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

Senior Design Project (ASSE III)
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Design of a Habitation Compound in a Difficult Soil

Presented by

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CHAPTER 1: INTRODUCTION

1.1. General

It is nowadays well reported that collapsible soils spread in many countries, including United States, Russia, China, South America (e.g. Brazil), South and North Africa (e.g. Egypt, Algeria), Middle East (e.g. Saudi Arabia) and many countries in Eastern Europe. In general, collapsible soils are located in arid and semi-arid regions around the world.

Many natural soils; especially sabkha soils, Aeolian, loess and silty sand with honeycomb structure will experience significant volume decrease upon wetting or rather will display collapsible behavior. Man-made fills, which were compacted at the optimum water content, may also develop a collapsible or metastable structure at low density (Tadepalli and Fredlund 1991). This special type of soil is characterized by abrupt reduction in strength, excessive and sudden settlement when it becomes wet leading to failure of the structure (Figure 1).

Due to the increasing use of water for irrigation, industrial projects and domestic purposes, foundations built on these types of soil (collapsible) have experienced uncalculated settlement, and accordingly significant damage to these structures. Construction on such a kind of soil is one of the prominent problems in geotechnical engineering.

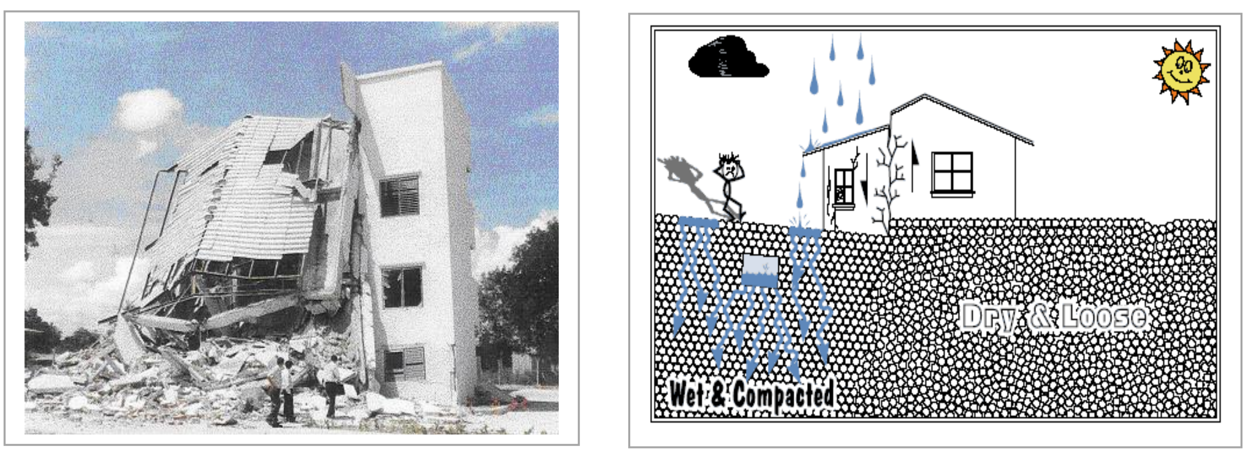


Figure 1. An example of a structure collapse failure

1.2. Project Objectives

The main objectives of this project are designing a habitation compound in a difficult soil such as sabkha soils. The project includes structural and geotechnical design of the different elements of a compound, such as apartment units, sport center, mosque, high rise water reservoir and local roads & parking.

The project is composed by the following stages or parts:

1- Development of the different architectural plans of the compound (including plans & faces views).
2- Structural design of an apartment unit (prototype), sport center, mosque and water reservoir.
3- Geotechnical design of different foundation system: foundations on replaced soil, dynamic compaction and piles.
4- Pavement design of a road & parking sections (prototype sections).
5- Development of the compound prototype.

1.3. Scope of the report

The present report is composed of six chapters. A detailed description of the project and its components (i.e. apartment units, sport center, mosque, high rise reservoir, parking and access roads) is presented in chapter 2. Soil report is introduced in chapter 3 including site investigation, laboratory works and soil profile. Following that, the structural and geotechnical design of the different elements of the compound are given in chapters 4 and 5, respectively. The report is achieved by some specific and general conclusions.

CHAPTER 2: Description of the project

2.1 Introduction

This chapter is mainly concerned by the description of the overall components or parts of the compound. The compound is constituted by many residential buildings and some related facilities. It is composed mainly by five different types of buildings, including: five stories apartment units, a two stories mosque, a men/women sport center (i.e. two stories gym) and a high rise reservoir for water consummation. Moreover, two small rooms and big hall are planned in the front and next to the main gate.

