the site is near the coast. This will require dewatering the site to be able to construct the foundation. We will consider a mat foundation system (a mat foundation, also called a raft foundation is essentially a continuous slab resting on the soil that extends over the entire footprint of the building, thereby supporting the building and transferring its weight to the ground) with piles to transfer the loads to the hard rock since the to reduce settlement of the foundation.

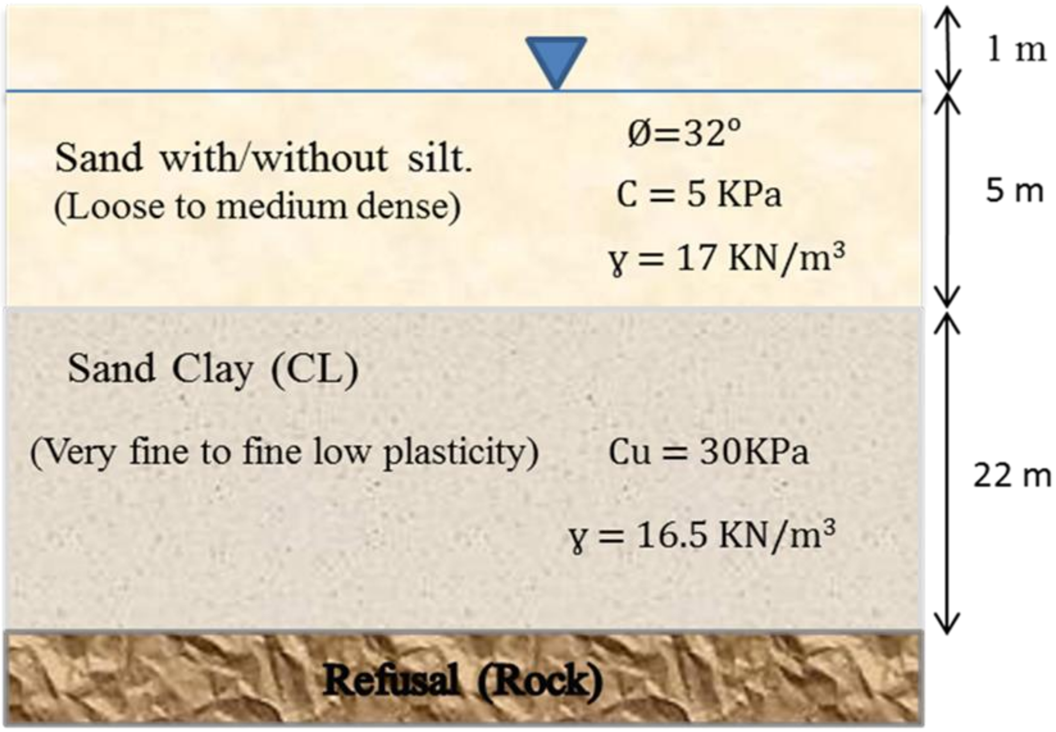


Figure 2.1-1 Soil Profile

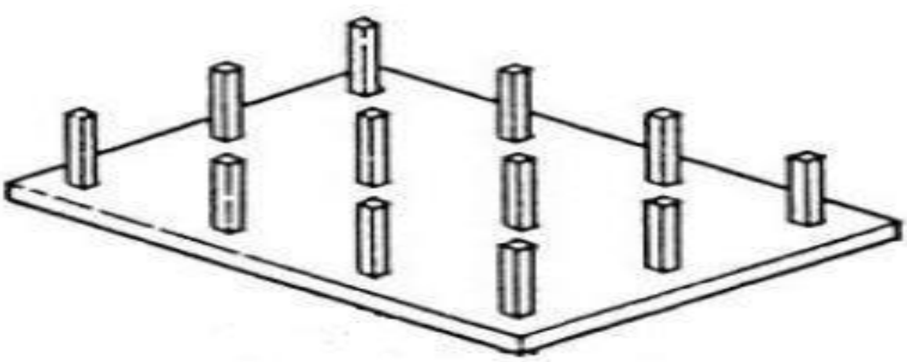


Figure 2.1-2 Mat Foundation

Table 2.1-2 Chemical Analysis

Sample number	Depth (m)	Chlorides %	Sulfates %
1	1.00	0.318	0.391
2	1.00	0.321	0.400
3	3.00	0.088	0.375
Average	-	0.24	0.38

- **0.24 % > 0.05 % (maximum allowable)**
- **0.2 % < 0.38 % < 2 % (Therefore sulfate attack hazard is severe).**

2.1.2 Material Constraints:

According to the geotechnical report, the amount of chemicals present in the water and the soil are of hazardous of higher concentration than that allowable. The amount of sulfates present is 0.38% this lays in the hazardous region; the chloride percentage is 0.24% which is almost five times the amount allowable. The sulfates will react with the concrete and affects the foundation; therefore, we will use type IV sulfate resistance Portland cement. Chloride on the other hand can penetrate the concrete and react with the steel reinforcements to corrode it. Therefore, the solution is to coat the steel rebar and the concrete mix after it gets hardened with a butane coat.

2.1.3 Environmental Constraints:

Considering that the top layer of sand is contaminated, it needs to be excavated and replaced. The excavated soil cannot be placed at any location since it contains contaminants, but needs to be moved to designated landfills for contaminated soil. Also, the water table is very high (one meter from the surface) so the soil needs to be dewatered and a permanent drainage system installed. However, the water is contaminated as well therefore it has to be transported to waste water treatment plant before being disposed of.

2.1.4 Structural Constraints:

The structural constrains that govern the project are those given by the proprietor, those specified in the design codes and the constraints induced by the design layout itself. The main constrains given from the owner is that the as the size must be limited to no more than six stories will fitting at least 1000 vehicles. The constraints given by the design code are to consider all load cases and design limitations. Also, the layout itself provides some limitations seeing as the largest span between columns is 17.7m, which is higher than the 12m that is recommended by construction engineers (Danish, 2019).

2.1.5 Cost Constraints:

All the previous constrains mentioned above, increase the cost of the project significantly. This is especially true for the cost associated with the removal, replacement, and disposal of the contaminated soil; since the soil will need to be moved to designated landfills that may not be near the construction site. The budget of the project will also be highly influenced by the cost of dewatering the soil and placing a permanent drainage system. This water must be sent to water treatment plants as should the drainage system in order for it to be treated before being disposed of in large water bodies.

2.2 Introduction for Design Principles and Planning

A RCC multi-story car park is a building structure that is designed by using reinforcement cement concrete where it is enclosed and an independent building. It is designed specifically with a number of floors were the cars can park temporarily. In most structures cars can flow between floors by interior , exterior or automated robot systems ramps (combination of ramp and elevator). In addition, some structures have either single accesses and exits where it can cause traffic congestion or multiple where it can avoid in and out traffic congestion.

Generally, as shown in figure (2.2-2-3-4) multi-story car parks floor system categorized into one of a three main layouts which are, flat deck, split level deck, sloping deck (ramped floor).

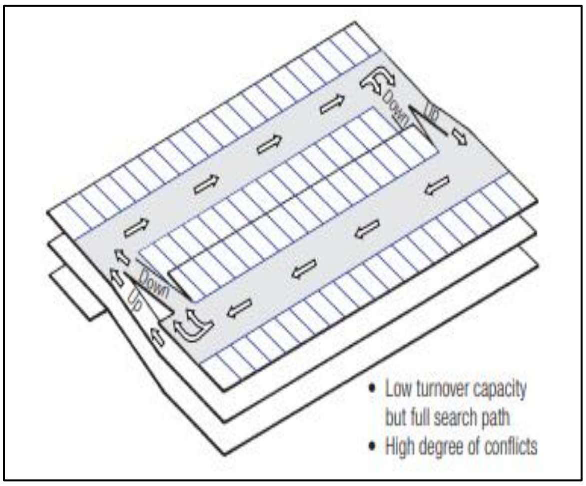


Figure 2.2-2 Split Level Deck (Irimia & Gottschling 2016)

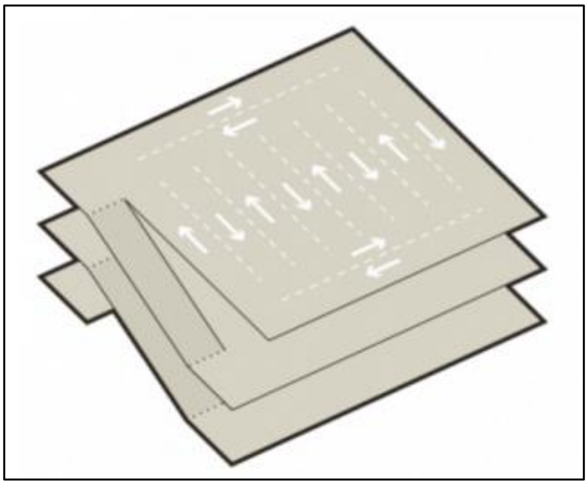


Figure 2.2-1 Flat Deck (Yurtoğlu, 2018)

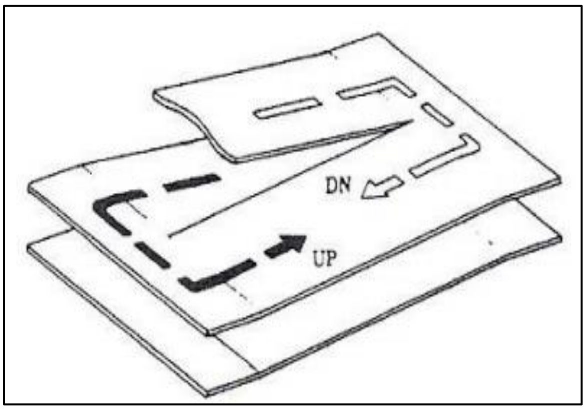


Figure 2.2-3 Sloping Deck (Duff, & Beasley, 1962)

2.3 Standard Codes and Parking Regulation

The parking layout has been chosen to be an angle parking where the cars will be aligned in an angle of 60˚. Normally the angle is aligned with the direction cars approach the parking space. It makes it a lot easier to drive into the parking space in addition that it can provide a good visibility. With angle parking it is ease to park in and out of parking spaces with respect to the smooth turn. Moreover, the figure and table below will show the dimension obtained from (MOMRA):

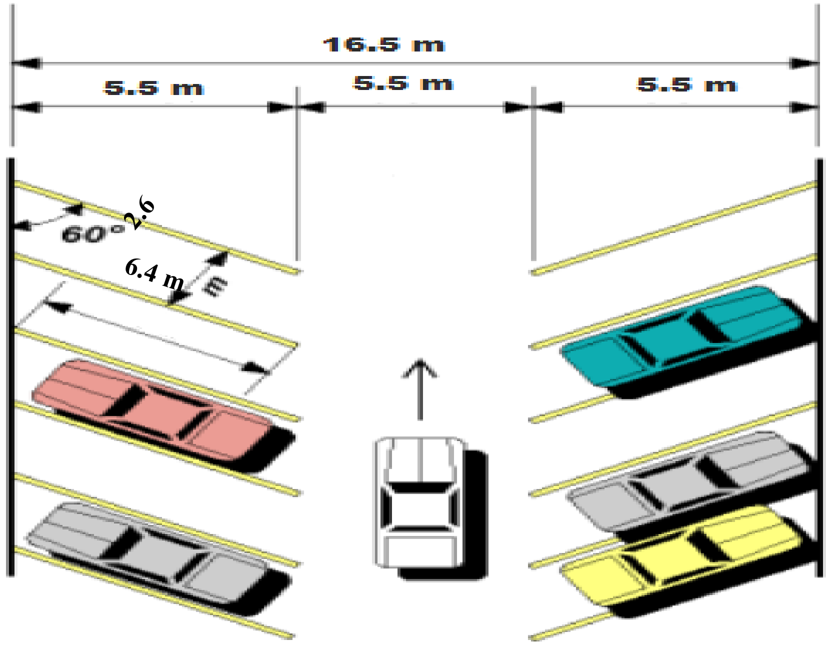


Figure 2.3-1 Detailed Parking dimensions Original from (V. A. 2014)

Table 2.3-1 Parking Dimension

Description	MOMRA Code Dimensions
Angle	60 Degree
Angular Parking space width	2.6 m
Horizontal Parking space width	5.5 m
Parking space length	6.4 m
Driving aisle width (1-way)	5.5 m
Two rows parking plus aisle width	16.5 m

2.4 Planning and Layout for the Floor System

The floor system layout of our multi-story car parking structure consists of a flat deck. In conducting this sort of design, the system has been chosen on the basis of its simplicity of construction. In addition, the decks are linked by internal ramps which are located at both sides of the structure. The flat deck system would be used appropriate for our car park where the dynamic capacity is not critical and the maximum use of space would be achieved.

Since our structure considered as a rectangular shape the design of the system will constructed simply where it will result in high efficiency of floor space per vehicle parking

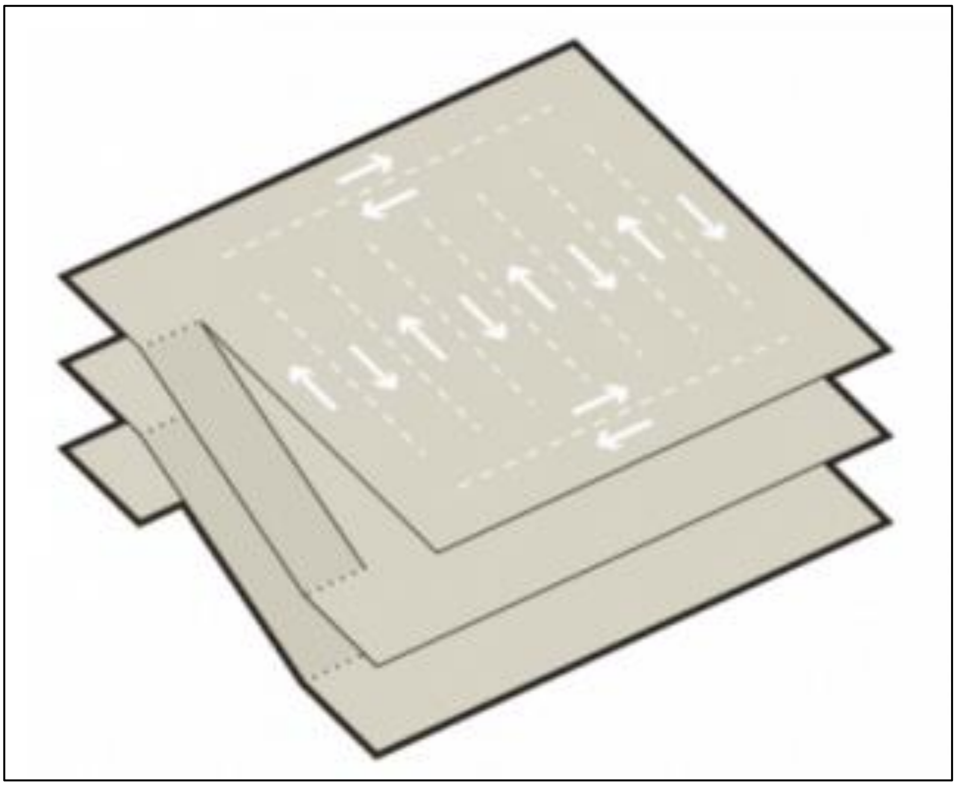


Figure 2.4-1 Flat Deck (Yurtoğlu, 2018)

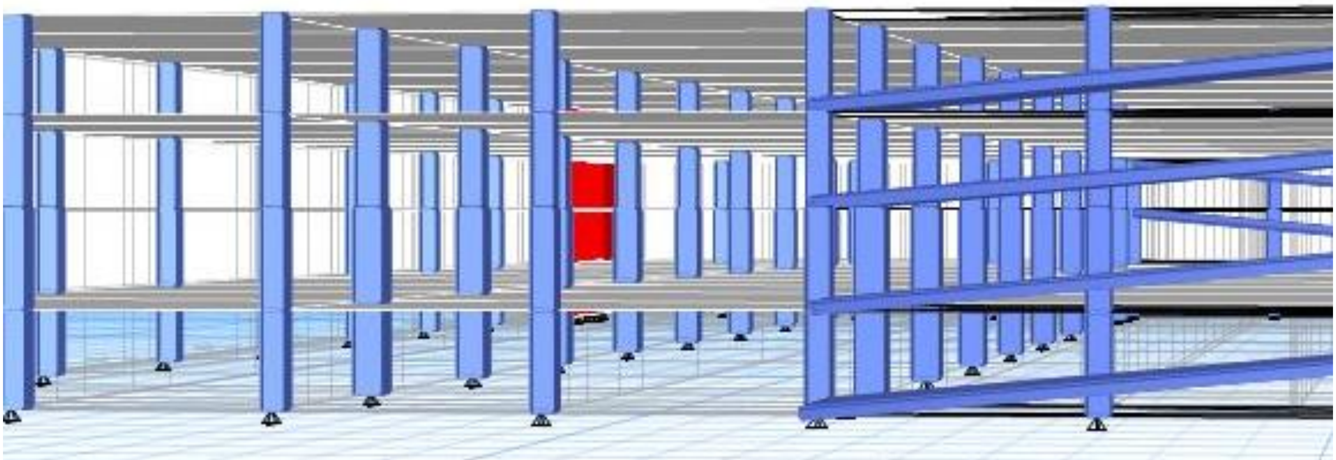


Figure 2.4-2 Floor System from ETABS

Standard and Regulation for Floor System (MOMRA Code**):**

Table 2.4-1 Floor Dimension

Description	**Dimensions**
Length	89.30 m
Width	83.80 m
Elevation	3.5 m

2.5 Planning and Layout for Ramp

Straight ramps have been selected to be designed in our structure since its shape is rectangular. In our design, up-and-down ramps are located at the ends of the structure. Ramps should fit well along the structure's longer side dimension. In order to satisfy the design code of the ramp, more horizontal distance is required to accommodate cars movement at the end of each ramp. Our ramp design considers as a parallel design system where up and down ramp slope located in the same direction so the cars must rotate in opposite direction. The reason behind choosing this system is cheaper in construction than others.

Table 2.5-1 Standard and Regulation for Ramp (MOMRA Code)

Description	Dimensions
Ramp slope	11% *
Angle of departure	11%
Angle of approaches	11%
Ramp Live load	5 KN/m^2
Ramp length	35 m
Ramp width	3.9 m

* Ramp slope 11% = each 1 m increase the height by 11 cm

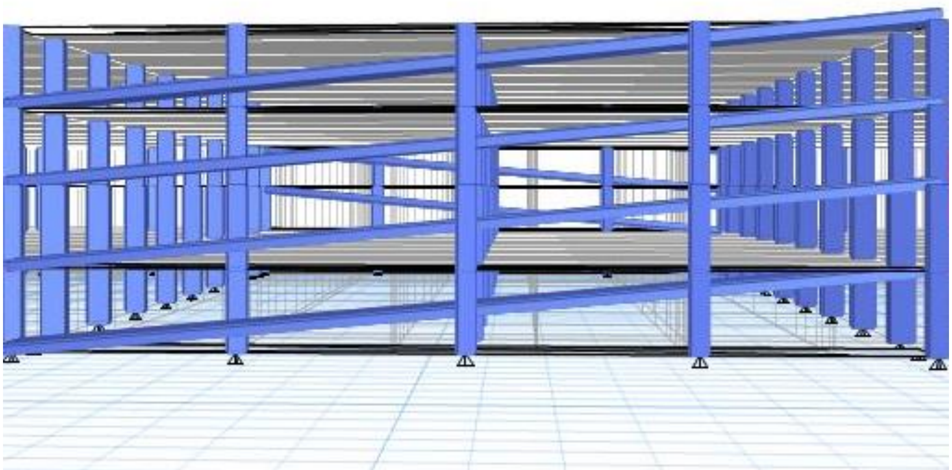


Figure 2.5-1 Ramp System from ETABS

3 CHAPTER 3: STRUCTURAL DESIGN

3.1 Introduction

The codes that have been used during the preliminary design, structural analysis, structural design, drawings and calculations were carried out based on building code requirements and specialized computer programs.

For the design, analysis and preliminary sizing of columns, slabs and other structural elements; as well as, the analysis of loads, the following codes were considered:

- Saudi Building Code (SBC).
- MOMRA: Ministry of Municipal and Rural Affairs
- ACI: The American Concrete Institute (ACI 318M-08) and commentary.

For the purposes of modeling and structural analysis of loads; ETABS was used to guarantee accuracy.

3.2 Loads

3.2.1 Dead Load:

The dead load includes loads that are relatively constant over time, Dead load is known as static or constant load; it is the load which is applied constantly on the Structure. The dead load includes loads that are relatively constant over time, including the weight of the structure itself including slabs, beams and columns. Building materials are not dead loads until constructed in permanent position.

$DL = \gamma\ t$, $\gamma = 24\ KN/m^3$, $t = 0.55$ m, $DL = 13.2\ KN/m^2$

3.2.2 Live Load:

Live load is produced by the occupancy of the structure, such as, people, machines, equipment, furniture and appliances. However, the construction and environmental loads are not included in this type of load. For this project, the live load values that are going to be used, accordance From SBC – 301chapter four tables 4.1:

The live load for garages and parking structures is 2 kN/m^2

3.3 Flat Slab Design:

We have decided to construct a flat slab system, a flat slab is one not supported by beams. The other defining feature of a slab is wither the slab is a two way or one way slab. Two way slabs are slabs that are supported on four sides, will the ratio of longer span to shorter span is less than 2. In two way slabs, load will be carried in both the directions. (MacGregor, 1996)

$$Ratio = \frac{Longer\ span}{shorter\ span} = {}^{17.7}/_{9.5} = 1.86 < 2$$

Therefor the slab is a two way slab.

Since there are no beams in our slabs, the slab thickness will be obtained from SBC 304

$$\textbf{Slab thickness} = \frac{\mathbf{L}}{\mathbf{33}} = \frac{\mathbf{17.7}}{\mathbf{33}} = \mathbf{0.53\ m} \approx \ \mathbf{0.55\ m}$$

The final slab thickness to be used is 550 mm.

3.4 Columns Design:

A column in structural engineering is a structural element that transmits, through compression, the weight of the structure above to other structural elements below (foundation and piles). In other words, a column is a compression member. For the purpose of wind or earthquake engineering, columns may be designed to resist lateral forces. Other compression members are often termed "columns" because of the similar stress conditions. Columns are frequently used to support beams or arches on which the upper parts of walls or ceilings rest.

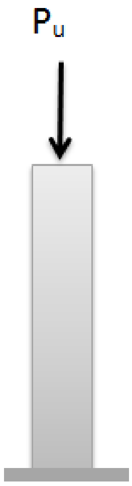
In our project we have long span and we don't have beams in the slap, so that means the columns are important in our project because the columns will transfer the weight of the structure to foundation. We will use the following equation to find dimension of columns.

3.4.1 Center Columns:

$$Pu = \emptyset r(0.85f'c(Ag - Ast) + fyAst)$$

The following data was used to calculate the column sizes:

As	Ag	Slab thickness	fc	fy	Area	Wu
2%	98%	0.55	28	420	168.15	19.04



3.4.1.1 3rd Floor

b=9.5m

L=17.7 m

$Wu = 1.2DL + 1.6LL$

Wu =19.04 kN/m^2

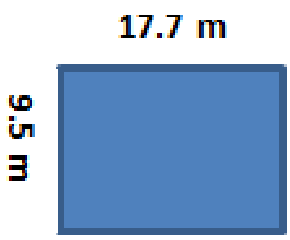


Figure 3.4-1 Tributary Area for Column

Pu = 3201.58 KN

$\frac{3135.5389 * 10^3}{0.65*0.8}$ = 0.85(28) (0.98Ag) +420(0.02Ag)

Ag = 194076.3 mm^2 take squawroot of Ag to find column dimensions

a = 440.5mm this calculation number but it is not practical so we will use 450mm.

$Ag' = a'^2 = (450)^2 = 22500mm$

The following calculations are for center column dimensions

P3	Ag	a	a'
3201.576	194076.3	440.5409	450

3.4.1.2 2nd Floor

$Pu = 2(3135.5389) + 0.9*17.7*9.5 = 6285.6578$ KN

$$\frac{6285.6578 \ *10^3}{0.65*0.8} = 0.85(28)\ (0.97Ag) + 420(0.03Ag)$$

a = 624.6733mm use 650mm

Ag'	P2	Ag	a	a'
0.2025	6437.202	390216.724	624.6733	650

3.4.1.3 1st Floor

$Pu = 3(3135.5389) + 0.9*17.7*9.5 = 9447.1167$ KN

$$\frac{9447.1167 \ *10^3}{0.65*0.8} = 0.85(28)\ (0.97Ag) + 420(0.03Ag)$$

a = 765.5799mm use 800mm

Ag	P1	Ag	a	a'
0.4225	9668.794	586112.565	765.5799	800

3.4.1.4 Ground Floor

Pu = 12623.1 KN

$\frac{12623.1 \ *10^3}{0.65*0.8}$ = 0.85(28) (0.97Ag) +420(0.03Ag)

a = 885.6432mm use 900mm

Ag	Pg	Ag	a	a'
0.64	12939.244	784363.964	885.6432	900

Table 3.4-1 Columns Dimensions

Center Columns	
Floor	Columns Dimensions
G	900 mm X 900 mm
1	800 mm X 800 mm
2	650 mm X 650 mm
3	450 mm X 450 mm

3.4.2 Edge Columns:

As	Ag	Slab thick	fc	fy	Area	Wu
2%	98%	0.55	28	420	84.075	19.04

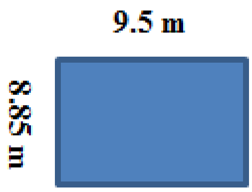


Figure 3.4-2 Tributary Area for Column

3.4.2.1 3rd Floor

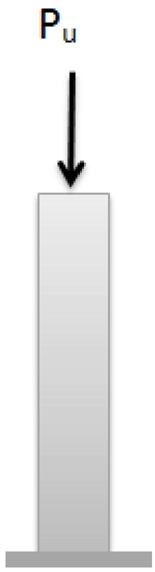
$$Pu = \emptyset r(0.85f'c(Ag - Ast) + fyAst)$$

b=8.85

L=9.5

w_u= 18.65

$Pu = 1567.7667\ KN$



$\frac{1567.7667*10^3}{0.65*0.8}$ = 0.85(28) (0.97Ag) +420(0.03Ag)

a = 311.5095mm use 350mm

The following calculations are for edge column dimensions

P4	Ag	a	a'
1600.788	97038.16	311.5095	350

3.4.2.2 2nd Floor

Pu = 2(1567.7667) + 0.9*8.85*9.5= 3143.1 KN

$\frac{3143.1*10^3}{0.65*0.8}$ = 0.85(28) (0.97Ag) +420(0.03Ag)

a = 441.249mm use 450mm

Ag'	P3	Ag	a	a'
0.1225	3211.875	194700.638	441.249	450

3.4.2.3 1st Floor

Pu = 3(1567.7667) + 0.9*8.85*9.5= 4727.89 KN

$\frac{4727.89*10^3}{0.65*0.8}$ = 0.85(28) (0.97Ag) +420(0.03Ag)

a = 540.3219mm use 550mm

Ag	P2	Ag	a	a'
0.2025	4816.111	291947.784	540.3219	550

3.4.2.4 Ground Floor

Pu = 6316.67749 KN

$$\frac{6316.67749 \ *10^3}{0.65*0.8} = 0.85(28)\ (0.97\text{Ag})\ + 420(0.03\text{Ag})$$

a = 624.0611 mm, use 650 mm.

Ag	P1	Ag	a	a'
0.3025	6424.592	389452.306	624.0611	650

Table 3.4-2 Columns Dimensions

Edge Columns	
Floor	Columns Dimensions
G	650 mm X 650 mm
1	550 mm X 550 mm
2	450 mm X 450 mm
3	350 mm X 350 mm

Note: We don’t have columns in the fourth floor because it is open area and doesn’t have ceiling and any weight on it.

However, after running the ETABS model with the specified dimensions for the edge columns, the edge columns were failing due to being over stressed. This could be due to the need for extra reinforcement in the columns that cannot be obtained with the current dimensions. Therefore, we have edited the edge column dimensions to the following (see table #) using a trial and error process to find the appropriate sizes:

Table 3.4-3 Modified Columns Dimensions

Edge Columns	
Floor	Columns Dimensions
G	650 mm X 650 mm
1	650 mm X 650 mm
2	600 mm X 600 mm
3	600 mm X 600 mm

Note: we unified each two floors to the same dimensions since the minimum accepted dimension was 600mm.

3.5 Shear Wall Design

Shear wall is defined as a structural member in a reinforced concrete framed structure that is designed to resist lateral forces such as, wind and seismic loads. It's providing a large strength and stiffness to buildings in order to reduce the damage to structure and lateral sway. Shear walls behavior depends upon: material used, wall thickness, wall length, wall positioning in building frame as well. The thickness of the shear wall should not be less than 150mm to avoid unusually thin sections, according to the (ACI code) "*The design methodology for concrete shear walls in 1963 ACI code (ACI 318, 1963) is based on working stress design; however, an ultimate strength design approach was also introduced. Chapter 22 of this code required a minimum thickness of 6 in. (150 mm) for walls up to two storeys in height. This minimum thickness is increased by 1 in. (25 mm) for each 25 ft (7.6 m) below the top two storey*".