The chapter will start by describing the overall component of the compound (i.e. the different parts of the compound). Then a description is given concerning the apartment units (including the architecture disposition and its emplacement). Following that a brief description of the gym, the mosque and high-rise reservoir is included. Finally, the chapter is achieved by some information concerning the different cars parking and access roads.

2.2 overall description of the project

The site of compound extends over an area of about 33000 m^2 (i.e. 315m $\times$105m) and is located fictively in Al-Azizia in the eastern province in Kingdom of Saudi Arabia. It is surrounded by three different streets. One of them will be used to communicate with the main gate of the compound, situated approximately in the middle of the 315 m side. The compound is composed by the following parts:

- Thirteen apartment units constructed parallel to each other, six units in one size of the main road and seven in the other size.

- A two floor mosque designed to accommodate both sex (men and women). The mosque has its own parking which is planned to accommodate a big number of cars.
- A sport center or a gym located nearby the mosque and it is composed mainly by a swimming pool and a large game room. It is located in the same side of the mosque.
- A fifteen meters high rise reservoir located on the opposite side of the mosque and the gym. It is designed to collect and distribute drinking water to the different facilities of the compound.
- Main and many access roads, and different cars parking will be constructed all around the compound.

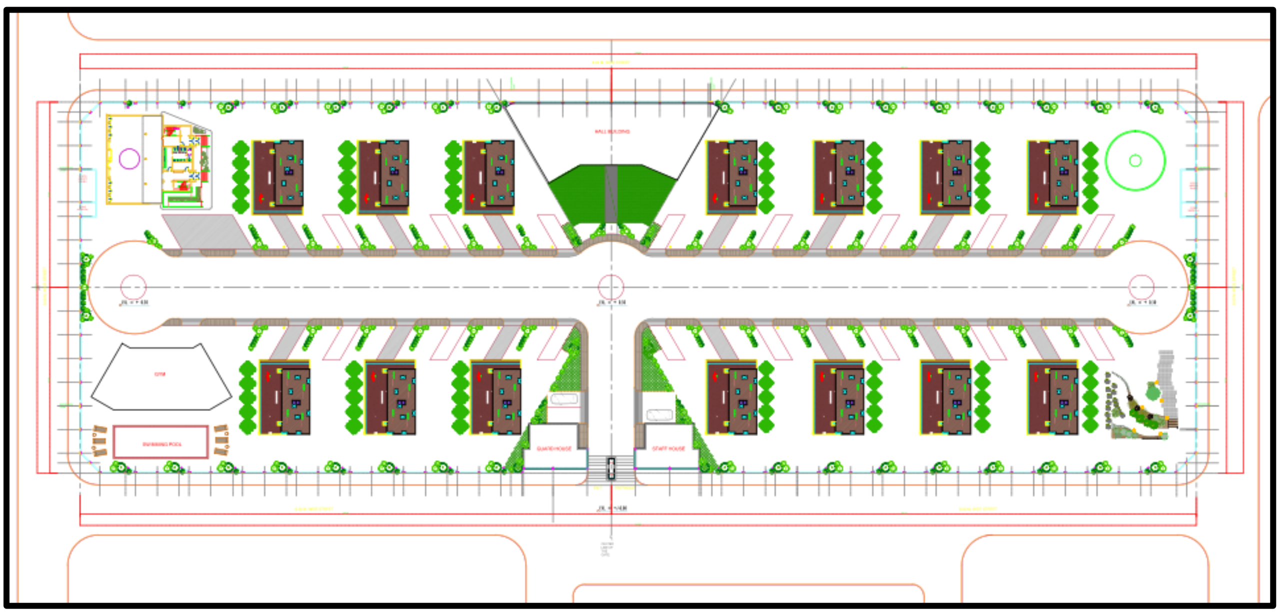


Figure 2.1. Top view of the Compound.

2.3 Apartment units

As mentioned previously, thirteen similar apartment units are designed for the compound. The plan area of each apartment unit will be around 670 m^2 (i.e. 21m × 32m). The units will be placed parallel to each other. Around the units some trees and small green areas will be implanted to give a nice view of the area.

The units are composed of five floors (i.e. stories). The ground floor in each unit is designed as a car parking area for the tenants. The parking garage has one main entrance communicating to the outside and one other door which communicate to the inside building. The first four stories above the ground floor are designed as apartments. Each story is composed of four similar apartments (i.e. apartments having the same plan area and number of rooms. However, the last story is divided into two apartments. There will be an emergency exit and an elevator for each building.

Each apartment is constituted of six rooms, a kitchen and bathrooms. The rooms have almost the same size with an average area of about 9 m^2 (3m×3m). However, the apartments of the last floor, having a plan area of 6 m^2, are divided into four rooms, a kitchen and bathrooms.

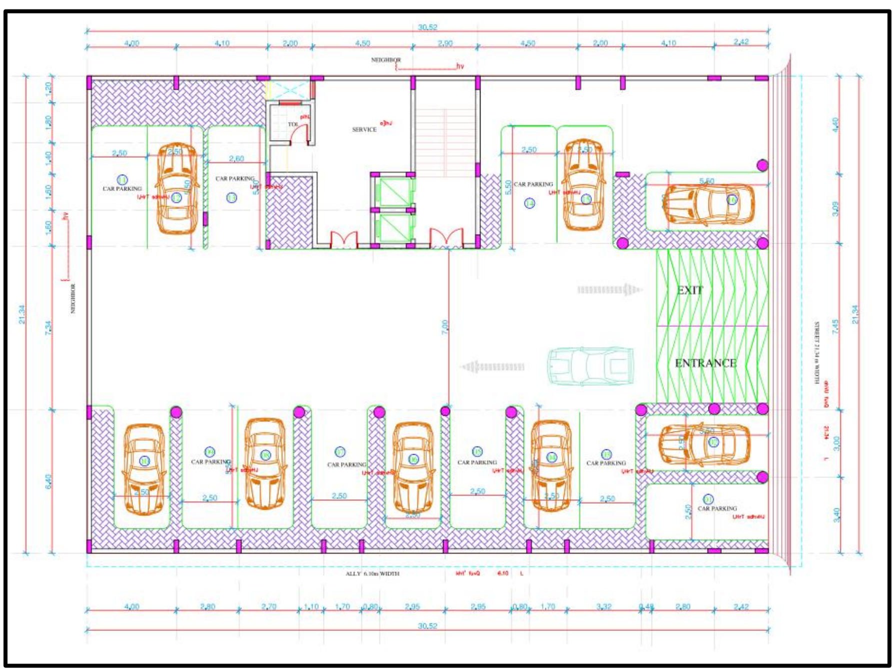


Figure 2.2. Plan of the ground floor (parking garage).

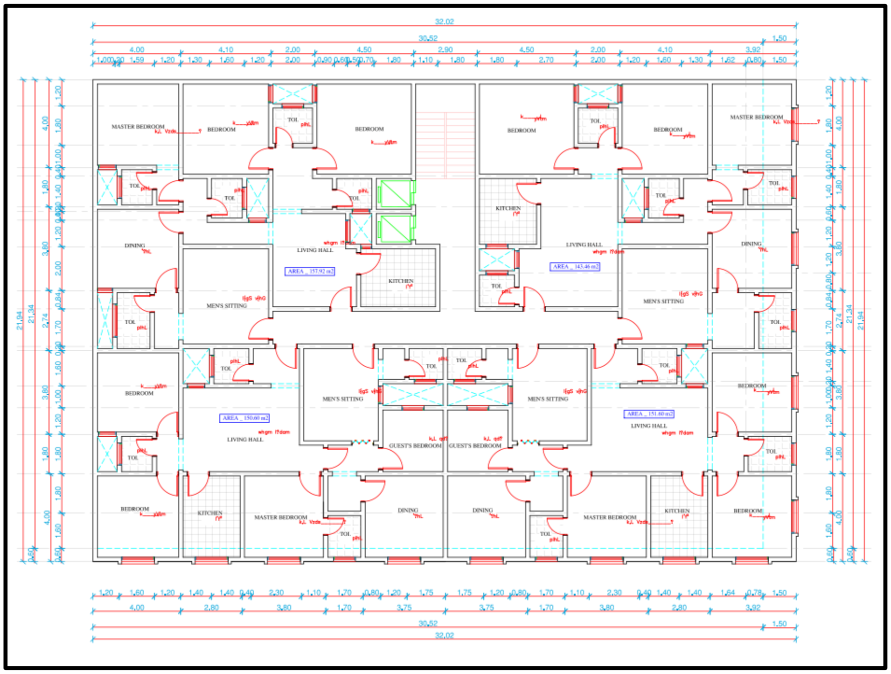


Figure 2.3. Plan of the apartments located in the first to the fourth floors.