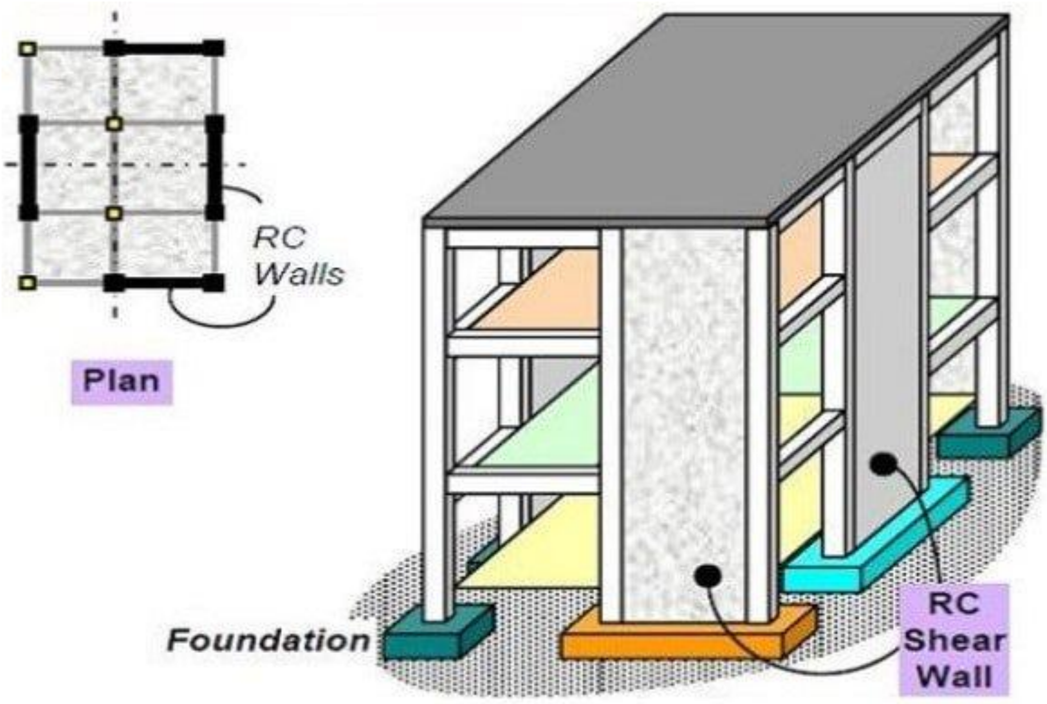


Figure 3.5-1 Reinforced Concrete Shear Wall (Murthy C.V.R., 2005)

The way of calculating the shear wall is the same as the column design. So according to the column calculation and the previous excel sheet we've found that:-

- Total length of all the shear wall sides = 24.7 m
- Effective area = 39.4m
- Ag = 18333501 mm^2 (from excel sheet)
- The thickness obtained is $\frac{Ag}{toal\ length} = \frac{18333501\ mm^2}{24700\ mm} = \quad 7.4\ mm \quad < \quad 150\ mm$

The thickness obtained above is obviously less than the minimum standard thickness which is 150 mm, so it's not acceptable. In this case the design should follow the standard code *(ACI 318, 1963)* with a minimum thickness of 150 mm.

3.6 Punching Shear

Punching shear is a typical flat plate failure that is characterized by the slab failing at the intersection point of the column. This results in the column breaking through the portion of the surrounding slab. This type of failure is one of the most critical problems to consider when determining the thickness of flat plates at the column-slab intersection. Accurate prediction of punching shear strength is a major concern and absolutely necessary for engineers so they can design a safe structure. (MacGregor, 1996)

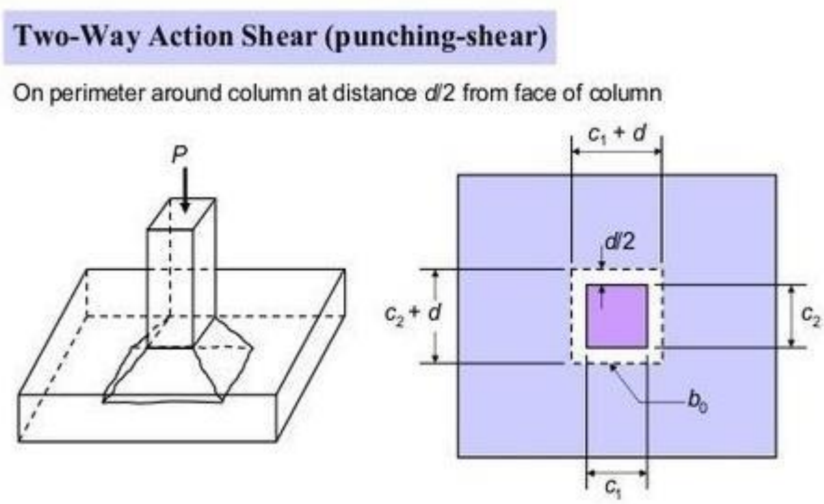


Figure 3.6-1 Two-Way Action Shear

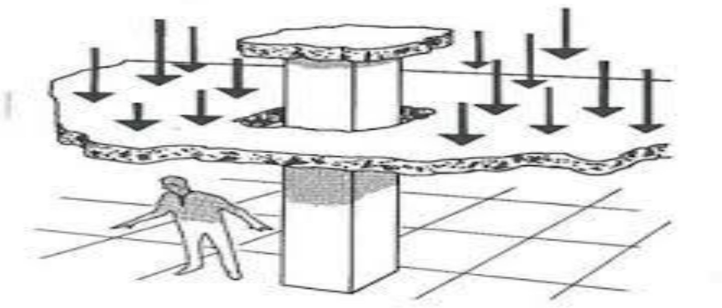


Figure 3.6-2 Slab Failure due to Punching Shear

Since we have a long space between columns and that means a huge area rest on each columns. So we have to check the punching shear in our project with this following equation

3.6.1 Central Columns:

To pass Ø Vc ≥ Vu given:

$$Vu = Wu\,(Atr - A'c) \text{ and } Vc = 4\sqrt{fc'}b * d$$

Where:

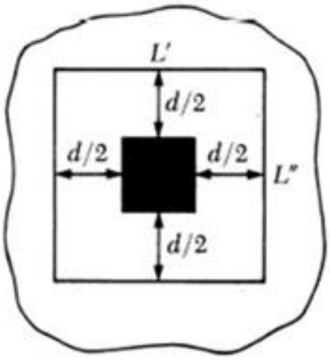


Figure 3.6-3 Adjusted Column Dimension

Ø: Reduction Factor= 0.75

A_{tr}: Tributary Area= 168.15 m^2

A'_c: Adjusted Column Dimension = 1.96 m^2

$f'c$: Concrete Compressive Strength = 28 MPa

Vc: Concrete Shear Strength

Vu: Ultimate Shear Force

wu: Ultimate Load = 1.6LL+1.2DL= 19.04 KN/ m^2.

d: Effective Depth = Slab Thickness – Concrete Cover – Bar Diameter

d = .55-.03-.02=19.685inch = 500 mm.

A: Column Dimensions

bo: Perimeter = 4 (a + d)

3.6.1.1 Ground floor:

a = 900mm, b = 5600mm

V_u = 3164.26 MPa

V_c = 59264.83MPa ØV$_c$ = 0.75 (59264.83) = 44448.62MPa

Ø $V_c > V_u$ (Therefore, Ground floor passes the punching shear)

3.6.1.2 First Floor:

a = 800mm

bo = 5200mm

V_u = 3169.40MPa

V_c = 55031.63MPa

ØV_c = 41273.72MPa

Ø $V_c > V_u$ (First floor passes the punching shear)

3.6.1.3 Second Floor:

a = 600

bo = 4600mm

V_u = 3176.40 MPa = 714.05 Kips

V_c = 48681.82MPa = 901.88 Kips

ØV_c = 36511.37MPa = 676.41 Kips

Ø $V_c < V_u$ (Second floor fails the punching shear)

3.6.1.4 Third Floor:

a = 450mm

bo: 3800mm

V_u = 3184.39 MPa = 715.85 Kips

V_c = 40215.42 MPa = 745.03 Kips

ØV_c = 30161.65MPa = 558.78 Kips

Ø $V_c < V_u$ (Third floor fails the punching shear)

Since the columns on second and third floor have failed in the punching shear, we may either adapt drop panel or increase columns dimensions. We chose to increase the dimensions of the columns. Moreover, since the top two columns will be of similar size, we have also decided to unify the bottom two column dimensions for better visual appearance.

Below is a table of our final column dimensions:

Table 3.6-1 Final Column Dimensions:

Central Columns	
Floor	Columns Dimensions
G	900 mm X 900 mm
1	900 mm X 900 mm
2	750 mm X 750 mm
3	750 mm X 750 mm

3.6.2 Edge Columns:

To pass Ø Vc $\geq$ Vu given:

$$Vu = Wu\,(Atr - A'c) \quad \text{and} \quad Vc = 4\sqrt{fc'}b * d$$

Where:

Atr: Tributary Area= 84.075 M2

A'c: Adjusted Column Dimension = 0.9025 M2

Ø: Reduction Factor= 0.75

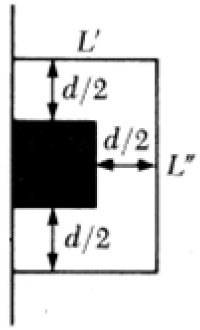


Figure 3.6-4 Adjusted Column Dimension

$f'c$: Concrete Compressive Strength = 28 Mpa

Vc: Concrete Shear Strength

Vu: Ultimate Shear Force

wu: Ultimate Load = LL*1.6+DL*1.2= 19.04 KN/ M2.

d: Effective Depth = Slab Thickness – Concrete Cover – Bar Diameter

d = .55-.03-.02=19.685inch = 500mm

A: Column Dimensions

bo: Perimeter = 4 (A+D)

3.6.2.1 Third Floor:

a = 350 mm

bo = 3400 mm

V_u = 1587.03MPa = 356.76 Kips

V_c = 35982.22 MPa = 666.61 Kips

$ØV_c$ = 26986.66 MPa = 499.96 Kips

$Ø V_c > V_u$ (The third floor passes the punching shear)

Since the minimum edge column dimension in our layout passes the punching shear check, it follows that the rest will also pass. This has been checked using an excel sheet.

3.7 Deflection

The deflection limit of our structure according to SBC 301 is given in table 1.4-1: deflection limits:

$$\partial = \frac{l}{240}$$

Considering that our long span is 17.7m then the deflection limit is 73.75 mm. However this is the maximum allowable local deflection and depends on elastic design of the project. According to Danish (2019), the workable deflection limit that is used in construction sites is limited to one inch (25mm).

The deflection was calculated using hand calculations found below:

$$\Delta = \frac{5wl^4}{384\, E\, I}$$

Where:

$l = 17.7m$

$w_u = 19.04kN/m^2$

$E = 4700\sqrt{28} = 24870\ MPa$

$I = \frac{bh^3}{12} = \frac{9.5(0.55)^3}{12} = 0.1317\ m^4$

Then $\Delta =$ 5.93 mm/floor

Total deflection = 23.72 mm.

This was compared to the deflection found in ETABS where this can be seen in the image below:

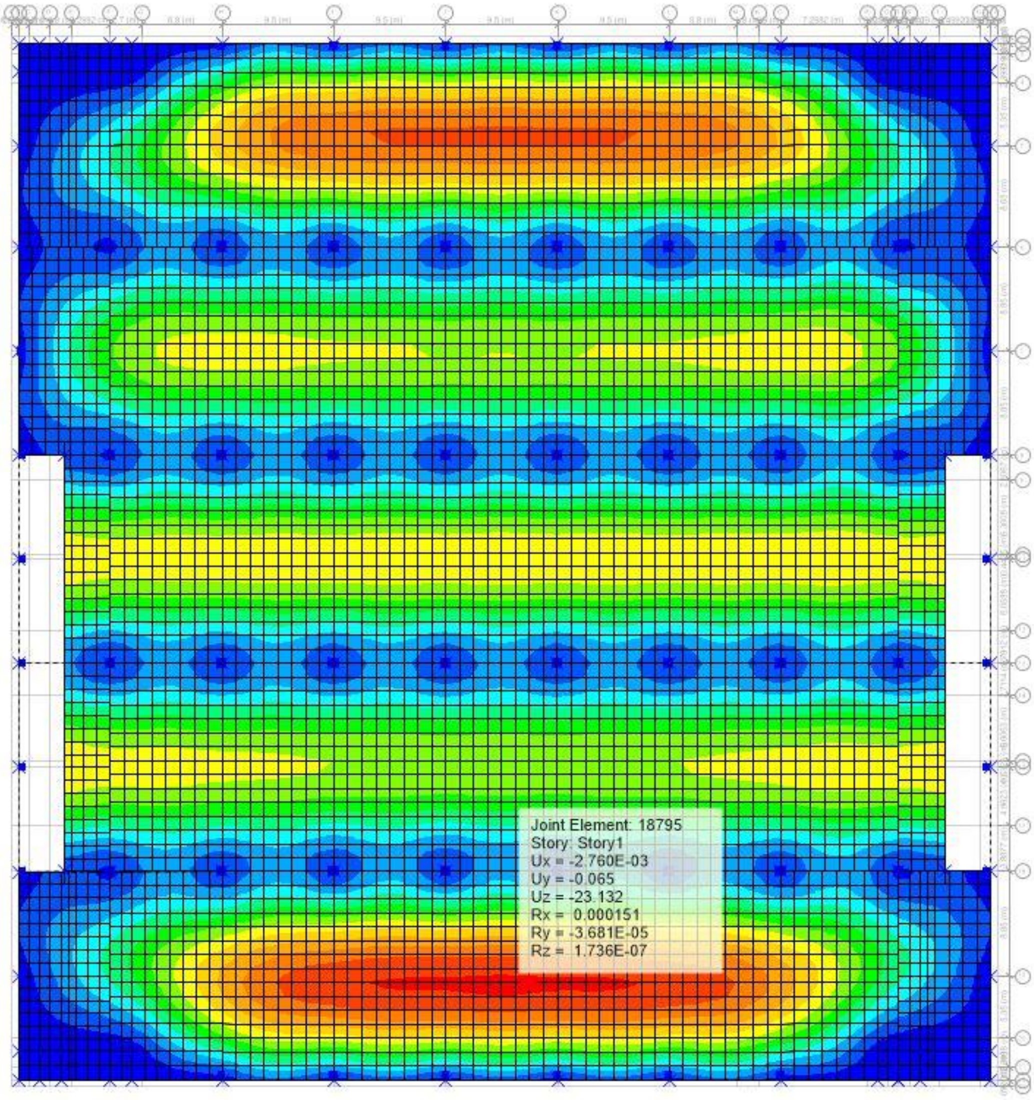


Figure 3.7-1 Maximum Deflection from ETABS

The maximum deflection according to ETABS is 23.13mm, which is similar to that calculated manually and within the allowable range.

3.8 Lateral Deflection:

Natural forces, such as wind and earthquake, cause the structure to deflect laterally and need to be taken into consideration during the design process. By decreasing the lateral deflection of the structure, the probability of failure will decrease if any natural events occur. The location of the project is near the coast and is in an open site, not surrounded be other large structures that may reduce the speed and effect of the wind. The region itself is a non-seismic region nevertheless, the deflection that one might cause will be considered. According to SBC, the maximum lateral deflection or drift allowable is 0.02h (table 10.12), where ‘h’ is the height of the building. Considering that each floor is 3.5min height, giving a total height of 14m, the maximum allowable lateral deflection will be 0.28m. ETABS considers the effect of lateral forces on each side or face of the structure and checks the drift that may occur. Using ETABS, the deflection of the structure will be as follows for both wind and seismic loads respectively:

3.8.1 Wind Load

Along the shorter face of the structure (84m) the following values where obtained:

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		mm	mm
Story4	14	Top	1.57	3.392
Story3	10.5	Top	1.126	0.175
Story2	7	Top	0.837	0.294
Story1	3.5	Top	0.496	0.156
Base	0	Top	0	0

.

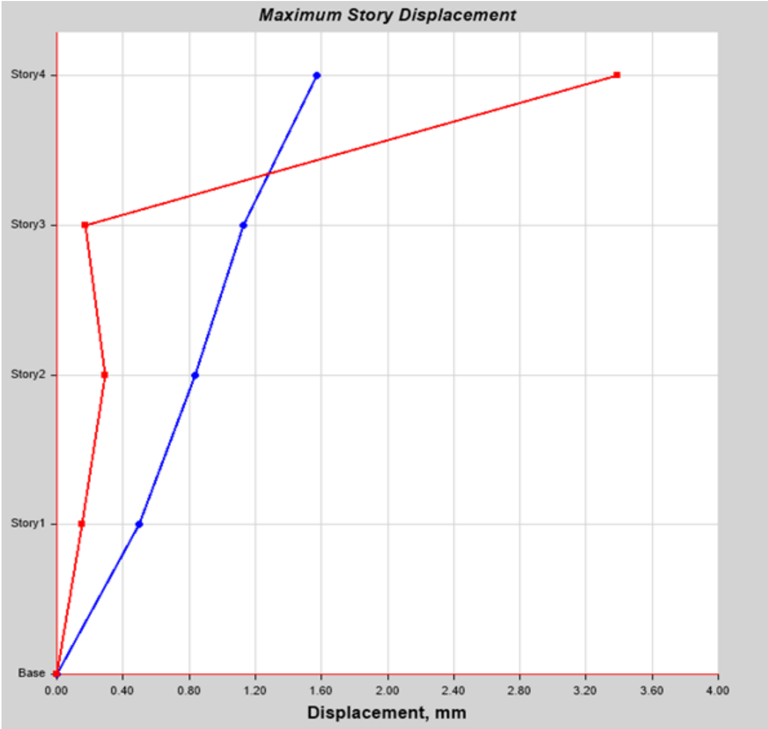


Figure 3.8-1 Wind Displacement in X-dir.

As seen in the table above the maximum deflection that will occur is 3.4 mm which is significantly less than the allowable of 280 mm.

Similarly, when considering the other side the values obtained where as follows:

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		mm	mm
Story4	14	Top	1.449	3.881
Story3	10.5	Top	0.185	0.555
Story2	7	Top	0.173	0.564
Story1	3.5	Top	0.1	0.292
Base	0	Top	0	0

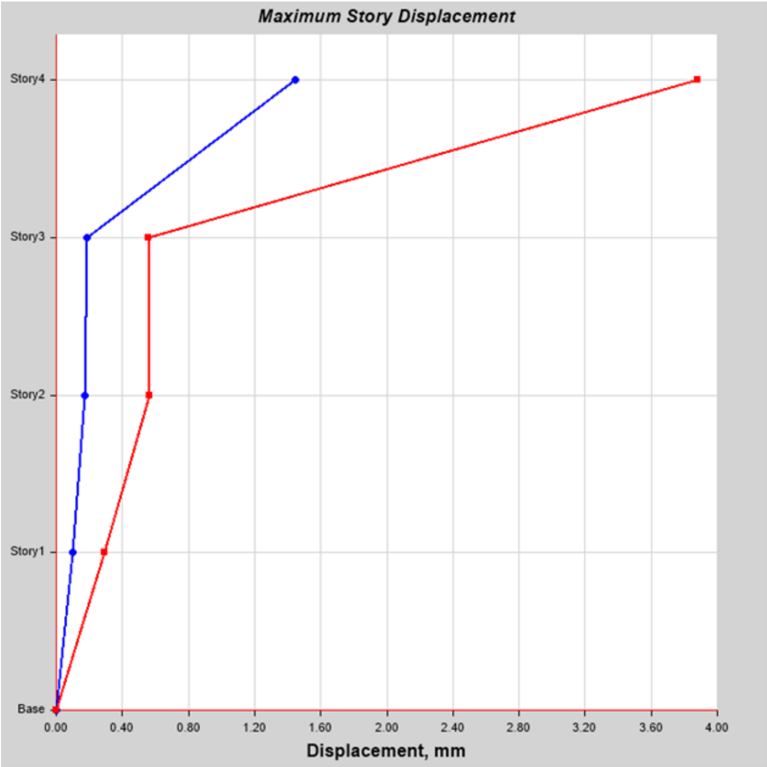


Figure 3.8-2 Wind Displacement in Y-dir.

Again the values obtained for the wind deflection are 3.88mm, way below the allowable specified limit of 280mm. Therefore, the building is safe in regards to wind deflection.

3.8.2 Earthquake

Again ETABS considers both faces of the structure. For the shorter side the following data was provided:

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		mm	mm
Story4	14	Top	5.796	3.673
Story3	10.5	Top	3.81	0.532
Story2	7	Top	2.482	0.35
Story1	3.5	Top	1.28	0.234
Base	0	Top	0	0

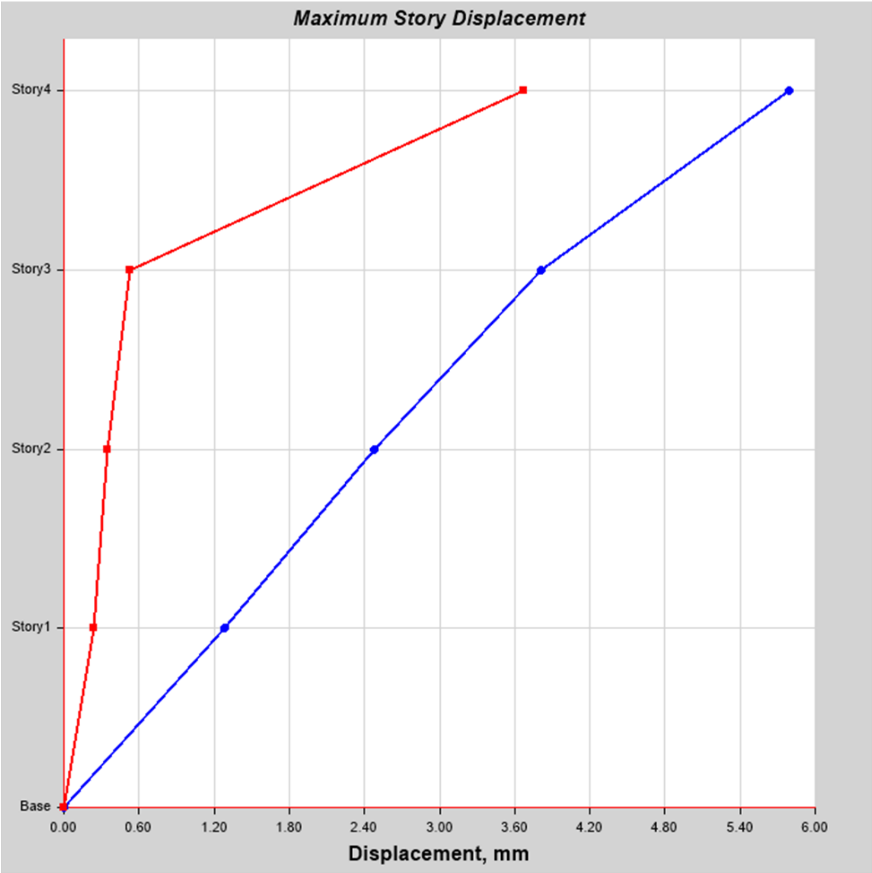


Figure 3.8-3 Seismic Displacement in X-dir.

Across the other face the following data was extracted:

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		mm	mm
Story4	14	Top	1.598	12.158
Story3	10.5	Top	0.469	7.038
Story2	7	Top	0.411	4.928
Story1	3.5	Top	0.318	2.487
Base	0	Top	0	0

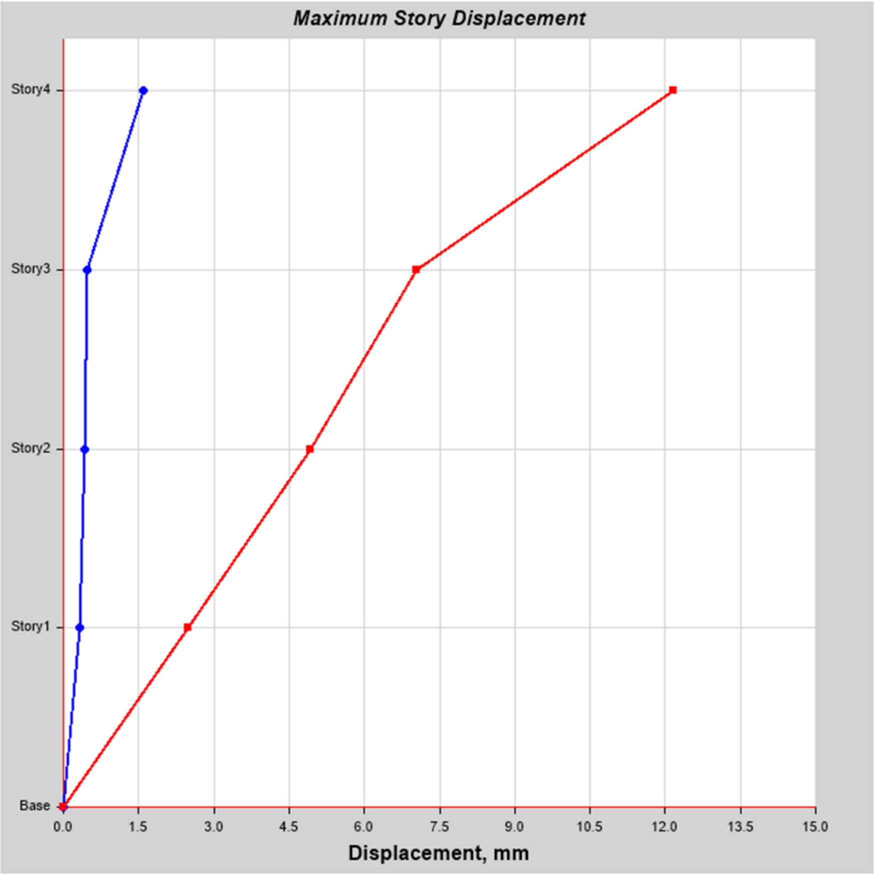


Figure 3.8-4 Seismic Displacement in Y-dir.

As seen in both tables above, the maximum lateral drift value was 12.2mm. This is less than the allowable limit of 280mm.