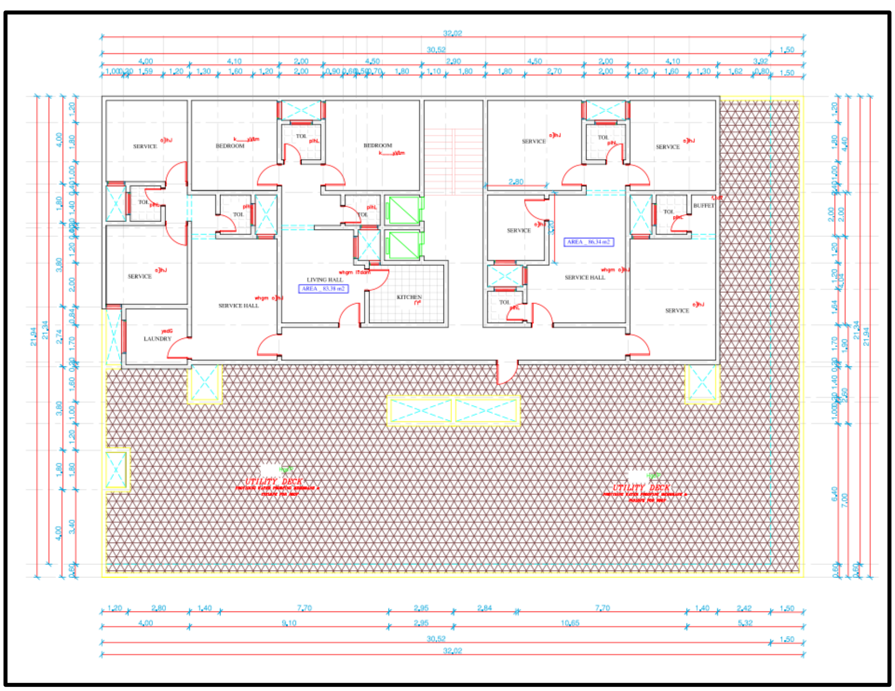


Figure 2.4. Plan of the apartments of the last floor.

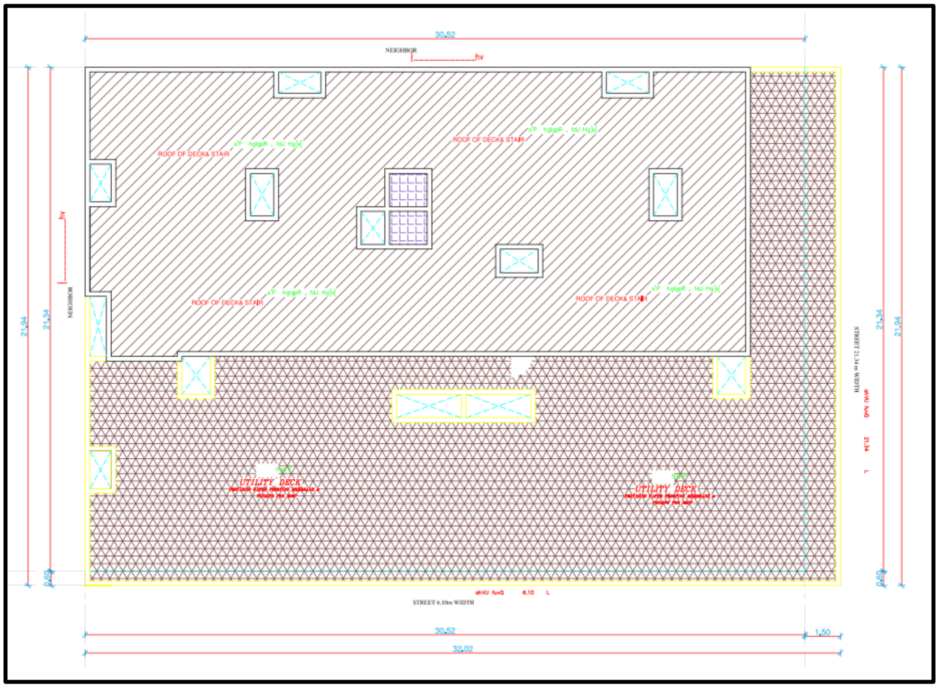


Figure 2.5. Top view of the apartment unit building.

2.4 Structure of the sport center (Gym)

The gym is located in zone A (north area of the compound). It is composed of two floors or stories and it has an average area of about (   ) m^2. In top view, the shape of the gym is in the form of three parallel lines. The gym is provided with one main entrance and an emergency exit located in the back of the building. The height of the building is around 9.0 m. The main parts of the gym are an Olympic swimming pool and a very large room designed for different indoor games such as six side soccer matches, and handball, basketball, and volleyball games.

The gym is provided will many washrooms and bathrooms situated in the ground floor of building. An elevator is also designed in the gym. The building will have a nice view since all its sides will be covered by glass.