4 CHAPTER 4: STEEL REINFORCEMENTS

4.1 Slab Reinforcement:

1. Obtain the maximum moment for the slab from ETABS.

   M_{max} = 330kN.m, fc' = 28 MPa, f_y = 420MPa

   b = width = 1000mm and **d** = actual effective depth = 500mm

2. Using the following equation from concrete book to find value of K

   $$K = \frac{M\ max}{0.9\ bd^2} = \frac{330(10^6)}{0.9(1000)(500^2)} = 1.466\ \text{N/mm}^2$$

3. Using the following equation to find steel ratio

   $$\rho = \frac{0.85}{\text{fy}} fc' \left(1 - \sqrt{1 - \frac{2\ Ru}{0.85\ fc'}}\right) = \frac{0.85\ (28)}{420}\left(1 - \sqrt{1 - \frac{2\ (1.466)}{0.85\ (28)}}\right) = 0.00360$$

4. Compute the A_{st} by multiply the steel ratio with b and d

   $$A_s = \rho bd = 0.00360(1000)(500) = 1800\ \text{mm}^2$$

5. Select number of bar and diameter from reinforcement table

   Using 14 # 13 = 1806 mm^2

6. Check for the $A_{st,\ min}$ and $A_{st,max}$ by using following equations from table A-5 of concrete book

   $$\rho_{Min} = \frac{3\ \sqrt{fc'}}{fy} = \frac{3\sqrt{28000}}{420000} = 0.0011 \gg A_{steel\ Min} = 0.0012\ (1000)(500) = 600\ \text{mm}^2$$

   $$\rho_{Max} = 0.75\ \left(\frac{0.85\ \beta\ fc'}{fy}\right)\left(\frac{600}{600+fy}\right) = 0.75\ \left(\frac{0.85\ (0.65)(28)}{420}\right)\left(\frac{600}{600+420}\right) = 0.02125$$

   $$A_{st\ max} = 0.02125\ (1000)(500) = 10625\text{mm}^2 \gg A_{st\ min} < A_{st} < A_{st\ max}$$

7. Spacing between bars:

   S = (L – 2C.C. – 2d_t – # of bar × d_b) / (# of bar – 1)

   *C.C. = concrete cover, *d_b: diameter of bar, d_t = tie diameter (none here)

8. Sketch the design showing the details of cross section with tie bars

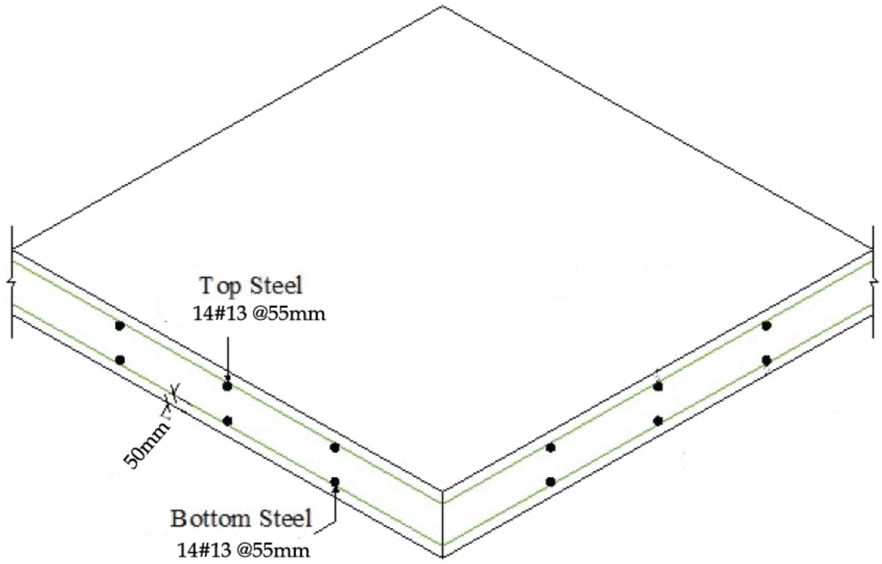


Figure 4.1-1 Slab Reinforcement

4.2 Ramp Reinforcement:

A similar procedure will be used, however for the ramp ETABS directly provides the area of steel required:

1. Obtain the Area of required for the ramp from ETABS

   $A_{s,\ required} = 4773\ mm^2$

2. Select number of bar and diameter from reinforcement table

   Using 6 # 32 = 4914 mm^2

3. Check for the $A_{st,\ min}$ and $A_{st,max}$ by using following equations from table A-5 of concrete book

$$\rho_{min} = \frac{3\ \sqrt{fc'}}{fy} = \frac{3\sqrt{28000}}{420000} = 0.0011 \gg A_{steel\ Min} = 0.0012\ (1000)(250) = 300\ mm^2$$

$$\rho_{max} = 0.75\ \left(\frac{0.85\ \beta\ fc'}{fy}\right)\left(\frac{600}{600+fy}\right) = 0.75\ \left(\frac{0.85\ (0.85)(28)}{420}\right)\left(\frac{600}{600+420}\right) = 0.02125$$

$$A_{st\ max} = 0.02125\ (1000)(250) = 5312.5\ mm^2 \gg A_{st\ Min} < A_{st} < A_{st\ max}$$

Using minimum clear spacing 50mm between bars (SBC304)

4. Sketch the design showing the details of cross section with tie bars

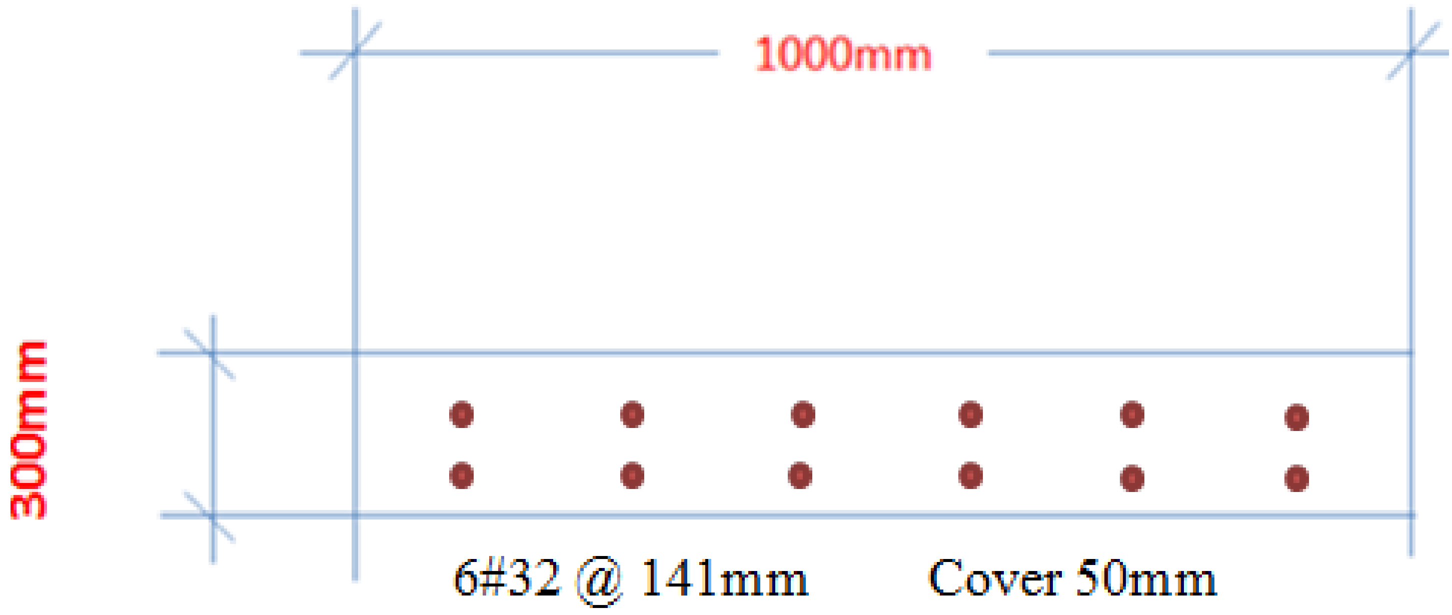


Figure 4.2-1 Ramp Reinforcement Cross Section

4.3 Column Reinforcement

For the reinforcement of columns, SBC specifies the following:

- **7.6.1** The minimum clear spacing between parallel bars in a layer shall be d_b, but not

less than 25 mm. See also Section 3.3.2.

- **7.10.5.2** Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or

wire diameters, or least dimension of the compression member

Also, from ACI the following gives the codes governing the ties:

Reinforcement Requirement (Lateral Ties)

(ACI 7.10.5.1)

Size $\geq$ # 10 mm bar if longitudinal bar $\leq$ #32 mm bar

$\geq$ # 12 mm bar if longitudinal bar $\geq$ # 36 mm bar

$\geq$ # 16 mm bar if longitudinal bars are bundled

Vertical Spacing: (ACI 7.10.5.2)

$S \leq 16\ D_b$ (D_b for longitudinal bars)

$S \leq 48\ D_t$ (D_t for tie bar)

$S \leq$ least lateral dimension of column

- ACI Code 10.9.1 requires $0.01 \leq \rho_g \leq 0.08$

4.3.1 Edge Columns

Table 4.3-1 Edge Column Dimensions

Edge Columns	
Floor	Columns Dimensions
G	650 mm X 650 mm
1	650 mm X 650 mm
2	600 mm X 600 mm
3	600 mm X 600 mm

Ground and 1st Floor:

A_g = 650 x 650 = 422500 mm^2

$A_{s,req}$ = 13865 mm^2 (From ETABS)

Choose: 10 #43 @ 118 mm c/c spacing

Gives: As = 14520 mm^2> 13865 mm^2

$S_{min} \leq 118$ mm $< S_{\max}$

For ties use #12 mm stirrups with 150 mm spacing

Checking $\rho_g = \frac{A_{st}}{A_g}$

$\frac{14520}{422500} = 0.01 \leq \mathbf{0.03} \leq 0.08$ (within the range)

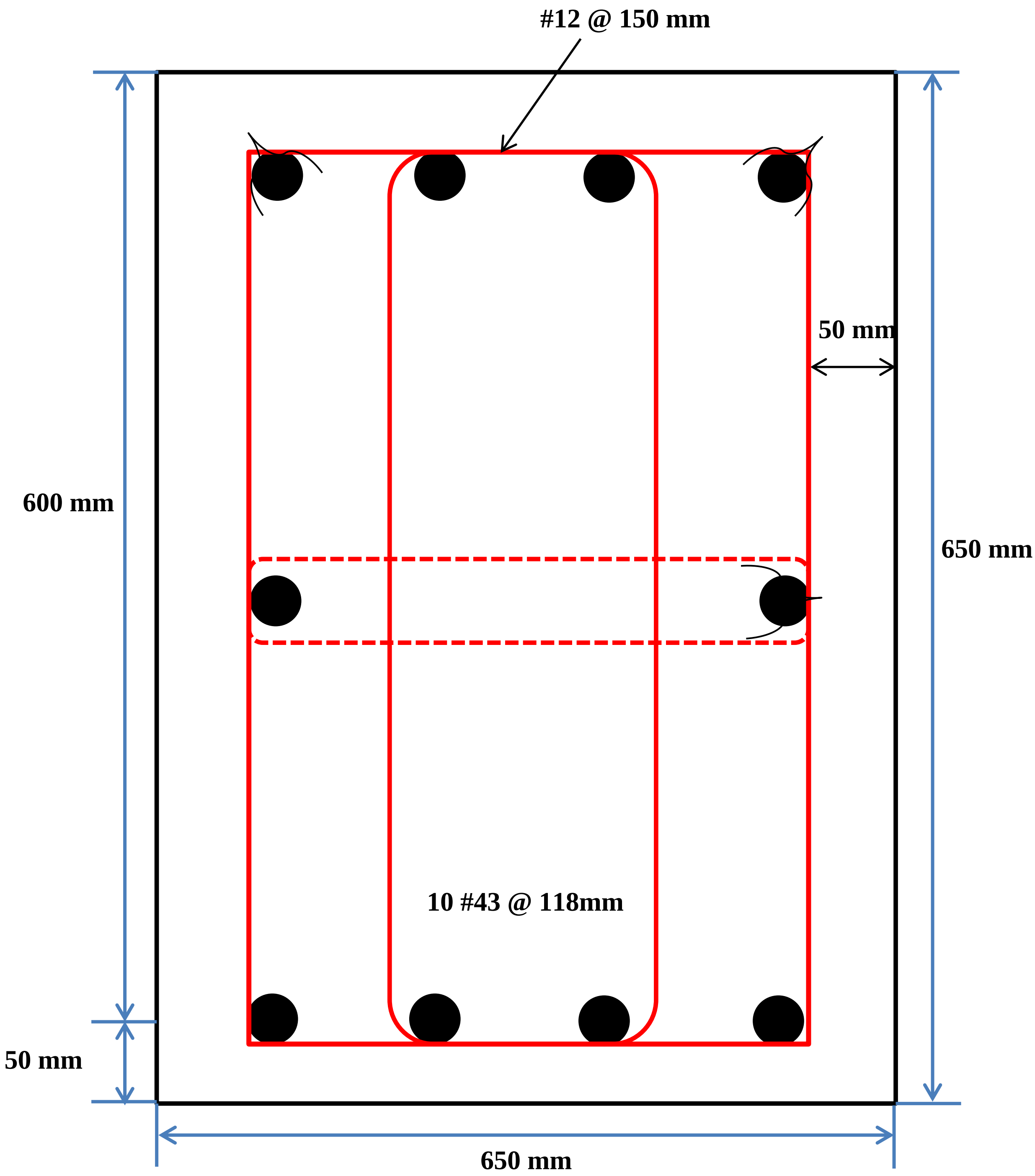


Figure 4.3-1 Column Reinforcement

2st Floor and 3st Floor:

A_g = 600 x 600 = 360000 mm^2

$A_{s,req}$ = 13349 mm^2 (From E-TABS)

Choose 10 #43 @ 118mm c/c spacing

Gives: As = 14520 mm^2 > 13349 mm^2

$S_{min} \leq 118$ mm $< S_{\max}$

Use #12 mm stirrups with 150 mm spacing

Checking $\rho_g = \frac{A_{st}}{A_g} = \frac{14520}{360000} = 0.04 \rightarrow 0.01 \leq \mathbf{0.04} \leq 0.08$ (within range)

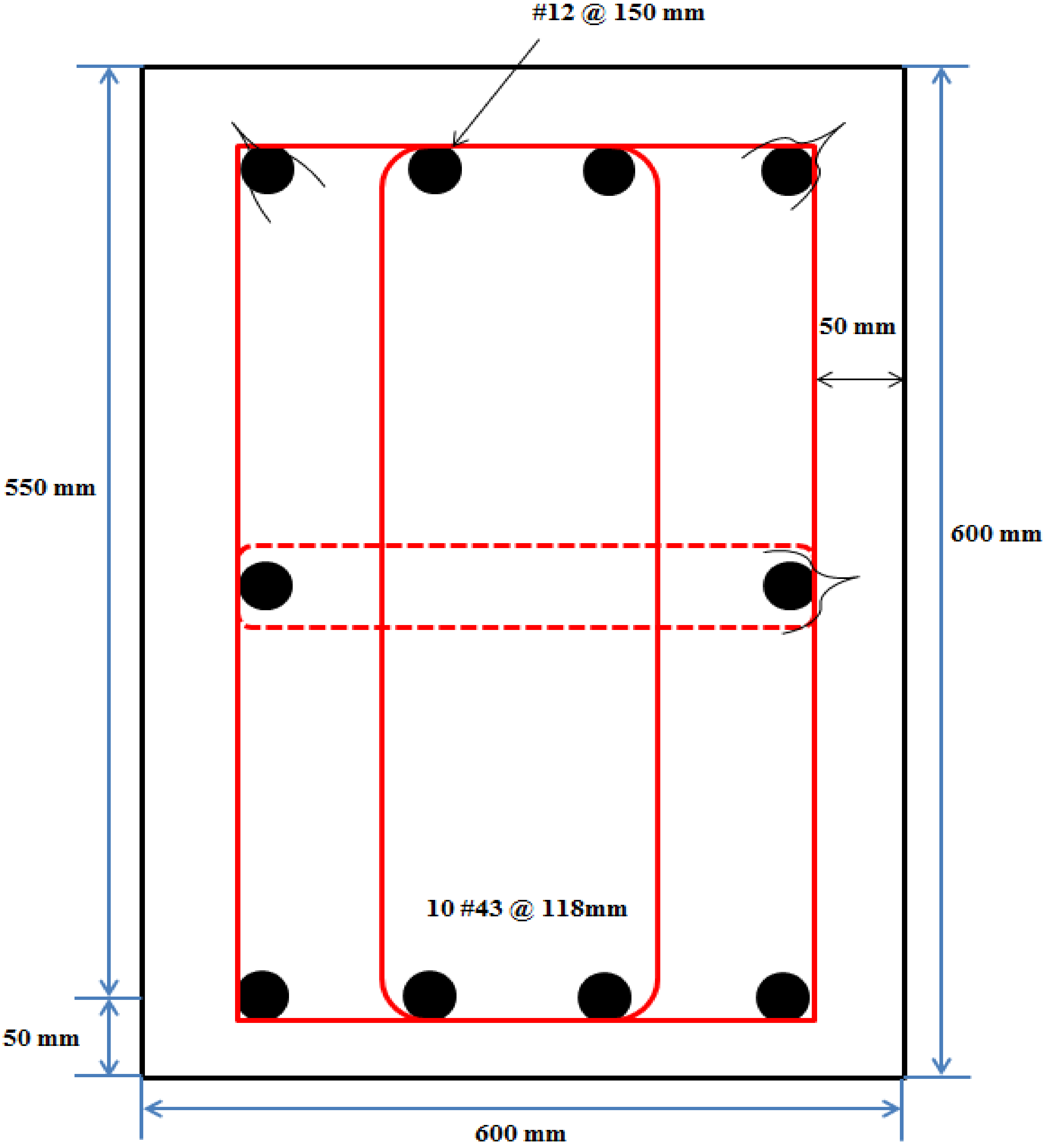


Figure 4.3-2 Column Reinforcement

4.3.2 Central Columns:

Table 4.3-2 Center Columns

Center Columns	
Floor	Columns Dimensions
G	900 mm X 900 mm
1	900 mm X 900 mm
2	750 mm X 750 mm
3	750 mm X 750 mm

<u>Ground and 1st Floor:</u>

A_g = 900 x 900 = 810000 mm^2

$A_{s,req}$ = 21495 mm^2 (From E-TABS)

Choose 10 #25 @ 225 mm c/c spacing

Gives: As = 25810 mm^2 > 21495 mm^2

Use #12 mm stirrups with 150 mm spacing

Checking $\rho_g = \frac{A_{st}}{A_g} = \frac{25810}{810000} = 0.03 \rightarrow 0.01 \leq \mathbf{0.03} \leq 0.08$ (within the range)

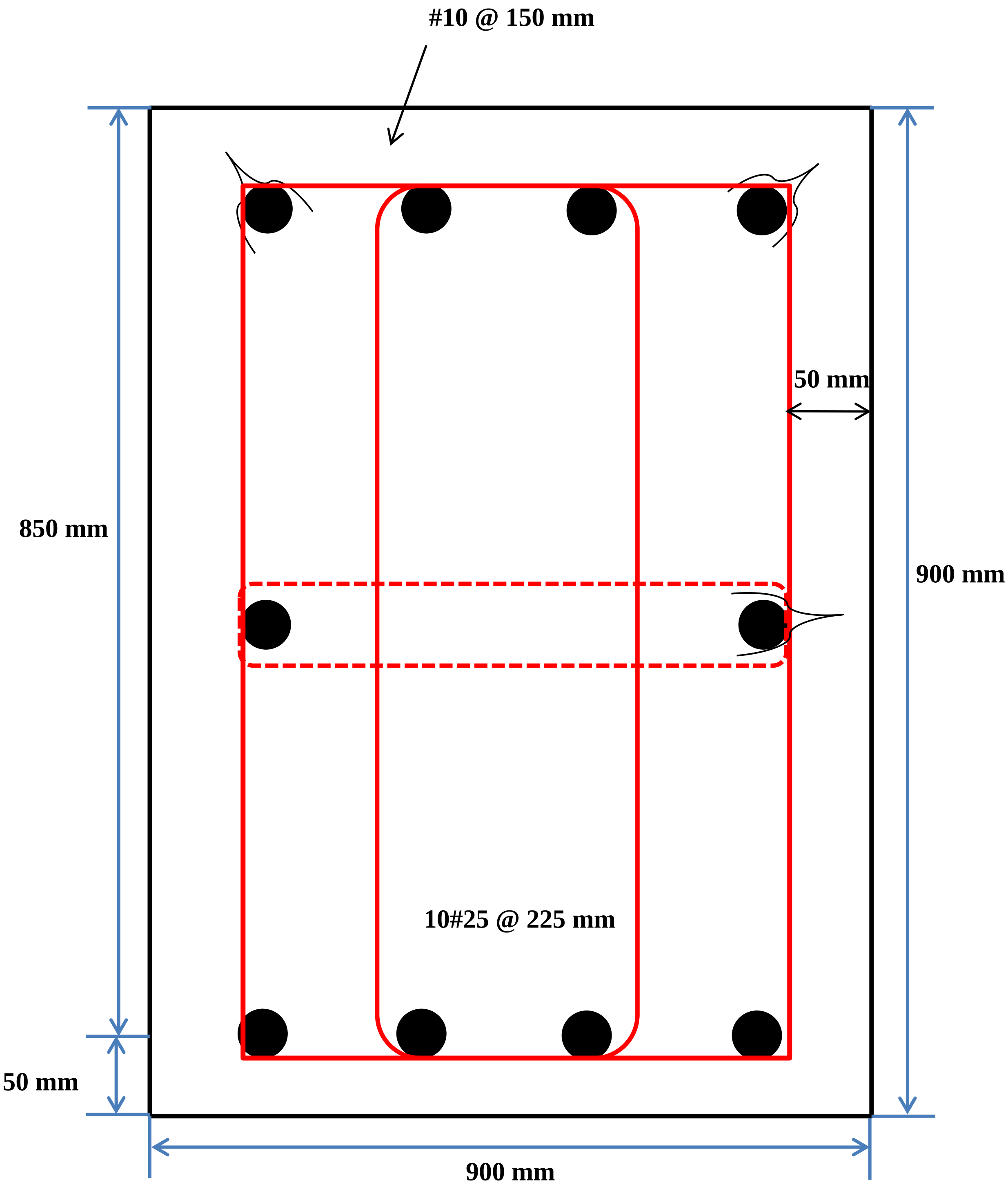


Figure 4.3-3 Column Reinforcement

2st Floor and 3st Floor:

A_g = 750 x 750 = 562500 mm^2

$A_{s,req}$ = 5625 mm^2 (From E-TABS)

Choose 10 #29 @ 170 mm c/c spacing

Gives: As = 6450 mm^2 > 5625 mm^2

Use #12 mm stirrups with 150 mm spacing

Checking $\rho_g = \frac{A_{st}}{A_g} = \frac{6450}{562500} = 0.011 \rightarrow 0.01 \leq \mathbf{0.011} \leq 0.08$ (Ok within the range)

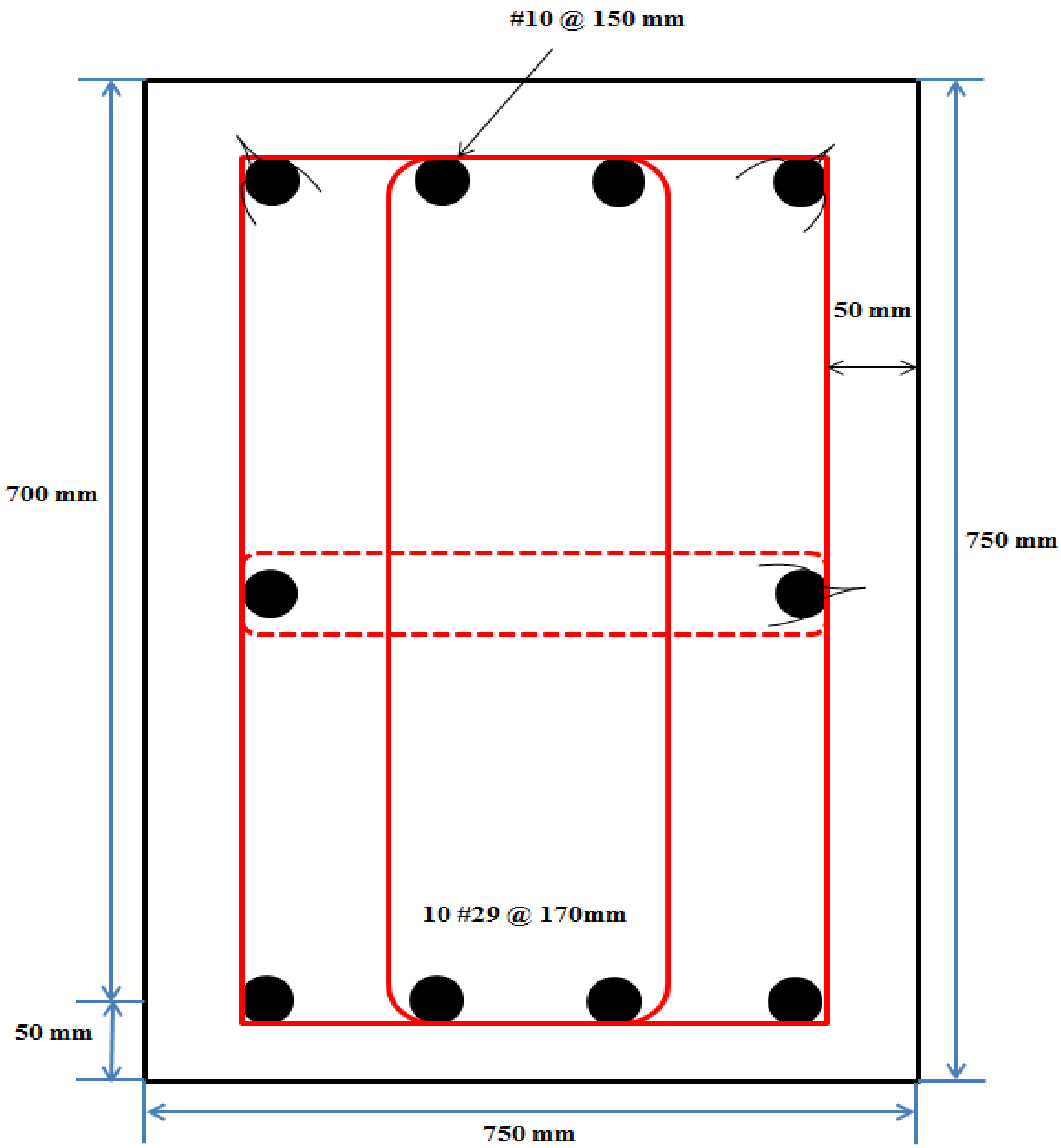


Figure 4.3-4 Column Reinforcement

4.4 Shear Wall Reinforcement

There are four shear walls located at each corner of the building. To design the reinforcement of the shear wall we will split it into segments, and then design the reinforcement requirement for each one. The thickness of the shear wall is 150 mm and we will design for one metric line.

Figure 4.4-1 Shear Wall Top View

Shape	Length
1	2.5m
2	2m
3	6m
4	4.7m
Total	24.7m

For

section 1:

L = 2.5 m

$A_{s,req}$ = 1500 mm^2 (From E-TABS)

Choose: 8 # 16

Gives: As = 1592 mm^2 > 1500 mm^2

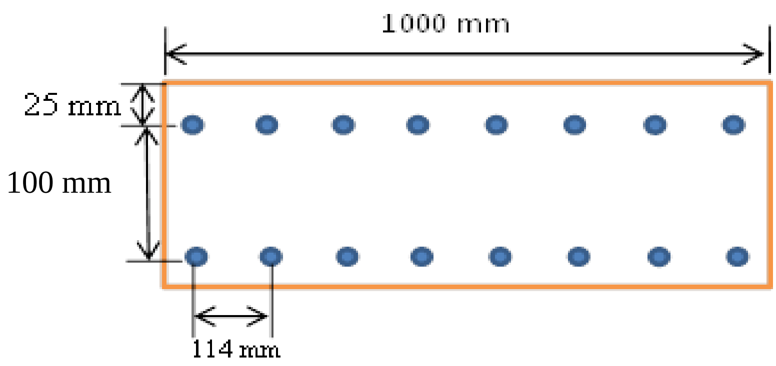


Figure 4.4-2 Section 1 Reinforcement

Spacing between top and bottom bars:

S = 150 – 2cc – 2(Tie) – # of bar in vertical line × d_b

*cc (concrete cover) = 25 mm, *d_b (diameter of bar),

S = 150- 2(25) - 2(10) -2×16 = 48 mm

48mm > d_b and 25 mm so ok ✓

Spacing in one metric line bars:

- S = 1000 -2(25)-2(10)-8(16) = 802 mm ÷ 7 = 114.5 = 114 mm

S_{min} = 114 mm > 25 mm

S_{min} = 114 mm > Bar Diameter

<u>For section 2:</u>

L = 2.5 m

$A_{s,req}$ = 1125 mm^2 (From E-TABS)

Choose 6 #16

Gives: As = 1194 mm^2 > 1125 mm^2

Spacing between top and bottom bars:

S = 150- 2(25) - 2(10) -2×16 = 48 mm

48mm > d_b 25 mm so ok ✓

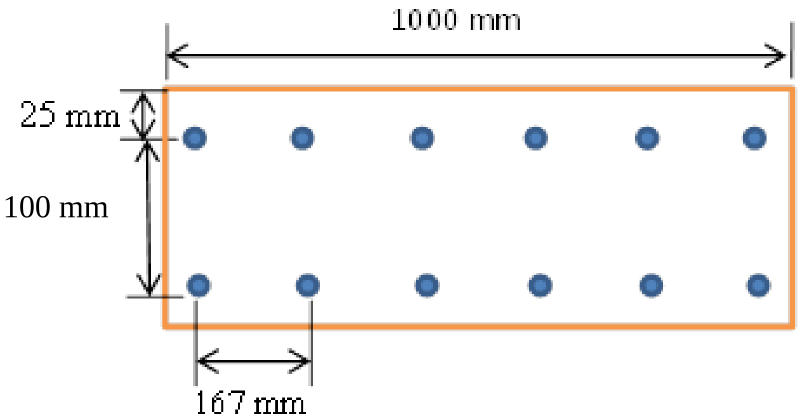


Figure 4.4-3 Section 2 Reinforcement

Spacing in one metric line bars:

S = 1000 -2(25)-2(10)-8(16) = 802 mm ÷ 7 = 166.8 = 167 mm

S_{min} = 167 mm > 25 mm

S_{min} = 167 mm > Bar Diameter

For section 3:

L = 6 m

$A_{s,req}$ = 3750 mm^2 (From E-TABS)

Choose 10 #22

Gives: As = 3870 mm^2 > 3750 mm^2

Spacing between top and bottom bars:

S = 150- 2(25) - 2(10) -2×22 = 36 mm

36 mm > d_b and 25 mm so ok ✓

Spacing in one metric line bars:

S = 1000 -2(50)-2(10)-10(22) = 710 mm = 78.8 = 79 mm

S_{min} = 79 mm > 25 mm

S_{min} = 79 mm > Bar Diameter

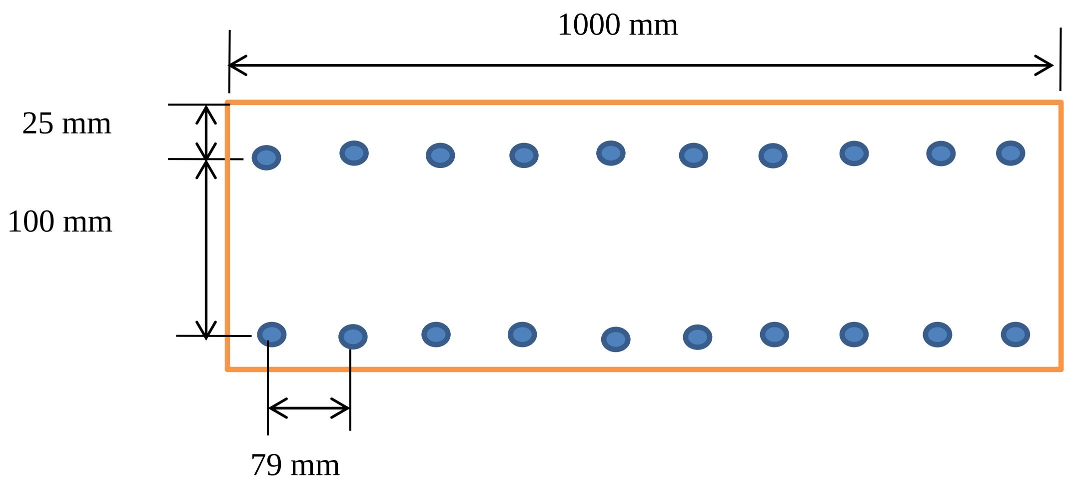


Figure 4.4-4 Section 3 Reinforcement

For section 4:

L = 4.7 m

$A_{s,req}$ = 2940 mm^2 (From E-TABS)

Choose 8 #22

Gives: As = 3096 mm^2 > 2940 mm^2

Spacing between top and bottom bars:

S = 150- 2(25) - 2(10) -2×22 = 36 mm

36mm > d_b and 25 mm so ok ✓

Spacing in one metric line bars:

S = 1000 -2(25)-2(10)-8(22)= 754 mm = 107.7 = 108 mm

S_{min} = 108 mm > 25 mm

S_{min} = 108 mm > Bar Diameter

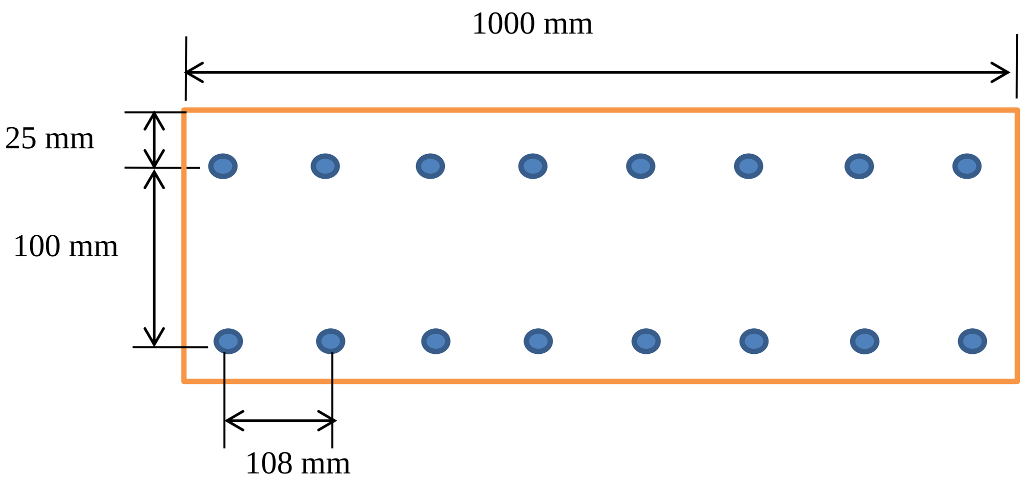


Figure 4.4-5 Section 4 Reinforcement

Final Reinforcement Details:

Table 4.4-1 Shear Wall Reinforcement

Shear Wall				
Total length	24.7 m			
Thickness	150 mm			
Shear Wall Section	1	2	3	4
A_{st} (req.)	1500 mm^2	1125 mm^2	3750 mm^2	2940 mm^2
Reinforcement (Each Side)	8#16	6#16	10#22	8#22
A_{st} (act.)	1592 mm^2	1194 mm^2	3870 mm^2	3096 mm^2

5 CHAPTER 5: GEOTECHNICAL DESIGN OF FOUNDATION SYSTEM

5.1 Soil Profile

As mentioned earlier, the soil profile consists of three layers (fig.5.1-1) the first layer is six meters of loose to medium dense sand called "Sabkha Soil", followed by 22 meters of very fine to fine low plasticity sandy clay, and finally a refusal layer of rock. The water table lies at one meter from the ground level, since the site is in a coastal region, requiring the site to be dewatered and a permanent drainage system to be installed. The water is highly contaminated with sulphate and chloride (Table 5.1-1), these contaminants have been transferred to the sand layer causing it to be above the allowable contamination limit. Due to these contaminants and the nature of the soil, the sand layer is very weak in terms of its bearing capacity; it therefore, needs to be replaced with a select fill in order to build on it.