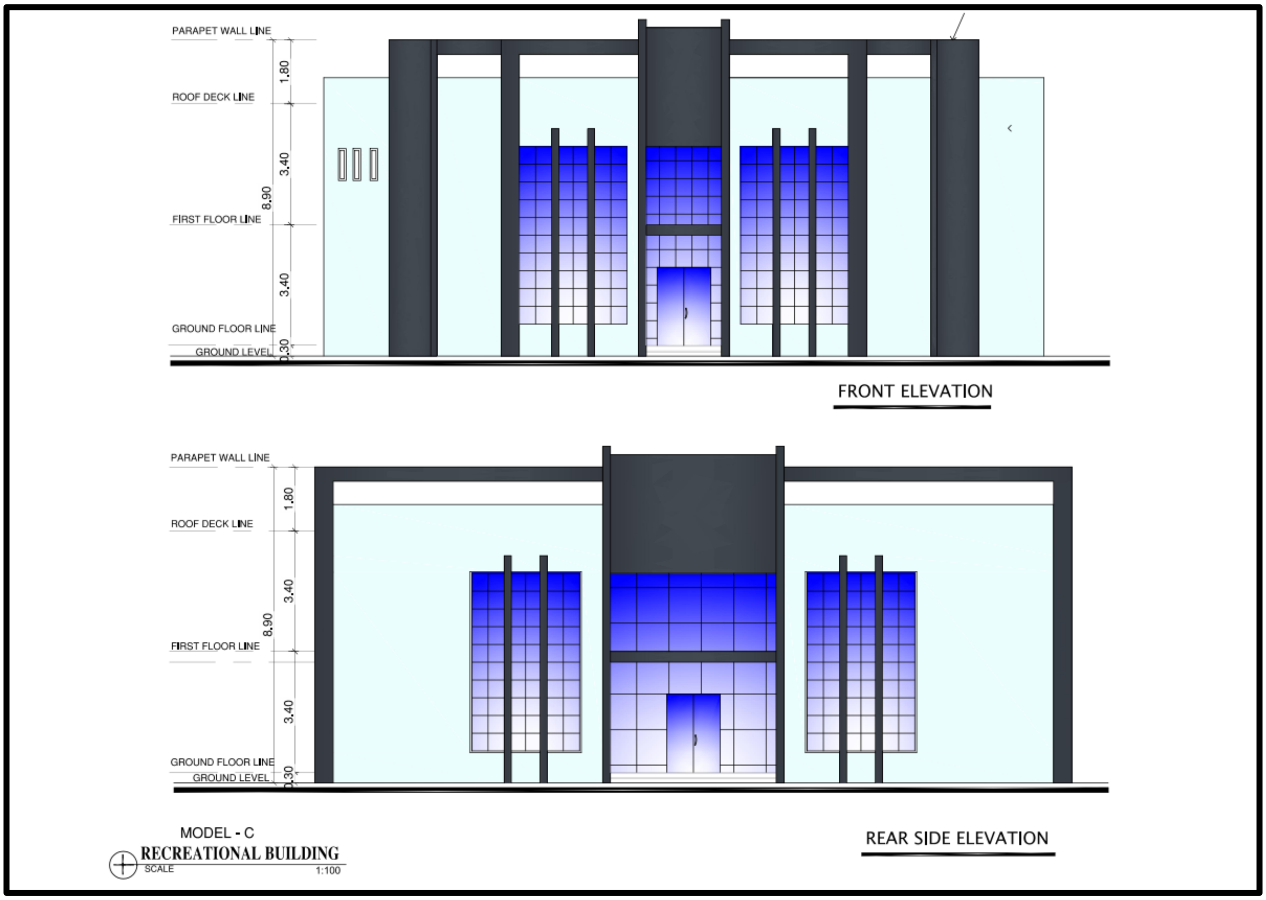


Figure 2.5. Front-side and back-side views of the gym.

2.5 Structure of the mosque

The mosque is also located in zone A (i.e. north side of the compound). Similarly to the gym, it is located in one corner of the compound and it is situated in the opposite side of the gym. The mosque is composed of two floors or stories, and designated for men and women (ground floor for men and first floor for women). The ground floor of the building has a plan area of about 315 m^2 (15m×21m), whereas, the area of the first floor is about half. Obviously, separate bathrooms are designed for each floor. Furthermore, the mosque is provided with a small outside car parking outside. The parking area is also sheared with the user of the gym.

Two large main entrances are designed for the mosque one for men in the ground floor and for women, connected by stairs to the second floor. Moreover, there will a small door in the right side of the mosque for Imam and conferences' guests. The walls of the mosque will be made mainly of small brown bricks and big glass windows located all the sides of the building except the entrance side.

Figure 2.6. Front and side views of the mosque.

2.6 High-rise reservoir

The height of the reservoir will be about 15 m and it will be located on zone C (i.e. in the south side of the compound). The height of the reservoir container or the water tank itself will be about 3 m; while, the structure handling the reservoir container (the water tank) will be almost 12 m. The handling structure is composed by eight cylindrical columns connected to the water tank and joined together by three different rings installed every three meters. The spacing between the different columns will be around 3 m. The water tank has a cylindrical shape with radius of 4 m and height of 3 m. The reservoir is designed to be higher than the different buildings (i.e. apartment units, gym and mosque) by about three meters. The water tank will store continually (i.e. continuous basis) more than 150 m^3 for serving the different buildings and green areas.