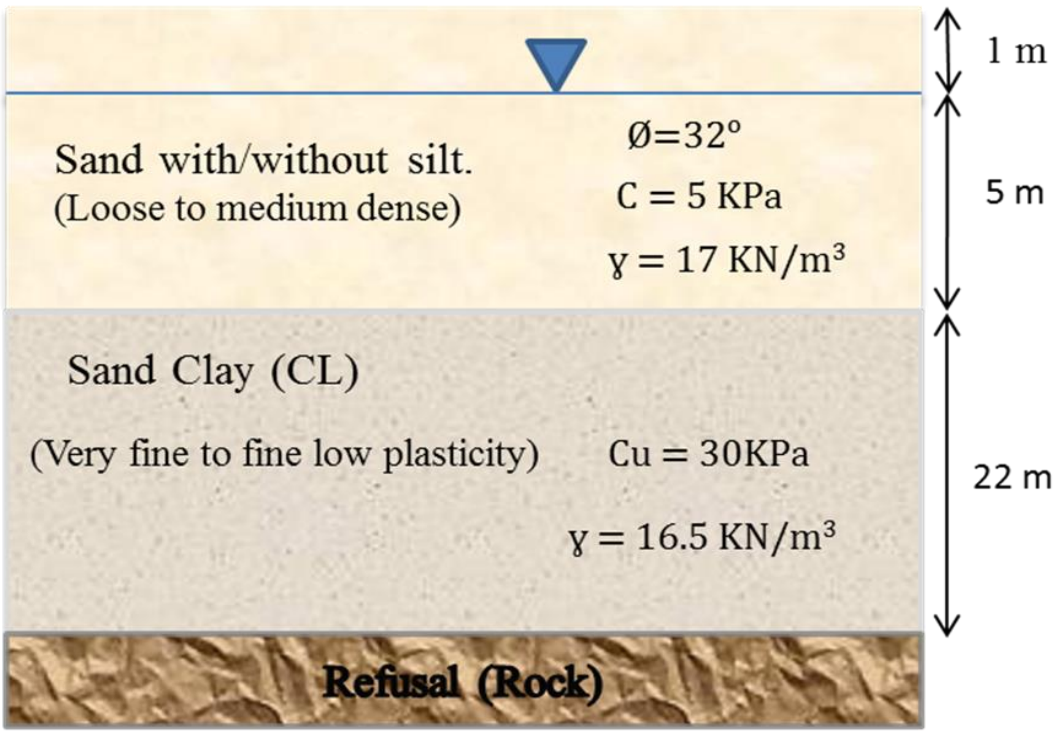


Figure 5.1-1 Soil Profile

Table 5.1-1 Chemical Analysis

Sample number	Depth (m)	Chlorides %	Sulfates %
1	1.00	0.318	0.391
2	1.00	0.321	0.400
3	3.00	0.088	0.375
Average	-	0.24	0.38

- **0.24 % > 0.05 % (maximum allowable)**
- **0.2 % < 0.38 % < 2 % (Therefore sulfate attack hazard is severe).**

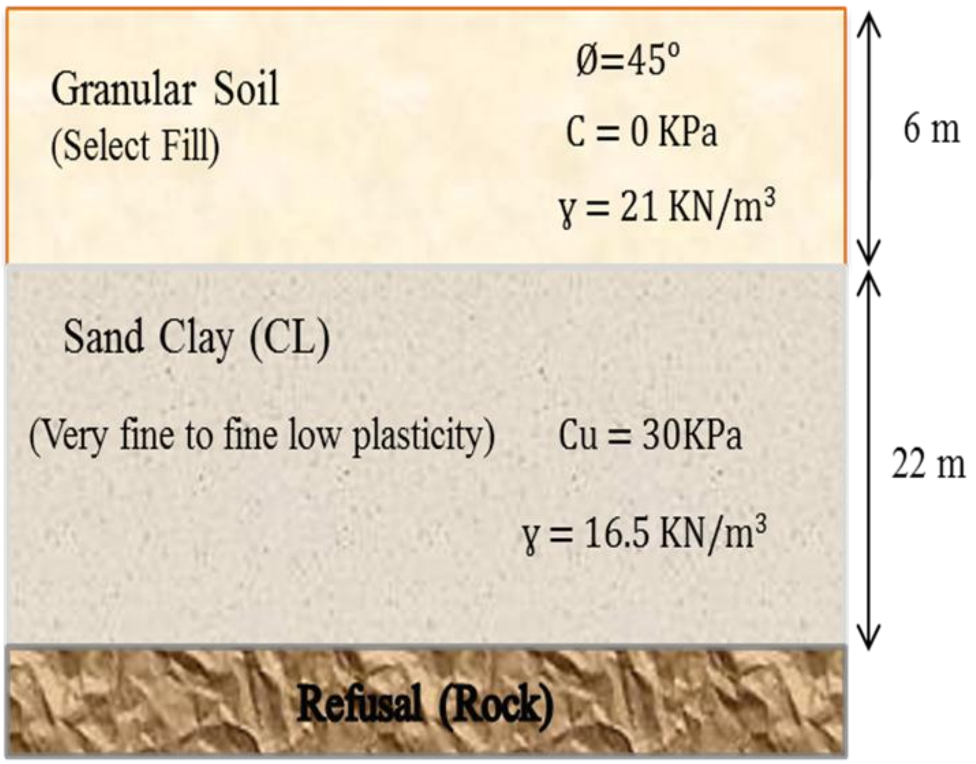


Figure 5.1-2 Modified Soil profile

5.2 Foundation System:

Since the second layer is a large clay layer, it is known through experience that it will experience a high settlement level. This will require the use of deep foundation system (piles) to transfer the loads to the hard rock in order to negate the settlement. For the foundation system itself, we will first consider an isolated foundation system, with piles of course. To check wither or not an isolated foundation system (footing) will be sufficient enough, we will calculate the required side "B" of the footing. Since the footing is supported with piles, the footing will simply act as a cap for the piles and not carry any load which will be mainly carried be the piles. Therefore, the dimensions of each footing should be within one meter each side. We will assume an embedment depth of 1.5m for the design of the footings as well as the data given in (fig 5.1-1) pertaining to the soil properties (Cu and γ). If the isolated foundation system is not sufficient, a mat foundation system will be considered.

5.3 Isolated Foundation System:

We will first consider an isolated foundation system. According to Terzaghi:

$$\frac{Q}{B^2} = FS\left(qN_q + CN_c + 0.5\,\gamma\, B\, N_\gamma \right)$$

$Q = Total\ Column\ Load$ = 14,000 KN (form ETABS)

$FS: Factor\ of\ Safety$

$q = \gamma\, D_f = 21 * 1.5 = 31.5\ {}^{\mathrm{KN}}/_{m^2}$

From table… using$\emptyset$:

N_q: 173.28 N_c = 172.28 $N\gamma$ = 325.34

Solving the above equation for “B” we obtain:

$$B = 1.88\text{m} \approx 2\text{ m}$$

Although the dimensions are relatively close to the design, it will we will check if the dimensions are sufficient enough to carry the forces in hands. Since using one pile under each footing by itself will not be sufficient.

Using piles with a diameter (d) = 0.6m; the maximum number of piles that can be inserted under one footing is four, one near each corner; the spacing between these piles will be 0.8m which is lower than the minimum value of 2.5d (Tahar, 2019). Also, even if the piles could fit under the footing, given four piles under each footing will not be sufficient enough to support the footings from the loads it will experience as can be later seen in the pile design calculation.

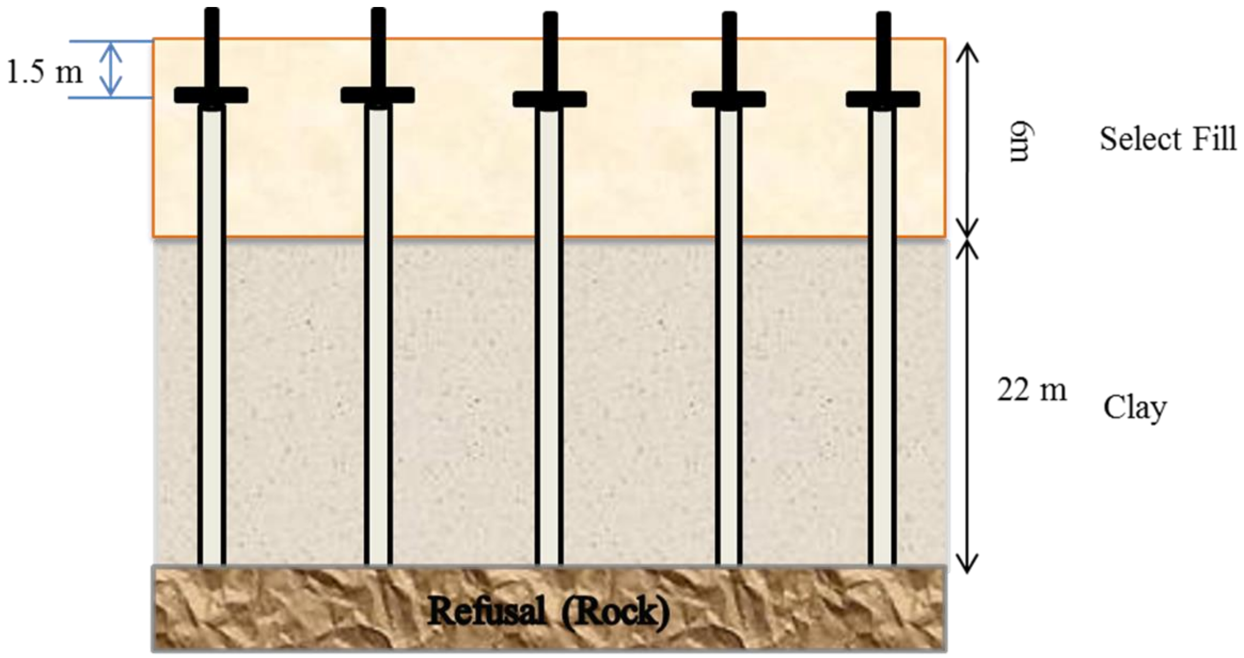


Figure 5.3-1 Isolated Foundation

5.4 Mat Foundation Design:

In order to design the mat foundation, we first need to derive an equation for the vertical force acting on each point; this is done by calculating the total load on the mat and checking if there exists an eccentricity in its location in relation to the center of the area. We will then calculate the bearing capacity of the soil; since contrary to the isolated foundation system, the mat foundation will be carrying the load, will the piles simply support the system from experiencing any large settlement. This loads will be transferred to the soil therefor the bearing capacity of the soil needs to be checked. We will then calculate the thickness of the mat foundation required based on the available loads, working with the critical strips in the x and y direction. Finally, we will design the reinforcement of the raft system based on the moments on the raft.

The methodology followed:

1. Calculating the soil bearing capacity. The pressure under each column must be less than the soil bearing capacity.
2. Calculating MAT foundation bearing capacity.
3. Summing the reactions coming from the columns.
4. Calculating the moment of inertia along X-axis & along axis to be able to calculate the pressure (q) under each column.
5. Computing the moment on X-axis & Y- axis.
6. Checking the pressure under each column if it is more or less than the soil bearing capacity.
7. We are providing supportive piles to increase the soil bearing capacity and reduce the settlement.
8. Calculating soil reaction which is the net soil bearing capacity that pushes the foundation upward.
9. Using ETABS to compute the shear and moment diagrams for each strip along X and Y directions.
10. Designing the MAT foundation depth based on the maximum shear value.
11. Getting the maximum and minimum moment for each strip from ETABS to calculate the required area of steel.

5.4.1 General Equation for Bearing Pressure

$$q = \frac{Q}{A} \pm \frac{M_x}{I_x} y \pm \frac{M_y}{I_y} x$$

$$I_x = \frac{bh^3}{12} = \frac{84 * 90^3}{12} = 5103000\ m^4 \qquad I_y = \frac{hb^3}{12} = \frac{90 * 84^3}{12} = 4445280\ m^4$$

From Excel: Q = 555946.9 KN , $x' = 37.58$ m , $y' = 38.56$ m

$$e_x = x' - \frac{B}{2} = 37.58 - \frac{84}{2} = -4.41\ m$$

$$e_y = y' - \frac{l}{2} = 38.56 - \frac{90}{2} = -6.43\ \mathrm{m}$$

$$M_y = Q * e_x = -2.45 * 10^6\ KN * m$$

$$M_x = Q * e_y = -3.57 * 10^6\ KN * m$$

So,

$$q = \frac{555946.9}{7560} - \frac{2.45 * 10^6}{5103000} y - \frac{3.57 * 10^6}{4445280} x \quad \rightarrow \quad q = 73.53 - 0.7\, y - 0.55\, x$$

<u>Calculating for Critical Strip:</u>

$$q_{c_3} = 73.53 - 0.7 \times 43.75 - 0.55 \times (-5) = 45.66\ \mathrm{KPa} \qquad (\mathrm{y-\ strip})$$

$$q_{c_{95}} = 73.53 - 0.7 \times (-42.65) - 0.55 \times (-5) = 106.14\ \mathrm{KPa} \qquad (\mathrm{y-\ strip})$$

$$q_{c_{45}} = 73.53 - 0.7 \times (-26.75) - 0.55 \times (-41.3) = 114.97\mathrm{KPa} \qquad (\mathrm{x-\ strip})$$

$$q_{c_{54}} = 73.53 - 0.7 \times (-26.75) - 0.55 \times (41.3) = 69.54\ \mathrm{KPa} \qquad (\mathrm{x-\ strip})$$

5.4.2 Bearing Capacity of the Soil:

(Meyerhof)

$$q_u = c\, N_c F_{cs} F_{cd} F_{ci} + q\, N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} B\gamma N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Where:

q_u : bearing capacity of the soil $(\frac{KN}{m^2})$

γ: unit weight of soil $(\frac{KN}{m^3})$

c: cohesion of soil $(\frac{KN}{m^2})$

q: effective stress at the bottom of the foundation = $\gamma\, D_f = (\frac{KN}{m^2})$

B: width of the foundation (m)

$N_c, N_q, and\ N_\gamma$ are factors obtained using Meyerhof's chart (fig.5.4-1)

$F_{cs}, F_{qs}, and\ \ F_{\gamma s}$ are shape factors

$F_{cd}, F_{qd}, and\ F_{\gamma d}$ are depth factors

$F_{ci}, F_{qi}\ , and\ F_{\gamma i}$ are inclination factors

$$F_{cs} = 1 + \frac{B\ N_q}{L\ N_c}$$

$$F_{cd} = 1 + 0.4\,\frac{D_f}{B}$$

$F_{ci} = (1 - \frac{\beta}{90})^2$, where, β is the inclination of the load on the foundation with respect to the vertical.

$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\emptyset$, where $\emptyset$ is the angel if shearing resistance.

$$F_{qd} = 1 + 2\tan\emptyset\,(1 - \sin\emptyset)^2\,\frac{D_f}{B}$$

$$F_{qi} = (1 - \frac{\beta}{90})^2$$

$$F_{\gamma s} = 1 - 0.4\,(\frac{B}{L})$$

$$F\_\gamma d = 1$$

$$F_{\gamma i} = (1 - \frac{\beta}{\emptyset})^2$$

Bearing Capacity Factors for General Bearing Capacity Equation

Table 3.3 Bearing Capacity Factors

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

Figure 5.4-1 Bearing Capacity Factors

Determine the soil bearing capacity

$$q_u = c\ N_c F_{cs} F_{cd} F_{ci} + q\ N_q F_{qs} F_{qd} F_{qi}\ + \frac{1}{2}\ B\gamma N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Since there exists an eccentricity in the location of the resultant load acting on the mat foundation; the dimensions B and L need to be modified.

$B' = B - 2e_x = 84 - 2\ (4.41\ ) = 75.18\ m$ and $L' = L - 2e_y = 90 - 2\ (6.43) = 77.14\ m$

$D_f = 1.5\ m$, c = 0 , $\emptyset = 45$, $\gamma = 21\ \frac{KN}{m^3}$, $\beta = 0$

From "Meyerhof's' chart" using $\emptyset = 45\ \rightarrow\ \boldsymbol{N_q = 134.87, N_\gamma = 271.75}$

So $q = 21\ \frac{KN}{m^3}\ (1.5\ m\ ) = 31.5\ \frac{KN}{m^2}$

$$F_{qs} = 1 + \left(\frac{75.18}{77.14}\right)\tan 45 = 1.97$$

$$F_{qd} = 1 + 2\tan 45\ (1 - \sin 45)^2\ \frac{1.5}{75.18} = 1.003$$

$$F_{qi} = (1 - \frac{\beta}{90})^2 = 1$$

$$F_{\gamma s} = 1 - 0.4\ \left(\frac{75.18}{77.14}\right) = 0.61$$

$$F_{\gamma d} = 1$$

$$F_{\gamma i} = (1 - \frac{\beta}{\emptyset})^2 = 1$$

Then:

$$q_u = 0 + 31.5\ (134.87)(1.97) + \frac{1}{2}(75.18)(21\ )(271.75)(0.61) = 139224.6\ \frac{KN}{m^2}$$

$$\therefore\ q_{all}\ = \frac{q_u}{FS} = \frac{139224.6}{3} = 46408.18\frac{KN}{m^2}$$

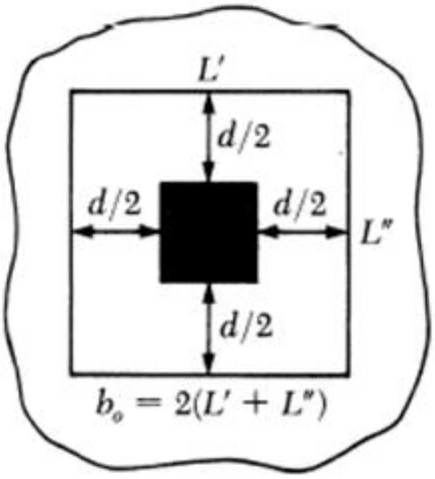
Since the soil bearing capacity is larger than the point soil pressure under each column with 1.5 as thickness of the mat foundation, we can consider 1.5 m depth for a mat foundation is suitable and safe.

5.4.3 Thickness of Mat Foundation:

The critical section for diagonal tension shear will be at the column C51. Column C51 is carrying 13930.28 KN.

Determining the effective depth by the following equation from American Concrete Institute (ACI) code:

$U = b_o\, d[\varphi(0.34)\sqrt{fc'}\,]$



$U\ =$ column load x load factor

$U\ = 13930.28(1.5)$

$U\ = 20895.42\ kN\ = 20.89\ MN$

$b_o = 4(a + d) =\ 4(0.9 + d) = 3.6 + 4d$

$U\ = b_o\, d[\varphi(0.34)\sqrt{fc'}\,]$

$20.89\ \ = (3.6 + 4d)d\ [0.85(0.34)\sqrt{28}\,]$

$d\ = 1.46m\ \ \approx 1.5\ m$

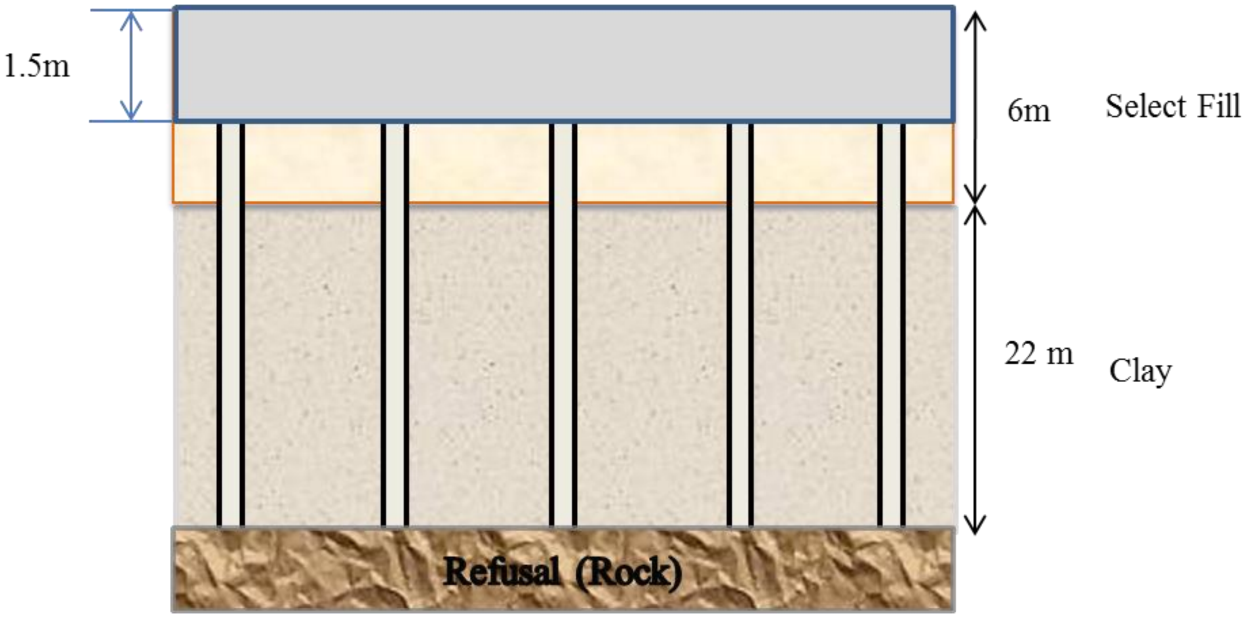


Figure 5.4-2 Mat Foundation

5.4.4 Reinforcement of Mat Foundation

In order to find the reinforcement required for the mat foundation. We first need to identify the critical strip in both the x and y direction. This is done by examining the forces on each strip. By checking the values, the critical strips where found to be as indicated in fig 5.4-3. Then using the appropriate geotechnical formulas, the soil pressure and total pressure load under each strip can be calculated to then obtain the load modification factor, which will be used to find the load per unit length. This is then inserted into ETABS along with the column loads to obtain the moment diagrams which will be used to find the top and bottom reinforcement.

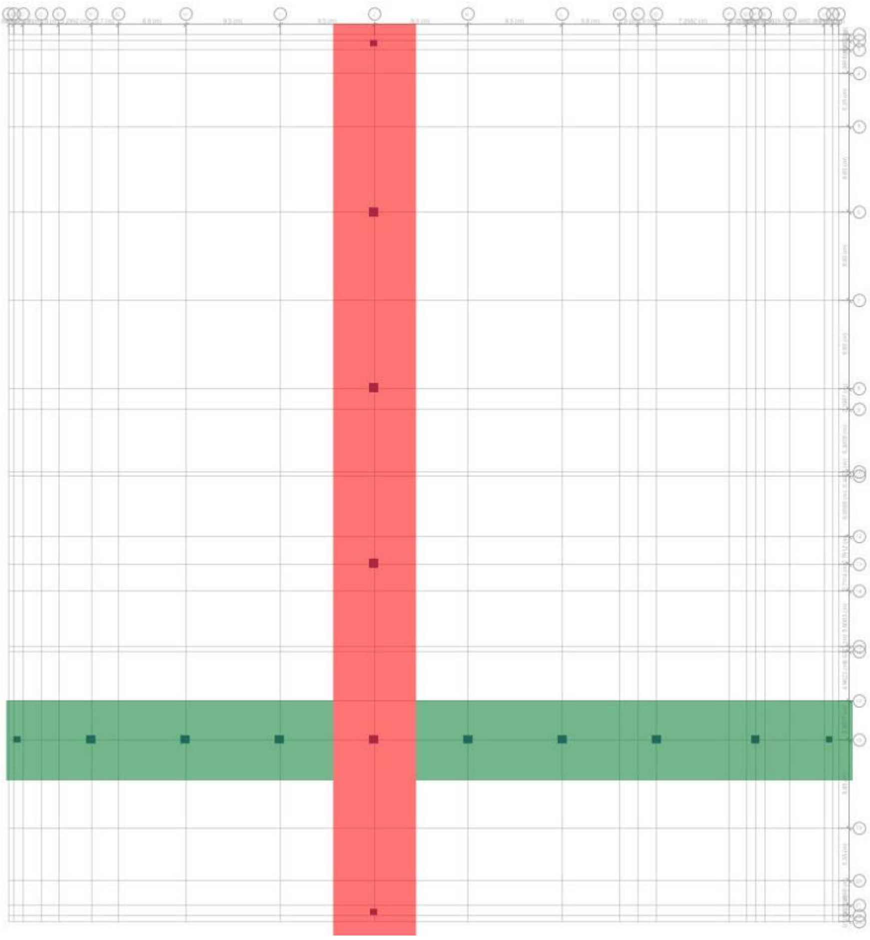


Figure 5.4-3 Critical Strips

Shear & Moment Diagram for "Y- Strip"

Average soil pressure: $q_{av} = \frac{(q_{c_3}+q_{c_{59}})}{2} = \frac{45.66+106.13}{2} = 75.89\ {}^{KN}/_{m^2}$

Total soil reaction: $q_{av}\ B\ \ L = 75.89 \times 17.7 \times 84 = 112840.7$ KN

$$Average\ load = \frac{load\ due\ to\ soil\ reaction + column\ loads}{2}$$

$= \frac{112840.7+64135.3}{2} = 88488\ KN$

Modified average soil pressure: $q_{av(modified)} = q_{av}(\frac{Average\ load}{Total\ soil\ reaction})$

$q_{av(modified)} = 75.89\ (\frac{88488}{112840.7}) = 59.51\ {}^{KN}/_{m^2}$

Column loads Factor: F $= \frac{Average\ load}{Column\ loads} = \frac{88488}{64135.3} = 1.38$

Load per unit length: $B\ q_{av(modified)} = 59.51(17.7) = 1053.4\ {}^{KN}/_{m^2}$

Shear & Moment Diagram for "X- Strip"

Average soil pressure: $q_{av} = \frac{(q_{c_{45}}+q_{c_{54}})}{2} = \frac{114+69.54}{2} = 91.77\ {}^{KN}/_{m^2}$

Total soil reaction: $q_{av}\ B\ \ L = 91.77 \times 9.5 \times 90 = 77853.1$ KN

$Average\ load = \frac{load\ due\ to\ soil\ reaction+column\ loads}{2} = \frac{109078.7+77853}{2} = 93465.9\ KN$

Modified average soil pressure: $q_{av(modified)} = q_{av}(\frac{Average\ load}{Total\ soil\ reaction})$

$q_{av(modified)} = 91.77\ (\frac{93465.9}{77853.1}) = 110.1\ {}^{KN}/_{m^2}$

Column loads Factor: F $= \frac{Average\ load}{Column\ loads} = \frac{93465.9}{109078.7} = 0.857$

Load per unit length: $B\ q_{av(modified)} = 91.77(9.5) = 1046.6\ {}^{KN}/_{m^2}$

Steel Reinforcement:

From ETABS, the following values where obtained from the moment diagram for both the x and y direction:

$M_{max} = 8761\ KN * m \quad (x - strip) \quad$ (Fig. 5.4-4)

$M_{min} = -4940\ KN * m \quad (x - strip) \quad$ (Fig. 5.4-4)

$M_{max} = 37029\ KN * m \quad (y - strip) \quad$ (Fig. 5.4-5)

$M_{min} = -20947\ KN * m \quad (y - strip) \quad$ (Fig. 5.4-5)

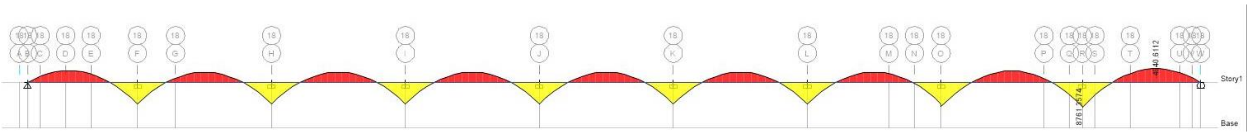


Figure 5.4-4 Moment Diagram in x-dir.

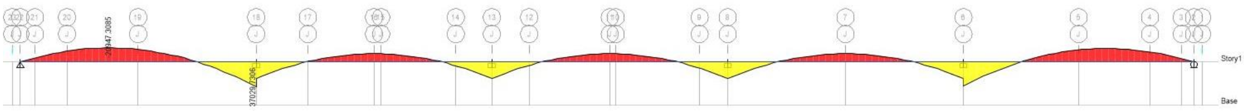


Figure 5.4-5 Moment Diagram in Y-dir.

Y-strip:

Calculating the required steel reinforcement for the maximum positive moment using the following equation from ACI:

$$M' = \frac{M\,max}{B} = \frac{37029}{17.7} = 2092\ \frac{kN.m}{m}$$

$$\text{Mu} = M'(load\ factor) = \varphi\ As\ fy\left(d - \frac{a}{2}\right) 2092(1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$$

$$a = \frac{As(fy)}{0.85fc'b} = \frac{As(413.7)}{0.85(28)(1)} = 17.38\ As \ \rightarrow As = 0.0575\ a$$

$$2092\ (1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$$

$2092\ (1.5) = 0.9\ (0.0575a)(413700)\left(1.5 - \frac{a}{2}\right) \quad \rightarrow \quad a = 0.1\ m$

$As = 0.0575\ a = 5.75 \times 10^{-3}\ \ m^2/m = 5750\ \ mm^2/m$

Use 9#29 @ 80 $mm\ c/c \rightarrow As = 5805\ \ mm^2$\

Calculating the required steel reinforcement for the <u>maximum negative moment</u>

$M' = \frac{M\ max}{B} = \frac{37029}{17.7} = 1183\ \frac{kN.m}{m}$

Mu $= M'(load\ factor) = \varphi\ As\ fy\left(d - \frac{a}{2}\right) 1183(1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$

$a = \frac{As(fy)}{0.85fc'b} = \frac{As(413.7)}{0.85(28)(1)} = 17.38\ As\ \rightarrow As = 0.0575\ a$

$1183\ (1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$

$1183\ (1.5) = 0.9\ (0.0575a)(413700)\left(1.5 - \frac{a}{2}\right) \quad \rightarrow \quad a = 0.05\ m$

$As = 0.0575\ a = 2.875 \times 10^{-3}\ \ m^2/m = 2875\ \ mm^2/m$

Use 8# 22 @ 100 $mm\ c/c \rightarrow As = 3096\ mm^2$\

<u>X-strip:</u>

Calculating the required steel reinforcement for the <u>maximum positive moment</u> using the following equation from ACI:

$M' = \frac{M\ max}{B} = \frac{8761}{9.5} = 922\ \frac{kN.m}{m}$

Mu $= M'(load\ factor) = \varphi\ As\ fy\left(d - \frac{a}{2}\right) 922(1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$

$a = \frac{As(fy)}{0.85fc'b} = \frac{As(413.7)}{0.85(28)(1)} = 17.38\ As\ \rightarrow As = 0.0575\ a$

$922\ (1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$

$922\ (1.5) = 0.9\ (0.0575a)(413700)\left(1.5 - \frac{a}{2}\right) \quad \rightarrow \quad a = 0.044\ m$

$As = 0.0575\ a = 2.53 \times 10^{-3}\ \ m^2/m = 2530\ \ mm^2/m$

Use 9#19 @ 90 $mm\ c/c \rightarrow As = 2556\ \ mm^2$

Calculating the required steel reinforcement for the maximum negative moment

$$M' = \frac{M\,max}{B} = \frac{4940}{9.5} = 520\ \frac{kN.m}{m}$$

$$\text{Mu} = M'(load\ factor) = \varphi\ As\ fy\left(d - \frac{a}{2}\right) 520(1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$$

$$a = \frac{As(fy)}{0.85 fc'b} = \frac{As(413.7)}{0.85(28)(1)} = 17.38\ As \rightarrow As = 0.0575\ a$$

$$520\ (1.5) = 0.9\ As\ (413700)(1.5 - \frac{a}{2})$$

$$520\ (1.5) = 0.9\ (0.0575a)(413700)\left(1.5 - \frac{a}{2}\right) \quad \rightarrow \quad a = 0.0245\ m$$

$$As = 0.0575\ a = 1.41 \times 10^{-3}\ m^2/m = 1410\ mm^2/m$$

Use 8# 16 @ 110 $mm\ c/c \rightarrow As = 1592\ mm^2$

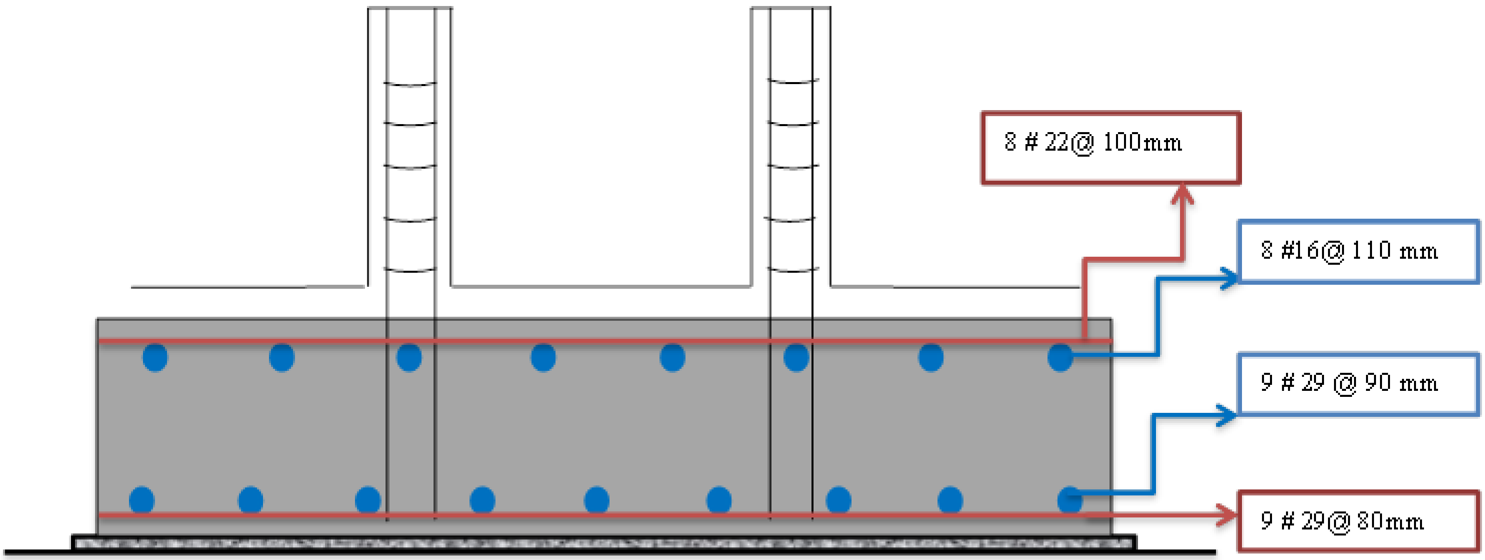


Figure 5.4-6 Raft Reinforcement

5.5 Pile Design

The ultimate bearing capacity of a pile consists of two main factors: point bearing capacity from the point of contact at the end of the pile, and frictional resistance from the soil surrounding the pile along its length. The main reason for the pile foundation is to reduce the settlement of the raft foundation from the clay layer as it has been said earlier in the chapter.

$$Qu = Qp + Qs \ and \ Qall = \frac{Qu}{4}$$

Where:

$Qu: ultimate\ bearing\ capacity$

$Qp: point\ bearing\ capacity = Ap * q_p = \frac{\pi d^2}{4} \times 0.08 R_c \quad using\ a\ diameter\ of\ 0.6m$

Rc $=$ is the compressive strength of the rock layer $=$ 11 MN (from soil report)

$Qs: frictional\ resistance \ = p\ l\ f \quad , \ f = \alpha\ Cu \qquad \therefore Qs = Cu * \alpha * p * l$

$Where:\ p = perimeter\ of\ pile\ , \quad l = length\ of\ pile\ , \ Cu = soil\ cohesion$

$\alpha: imperical\ adhesoin\ factor = 1 \quad (from\ fig\ 5.5-1)$

$$Qp = \frac{\pi(0.6)^2}{4} \times 0.08 \times 11000 = 248.8KN$$

$Q_s = 30(1)(\pi \times 0.6)(26.5) = 1498.5$ kPa [for one pile]

$$\therefore Qu = 248.8 + 1244 = 1747\ KN \rightarrow Qall = \frac{1747}{4}$$

$$= 436.8\ KN$$

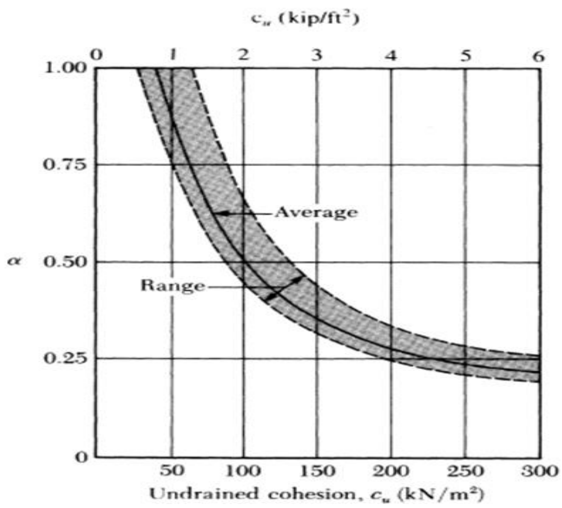


Figure 5.5-1 Adhesion Factor

Number of piles = 62 piles (one under each column)

6 CHAPTER 6: SOFTWARE BASED DESIGN

6.1 AutoCAD:

Auto-CAD is Automatic Computer-Aided Drafting software that has been developed by Autodesk that enables drafters, architects, engineers, and other professionals to create two-dimensional (2D) and three-dimensional (3D) models of mesh and solid surfaces. AutoCAD is one of the recommended design software applications because it can minimize the human errors and can be used to bring engineering ideas to life with the accuracy they require. So in this stead, AutoCAD is used by professionals across many industries to do everything from designing and building. It's serves as software for designing the plan of roadways, multi-story structure…, etc.

In our project, Auto-CAD provided us with storage and accessibility, 3D view, revisions and modifications, speed, and accuracy.

It has been used in redrawing the gridlines besides modifying the layout with new dimensions. The layout modified by duplicating parking spaces with new dimension that follows (MOMRA) code. Furthermore, it has been used in drawing ramps, columns, elevators including its location and dimensions for each floor.

6.2 ETABS:

Stand for, Extended Three-dimensional Analysis of Building Systems. It's being used as an engineering software application for multi- Story building structural analysis as well as structural design.

Load application based on various codes, modeling tools and templates, various analysis system and solution strategies; all manages with the grid like geometry unique to this type of structure. Design of steel and concrete frames (with automated optimization), composite beams, composite columns, steel joists, and shear walls is included, as is the capacity check for steel connections and base plates. Models may be realistically rendered, and all results can be shown directly on the structure. Comprehensive and customizable reports are available for all analysis and design output, and schematic construction drawings of framing plans, schedules, details, and cross-sections may be generated for concrete and steel structure.

After finishing with the preliminary design with the initial dimension for columns slab thickness shear wall etc. we draw the actual structure with all its members.

ETABS was used for:

1. Drawing the grid line.
2. Define the dimension for the members.
3. Assign the members (Column. Slabs, ramp, shear wall).
4. Define load combination (seismic, wind) loads.
5. Run analysis to make sure that all members are passes.
6. Redesign for any failure in the members.
7. Obtain the area of steal required.
8. Check the (deflection, lateral deflection).
9. Obtain design moment for slab reinforcement.
10. Column reaction for geotechnical design.
11. Moment and shear diagram for mat foundation design.
12. 3D modeling for the structure.

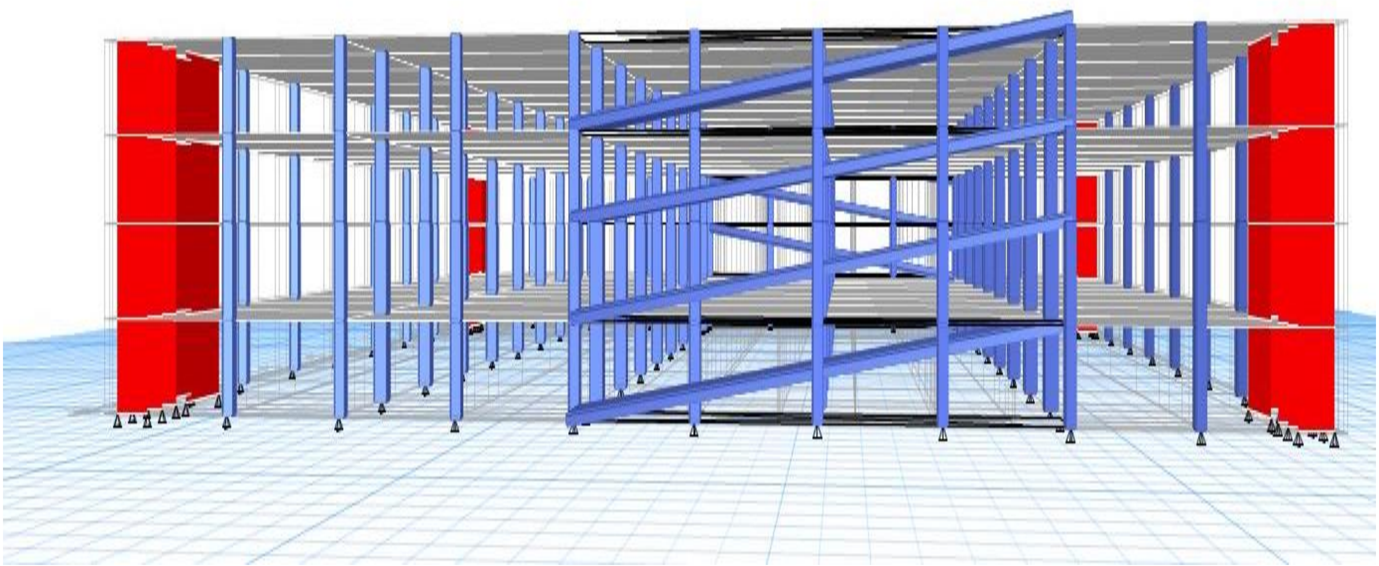


Figure 6.2-1 3D Side View

6.3 SAFE

It is the ultimate tool for designing concrete floor and foundation systems. From framing layout all the way through to detail drawing production, SAFE integrates every aspect of the engineering design process in one easy and intuitive environment. SAFE provides unmatched benefits to the engineer with its truly unique combination of power, comprehensive capabilities, and ease-of-use.

Laying out models is quick and efficient with the sophisticated drawing tools, or uses one of the import options to bring in data from CAD, spreadsheet, or database programs. Slabs or foundations can be of any shape, and can include edges shaped with circular and spline curves.

Post-tensioning may be included in both slabs and beams to balance a percentage of the self-weight. Suspended slabs can include flat, two-way, waffle, and ribbed framing systems. Models can have columns, braces, walls, and ramps connected from the floors above and below. Walls can be modeled as either straight or curved.

Safe was used in the project for:

1. Comparison between hand calculations with the software.
2. Obtaining the moment diagram and shear reinforcement.

7 CHAPTER 7: COST ESTIMATION

1. Find the volume of concrete for center columns

	Center Columns			
Floors	Number of Columns	Area of Columns (m^2)	Height of Columns (m)	Volume of column (m^3)
G	32	0.81	3.5	82.944
1	32	0.81	3.5	82.944
2	32	0.5625	3.5	57.6
3	32	0.5625	3.5	57.6
			Total	281.088

2. Find the volume of concrete for edge columns

	Edge Columns			
Floors	Number of Columns	Area of Columns (m^2)	Height of Columns (m)	Volume of column (m^3)
G	28	0.4225	3.5	37.856
1	28	0.4225	3.5	37.856
2	28	0.36	3.5	32.256
3	28	0.36	3.5	32.256
			Total	140.224

3. Find the volume of concrete for shear wall

	Shear Wall			
Floors	Number of Shear Wall	Area of Shear Wall (m^2)	Height of Shear Wall (m)	Volume of Shear Wall (m^3)
G	4	3.705	3.5	47.424
1	4	3.705	3.5	47.424
2	4	3.705	3.5	47.424
3	4	3.705	3.5	47.424
			Total	189.696

4. Find the volume of concrete for slab

Slab					
Floors	Number of Columns	Area of Slab (m^2)	Height of Slab (m)		Volume of Slab (m^3)
G	1	7560	0.55		4158
1	1	7560	0.55		4158
2	1	7560	0.55		4158
3	1	7560	0.55		4158
				Total	16632

5. Find the volume of concrete for foundation

Foundation				
Number	Area (m^2)	Height (m)		Volume (m^3)
1	7560	1.5		11340
			Total	11340

6. Find the volume of concrete for piles

Piles				
Number of Piles	Area of Piles (m^2)	Height of Piles (m)		Volume of Piles (m^3)
62	0.2827	26.5		464.4761
			Total	464.4761

7. Take sum of all concrete volumes (Center Columns + Edge Columns + Shear Wall + Slab + Foundation + piles)

Total volume of concrete = 29047.48m^3

8. Find the weight of reinforcement

Using 2% of volume of concrete to determine weight of Reinforcement

Reinforcement volume = Total volume of concrete * 2%= 581 m^3

Reinforcement weight = 581 * 8050 = 4677.05 tons

28Mpa concrete type v price is 199.5 SR per m^3. (AL KIFAH Company)

Total volume cost per cubic meter

Cost of concrete = Total volume of concrete * price per cubic meter

Cost of concrete = 29048$m^3 * 199.5\ {SR}/{m^3}$ = 5,795,076 SR

Cost of Steel is 2200 SR per ton. (STEEL BROTHERS HOLDING Company)

Total weight cost per unit ton

Cost of reinforcement = Weight of steel * price per ton

Cost of reinforcement = 4677.05 Tons * 2300 ${SR}/{ton}$= 10,757,215 SR

Cost of Finishing

(AWALLI ALBROUJ Company)

Type	Finishing %
Villas	80%
Malls	70%
Schools	60%
Houses	55%
Parking's	15%

Cost of finishing = 15% * Total cost

Cost of finishing = 15% * (10,757,215 +5,795,076) = 2,482,843.65 SR

Final Estimation

These costs only include the material cost.

Total Cost of the building = cost of finishing + cost of reinforcement + cost of concrete

Cost = 2,482,843.65 + 10,757,215 + 5,795,076 **= 19,035,134.35 SR**

Cost of Concrete	5,795,076 SR
Cost of Reinforcement	10,757,215 SR
Cost of Finishing	2,482,843.65 SR
Total cost	**19,035,134.35 SR**

8 CHAPTER 8: CONCLUSION

There is a significant shortage in car parks inside Prince Mohammed Bin Fahd University (PMU). Peak period from 8:00 am to 3:00 pm is the most significant time where students are seen roaming the parking lots looking for space. Other students who decide to park in front of the male gate block the road and reduce the number of lanes from three to one. With the increasing number of students in the university and plans to open new majors within the campus, his problem will propagate further. The main purpose for this project was to combat this problem and provide safe and accessible parking areas.

The structure is reinforced concrete structure that consists of five floors, with the top floor being open. Each floor has an area of 84x90 m^2 and has a capacity of 214 car parks. The main elements are the slab and columns, where edge and central columns differ in size due to the different loads on them. The structure was first designed using hand calculations, which were then reexamined using software, mainly ETABS.

The main problem that we encountered was due to the relatively large spans between columns (max 17.7m), this lead to a thick slab (0.55m) and later on forced us to increase the sizes of the columns due to them failing through shear. The deflection was also a factor that needed to be put into consideration due to the spans.

The project was successfully designed and approximate cost estimation for the structure was calculated at 19 million Saudi Riyals.

As a recommendation for future development and improvement, a pathway to the university from the parking lot should be designed to facilitate the safe movement of students and faculty to and from the parking lot. This can take the form a bridge, underground passage, or regular walkway. Also, a secondary entrance and exit to the parking lot that leads directly to the third floor may be designed to ease the flow of traffic into the parking lot.

9 References:

Journal

1. Irimia R, Gottschling M (2016) Taxonomic revision of Rochefortia Sw. (Ehretiaceae, Boraginales). Biodiversity Data Journal 4: E7720. https://doi.org/10.3897/BDJ.4.e7720. (n.d.). Figure 2f from: doi:10.3897/bdj.4.e7720.figure2f

2. Duff, J. T., & Beasley, L. T. (1962). Informal Discussion. Multi-Storey Car Parking Garages. Proceedings of the Institution of Civil Engineers, 21(3). doi:10.1680/iicep.1962.05070

3. Yurtoğlu, N. (2018). History Studies International Journal of History, 10(7), 241-264. doi:10.9737/hist.2018.658Http://www.historystudies.net/dergi//birinci-dunya-savasinda-bir-asayis-sorunu-sebinkarahisar-ermeni-isyani20181092a4a8f.pdf.

4. V. A. (2014). Performance Evaluation Of A Multi Storey Car Parking Structure Under Strength Irregularities By Push Over Analysis. International Journal of Research in Engineering and Technology, 03(15), 923-928. doi:10.15623/ijret.2014.0315174

5. Sardar, S. (2015). Types of foundation in building construction and civil engineering. Retrieved from https://unatnasce.wordpress.com/2015/10/14/types-of-foundation-in-building-construction-and-civil-engineering/

6. "Neubau Parkhaus SMA / HHS Planer + Architekten AG" 24 Mar 2014. Arch Daily. Accessed 22 Jan 2019. <https://www.archdaily.com/489026/neubau-parkhaus-sma-hhs-planer-architekten-ag/> ISSN 0719-8884

7. Ministry of Municipality and Rural Affairs, 2017. Technical guidelines for car parks. https://www.momra.gov.sa/GeneralServ/Specs/spec0027.asp

8. Wight, J. K. (2016). *Reinforced concrete: Mechanics and design*. Boston: Pearson.

9. (n.d.). Retrieved from http://www.mme.gov.qa/cui/index.dox?siteID=2

10. Rahul R., (2016). *What are one way shear and two way punching shear in footing?* Retrieved Feb 23, 2019. From https://www.quora.com/What-are-one-way-shear-and-two-way-punching-shear-in-footing.
11. Shakri K., Khairuddin N. (). *Multi Storey Car Parking.* Retrieved March 20, 2019, From https://www.sefindia.org/forum/files/2194924_8_multi_storey_car_parking_159.pdf

Books or Website

1. Saudi Building Code (SBC 304).

2. Saudi Building Code (SBC 301).

3. MOMRA

4. Hassoun, N., & Al-Manaseer, A. (2015). *Structural concrete: Theory and Design*. Hoboken (New Jersey): Wiley.

Companies:

1. Awalli Albrouj Company
2. Al Kifah Company
3. Al Atiah Company

- Foundation analysis and design, lecture slides, Dr. Ayadat 2019 (chapter 8 group piles).
- Introduction to concrete analysis and design, lecture slides,

10 APPENDIX I SBC 301

TABLE 1.4-1: DEFLECTION LIMITS[a, b, c, g]

Construction	L	W[e]	D+L[f]
Roof members[d] Supporting plaster ceiling Supporting nonplaster ceiling Not supporting ceiling	 l/360 l/240 l/180	 l/360 l/240 l/180	 l/240 l/180 l/120
Floor members	l/360		l/240
Exterior walls and interior partitions: With brittle finishes With flexible finishes		 l/240 l/120	
Farm buildings			l/180
Greenhouses			l/120

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed l/60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed l/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed l/90. For roofs, this exception only applies when the metal sheets have no roof covering.
b. Interior partitions not exceeding 1.8 m in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 4.11.
c. See SBC 201 for glass supports.
d. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Chapter 8 for rain and ponding requirements and SBC 201 for roof drainage requirements.
e. The wind load is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.
f. For steel structural members, the dead load shall be taken as zero.
g. For cantilever members, l shall be taken as twice the length of the cantilever.

1.4.3.2 **Reinforced Concrete.** The deflection of reinforced concrete structural members shall not exceed that permitted by SBC 304.

SECTION 1.5

GENERAL STRUCTURAL INTEGRITY

1.5.0 Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. This shall be achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. This shall be accomplished by providing sufficient continuity, redundancy, or energy dissipating capacity (ductility), or a combination thereof, in the members of the structure.

TABLE 1.6-1:

CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND AND EARTHQUAKE LOADS

Nature of Occupancy	Category
1) Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: a) Agricultural facilities b) Certain temporary facilities c) Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
1) Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: a) Buildings and other structures where more than 300 people congregate in one area b) Buildings and other structures with day care facilities with capacity greater than 150 c) Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250 d) Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Jails and detention facilities g) Power generating stations and other public utility facilities not included in Category IV 2) Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. 3) Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the hazardous material does not pose a threat to the public.	III

1) Buildings and other structures designated as essential facilities including, but not limited to: a) Hospitals and other health care facilities having surgery or emergency treatment facilities b) Fire, rescue, ambulance, and police stations and emergency vehicle garages c) Designated earthquake, hurricane, or other emergency shelters d) Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response e) Power generating stations and other public utility facilities required in an emergency f) Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency g) Aviation control towers, air traffic control centers, and emergency aircraft hangars h) Water storage facilities and pump structures required to maintain water pressure for fire suppression i) Buildings and other structures having critical national defense functions 2) Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. 3) Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

SECTION 2.2

SYMBOLS AND NOTATIONS

D = dead load;

E = earthquake load;

F = load due to fluids with well-defined pressures and maximum heights;

Fa = flood load;

H = load due to lateral earth pressure, ground water pressure, or pressure of

bulk materials;

L = live load;

Lr = roof live load;

P = ponding load;

R = rain load;

T = self-straining force;

W = wind load;

SECTION 2.3

COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 Applicability. The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1.4 (D + F) **(Eq. 2.3.2-1)**

1.2 (D + F + T) + 1.6 (L + H) + 0.5 (Lr or R) **(Eq. 2.3.2-2)**

1.2 D + 1.6 (Lr or R) + (f1L or 0.8 W) **(Eq. 2.3.2-3)**

1.2D + 1.6W + f1L + 0.5 (Lr or R) **(Eq. 2.3.2-4)**

1.2D + 1.0 E + f1L **(Eq. 2.3.2-5)**

0.9D + 1.6W + 1.6H **(Eq. 2.3.2-6)**

0.9D + 1.0E + 1.6H **(Eq. 2.3.2-7)**

where

f1 = 1.0 for areas occupied as places of public assembly, for live loads in excess of 5.0 kN/m2, and for parking garage live load.

f1 = 0.5 for other live loads.

DEAD LOADS

SECTION 3.1

DEFINITION

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.

SECTION 3.2

WEIGHTS OF MATERIALS AND CONSTRUCTIONS

In determining dead loads for purposes of design, the actual weights of materials and constructions shall be used provided that in the absence of definite information, values approved by the authority having jurisdiction shall be used. The minimum design dead loads are shown in Table 3-1 and Table 3-1(a) and the minimum densities for design loads from materials are shown in Table 3-2.

SECTION 3.3

WEIGHT OF FIXED SERVICE EQUIPMENT

In determining dead loads for purposes of design, the weight of fixed service equipment such as plumbing stacks and risers, electrical feeders, heating, ventilating and air conditioning systems (HVAC) and fire sprinkler systems shall be included.

LIVE LOADS

SECTION 4.1

DEFINITION

Live loads are those loads produced by the use and occupancy of the building or other structure and do not include construction or environmental loads such as wind load, rain load, earthquake load, flood load, or dead load. Live loads on a roof are those produced (1) during maintenance by workers, equipment, and materials, and (2) during the life of the structure by movable objects such as planters and by people.