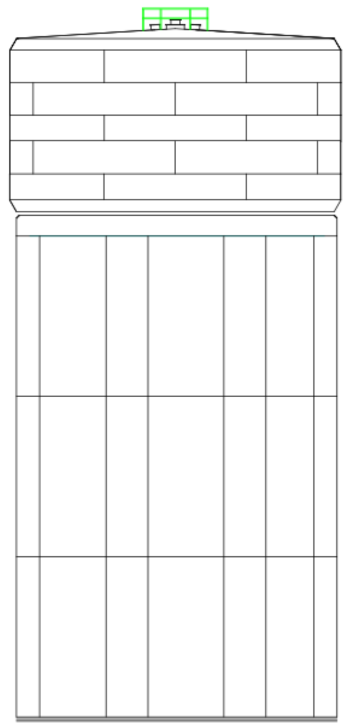


Figure 2.7. Side view of the High-Rise Reservoir.

2.7 Parking and access roads

The road network of the compound is composed by a main road and some access roads. The main road is about 300m long and it has a width of two times four meters. In addition, the road has three different roundabouts to make cars flow and/or return more easily and smoothly. The road is provided with two sidewalks and equipped in its middle by a path design to accommodate some trees and standard and floor lamps. Moreover, many access roads to the different buildings are designed. These secondary roads are design to have a width of four meters. All the roads are equipped with appropriate traffic signs, including signs considering children mobility. It is worthy to note that the compound is provided with two enter/exit gates. In addition, it has two rooms one for security and the other for staff of the compound.

The compound is designed to accommodate four main cars parkings. These cars parking are distributed uniformly all around the compound. Furthermore, two bus stops were provided inside the habitation area.

Chapter 3 : Soil Report

3.1 Introduction

Collapsible soils are vulnerable to large volumetric strains when they become saturated. many soil types fall in the general category of collapsible soils, including loess, a well-known aeolian deposit, present throughout most of the eastern province coast. Loess is characterized by relatively low density and cohesion, appreciable strength and stiffness in the dry state, but is susceptible to significant deformations as a consequence of wetting.

Many natural soils; especially sabkha soils, Aeolian, loess and silty sand with honeycomb structure will experience significant volume decrease upon wetting or rather will display collapsible behavior. Man-made fills, which were compacted at the optimum water content, may also develop a collapsible or metastable structure at low density (Tadepalli and Fredlund 1991). Due to the increasing use of water for irrigation, industrial projects and domestic purposes, foundations built on these types of soil (collapsible) have experienced uncalculated settlement, and accordingly significant damage to these structures.

This chapter is concerned by the description of the site and laboratory investigations, soil report and ground profile of the site of the compound. The information obtained in these investigations are used to design the foundation systems for the different components of the compound.

3.2 Site Investigation

A site investigation is a process to determine the soil profile (i.e. the different layers of the natural soil deposits that underlie a proposed structure) and also their physical properties. Generally, two stages of a site investigation has to be considered, in this case: a preliminary site investigation and a detailed site investigation.

On the preliminary site investigation phase, a few borings are made or a test pit is opened to establish in a general approach the stratification, types of soil to be projected, and possibly the location of the groundwater table. One or more borings should be taken to rock, or competent stratum, if the initial borings indicate the upper soil is loose or highly compressible.

After the preliminary site investigation has established the possibility of the project, a more detailed exploration is undertaken. The preliminary borings and data are used as a source for locating additional borings, which should be confirmatory in nature, and determining the additional samples required.

The approximate required minimum depth of the borings should be predetermined and the estimated depths can be changed during the drilling operation depending on the subsoil encounter.

For the present project, a total of fourteen (14) vertical boreholes with soil sampling were performed and are identified as BH-01 to BH-14.

Drillings were carried out using a drilling machine type Simco assembled on trailer and equipped for ground sampling. Boreholes reached depths between 3.0 m and 11.3 m. During the drilling operation, remoulded samples of soil were obtained at interval of 0.6 to 1.5 m depth.

Moreover, piezometers were installed in the holes of five boreholes (BH-01, BH-02, BH-07, BH-09 and BH-14), before the withdrawal of the casing, for future observation of ground water level.

The locations of the different boreholes are indicated in Figure 1. Somme data from the boring operation are included in Figure 2.