TABLE 4-1:
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS

Occupancy or Use	Uniform kN/m^2	Conc. kN
Apartments (see residential)		
Access floor systems Office use Computer use	 2.5 5	 9 9
Armories and drill rooms	7.5	
Assembly areas and theaters • Fixed seats (fastened to floor) • Lobbies • Movable seats • Platforms (assembly) • Stage floors	 3 5 5 5 7.5	
Balconies (exterior) On one- and two-family residences only, and not exceeding 10 m^2	5 3	
Bowling alleys, poolrooms, and similar recreational areas	4	
Catwalks for maintenance access	2	1.5
Corridors First floor Other floors, same as occupancy served except as indicated	 5	
Mosques	5	
Decks (patio and roof) Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	5	
Dwellings (see residential)		
Elevator machine room grating (on area of 2500 mm^2)		1.5
Finish light floor plate construction (on area of 650 mm^2)		1
Fire escapes	5	
Fixed ladders	See Section 4.4	
Garages (passenger vehicles only) Trucks and buses	2 Note (2)	Note (1) Note (2)
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	5 Note (4)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals • Operating rooms, laboratories • Private rooms • Wards • Corridors above first floor	 3 2 2 4	 4.5 4.5 4.5 4.5
Hotels (see residential)		
Libraries • Reading rooms • Stack rooms • Corridors above first floor	 3 7.5 Note (3) 4	 4.5 4.5 4.5
Manufacturing • Light • Heavy	 6 12	 9 13.5
Marquees and canopies	4	
Office buildings • File and computer rooms shall be designed for heavier loads based on anticipated occupancy: • Lobbies and first floor corridors • Offices • Corridors above first floor	 5 2.5 4.0	 9 9 9
Penal institutions Cell blocks Corridors	 2 5	

Occupancy or Use	Uniform kN/m^2	Conc. kN
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	0.5	
Uninhabitable attics with storage	1.0	
Habitable attics and sleeping areas	1.5	
All other areas except stairs and balconies	2.0	
Hotels and multifamily houses		
Private rooms and corridors serving them	2.0	
Public rooms and corridors serving them	5.0	
Reviewing stands, grandstands, and bleachers	5.0 Note (4)	
Roofs	See Sections 4.3 and 4.9	
Schools		
Classrooms	3	4.5
Corridors above first floor	4	4.5
First floor corridors	5	4.5
Scuttles, skylight ribs, and accessible ceilings		10
Sidewalks, vehicular driveways, and yards subject to trucking	12 Note (5)	36 Note (6)
Stadiums and arenas		
Bleachers	5 Note (4)	
Fixed Seats (fastened to floor)	3 Note (4)	
Stairs and exit-ways	5	Note (7)
One- and two-family residences only	2	
Storage areas above ceilings	1	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	6	
Heavy	12	
Stores		
Retail		
First floor	5	4.5
Upper floors	4	4.5
Wholesale, all floors	6	4.5
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit-ways)	3	
Yards and terraces, pedestrians	5	

Notes

(1) Floors in garages or portions of building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 13.0 kN acting on an area of 100 mm by 100 mm, footprint of a jack; (2) for mechanical parking structures without slab or deck which are used for storing passenger car only, 10 kN per wheel.

(2) Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

(3) The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations:
   a. The nominal bookstack unit height shall not exceed 2300 mm;
   b. The nominal shelf depth shall not exceed 300 mm for each face; and
   c. Parallel rows of double-faced bookstacks shall be separated by aisles not less than 900 mm wide.

(4) In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 0.4 kN/linear m of seat applied in a direction parallel to each row of seats and 0.15 kN/linear m of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

(5) Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

(6) The concentrated wheel load shall be applied on an area 100 mm by 100 mm, footprint of a jack.

(7) Minimum concentrated load on stair treads on area of 2500 mm^2 is 1.5 kN

SECTION 4.2

UNIFORMLY DISTRIBUTED LOADS

4.2.1 Required Live Loads. The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall in no case be less than the minimum uniformly distributed unit loads required by Table 4-1 and Table 4-2.

WIND LOADS

SECTION 6.1

GENERAL

6.1.1 Scope. Buildings and other structures, including the main wind force-resisting system and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein.

6.1.2 Allowed Procedures. The design wind loads for buildings and other structures, including the main wind force-resisting system and component and cladding elements thereof, shall be determined using one of the following procedures: (1) Method 1 – Simplified Procedure as specified in Section 7.1 for buildings meeting the requirements specified therein; (2) Method 2 – Analytical Procedure as specified in Section 7.2 for buildings meeting the requirements specified therein; (3) Method 3 – Wind Tunnel Procedure according to Section 7.3.

6.1.3 Wind Pressures Acting on Opposite Faces of Each Building Surface. In the calculation of design wind loads for the main wind force-resisting system and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

6.1.4 Minimum Design Wind Loading. The design wind load, determined by any one of the procedures specified in Section 6.1.2, shall be not less than specified in this Section.

6.1.4.1 Main Wind Force-Resisting System

The wind load to be used in the design of the main wind force-resisting system for an enclosed or partially enclosed building or other structure shall not be less than 0.5 kN/m2 multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 0.5 kN/m2 multiplied by the area Af, as defined in section 6.3.

SECTION 6.2

DEFINITIONS

The following definitions apply only to the provisions of Chapters 6 and 7:

Approved. Acceptable to the authority having jurisdiction.

Basic Wind Speed, V. 3-second gust speed at 10 m above the ground in Exposure C (see Section 6.4.2) as determined in accordance with Section 6.4.1.

Building, Enclosed. A building that does not comply with the requirements for open or partially enclosed buildings.

Building Envelope. Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

Building and Other Structure, Flexible. Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

Building, Low-Rise. Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h is less than or equal to 18 m; and

2. Mean roof height h does not exceed least horizontal dimension.

Building, Open. A building having each wall at least 80% open. This condition is expressed for each wall by the equation Ao > 0.8Ag where

Ao = total area of openings in a wall that receives positive external pressure, in m2

Ag = the gross area of that wall in which Ao is identified, in m2

Building, Partially Enclosed. A building that complies with the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%, and

2. The total area of openings in a wall that receives positive external pressure exceeds 0.4 m2 or 1% of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20%.

These conditions are expressed by the following equations:

1. Ao > 1.10 Aoi

2. Ao > 0.4 m2 or > 0.01 Ag, whichever is smaller, and Aoi /Agi □ 0.20

where

Ao, Ag are as defined for Open Building

Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao, in m2.

Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag, in m2.

Building or Other Structure, Regular Shaped. A building or other structure having no unusual geometrical irregularity in spatial form.

TABLE 10.12:
ALLOWABLE STORY DRIFT, Δ_a[a]

Structure	Occupancy Category		
	I, II	**III**	**IV**
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 10.12 is not waived.

[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

11 APPENDIX II SBC 304

SECTION 7.6

SPACING LIMITS FOR REINFORCEMENT

7.6.1 The minimum clear spacing between parallel bars in a layer shall be *b d* , but not less than 25 mm. See also Section 3.3.2.

7.6.2 Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 25 mm.

7.6.3 In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall be not less than *b* 1.5*d* nor less than 40 mm. See also Section 3.3.2.

7.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

7.6.5 In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than two times the wall or slab thickness, nor farther apart than 300 mm.

SECTION 7.7

CONCRETE PROTECTION FOR REINFORCEMENT

7.7.1 Cast-in-place concrete (nonprestressed). The following minimum concrete cover shall be provided for reinforcement, but shall not be less than required by Section 7.7.5 and 7.7.7:

Minimum

cover, mm

(a) Concrete cast against and permanently exposed to earth 75

(b) Concrete exposed to earth or weather:

Dia 20 mm bars and larger... 50

Dia 18 mm bar, WD 12.0 wire, and smaller 40

(c) Concrete not exposed to weather or in contact with ground:

Slabs, walls, joists:

Dia 40 mm bars and larger.. 40

Bars with diameters smaller than 40 mm 20

Beams, Columns:

Primary reinforcement, ties, stirrups, spirals 40

Shells, folded plate members:

Dia 20 mm bar and Larger... 20

Dia 18 mm WD 12.0 wire, and smaller..................................... 15

7.7.2 Cast-in-place concrete (prestressed). The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, but shall not be less than required by Section 7.7.5, 7.7.5.1, and 7.7.7:

Minimum

cover, mm

(a) Concrete cast against and permanently exposed to earth 75

(b) Concrete exposed to earth or weather:

Wall panels, slabs, joists.. 25

Other members 40

(c) Concrete not exposed to weather or in contact with ground:

Slabs, walls, joists... 40

Bars with diameters smaller than 40 mm 20

Beams, columns:

Primary reinforcement... 40

Ties, stirrups, spirals 25

Shells, folded plate members:

Dia 18 mm bar, WD 12.0 wire, and smaller 10

Other reinforcement .... *b d* but not less than 20

7.10.5 Ties. Tie reinforcement for compression members shall conform to the following:

7.10.5.1 All nonprestressed bars shall be enclosed by lateral ties, at least Dia 10 mm in size for longitudinal bars Dia 32 mm or smaller, and at least Dia 12 mm in size for Dia 32 mm bars and larger and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area shall be permitted.

7.10.5.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

7.10.5.3 Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 deg and no bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted.

7.10.5.4 Ties shall be located vertically not more than one-half a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than one-half a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

TABLE B.3 Grades of Reinforcing Bars and Metric Bar Sizes Available for Each

ASTM No.	Steel Grade (MPa)	Bar Sizes
A615M Billet	300	#10–#19
	420	#10–#57
	520	#19–#57
A616M Rail	350	#10–#36
	420	#10–#36
A617M Axle	300	#10–#36
	420	#10–#36
A706M Low-Alloy	420	#10–#57

TABLE B.4 Areas of Groups of Standard Metric Bars (mm^2)

Bar Designation	Number of Bars								
	2	3	4	5	6	7	8	9	10
#10	142	213	284	355	426	497	568	639	710
#13	258	387	516	645	774	903	1032	1161	1290
#16	398	597	796	995	1194	1393	1592	1791	1990
#19	568	852	1136	1420	1704	1988	2272	2556	2840
#22	774	1161	1548	1935	2322	2709	3096	3483	3870
#25	1020	1530	2040	2550	3060	3570	4080	4590	5100
#29	1290	1935	2580	3225	3870	4515	5160	5805	6450
#32	1638	2457	3276	4095	4914	5733	6552	7371	8190
#36	2012	3018	4024	5030	6036	7042	8048	9054	10 060
#43	2904	4356	5808	7260	8712	10 162	11 616	13 068	14 520
#57	5162	7743	10 324	12 905	15 486	18 067	20 648	23 229	25 810

Bar Designation	Number of Bars								
	11	12	13	14	15	16	17	18	19	20
#10	781	852	923	994	1065	1136	1207	1278	1349	1420
#13	1419	1548	1677	1806	1935	2064	2193	2322	2451	2580
#16	2189	2388	2587	2786	2985	3184	3383	3582	3781	3980
#19	3124	3408	3692	3976	4260	4544	4828	5112	5396	5680
#22	4257	4644	5031	5418	5805	6192	6579	6966	7353	7740
#25	5610	6120	6630	7140	7650	8160	8670	9180	9690	10 200
#29	7095	7740	8385	9030	9675	10 320	10 965	11 610	12 255	12 900
#32	9009	9828	10 647	11 466	12 285	13 104	13 913	14 742	15 561	16 380
#36	11 066	12 072	13 078	14 084	15 090	16 096	17 102	18 108	19 114	20 120
#43	15 972	17 424	18 876	20 328	21 780	23 232	24 684	26 136	27 588	29 040
#57	28 391	30 972	33 553	36 134	38 715	41 296	43 877	46 458	49 039	51 620

SECTION 9.1

GENERAL

9.1.1 Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in SBC 304.

9.1.2 Members also shall meet all other requirements of SBC 304 to ensure adequate performance at service load levels.

SECTION 9.2

REQUIRED STRENGTH

9.2.1 Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

$U = 1.4\,(D + F)$ (9-1)

$U = 1.4\,(D + F + T) + 1.7(L + H) + 0.5\,(Lr \text{ or } R)$ (9-2)

$U = 1.2D + 1.6\,(Lr \text{ or } R) + (1.0\,L \text{ or } 0.8\,W)$ (9-3)

$U = 1.2D + 1.6W + 1.0L + 0.5(Lr \text{ or } R)$ (9-4)

$U = 1.2D + 1.0E + 1.0L$ (9-5)

$U = 0.9D + 1.6W + 1.6H$ (9-6)

$U = 0.9D + 1.0E + 1.6H$ (9-7)

SECTION 9.5

CONTROL OF DEFLECTIONS

9.5.1 Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect strength or serviceability of a structure.

TABLE 9.5(b)
MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/180$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)**	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

** Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

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

Yield strength f_y MPa*	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
300	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
520	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

* For values of reinforcement yield strength between the values given in the table, minimum thickness shall be determined by linear interpolation.

† Drop panel is defined in 13.3.7.1 and 13.3.7.2

‡ Slabs with beams between columns along exterior edges. The value of α for the edge beam shall not be less than 0.8.

SECTION 13.3

SLAB REINFORCEMENT

13.3.1 Area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections, but shall not be less than required by 7.12.

13.3.2 Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area of cellular or ribbed construction.

In the slab over cellular spaces, reinforcement shall be providedas required by 7.12.

13.3.3 Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

13.3.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Chapter 12.

13.3.5 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

13.3.6 In slabs with beams between supports with a value of α greater than 1.0, special top and bottom slab reinforcement shall be provided at exterior corners in accordance with Sections 13.3.6.1 through 13.3.6.4.

13.3.6.1 The special reinforcement in both top and bottom of slab shall be sufficient to resist a moment per meter of width equal to the maximum positive moment in the slab.

13.3.6.2 The moment shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the comer in the bottom of the slab.

13.3.8 Details of reinforcement in slabs without beams

13.3.8.1 In addition to the other requirements of 13.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 13.3.8.

13.3.8.2 Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Fig. 13.3.8 shall be based on requirements of the longer span.

13.3.8.3 Bent bars shall be permitted only when depth-span ratio permits use of bends of 45 deg or less.

13.3.8.4 In frames where two-way slabs act as primary members resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Fig. 13.3.8.

13.3.8.5 All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class A tension splices or with mechanical or welded splices satisfying 12.14.3. Splices shall be located as shown in Fig: 13.3.8. At least two of the column strip bottom bars or wires in each direction shall pass within the column core and shall be anchored at exterior support.

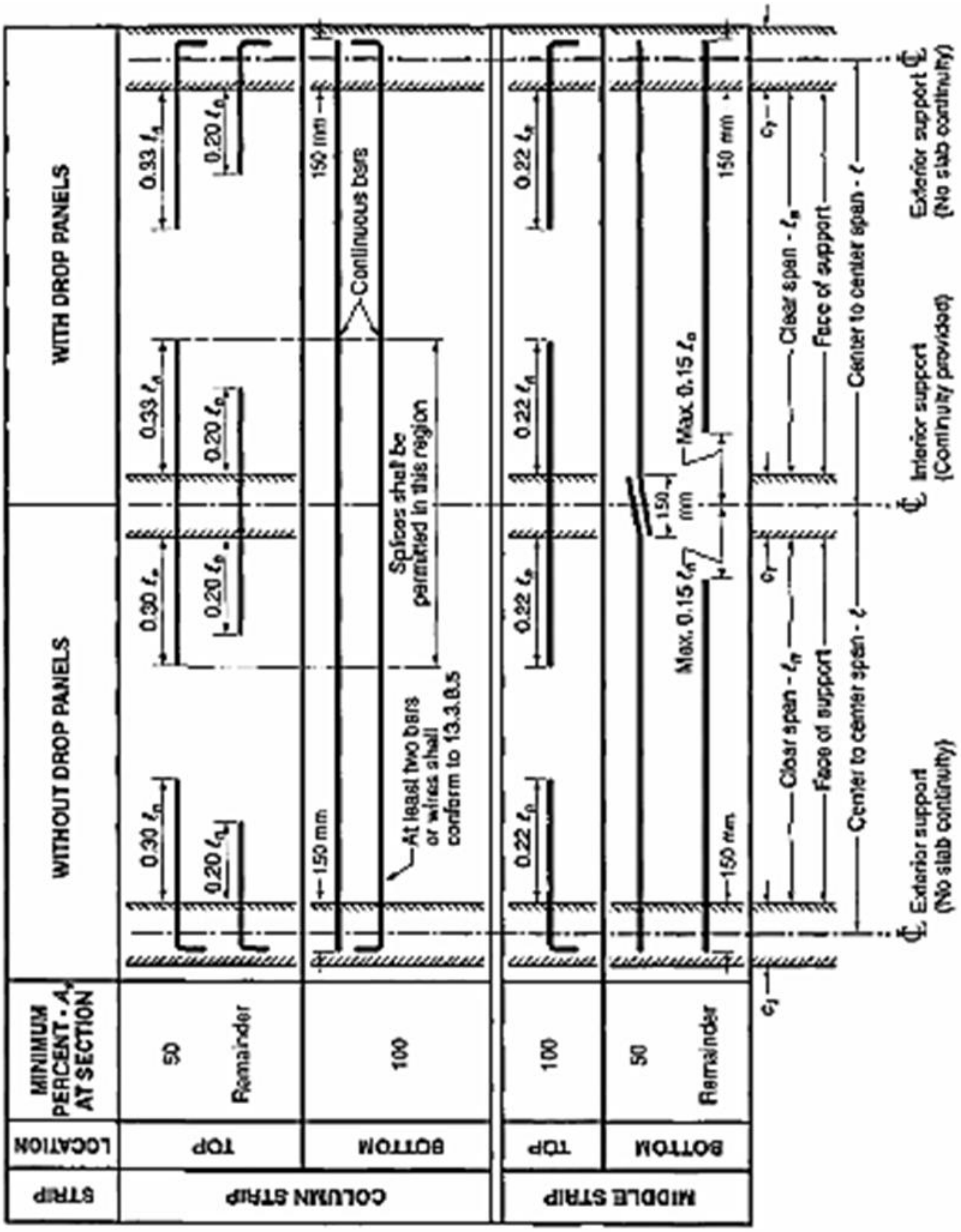


Fig. 13.3.8 – Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

12 APPENDIX III SBC 303

Geotechnical Design

Appendix 1.2.1 Spread Footings

- Spread footings shall be designed and constructed in accordance with Sections 5.1 through 5.6. Spread footings shall be designed and constructed in accordance with Sections 5.4.1 through 5.4.4.

- The minimum depth of footing below the natural ground level shall not be less than 1.2 m for cohesionless soils, 1.5 m for silty and clay soils and 600 mm to 1200 mm for rocks depending on strength and integrity of the rock formations.

- Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The minimum width of footings shall be 300 mm.

- Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings.

- Settlements shall be estimated based on methods of analysis approved by the building official. The least value found from Tables 5.1 and 5.2 shall be taken as the allowable differential settlement.

Table 5.1: Maximum Allowable Total Settlement

Footing Type	**Total Settlement (mm)**	
	Clay	**Sand**
Spread Footings	60	40
Mat Foundations	80	60

Table 5.2: Maximum Allowable Angular Torsion

Building Type	*L/H*	*ρ/l*
Multistory reinforced concrete structures founded on mat foundation	---	0.0015
Steel frame structure with side sway	---	0.008
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	---	0.002-0.003
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	≥ 5 ≤ 3	0.002 0.001
Slip and high structures as silos and water tanks founded on stiff mat foundations	---	0.002
Cylindrical steel tank with fixed cover and founded on flexible footing	---	0.008
Cylindrical steel tank with portable cover and founded on flexible footing	---	0.002-0.003
Rail for supporting hanged lift	---	0.003

L = Building length

l = Span between adjacent footings

H = Overall height of the structure

δ = Differential settlement

- Factor of safety shall not be less than 3 for permanent structures and 2 for temporary structures. Consideration shall be given to all possible circumstances including, but not limited to, flooding of foundation soil, removal of existing overburden by scour or excavation, and change in groundwater table level.

- The design, materials and construction of concrete footings shall comply with Sections 5.4.2.1 through 5.4.2.8 and the provisions of SBC 304 where applicable. Concrete in footings shall have a specified compressive strength (f'_c) of not less than 20 MPa at 28 days.

- Concrete and masonry foundation walls shall be designed in accordance with SBC 304 or SBC 305. Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 6.1 through 6.3 are permitted to be designed and constructed in accordance with Sections 6.2 through 6.6.

- Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 13.4.2 and 13.4.3. Computations of lateral earth pressures shall comply with the provisions of Sections 7.2.1 through 7.2.6. The values set forth in Tables 7-2 and 7-3 shall be used in computations that include effects of wall friction.

Table 7.2: Ultimate Friction factors For Dissimilar Materials

Interface Materials	Friction Factor, $\tan\delta^*$
Clean sound rock	0.7
Clean gravel, gravel-sand mixtures, coarse sand	0.55 - 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 – 0.55
Clean fine sand, silty or clayey fine to medium sand	0.35 – 0.45
Fine sandy silt, nonplastic silt	0.30 – 0.35
Very stiff and hard residual or preconsolidated clay	0.40 - 0.50
Medium stiff and stiff clay and silty clay	0.30 – 0.35

*Values for δ shall not exceed one-half the angle of internal friction of the backfill soils for steel and precast concrete

And two-third the angle of internal friction of the backfill soils for cast-in place concrete

- The allowable soil pressure shall be determined in accordance with the provisions of Chapter 4. The factor of safety with respect to bearing capacity shall not be less than 3.

Retaining Wall

- Retaining walls shall be designed to ensure stability against overturning, sliding, and stability of supporting ground. Stability analyses shall conform to the provisions of Sections 7.4.1 through 7.4.4.

- The retaining wall shall be proportioned so that the factor of safety against overturning is not less than 1.5. The value of angular distortion (settlement/length of structure) of retaining walls shall not exceed 0.002 radians.

- Where retaining walls are underlain by weak soils, the overall stability of the soil mass containing the retaining wall shall be checked with respect to the most critical surface of sliding. The stability analysis shall be made for after construction and for long-term conditions. The factor of safety for the overall stability of the soil mass containing the wall shall not be less than 2.

- Thickness of the upper part of the wall shall not be less than 300 mm, whereas thickness of the lower part of the wall shall be enough to resist shear without reinforcement. Depth of wall foundation shall be located below line of seasonal changes and shall be deep enough to provide adequate bearing capacity and soil sliding resistance. The wall foundation shall be proportioned such that the wall does not slide or overturn, the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The base and other dimensions shall be such that the resultant falls within the middle third of the base. Where additional front clearance is needed, it shall be permitted to construct counterfort retaining walls without a toe provided that the sliding and overturning stability requirements stated in Sections 7.4.1 and 7.4.2 are met.

Combined Footing and Mat Foundation

- Analysis and design of combined footings and mats shall conform to all requirements of ACI 336.2R Suggested Analysis and Design Procedures for Combined Footings and Mats except as modified by Chapter 8. All provisions of SBC 303 not specifically excluded, and not in conflict with the provisions of Chapter 8 shall apply to combined footings and mats, where applicable. Design of combined or mat foundations shall be based on the Strength Design Method of SBC 304.

- Combined footings and mats shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Section 4.8 SBC 301, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shallcomply with load combinations specified in SBC 304.

- Soil contact pressure acting on a combined footing or mat and the internal stresses produced by them shall be determined from one of the load combinations given in Section 2.4 SBC 301**,** whichever produces the maximum value for the element under investigation.

- The maximum unfactored design contact pressures shall not exceed the allowable soil pressure as obtained from Chapter 4 or cause settlements that exceed the values set forth in Table 5.1 and 8.3. Where wind or earthquake forces form a part of the load combination, the allowable soil pressure may be increased as allowed by the Saudi Building Code or approved by the building official.

- Contact pressures at the base of combined footings and mats shall be determined in accordance with Sections 8.4.1.1 through 8.4.1.3.

- Settlements of combined footings and mats shall conform to the provisions of Sections 8.5.1 through 8.5.3. Total settlement of combined footings and mats shall not exceed the value set forth in Table 5.1. Differential settlements for combined footings shall not exceed the values set forth in Table 5.2. For mats the differential settlement shall be taken as three-fourths the total settlement if this is not more than 50 mm or determined based on relative stiffness, $r\ k$, as shown in Table 8.3.

- Combined footings shall be designed and constructed in accordance with Sections 8.6.1 through 8.6.3.

Table 8.3: Maximum Allowable Differentials Settlement of Mats

k_r	**Shape**	**Differential Settlement (mm)**
0	Rectangular Base	$0.5\times\Delta H^{*}$
	Square Base	$0.35\times\Delta H$
0.5		$0.1\times\Delta H$
>0.5	Rigid mat: no differential settlement	

*ΔH = Total settlement estimated based on approved methods of analysis but shall not exceed values in Table 5.1.

- Continuous foundations shall be designed and constructed in accordance with Sections 8.7.1 through 8.7.3.

- Mats shall be designed and constructed in accordance with Sections 8.9.1 through 8.9.3. Mats may be designed and analyzed as either rigid bodies or as flexible plates supported by an elastic foundation (the soil).

- A mat may be designed using the Strength Design Method of SBC 304. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. The mat plan shall be proportioned using unfactored loads and any overturning moments.

- The pressure diagram is considered linear and computed from Equation 8-2 and SBC 303 2007 8/11 shall be less than allowable load bearing. Loads shall include the effect of any column moments and any overturning moment due to wind or other effects. Anymoments applied to the mat from columns or overturning, etc., shall be included when computing the eccentricity.

- The contact pressure shall not exceed the allowable load bearing determined from Chapter 4. The allowable soil pressure may be furnished as one or more values depending on long-term loading or including transient loads such as wind. The soil pressure furnished by the geotechnical engineer shall be factored to a pseudo "ultimate" value by multiplying the allowable pressure by the ratio of the sum of factored design loads to the sum of the unfactored design loads.

- The minimum mat thickness based on punching shear at critical columns shall be computed based on column load and shear perimeter. The depth of the mat shall be found without using shear reinforcement and determined on the basis of diagonal-tension shear as noted in SBC 304 Chapter 15.

Underground Water-Retention Structures

- Walls or portions thereof that retain earth and enclose interior spaces and floors below grade, and underground water-retention structures shall be waterproofed and damp proofed in accordance with provisions of this Chapter, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy. Ventilation for crawl spaces shall comply with Section 7.3.4 SBC 201.

- Underground water-retention structures shall meet the provisions of Sections 13.5.1 through 13.5.4.

- For design and constructions of underground water-retention structures, provisions of SBC 304 and ACI 350 Environmental Engineering Concrete Structures shall govern, where applicable.

Deep Foundations (Piles)

- Piles are permitted to be designed in accordance with provisions for piers in Chapter 14 and Sections 17.3 through 17.10 where either of the following conditions exists, subject to the approval of the building official:

- Group R-3 and U occupancies not exceeding two stories of light-frame construction, or
- Where the surrounding foundation materials furnish adequate lateral support for the pile.

- The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.

- The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 360 kN. For allowable loads above 360 kN, the wave equation method of analysis shall be used to estimate pile drivability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 14.8.3.

- Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 14.8.3 divided by a factor of safety of two.

- Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers.

- Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

- Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load.

- The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 25 mm at the ground surface.

- Allowable stresses greater than those specified for piers or for each pile type in Chapters 14 and 15 are permitted where supporting data justifying such higher stresses is filed with the building official.

- The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

- The materials, reinforcement and installation of precast concrete piles shall conform to Sections 15.1.1.1 through 15.1.1.4.

- Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

- The minimum lateral dimension shall be 200 mm. Corners of square piles shall be chamfered.

- Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 100 mm apart, center to center, for a distance of 600 mm from the ends of the pile; and not more than 150 mm elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 25 mm center to center.

- Precast nonprestressed concrete piles shall conform to Sections 15.1.2.1 through 15.1.2.5. Concrete shall have a 28-day specified compressive strength ($'fc$) of not less than 20 MPa.

- The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.

- The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (fc) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel ($f y$) or a maximum of 210 MPa. The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel ($f y$) or a maximum of 165 MPa.

- Precast prestressed concrete piles shall conform to the requirements of Sections 15.1.3.1 through 15.1.3.5.

- Prestressing steel shall conform to ASTM A416. Concrete shall have a 28-day specified compressive strength (fc) of not less than 35 MPa.

- Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 3 MPa for piles up to 9 meters in length, 4 MPa for piles up to 15 meters in length and 5 MPa for piles greater than 15 m in length. Effective prestress shall be based on an assumed loss of 210 MPa in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in SBC 304.

- Structural steel piles shall conform to the requirements of Sections 15.2.2 through 15.2.5.

- Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A36, ASTM A252, ASTM A283, ASTM A572, ASTM A588 or ASTM A913.

- The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength ($f y$).

- Sections of H-piles shall comply with the following:

- The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
- The nominal depth in the direction of the web shall not be less than 200 mm.
- Flanges and web shall have a minimum nominal thickness of 10 mm.

- Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 200 mm. The pipe shall have a minimum of 220 mm2 of steel in cross section to resist each 1360 N-m of pile hammer energy or the equivalent strength for steels having a yield strength greater than 240 MPa. Where pipe wall thickness less than 5 mm is driven open ended, a suitable cutting shoe shall be provided.

- The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 16.1.1 through 16.1.3.

- Concrete shall have a 28-day specified compressive strength (' fc) of not less than 20 MPa. Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 100 mm and not more than 150 mm. Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

- Except for steel dowels embedded 1.5 m or less in the pile and as provided in Section 16.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state.

- Enlarged base piles shall conform to the requirements of Sections 16.2.1 through 16.2.5.

- The maximum size for coarse aggregate for concrete shall be 20 mm. Concrete to be compacted shall have a zero slump.

- The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25% of the 28-day specified compressive strength (*fc*). Where the concrete is placed in a permanent steel casing, the maximum allowable concrete stress shall be 33% of the 28-day specified compressive strength (*fc*).