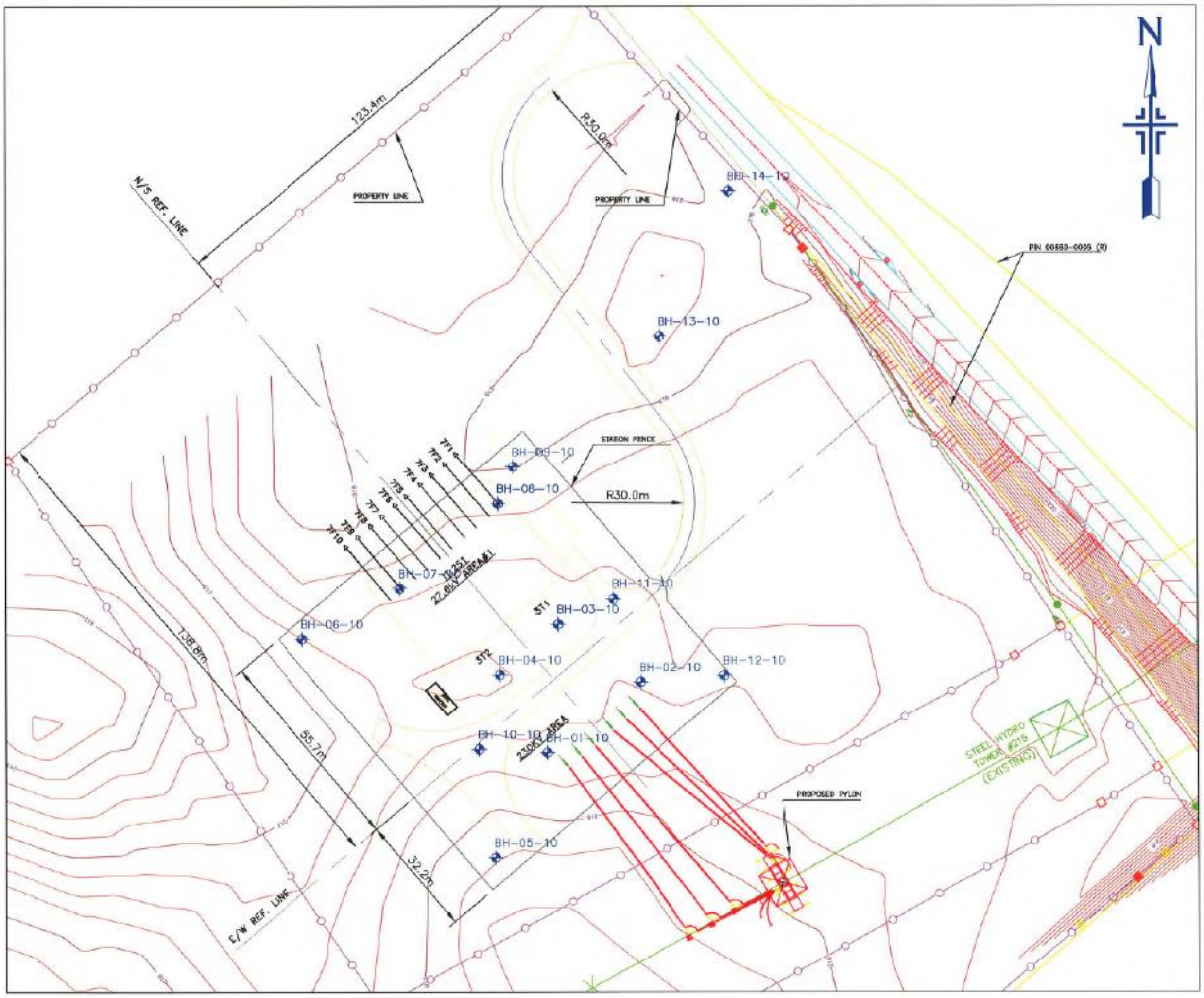
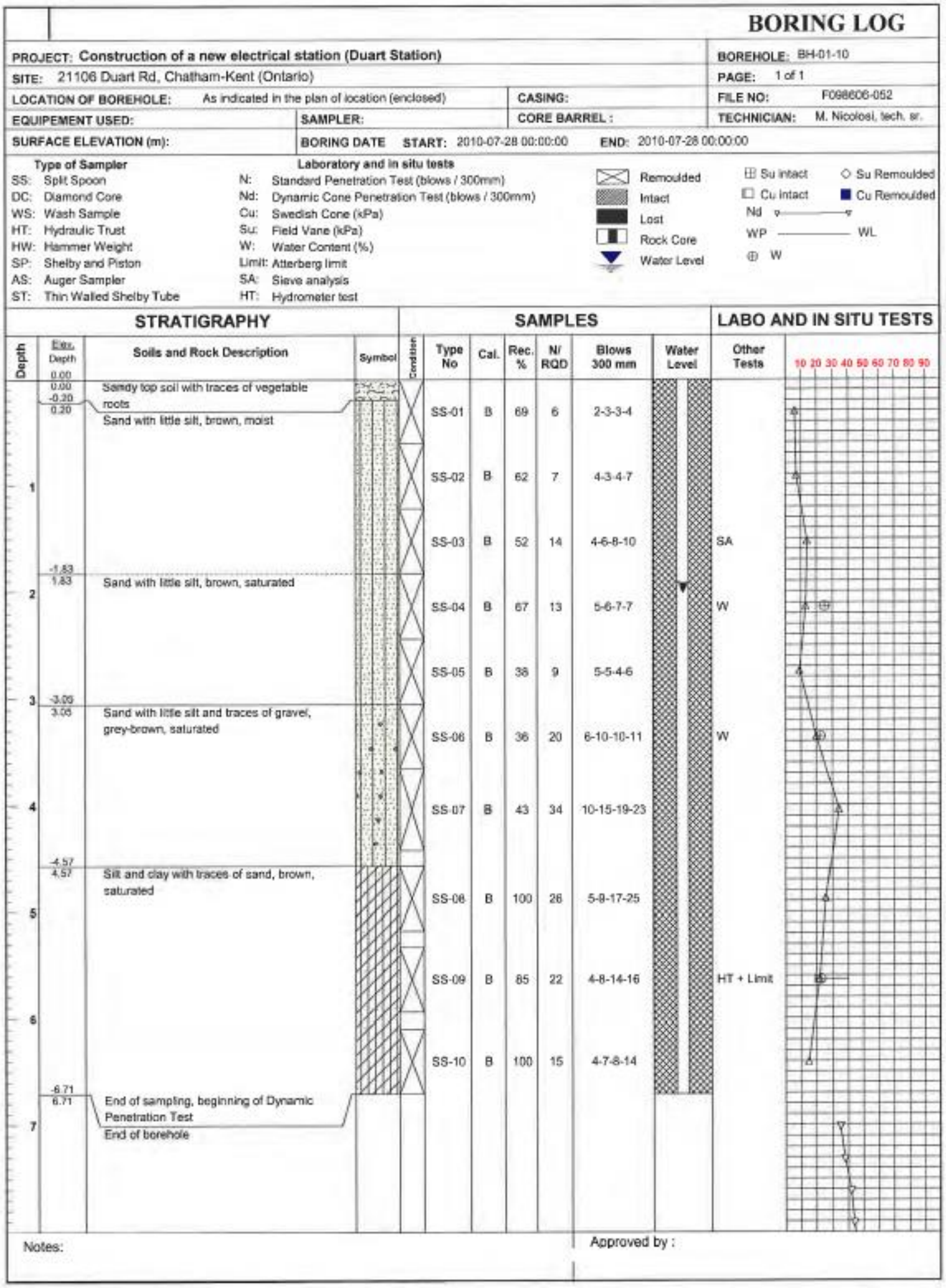