- Drilled or augered uncased piles shall conform to Sections 16.3.1 through 16.3.5.

- The allowable design stress in the concrete of drilled uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (' *fc*). The allowable design stress in the concrete of augered cast-in-place piles shall not exceed 25 percent of the 28-day specified compressive strength (*fc*). The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or (175 MPa).

- The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 mm.

- Driven uncased piles shall conform to Sections 16.4.1 through 16.4.4.

- The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (' *fc*) applied to a cross sectional area not greater than the inside area of the drive casing or mandrel.

- The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 mm.

- Steel-cased piles shall comply with the requirements of Sections 16.5.1 through 16.5.4.

- Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 200 mm.

- The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength ($'fc$). The allowable concrete compressive stress shall be 0.40 ($'fc$) for that portion of the pile meeting the conditions specified in Sections 16.5.2.1 through 16.5.2.4.

- The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage 1.75 mm minimum.

- Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 16.6.1 through 16.6.5.

- Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 16.1.1. The maximum coarse aggregate size shall be 20 mm.

- The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (fc). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (fy), provided fy shall not be assumed greater than 250 MPa for computational purposes.

- Caisson piles shall conform to the requirements of Sections 16.7.1 through 16.7.6.

- Composite piles shall conform to the requirements of Sections 16.8.2 through 16.8.5.

- Isolated and multiple piers used as foundations shall conform to the requirements of Sections 17.2 through 17.10, as well as the applicable proves.

Slope Stability

- The minimum acceptable static factor of safety for cut, fill, and natural slopes is 1.3 in the absence of earthquake.

- The minimum dynamic factor of safety for cut, fill, and natural slopes is 1.1 for the case of an earthquake.

- Safety factor is defined as the quotient of the sum of forces tending to resist failure divided by the sum of forces tending to cause failure. 1.

- New buildings and additions to buildings may be constructed on or adjacent to a cut, fill, or natural slope provided that:

- The slopes have an evaluated safety factor of at least 1.5 against deep-seated static failure.
- The slopes ascending above proposed structures have an evaluated safety factor of at least 1.5 against surficial failure or adequately designed protective devices are recommended that will protect the construction from the hazard of mud and debris flow.

- Minor additions or alterations may be made to existing structures where acceptable devices are provided to mitigate potential damage from failure of adjacent slopes and where the hazard to life or property is not increased.

13 APPENDIX IV EXCEL

Pile Design:

Diameter (m)	Ap (m^2)	Rc (KN)	qp (Kpa)	Qp (KN)	Cu	α	P (m)	L (m)
0.3	0.0707	11000	880	62.20	30	1	0.942	26.5
0.4	0.1257	11000	880	110.58	30	1	1.257	26.5
0.5	0.1963	11000	880	172.79	30	1	1.571	26.5
0.6	0.2827	11000	880	248.81	30	1	1.885	26.5

Qs (KN)	Qu (KN)	Qall (KN)
749.27	811.47	202.87
999.03	1109.61	277.40
1248.78	1421.57	355.39
1498.54	1747.35	582.45

Column Loads: (KN)

C-1	4098.48	C-32	1607.27
C-2	5542.96	C-33	1598.7
C-3	5423.61	C-34	11870
C-4	5419.76	C-35	13167
C-5	5523.8	C-36	12758.08
C-6	4500.68	C-37	12714.32
C-7	2376.36	C-38	12709
C-8	2354.38	C-39	12714.21
C-9	2109	C-40	13154.34
C-10	10727.98	C-41	12178.97
C-11	13405.98	C-42	1598.82
C-12	13807.01	C-43	1609.5
C-13	13751.93	C-44	1636.86
C-14	13745.28	C-45	1952.25
C-15	13774.53	C-46	10928
C-16	13838.06	C-47	13377.46
C-17	11108.4	C-48	13956
C-18	2064.3	C-49	13927.95
C-19	3564.61	C-50	13920.73
C-20	3549.63	C-51	13930.28
C-21	1852.67	C-52	13827.52
C-22	11401	C-53	11307.85
C-23	13003.14	C-54	1950.7
C-24	12804.94	C-55	2589.91
C-25	12748	C-56	2561.55
C-26	12742.8	C-57	4159.57
C-27	12757.64	C-58	5677.82
C-28	13374.44	C-59	5569.53
C-29	11781.24	C-60	5564.09
C-30	1794.78	C-61	5671.04
C-31	1612.73	C-62	4585.52

Shear Wall Loads: (KN)

W1-1	548.134	W2-1	1232.99	W3-1	149.6	W4-1	1178.42
W1-2	847.43	W2-2	2037.52	W3-2	218.22	W4-2	964.48
W1-3	1803.2	W2-3	1784.27	W3-3	286.67	W4-3	388.62
W1-4	2287.18	W2-4	2139.89	W3-4	2285.29	W4-4	529.11
W1-5	146.67	W2-5	1094.12	W3-5	2905.3	W4-5	987.59
W1-6	209.94	W2-6	923.4	W3-6	522.68	W4-6	1201.91
W1-7	276.79	W2-7	364.17	W3-7	827.772	W4-7	1991.27
W1-8	2217.98	W2-8	501.6	W3-8	1780.5	W4-8	1765.56
W1-9	2787.37	W2-9	949.98	W3-9	2315.73	W4-9	2160.6

Mat Foundation Design:

Qtotal	Ix	Iy	A	B	L
555947	5103000	4445280	7560	84	90

X-1	6261.36225
X-2	330213.303
X-3	1031415.97
X-4	1700808.44
X-5	2299251.94
X-6	2907010.28
X-7	3530776.78
X-8	4072104.04
X-9	3448089.8
X-10	1570567.52
Total	**20896499.4**

Y-1	10148.96
Y-2	43272.26
Y-3	1881608
Y-4	84730
Y-5	3650997
Y-6	141036
Y-7	5489323
Y-8	437525.8
Y-9	6654209
Y-10	373728.5
Y-11	2674139
Total	**21440718**

X'	37.5872207	Y'	38.56612
e_x	-4.4127793	e_y	-6.43388
My	-2453271	Mx	-3576893

Foundation Reinforcement:

Y strip	Total Soil Reaction (KN)	Total Column Load (KN)	Average Load (KN)	Modified Average Soil Pressure (KPa)	F	Soil pressure per meter (KPa)	B (m)	Max Positive Moment (KN.m)	Max Negative Moment (KN.m)
	95373.62	64135.34	79754.479	53.77	1.24	951.72	17.7	2092	-1183

X strip	Total Soil Reaction (KN)	Total Column Load (KN)	Average Load (KN)	Modified Average Soil Pressure (KPa)	F	Soil pressure per meter (KPa)	B (m)	Max Positive Moment (KN.m)	Max Negative Moment (KN.m)
	69140.525	109078.74	89109.633	105.04	0.817	997.87	9.5	922	-520

X-strip:

Column Loads (KN)	Modified Columns Loads (KN)
1952.25	1594.85
10928	8927.40
13377.46	10928.44
13956	11401.07
13927.95	11378.15
13920.73	11372.25
13930.28	11380.06
13827.52	11296.11
11307.85	9237.72
1950.7	1593.58

Y-strip:

Column Loads (KN)	Modified Columns Loads (KN)
5423.61	6744.44
13751.93	17100.99
12748	15852.57
12714.32	15810.69
13927.95	17319.88
5569.53	6925.90

Column Dimension:

	As	Ag	Slab thick	fc	fy	Area	Wu	P4	Ag	a	a'
Edge	3%	97%	0.55	28	420	84.075	19.04	1600.788	86264.6	293.708	300
Center	3%	97%	0.55	28	420	168.15	19.04	3201.576	172529	415.366	450

Ag'	P3	Ag	a	a'
0.0900	3209.143	172936.960	415.857	450
0.2025	6437.202	346893.329	588.977	600

Ag	P2	Ag	a	a'
0.2025	4813.378	259387.346	509.301	550
0.36	9660.571	520596.884	721.524	750

Ag	P1	Ag	a	a'
0.3025	6421.860	346066.532	588.274	600
0.5625	12915.350	695993.167	834.262	850

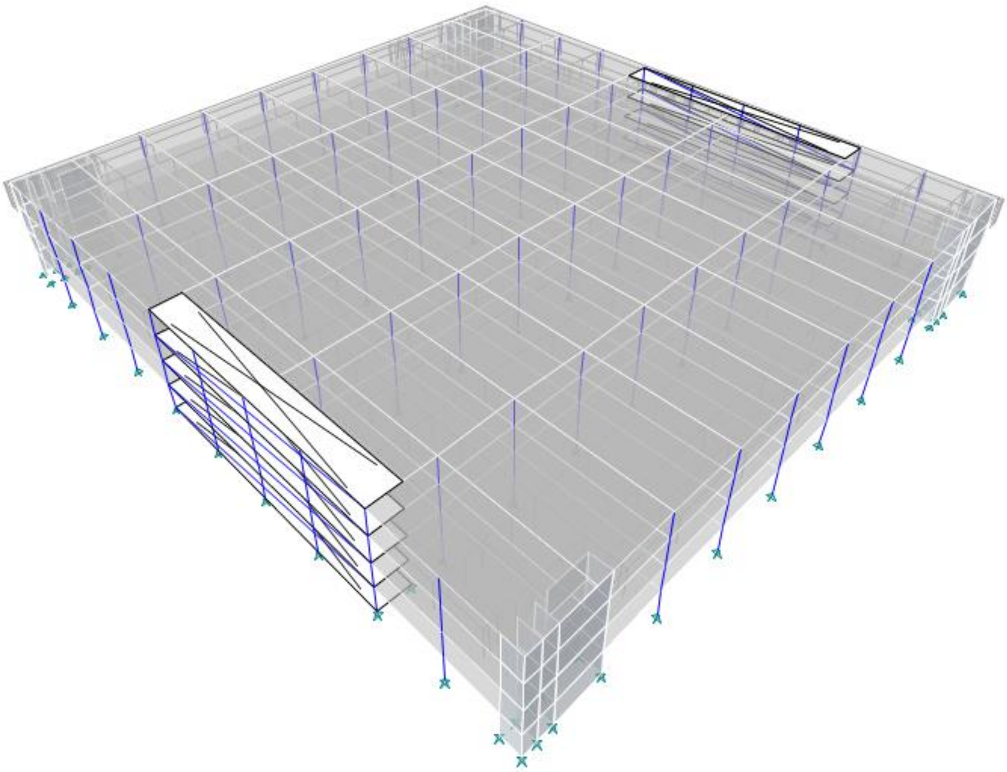
	As4	As3	As2	As1
Edge	2363.5499	3412.2372	5191.8143	9545.0032
Center	3375.5096	9619.47334	13100.7582	19287.5118
% As	2.740%	1.97%	2.00%	2.76%
	1.956%	2.77%	2.52%	2.77%

Punching Shear:

Center	Gama RC (KN/m3)	LL (KN/m2)	Area (m2)	t (mm)	fc' (Mpa - PSI)	C.C. (mm)	bar d (mm)	Wu (KN)
SI	24	2	168.15	550	28	30	20	19.04
Imperical					4000			
Edge								
SI	24	2	84.075	550	28	30	20	19.04
Imperical					4000			
Center	a (mm -inch)	d (mm - inch)	bo (mm - inch)	A'col (m2)	Vu	Vc	⌀Vc	
SI	750	500	5000	1.5625	3171.83	52915.03	39686.27	
Imperical	29.528	19.685	196.850		713.03	980.31	735.23	pass
Edge								
SI	200	500	2800	0.49	1591.46	29632.41	22224.31	
Imperical	7.874	19.685	110.236		357.76	548.97	411.73	pass

14 APPENDIX V ETABS

ETABS® 2016
Integrated Building Design Software



Summary Report

Model File: projct final, Revision 0
4/30/2019

1 Structure Data

This chapter provides model geometry information, including items such as story levels, point coordinates, and element connectivity.

1.1 Story Data

Table 1.1 - Story Data

Name	Height mm	Elevation mm	Master Story	Similar To	Splice Story
Story4	3500	14000	Yes	None	No
Story3	3500	10500	No	Story4	No
Story2	3500	7000	No	Story4	No
Story1	3500	3500	No	Story4	No
Base	0	0	No	None	No

2 Loads

This chapter provides loading information as applied to the model.

2.1 Load Patterns

Table 2.1 - Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
windx	Wind	0	ASCE 7-10
windy	Wind	0	ASCE 7-10
earthquackex	Seismic (Drift)	0	ASCE 7-10
earthquackey	Seismic (Drift)	0	ASCE 7-10
live1	Live	0	

2.2 Load Cases

Table 2.2 - Load Cases - Summary

Name	Type
Dead	Linear Static
Live	Linear Static
windx	Linear Static
windy	Linear Static
earthquackex	Linear Static
earthquackey	Linear Static
pdeltax	Nonlinear Static
pdeltay	Nonlinear Static
live1	Linear Static

3 Analysis Results

This chapter provides analysis results.

3.1 Structure Results

Table 3.1 - Base Reactions

Load Case/Combo	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m	Y m	Z m
Dead	7.287E-07	0	401525.8307	28993538	-36331342	-7.545E-05	0	0	0
Live	0	0	57465.4423	4146413	-5201166	-1.19E-05	0	0	0
windx	-1040.6443	0	0	-0.002213	-7531.2185	74048.8346	0	0	0
windy	0	-1075.3953	-0.000114	7790.3411	0.0141	-97170.553	0	0	0
earthquackex	-24964.2141	0	-6.703E-05	-0.0781	-235475	1803360	0	0	0
earthquackey	0	-21602.5921	-0.003323	205032.9146	0.412	-1954655	0	0	0
pdeltax Max	-1664.9825	0.0062	361373.2493	26094192	-32710301	118475.1286	0	0	0
pdeltax Min	-1664.9825	0.0062	361373.2493	26094192	-32710301	118475.1286	0	0	0
pdeltay Max	-0.0099	-1720.5019	361373.2498	26106762	-32698199	-155461	0	0	0
pdeltay Min	-0.0099	-1720.5019	361373.2498	26106762	-32698199	-155461	0	0	0
live1	0	0	32.8	2992.1787	-4107.1745	0	0	0	0
Comb1	1.059E-06	0	573775.7045	41426506	-51919476	-0.0001096	0	0	0
DL+EQX	-24964.2141	0	401525.8306	28993538	-36566817	1803360	0	0	0
DL+EQY	6.398E-07	-21602.5921	401525.8273	29198571	-36331342	-1954655	0	0	0
DL+WINDX	-1665.0309	0	361373.2476	26094185	-32710258	118478.1353	0	0	0
DL+WINDY	6.46E-07	-1720.6324	361373.2474	26106649	-32698208	-155473	0	0	0
SL	8.44E-07	0	458991.273	33139951	-41532508	-8.736E-05	0	0	0
DWal1	1.02E-06	0	562136.1629	40590954	-50863879	-0.0001056	0	0	0
DWal2	1.064E-06	0	573828.1845	41431294	-51926048	-0.0001101	0	0	0
DWal3	-1040.6443	0	539329.2391	38941651	-48810415	74048.8345	0	0	0
DWal4	1040.6443	0	539329.2391	38941651	-48795353	-74048.8347	0	0	0
DWal5	9.869E-07	-1075.3953	539329.239	38949441	-48802884	-97170.5531	0	0	0
DWal6	9.991E-07	1075.3953	539329.2392	38933861	-48802884	97170.5529	0	0	0
DWal7	-1040.6443	0	361373.2476	26094185	-32705739	74048.8345	0	0	0
DWal8	1040.6443	0	361373.2476	26094185	-32690677	-74048.8347	0	0	0
DWal9	6.497E-07	-1075.3953	361373.2475	26101975	-32698208	-97170.553	0	0	0
DWal10	6.62E-07	1075.3953	361373.2477	26086394	-32698208	97170.5529	0	0	0
ultimate D+L1	8.797E-07	0	481883.4768	34797034	-43604182	-9.109E-05	0	0	0
D+L1	7.32E-07	0	401558.6307	28996531	-36335449	-7.579E-05	0	0	0

3.2 Story Results

Table 3.2 - Story Drifts

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story4	Dead	Y	0.001084	16	94.9867	27.9999	14
Story4	Live	Y	0.000161	16	94.9867	27.9999	14
Story4	windx	X	4.179E-05	35	131.7867	54.5499	14
Story4	windx	Y	5.943E-06	1	49.1867	116.0999	14
Story4	windy	Y	7E-05	83	50.9867	116.0999	14
Story4	earthquackex	X	0.000895	150	58.7859	30.3997	14
Story4	earthquackey	Y	0.001903	88	58.7859	113.7008	14
Story4	pdeltax Max	X	0.000411	4	49.1867	89.9499	14
Story4	pdeltax Max	Y	0.000978	16	94.9867	27.9999	14
Story4	pdeltax Min	X	0.000411	4	49.1867	89.9499	14
Story4	pdeltax Min	Y	0.000978	16	94.9867	27.9999	14
Story4	pdeltay Max	Y	0.001003	16	94.9867	27.9999	14
Story4	pdeltay Min	Y	0.001003	16	94.9867	27.9999	14
Story4	live1	X	1.134E-07	31	131.7867	89.9499	14
Story4	live1	Y	1.201E-07	28	131.7867	116.0999	14
Story4	Comb1	Y	0.001558	16	94.9867	27.9999	14
Story4	DL+EQX	X	0.000901	150	58.7859	30.3997	14

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story4	DL+EQX	Y	0.001076	16	94.9867	27.9999	14
Story4	DL+EQY	Y	0.001923	132	122.1856	30.3997	14
Story4	DL+WINDX	X	0.00041	4	49.1867	89.9499	14
Story4	DL+WINDX	Y	0.000975	16	94.9867	27.9999	14
Story4	DL+WINDY	Y	0.001	16	94.9867	27.9999	14
Story4	SL	Y	0.001245	16	94.9867	27.9999	14
Story4	DWal1	Y	0.001518	16	94.9867	27.9999	14
Story4	DWal2	Y	0.001558	16	94.9867	27.9999	14
Story4	DWal3	Y	0.001461	16	94.9867	27.9999	14
Story4	DWal4	X	0.000616	31	131.7867	89.9499	14
Story4	DWal4	Y	0.001462	16	94.9867	27.9999	14
Story4	DWal5	Y	0.001477	16	94.9867	27.9999	14
Story4	DWal6	X	0.00061	31	131.7867	89.9499	14
Story4	DWal6	Y	0.001447	16	94.9867	27.9999	14
Story4	DWal7	Y	0.000975	16	94.9867	27.9999	14
Story4	DWal8	X	0.000413	31	131.7867	89.9499	14
Story4	DWal8	Y	0.000976	16	94.9867	27.9999	14
Story4	DWal9	Y	0.000991	16	94.9867	27.9999	14
Story4	DWal10	X	0.000407	31	131.7867	89.9499	14
Story4	DWal10	Y	0.000961	16	94.9867	27.9999	14
Story4	ultimate D+L1	Y	0.001301	16	94.9867	27.9999	14
Story4	D+L1	Y	0.001084	16	94.9867	27.9999	14
Story3	Dead	X	4.698E-05	4	49.1867	89.9499	10.5
Story3	Dead	Y	0.000101	24	94.9867	116.0999	10.5
Story3	Live	X	6.996E-06	4	49.1867	89.9499	10.5
Story3	Live	Y	1.503E-05	24	94.9867	116.0999	10.5
Story3	windx	X	3.666E-05	7	49.1867	63.3999	10.5
Story3	windy	Y	2.567E-05	5	49.1867	81.0999	10.5
Story3	earthquackex	X	0.000359	6	49.1867	72.2499	10.5
Story3	earthquackey	Y	0.000718	10	49.1867	36.8499	10.5
Story3	pdeltax Max	X	8.3E-05	7	49.1867	63.3999	10.5
Story3	pdeltax Min	X	8.3E-05	7	49.1867	63.3999	10.5
Story3	pdeltay Max	Y	0.000129	24	94.9867	116.0999	10.5
Story3	pdeltay Min	Y	0.000129	24	94.9867	116.0999	10.5
Story3	live1	X	1.816E-07	31	131.7867	89.9499	10.5
Story3	live1	Y	7.221E-08	48	123.9867	98.7999	10.5
Story3	Comb1	X	6.8E-05	4	49.1867	89.9499	10.5
Story3	Comb1	Y	0.000145	24	94.9867	116.0999	10.5
Story3	DL+EQX	X	0.000385	7	49.1867	63.3999	10.5
Story3	DL+EQY	Y	0.000795	24	94.9867	116.0999	10.5
Story3	DL+WINDX	X	8.3E-05	7	49.1867	63.3999	10.5
Story3	DL+WINDY	Y	0.000128	24	94.9867	116.0999	10.5
Story3	SL	X	5.4E-05	4	49.1867	89.9499	10.5
Story3	SL	Y	0.000116	24	94.9867	116.0999	10.5
Story3	DWal1	X	6.6E-05	4	49.1867	89.9499	10.5
Story3	DWal1	Y	0.000142	24	94.9867	116.0999	10.5
Story3	DWal2	X	6.8E-05	4	49.1867	89.9499	10.5
Story3	DWal2	Y	0.000145	24	94.9867	116.0999	10.5
Story3	DWal3	X	7.3E-05	7	49.1867	63.3999	10.5
Story3	DWal3	Y	0.000136	24	94.9867	116.0999	10.5
Story3	DWal4	X	8.3E-05	34	131.7867	63.3999	10.5
Story3	DWal4	Y	0.000137	24	94.9867	116.0999	10.5
Story3	DWal5	X	6.3E-05	4	49.1867	89.9499	10.5
Story3	DWal5	Y	0.000159	24	94.9867	116.0999	10.5
Story3	DWal6	X	6.4E-05	4	49.1867	89.9499	10.5
Story3	DWal6	Y	0.000156	15	85.4867	27.9999	10.5
Story3	DWal7	X	6.1E-05	7	49.1867	63.3999	10.5
Story3	DWal8	X	6.7E-05	34	131.7867	63.3999	10.5
Story3	DWal9	X	4.209E-05	4	49.1867	89.9499	10.5

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story3	DWal9	Y	0.000114	24	94.9867	116.0999	10.5
Story3	DWal10	X	4.247E-05	4	49.1867	89.9499	10.5
Story3	DWal10	Y	0.000112	15	85.4867	27.9999	10.5
Story3	ultimate D+L1	X	5.6E-05	4	49.1867	89.9499	10.5
Story3	ultimate D+L1	Y	0.000121	24	94.9867	116.0999	10.5
Story3	D+L1	X	4.697E-05	4	49.1867	89.9499	10.5
Story3	D+L1	Y	0.000101	24	94.9867	116.0999	10.5
Story2	Dead	X	2.464E-05	99	122.1856	113.7008	7
Story2	Dead	Y	4.468E-05	15	85.4867	27.9999	7
Story2	Live	X	3.712E-06	99	122.1856	113.7008	7
Story2	Live	Y	6.647E-06	15	85.4867	27.9999	7
Story2	windx	X	5.8E-05	7	49.1867	63.3999	7
Story2	windy	Y	2.792E-05	7	49.1867	63.3999	7
Story2	earthquackex	X	0.000351	34	131.7867	63.3999	7
Story2	earthquackey	Y	0.000732	1	49.1867	116.0999	7
Story2	pdeltax Max	X	9.9E-05	7	49.1867	63.3999	7
Story2	pdeltax Min	X	9.9E-05	7	49.1867	63.3999	7
Story2	pdeltay Max	X	2.225E-05	99	122.1856	113.7008	7
Story2	pdeltay Max	Y	7.8E-05	16	94.9867	27.9999	7
Story2	pdeltay Min	X	2.225E-05	99	122.1856	113.7008	7
Story2	pdeltay Min	Y	7.8E-05	16	94.9867	27.9999	7
Story2	live1	X	1.098E-06	31	131.7867	89.9499	7
Story2	Comb1	X	3.551E-05	99	122.1856	113.7008	7
Story2	Comb1	Y	6.4E-05	15	85.4867	27.9999	7
Story2	DL+EQX	X	0.000358	6	49.1867	72.2499	7
Story2	DL+EQY	Y	0.000735	1	49.1867	116.0999	7
Story2	DL+WINDX	X	9.9E-05	7	49.1867	63.3999	7
Story2	DL+WINDY	X	2.218E-05	99	122.1856	113.7008	7
Story2	DL+WINDY	Y	7.8E-05	16	94.9867	27.9999	7
Story2	SL	X	2.836E-05	99	122.1856	113.7008	7
Story2	SL	Y	5.1E-05	15	85.4867	27.9999	7
Story2	DWal1	X	3.45E-05	99	122.1856	113.7008	7
Story2	DWal1	Y	6.3E-05	15	85.4867	27.9999	7
Story2	DWal2	X	3.557E-05	99	122.1856	113.7008	7
Story2	DWal2	Y	6.4E-05	15	85.4867	27.9999	7
Story2	DWal3	X	6.7E-05	7	49.1867	63.3999	7
Story2	DWal3	Y	6E-05	15	85.4867	27.9999	7
Story2	DWal4	X	7.6E-05	34	131.7867	63.3999	7
Story2	DWal4	Y	6.1E-05	15	85.4867	27.9999	7
Story2	DWal5	X	3.332E-05	99	122.1856	113.7008	7
Story2	DWal5	Y	8.4E-05	15	85.4867	27.9999	7
Story2	DWal6	X	3.332E-05	99	122.1856	113.7008	7
Story2	DWal6	Y	7E-05	24	94.9867	116.0999	7
Story2	DWal7	X	6.4E-05	7	49.1867	63.3999	7
Story2	DWal7	Y	3.971E-05	15	85.4867	27.9999	7
Story2	DWal8	X	7E-05	34	131.7867	63.3999	7
Story2	DWal8	Y	4.072E-05	15	85.4867	27.9999	7
Story2	DWal9	X	2.218E-05	99	122.1856	113.7008	7
Story2	DWal9	Y	6.4E-05	16	94.9867	27.9999	7
Story2	DWal10	X	2.218E-05	99	122.1856	113.7008	7
Story2	DWal10	Y	5.4E-05	24	94.9867	116.0999	7
Story2	ultimate D+L1	X	2.963E-05	99	122.1856	113.7008	7
Story2	ultimate D+L1	Y	5.4E-05	15	85.4867	27.9999	7
Story2	D+L1	X	2.468E-05	99	122.1856	113.7008	7
Story2	D+L1	Y	4.464E-05	15	85.4867	27.9999	7
Story1	Dead	X	3.073E-05	132	122.1856	30.3997	3.5
Story1	Dead	Y	4.984E-05	16	94.9867	27.9999	3.5
Story1	Live	X	4.555E-06	99	122.1856	113.7008	3.5
Story1	Live	Y	7.41E-06	16	94.9867	27.9999	3.5

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story1	windx	X	9E-05	7	49.1867	63.3999	3.5
Story1	windy	Y	3.139E-05	5	49.1867	81.0999	3.5
Story1	earthquackex	X	0.000357	32	131.7867	81.0999	3.5
Story1	earthquackey	Y	0.000704	5	49.1867	81.0999	3.5
Story1	pdeltax Max	X	0.000142	34	131.7867	63.3999	3.5
Story1	pdeltax Min	X	0.000142	34	131.7867	63.3999	3.5
Story1	pdeltay Max	X	2.87E-05	132	122.1856	30.3997	3.5
Story1	pdeltay Max	Y	8.4E-05	16	94.9867	27.9999	3.5
Story1	pdeltay Min	X	2.87E-05	132	122.1856	30.3997	3.5
Story1	pdeltay Min	Y	8.4E-05	16	94.9867	27.9999	3.5
Story1	live1	X	9.889E-07	31	131.7867	89.9499	3.5
Story1	live1	Y	5.096E-07	48	123.9867	98.7999	3.5
Story1	Comb1	X	4.411E-05	132	122.1856	30.3997	3.5
Story1	Comb1	Y	7.2E-05	16	94.9867	27.9999	3.5
Story1	DL+EQX	X	0.000366	4	49.1867	89.9499	3.5
Story1	DL+EQY	Y	0.000711	5	49.1867	81.0999	3.5
Story1	DL+WINDX	X	0.000142	34	131.7867	63.3999	3.5
Story1	DL+WINDY	X	2.864E-05	132	122.1856	30.3997	3.5
Story1	DL+WINDY	Y	8.3E-05	16	94.9867	27.9999	3.5
Story1	SL	X	3.525E-05	132	122.1856	30.3997	3.5
Story1	SL	Y	5.7E-05	16	94.9867	27.9999	3.5
Story1	DWal1	X	4.303E-05	132	122.1856	30.3997	3.5
Story1	DWal1	Y	7E-05	16	94.9867	27.9999	3.5
Story1	DWal2	X	4.414E-05	132	122.1856	30.3997	3.5
Story1	DWal2	Y	7.2E-05	16	94.9867	27.9999	3.5
Story1	DWal3	X	8.7E-05	34	131.7867	63.3999	3.5
Story1	DWal3	Y	6.7E-05	16	94.9867	27.9999	3.5
Story1	DWal4	X	9.5E-05	7	49.1867	63.3999	3.5
Story1	DWal4	Y	6.7E-05	16	94.9867	27.9999	3.5
Story1	DWal5	X	4.203E-05	132	122.1856	30.3997	3.5
Story1	DWal5	Y	9.1E-05	16	94.9867	27.9999	3.5
Story1	DWal6	X	4.187E-05	99	122.1856	113.7008	3.5
Story1	DWal6	Y	7.9E-05	23	85.4867	116.0999	3.5
Story1	DWal7	X	8.8E-05	34	131.7867	63.3999	3.5
Story1	DWal7	Y	4.464E-05	16	94.9867	27.9999	3.5
Story1	DWal8	X	9.3E-05	7	49.1867	63.3999	3.5
Story1	DWal9	X	2.827E-05	132	122.1856	30.3997	3.5
Story1	DWal9	Y	6.9E-05	16	94.9867	27.9999	3.5
Story1	DWal10	X	2.814E-05	99	122.1856	113.7008	3.5
Story1	DWal10	Y	6.1E-05	23	85.4867	116.0999	3.5
Story1	ultimate D+L1	X	3.691E-05	132	122.1856	30.3997	3.5
Story1	ultimate D+L1	Y	6E-05	16	94.9867	27.9999	3.5
Story1	D+L1	X	3.075E-05	132	122.1856	30.3997	3.5
Story1	D+L1	Y	4.98E-05	16	94.9867	27.9999	3.5

Table 3.3 - Story Forces

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story4	Dead	Top	0	0	0	0	0	0
Story4	Dead	Bottom	91712.8022	29.2085	-81.084	-5655.172	6625890	-8299365
Story4	Live	Top	0	0	0	0	0	0
Story4	Live	Bottom	13341.6149	4.4507	-11.6957	-676.8986	963270.4813	-1207377
Story4	windx	Top	0	0	0	0	0	0
Story4	windx	Bottom	-0.1069	-108.15	-3.4939	7439.2641	19.2032	-79.9121
Story4	windy	Top	0	0	0	0	0	0
Story4	windy	Bottom	-0.3915	-4.157	-275.4465	-24468.2762	-332.8355	37.8045
Story4	earthquackex	Top	0	-223.8791	0	15282.9604	0	0
Story4	earthquackex	Bottom	-1.3289	-2790.2497	-73.3462	196909.6666	355.1922	-3949.0442
Story4	earthquackey	Top	0	0	-196.9658	-17805.9655	0	0

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story4	earthquackey	Bottom	-14.0638	-58.8522	-6918.6807	-617405	-10110.6246	1347.1569
Story4	pdeltax Max	Top	2.055E-06	2.257E-05	2.649E-06	-0.0002075	0.0001145	-0.0001486
Story4	pdeltax Max	Bottom	82545.6716	-146.2438	-78.6624	6773.3579	5963632	-7469966
Story4	pdeltax Min	Top	2.055E-06	2.257E-05	2.649E-06	-0.0002075	0.0001145	-0.0001486
Story4	pdeltax Min	Bottom	82545.6716	-146.2438	-78.6624	6773.3579	5963632	-7469966
Story4	pdeltay Max	Top	0	2.465E-06	4.223E-05	0.004657	0.0001329	-0.0003478
Story4	pdeltay Max	Bottom	82545.2348	19.4853	-514.0122	-44250.8288	5963064	-7469771
Story4	pdeltay Min	Top	0	2.465E-06	4.223E-05	0.004657	0.0001329	-0.0003478
Story4	pdeltay Min	Bottom	82545.2348	19.4853	-514.0122	-44250.8288	5963064	-7469771
Story4	live1	Top	0	0	0	0	0	0
Story4	live1	Bottom	-0.0058	0.0103	0.1461	11.729	-0.2058	0.7643
Story4	Comb1	Top	0	0	0	0	0	0
Story4	Comb1	Bottom	131401.9464	42.1713	-116.0139	-7869.2441	9492300	-11891041
Story4	DL+EQX	Top	0	-223.8791	0	15282.9604	0	0
Story4	DL+EQX	Bottom	91711.4733	-2761.0412	-154.4301	191254.4946	6626245	-8303314
Story4	DL+EQY	Top	0	0	-196.9658	-17805.9655	0	0
Story4	DL+EQY	Bottom	91698.7383	-29.6437	-6999.7647	-623061	6615779	-8298018
Story4	DL+WINDX	Top	0	0	0	0	0	0
Story4	DL+WINDX	Bottom	82541.351	-146.7523	-78.5658	6813.1677	5963331	-7469556
Story4	DL+WINDY	Top	0	0	0	0	0	0
Story4	DL+WINDY	Bottom	82540.8955	19.6365	-513.6899	-44238.8968	5962768	-7469368
Story4	SL	Top	0	0	0	0	0	0
Story4	SL	Bottom	105054.4171	33.6592	-92.7797	-6332.0706	7589160	-9506742
Story4	DWal1	Top	0	0	0	0	0	0
Story4	DWal1	Bottom	128397.923	40.8919	-113.5175	-7917.2408	9276246	-11619111
Story4	DWal2	Top	0	0	0	0	0	0
Story4	DWal2	Bottom	131401.9372	42.1879	-115.7803	-7850.4777	9492300	-11891040
Story4	DWal3	Top	0	0	0	0	0	0
Story4	DWal3	Bottom	123396.8648	-68.6387	-112.3443	-12.1119	8914357	-11166694
Story4	DWal4	Top	0	0	0	0	0	0
Story4	DWal4	Bottom	123397.0785	147.6612	-105.3566	-14890.64	8914319	-11166534
Story4	DWal5	Top	0	0	0	0	0	0
Story4	DWal5	Bottom	123396.5802	35.3543	-384.2969	-31919.6522	8914005	-11166576
Story4	DWal6	Top	0	0	0	0	0	0
Story4	DWal6	Bottom	123397.3632	43.6682	166.596	17016.9003	8914671	-11166652
Story4	DWal7	Top	0	0	0	0	0	0
Story4	DWal7	Bottom	82541.4151	-81.8623	-76.4695	2349.6093	5963320	-7469509
Story4	DWal8	Top	0	0	0	0	0	0
Story4	DWal8	Bottom	82541.6288	134.4376	-69.4817	-12528.9189	5963282	-7469349
Story4	DWal9	Top	0	0	0	0	0	0
Story4	DWal9	Bottom	82541.1304	22.1307	-348.422	-29557.931	5962968	-7469391
Story4	DWal10	Top	0	0	0	0	0	0
Story4	DWal10	Bottom	82541.9134	30.4446	202.4709	19378.6214	5963634	-7469466
Story4	ultimate D+L1	Top	0	0	0	0	0	0
Story4	ultimate D+L1	Bottom	110055.3533	35.0668	-97.0671	-6767.44	7951067	-9959237
Story4	D+L1	Top	0	0	0	0	0	0
Story4	D+L1	Bottom	91712.7964	29.2189	-80.9379	-5643.443	6625889	-8299364
Story3	Dead	Top	91712.8022	29.2085	-81.084	-5655.172	6625890	-8299365
Story3	Dead	Bottom	183455.9729	94.7015	-96.616	-13616.7032	13254113	-16600953
Story3	Live	Top	13341.6149	4.4507	-11.6957	-676.8986	963270.4813	-1207377
Story3	Live	Bottom	26705.2814	14.0405	-14.3361	-1951.8131	1927844	-2417395
Story3	windx	Top	-0.1069	-108.15	-3.4939	7439.2641	19.2032	-79.9121
Story3	windx	Bottom	-0.0469	-157.8655	-6.4692	10725.9488	67.0183	-366.9408
Story3	windy	Top	-0.3915	-4.157	-275.4465	-24468.2762	-332.8355	37.8045
Story3	windy	Bottom	-1.0285	-3.0832	-370.3903	-33233.7641	-321.0915	99.9191
Story3	earthquackex	Top	-1.3289	-3126.0156	-73.3462	219830.7635	355.1922	-3949.0442
Story3	earthquackex	Bottom	5.5676	-3700.5825	-107.3706	260184.7752	1494.2318	-14394.0317
Story3	earthquackey	Top	-14.0638	-58.8522	-7210.2059	-643760	-10110.6246	1347.1569
Story3	earthquackey	Bottom	-31.5567	-76.6184	-8956.304	-804161	-10885.2746	2871.6576

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story3	pdeltax Max	Top	82545.6716	-146.2436	-78.662	6773.3726	5963632	-7469966
Story3	pdeltax Max	Bottom	165115.4272	-165.7978	-97.2106	4807.0799	11929167	-14941935
Story3	pdeltax Min	Top	82545.6716	-146.2436	-78.662	6773.3726	5963632	-7469966
Story3	pdeltax Min	Bottom	165115.4272	-165.7978	-97.2106	4807.0799	11929167	-14941935
Story3	pdeltay Max	Top	82545.2348	19.485	-514.0093	-44250.5461	5963064	-7469771
Story3	pdeltay Max	Bottom	165113.861	79.8021	-677.1167	-65168.7989	11928544	-14941172
Story3	pdeltay Min	Top	82545.2348	19.485	-514.0093	-44250.5461	5963064	-7469771
Story3	pdeltay Min	Bottom	165113.861	79.8021	-677.1167	-65168.7989	11928544	-14941172
Story3	live1	Top	-0.0058	0.0103	0.1461	11.729	-0.2058	0.7643
Story3	live1	Bottom	-0.004123	-0.2435	0.2854	48.5475	0.1237	0.0167
Story3	Comb1	Top	131401.9464	42.1713	-116.0139	-7869.2441	9492300	-11891041
Story3	Comb1	Bottom	262875.6177	136.1066	-138.8769	-19462.9448	18989486	-23788975
Story3	DL+EQX	Top	91711.4733	-3096.807	-154.4301	214175.5915	6626245	-8303314
Story3	DL+EQX	Bottom	183461.5404	-3605.881	-203.9865	246568.072	13255607	-16615347
Story3	DL+EQY	Top	91698.7383	-29.6437	-7291.2898	-649415	6615779	-8298018
Story3	DL+EQY	Bottom	183424.4162	18.0831	-9052.92	-817778	13243228	-16598081
Story3	DL+WINDX	Top	82541.351	-146.7523	-78.5658	6813.1677	5963331	-7469556
Story3	DL+WINDX	Bottom	165110.3005	-167.3534	-97.305	4906.4852	11928809	-14941445
Story3	DL+WINDY	Top	82540.8955	19.6365	-513.6899	-44238.8968	5962768	-7469368
Story3	DL+WINDY	Bottom	165108.73	80.2982	-679.5788	-65429.0555	11928188	-14940698
Story3	SL	Top	105054.4171	33.6592	-92.7797	-6332.0706	7589160	-9506742
Story3	SL	Bottom	210161.2542	108.742	-110.952	-15568.5163	15181957	-19018347
Story3	DWal1	Top	128397.923	40.8919	-113.5175	-7917.2408	9276246	-11619111
Story3	DWal1	Bottom	256838.362	132.5821	-135.2624	-19063.3845	18555758	-23241334
Story3	DWal2	Top	131401.9372	42.1879	-115.7803	-7850.4777	9492300	-11891040
Story3	DWal2	Bottom	262875.6111	135.717	-138.4203	-19385.2688	18989487	-23788975
Story3	DWal3	Top	123396.8648	-68.6387	-112.3443	-12.1119	8914357	-11166694
Story3	DWal3	Bottom	246852.3978	-30.4267	-136.459	-7517.3607	17832847	-22338905
Story3	DWal4	Top	123397.0785	147.6612	-105.3566	-14890.64	8914319	-11166534
Story3	DWal4	Bottom	246852.4916	285.3043	-123.5207	-28969.2582	17832713	-22338171
Story3	DWal5	Top	123396.5802	35.3543	-384.2969	-31919.6522	8914005	-11166576
Story3	DWal5	Bottom	246851.4162	124.3556	-500.3801	-51477.0736	17832459	-22338438
Story3	DWal6	Top	123397.3632	43.6682	166.596	17016.9003	8914671	-11166652
Story3	DWal6	Bottom	246853.4732	130.522	240.4004	14990.4547	17833101	-22338638
Story3	DWal7	Top	82541.4151	-81.8623	-76.4695	2349.6093	5963320	-7469509
Story3	DWal7	Bottom	165110.3287	-72.6341	-93.4235	-1529.0841	11928769	-14941224
Story3	DWal8	Top	82541.6288	134.4376	-69.4817	-12528.9189	5963282	-7469349
Story3	DWal8	Bottom	165110.4225	243.0969	-80.4852	-22980.9817	11928635	-14940491
Story3	DWal9	Top	82541.1304	22.1307	-348.422	-29557.931	5962968	-7469391
Story3	DWal9	Bottom	165109.3471	82.1481	-457.3447	-45488.797	11928381	-14940758
Story3	DWal10	Top	82541.9134	30.4446	202.4709	19378.6214	5963634	-7469466
Story3	DWal10	Bottom	165111.404	88.3146	283.4359	20978.7313	11929023	-14940957
Story3	ultimate D+L1	Top	110055.3533	35.0668	-97.0671	-6767.44	7951067	-9959237
Story3	ultimate D+L1	Bottom	220147.1608	113.2522	-115.4826	-16262.3679	15904936	-19921143
Story3	D+L1	Top	91712.7964	29.2189	-80.9379	-5643.443	6625889	-8299364
Story3	D+L1	Bottom	183455.9687	94.458	-96.3306	-13568.1557	13254113	-16600953
Story2	Dead	Top	183455.9729	94.7015	-96.616	-13616.7032	13254113	-16600953
Story2	Dead	Bottom	276055.1527	59.2586	-78.8572	-10950.2789	19944222	-24979764
Story2	Live	Top	26705.2814	14.0405	-14.3361	-1951.8131	1927844	-2417395
Story2	Live	Bottom	40058.5721	8.741	-11.6786	-1560.7294	2891987	-3625737
Story2	windx	Top	-0.0469	-157.8655	-6.4692	10725.9488	67.0183	-366.9408
Story2	windx	Bottom	0.4026	-193.4372	-7.0006	13239.7052	144.2822	-703.535
Story2	windy	Top	-1.0285	-3.0832	-370.3903	-33233.7641	-321.0915	99.9191
Story2	windy	Bottom	-1.7228	0.6217	-433.0461	-39574.7443	-267.5643	161.0457
Story2	earthquackex	Top	5.5676	-3945.3651	-107.3706	276978.572	1494.2318	-14394.0317
Story2	earthquackex	Bottom	22.0015	-3737.8576	-91.8481	264703.2646	3240.8301	-26415.6245
Story2	earthquackey	Top	-31.5567	-76.6184	-9164.9131	-823018	-10885.2746	2871.6576
Story2	earthquackey	Bottom	-52.1137	33.937	-9343.3638	-855073	-10164.4987	4661.4301
Story2	pdeltax Max	Top	165115.4272	-165.7985	-97.2114	4807.0545	11929167	-14941935

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story2	pdeltax Max	Bottom	248454.2682	-252.8783	-81.9512	11111.657	17950310	-22483307
Story2	pdeltax Min	Top	165115.4272	-165.7985	-97.2114	4807.0545	11929167	-14941935
Story2	pdeltax Min	Bottom	248454.2682	-252.8783	-81.9512	11111.657	17950310	-22483307
Story2	pdeltay Max	Top	165113.8611	79.8017	-677.1248	-65169.5118	11928544	-14941172
Story2	pdeltay Max	Bottom	248450.8453	53.5984	-758.5872	-72655.6771	17949653	-22481896
Story2	pdeltay Min	Top	165113.8611	79.8017	-677.1248	-65169.5118	11928544	-14941172
Story2	pdeltay Min	Bottom	248450.8453	53.5984	-758.5872	-72655.6771	17949653	-22481896
Story2	live1	Top	-0.004123	-0.2435	0.2854	48.5475	0.1237	0.0167
Story2	live1	Bottom	-0.0595	-1.2484	-1.6846	-107.1194	0.3255	2.6068
Story2	Comb1	Top	262875.6177	136.1066	-138.8769	-19462.9448	18989486	-23788975
Story2	Comb1	Bottom	395359.8987	85.096	-113.3144	-15637.5017	28560247	-35776896
Story2	DL+EQX	Top	183461.5404	-3850.6636	-203.9865	263361.8688	13255607	-16615347
Story2	DL+EQX	Bottom	276077.1543	-3678.599	-170.7053	253752.9857	19947463	-25006180
Story2	DL+EQY	Top	183424.4162	18.0831	-9261.529	-836635	13243228	-16598081
Story2	DL+EQY	Bottom	276003.039	93.1955	-9422.2211	-866023	19934058	-24975103
Story2	DL+WINDX	Top	165110.3005	-167.3534	-97.305	4906.4852	11928809	-14941445
Story2	DL+WINDX	Bottom	248450.2817	-256.1668	-82.1725	11328.2774	17950031	-22482914
Story2	DL+WINDY	Top	165108.73	80.2982	-679.5788	-65429.0555	11928188	-14940698
Story2	DL+WINDY	Bottom	248446.8811	54.3274	-763.8452	-73174.8418	17949372	-22481530
Story2	SL	Top	210161.2542	108.742	-110.952	-15568.5163	15181957	-19018347
Story2	SL	Bottom	316113.7249	67.9996	-90.5358	-12511.0083	22836210	-28605501
Story2	DWal1	Top	256838.362	132.5821	-135.2624	-19063.3845	18555758	-23241334
Story2	DWal1	Bottom	386477.2138	82.962	-110.4001	-15330.3904	27921911	-34971670
Story2	DWal2	Top	262875.6111	135.717	-138.4203	-19385.2688	18989487	-23788975
Story2	DWal2	Bottom	395359.8036	83.0985	-116.0098	-15808.8928	28560247	-35776892
Story2	DWal3	Top	246852.3978	-30.4267	-136.459	-7517.3607	17832847	-22338905
Story2	DWal3	Bottom	371325.0986	-114.8343	-114.9925	-1568.4782	26825199	-33602155
Story2	DWal4	Top	246852.4916	285.3043	-123.5207	-28969.2582	17832713	-22338171
Story2	DWal4	Bottom	371324.2933	272.0402	-100.9912	-28047.8887	26824910	-33600748
Story2	DWal5	Top	246851.4162	124.3556	-500.3801	-51477.0736	17832459	-22338438
Story2	DWal5	Bottom	371322.9732	79.2246	-541.0379	-54382.9277	26824787	-33601290
Story2	DWal6	Top	246853.4732	130.522	240.4004	14990.4547	17833101	-22338638
Story2	DWal6	Bottom	371326.4187	77.9813	325.0542	24766.5608	26825322	-33601612
Story2	DWal7	Top	165110.3287	-72.6341	-93.4235	-1529.0841	11928769	-14941224
Story2	DWal7	Bottom	248450.0401	-140.1045	-77.9721	3384.4542	17949944	-22482491
Story2	DWal8	Top	165110.4225	243.0969	-80.4852	-22980.9817	11928635	-14940491
Story2	DWal8	Bottom	248449.2348	246.7699	-63.9709	-23094.9563	17949656	-22481084
Story2	DWal9	Top	165109.3471	82.1481	-457.3447	-45488.797	11928381	-14940758
Story2	DWal9	Bottom	248447.9147	53.9544	-504.0176	-49429.9953	17949532	-22481627
Story2	DWal10	Top	165111.404	88.3146	283.4359	20978.7313	11929023	-14940957
Story2	DWal10	Bottom	248451.3602	52.711	362.0746	29719.4933	17950068	-22481949
Story2	ultimate D+L1	Top	220147.1608	113.2522	-115.4826	-16262.3679	15904936	-19921143
Story2	ultimate D+L1	Bottom	331266.0881	69.1129	-97.324	-13311.7257	23933067	-29975713
Story2	D+L1	Top	183455.9687	94.458	-96.3306	-13568.1557	13254113	-16600953
Story2	D+L1	Bottom	276055.0933	58.0102	-80.5418	-11057.3983	19944223	-24979762
Story1	Dead	Top	276055.1527	59.2586	-78.8572	-10950.2789	19944222	-24979764
Story1	Dead	Bottom	401525.8307	7.287E-07	0	-7.545E-05	28993538	-36331342
Story1	Live	Top	40058.5721	8.741	-11.6786	-1560.7294	2891987	-3625737
Story1	Live	Bottom	57465.4423	0	0	-1.19E-05	4146413	-5201166
Story1	windx	Top	0.4026	-193.4372	-7.0006	13239.7052	144.2822	-703.535
Story1	windx	Bottom	0	-1040.6443	0	74048.8346	-0.002213	-7531.2185
Story1	windy	Top	-1.7228	0.6217	-433.0461	-39574.7443	-267.5643	161.0457
Story1	windy	Bottom	-0.000114	0	-1075.3953	-97170.553	7790.3411	0.0141
Story1	earthquackex	Top	22.0015	-3870.9643	-91.8481	273872.2995	3240.8301	-26415.6245
Story1	earthquackex	Bottom	-6.703E-05	-24964.2141	0	1803360	-0.0781	-235475
Story1	earthquackey	Top	-52.1137	33.937	-9453.2462	-865005	-10164.4987	4661.4301
Story1	earthquackey	Bottom	-0.003323	0	-21602.5921	-1954655	205032.9146	0.412
Story1	pdeltax Max	Top	248454.2682	-252.8743	-81.9507	11111.43	17950310	-22483307
Story1	pdeltax Max	Bottom	361373.2493	-1664.9825	0.0062	118475.1286	26094192	-32710301

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story1	pdeltax Min	Top	248454.2682	-252.8743	-81.9507	11111.43	17950310	-22483307
Story1	pdeltax Min	Bottom	361373.2493	-1664.9825	0.0062	118475.1286	26094192	-32710301
Story1	pdeltay Max	Top	248450.8453	53.5992	-758.6069	-72657.5418	17949653	-22481896
Story1	pdeltay Max	Bottom	361373.2498	-0.0099	-1720.5019	-155461	26106762	-32698199
Story1	pdeltay Min	Top	248450.8453	53.5992	-758.6069	-72657.5418	17949653	-22481896
Story1	pdeltay Min	Bottom	361373.2498	-0.0099	-1720.5019	-155461	26106762	-32698199
Story1	live1	Top	-0.0595	-1.2484	-1.6846	-107.1194	0.3255	2.6068
Story1	live1	Bottom	32.8	0	0	0	2992.1787	-4107.1745
Story1	Comb1	Top	395359.8987	85.096	-113.3144	-15637.5017	28560247	-35776896
Story1	Comb1	Bottom	573775.7045	1.059E-06	0	-0.0001096	41426506	-51919476
Story1	DL+EQX	Top	276077.1543	-3811.7057	-170.7053	262922.0206	19947463	-25006180
Story1	DL+EQX	Bottom	401525.8306	-24964.2141	0	1803360	28993538	-36566817
Story1	DL+EQY	Top	276003.039	93.1955	-9532.1034	-875955	19934058	-24975103
Story1	DL+EQY	Bottom	401525.8273	6.398E-07	-21602.5921	-1954655	29198571	-36331342
Story1	DL+WINDX	Top	248450.2817	-256.1668	-82.1725	11328.2774	17950031	-22482914
Story1	DL+WINDX	Bottom	361373.2476	-1665.0309	0	118478.1353	26094185	-32710258
Story1	DL+WINDY	Top	248446.8811	54.3274	-763.8452	-73174.8418	17949372	-22481530
Story1	DL+WINDY	Bottom	361373.2474	6.46E-07	-1720.6324	-155473	26106649	-32698208
Story1	SL	Top	316113.7249	67.9996	-90.5358	-12511.0083	22836210	-28605501
Story1	SL	Bottom	458991.273	8.44E-07	0	-8.736E-05	33139951	-41532508
Story1	DWal1	Top	386477.2138	82.962	-110.4001	-15330.3904	27921911	-34971670
Story1	DWal1	Bottom	562136.1629	1.02E-06	0	-0.0001056	40590954	-50863879
Story1	DWal2	Top	395359.8036	83.0985	-116.0098	-15808.8928	28560247	-35776892
Story1	DWal2	Bottom	573828.1845	1.064E-06	0	-0.0001101	41431294	-51926048
Story1	DWal3	Top	371325.0986	-114.8343	-114.9925	-1568.4782	26825199	-33602155
Story1	DWal3	Bottom	539329.2391	-1040.6443	0	74048.8345	38941651	-48810415
Story1	DWal4	Top	371324.2933	272.0402	-100.9912	-28047.8887	26824910	-33600748
Story1	DWal4	Bottom	539329.2391	1040.6443	0	-74048.8347	38941651	-48795353
Story1	DWal5	Top	371322.9732	79.2246	-541.0379	-54382.9277	26824787	-33601290
Story1	DWal5	Bottom	539329.239	9.869E-07	-1075.3953	-97170.5531	38949441	-48802884
Story1	DWal6	Top	371326.4187	77.9813	325.0542	24766.5608	26825322	-33601612
Story1	DWal6	Bottom	539329.2392	9.991E-07	1075.3953	97170.5529	38933861	-48802884
Story1	DWal7	Top	248450.0401	-140.1045	-77.9721	3384.4542	17949944	-22482491
Story1	DWal7	Bottom	361373.2476	-1040.6443	0	74048.8345	26094185	-32705739
Story1	DWal8	Top	248449.2348	246.7699	-63.9709	-23094.9563	17949656	-22481084
Story1	DWal8	Bottom	361373.2476	1040.6443	0	-74048.8347	26094185	-32690677
Story1	DWal9	Top	248447.9147	53.9544	-504.0176	-49429.9953	17949532	-22481627
Story1	DWal9	Bottom	361373.2475	6.497E-07	-1075.3953	-97170.553	26101975	-32698208
Story1	DWal10	Top	248451.3602	52.711	362.0746	29719.4933	17950068	-22481949
Story1	DWal10	Bottom	361373.2477	6.62E-07	1075.3953	97170.5529	26086394	-32698208
Story1	ultimate D+L1	Top	331266.0881	69.1129	-97.324	-13311.7257	23933067	-29975713
Story1	ultimate D+L1	Bottom	481883.4768	8.797E-07	0	-9.109E-05	34797034	-43604182
Story1	D+L1	Top	276055.0933	58.0102	-80.5418	-11057.3983	19944223	-24979762
Story1	D+L1	Bottom	401558.6307	7.32E-07	0	-7.579E-05	28996531	-36335449

3.3 Modal Results

Table 3.4 - Modal Periods and Frequencies

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	1	0.592	1.69	10.6162	112.7033
Modal	2	0.4	2.499	15.7015	246.5372
Modal	3	0.294	3.403	21.3787	457.0507
Modal	4	0.182	5.482	34.4473	1186.6179
Modal	5	0.166	6.023	37.8436	1432.141
Modal	6	0.132	7.597	47.7322	2278.3637
Modal	7	0.13	7.69	48.3159	2334.4304
Modal	8	0.116	8.591	53.977	2913.5165
Modal	9	0.11	9.121	57.3067	3284.0624
Modal	10	0.109	9.187	57.7229	3331.9337
Modal	11	0.087	11.474	72.091	5197.1089
Modal	12	0.087	11.488	72.1781	5209.678

Table 3.5 - Modal Participating Mass Ratios (Part 1 of 2)

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	0.592	0.0004126	0.8178	0	0.0004126	0.8178	0
Modal	2	0.4	0.8462	0.0003876	0	0.8467	0.8182	0
Modal	3	0.294	3.321E-05	0.0003287	0	0.8467	0.8185	0
Modal	4	0.182	0.0002596	0.1442	0	0.847	0.9627	0
Modal	5	0.166	0.1193	0.0004399	0	0.9662	0.9631	0
Modal	6	0.132	1.885E-05	0	0	0.9663	0.9631	0
Modal	7	0.13	0.004429	4.291E-05	0	0.9707	0.9632	0
Modal	8	0.116	7.736E-06	6.342E-05	0	0.9707	0.9632	0
Modal	9	0.11	0.0165	4.381E-05	0	0.9872	0.9633	0
Modal	10	0.109	2.127E-05	0.0271	0	0.9872	0.9904	0
Modal	11	0.087	4.977E-06	2.522E-06	0	0.9872	0.9904	0
Modal	12	0.087	0.0001876	4.209E-06	0	0.9874	0.9904	0

Table 3.5 - Modal Participating Mass Ratios (Part 2 of 2)

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0.1955	1.092E-05	0.000179	0.1955	1.092E-05	0.000179
Modal	2	3.933E-05	0.1646	1.378E-05	0.1955	0.1646	0.0001928
Modal	3	2.316E-05	0	0.8151	0.1955	0.1646	0.8153
Modal	4	0.6545	0.0008122	0.000265	0.8501	0.1655	0.8156
Modal	5	0.0009253	0.6847	8.236E-05	0.851	0.8501	0.8157
Modal	6	0	0.0001562	0.0074	0.851	0.8503	0.823
Modal	7	0.0003512	0.0276	0.0008802	0.8513	0.8779	0.8239
Modal	8	0.0003029	7.783E-05	0.135	0.8516	0.878	0.9589
Modal	9	0.0001563	0.0652	2.974E-06	0.8518	0.9431	0.9589
Modal	10	0.1039	8.555E-05	0.0003531	0.9557	0.9432	0.9593
Modal	11	7.548E-06	8.827E-05	9.667E-05	0.9557	0.9433	0.9594
Modal	12	4.291E-06	0.00157	0.0003303	0.9557	0.9449	0.9597

Table 3.6 - Modal Load Participation Ratios

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	99.97	98.74
Modal	Acceleration	UY	99.99	99.04
Modal	Acceleration	UZ	0	0

Table 3.7 - Modal Direction Factors

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	0.592	0	0.999	0	0
Modal	2	0.4	1	0	0	0
Modal	3	0.294	0	0.001	0	0.999
Modal	4	0.182	0.001	0.998	0	0
Modal	5	0.166	0.998	0.002	0	0
Modal	6	0.132	0.002	0	0	0.998
Modal	7	0.13	0.878	0.01	0	0.111
Modal	8	0.116	0.002	0.011	0	0.987
Modal	9	0.11	0.998	0.001	0	0.001
Modal	10	0.109	0	0.999	0	0.001
Modal	11	0.087	0.001	0	0	0.999
Modal	12	0.087	0.23	0.001	0	0.769