Figure 4.1. Location of the different boreholes


BORING LOG

PROJECT: Construction of a new electrical station (Duart Station)
SITE: 21106 Duart Rd, Chatham-Kent (Ontario)
LOCATION OF BOREHOLE: As indicated in the plan of location (enclosed)
CASING:
EQUIPEMENT USED:
SAMPLER:
CORE BARREL :
SURFACE ELEVATION (m):
BORING DATE START: 2010-07-28 00:00:00 END: 2010-07-28 00:00:00
BOREHOLE: BH-01-10
PAGE: 1 of 1
FILE NO: F098606-052
TECHNICIAN: M. Nicolosi, tech. sr.

Type of Sampler
SS: Split Spoon
DC: Diamond Core
WS: Wash Sample
HT: Hydraulic Trust
HW: Hammer Weight
SP: Shelby and Piston
AS: Auger Sampler
ST: Thin Walled Shelby Tube

Laboratory and in situ tests
N: Standard Penetration Test (blows / 300mm)
Nd: Dynamic Cone Penetration Test (blows / 300mm)
Cu: Swedish Cone (kPa)
Su: Field Vane (kPa)
W: Water Content (%)
Limit: Atterberg limit
SA: Sieve analysis
HT: Hydrometer test

Remoulded
Intact
Lost
Rock Core
Water Level

Su intact
Su Remoulded
Cu intact
Cu Remoulded
Nd
WP
WL
W

Elev. Depth	Soils and Rock Description	Type No	Cal.	Rec. %	N/ RQD	Blows 300 mm	Other Tests
0.00 / -0.20 / 0.20	Sandy top soil with traces of vegetable roots						
	Sand with little silt, brown, moist	SS-01	B	69	6	2-3-3-4	
		SS-02	B	62	7	4-3-4-7	
		SS-03	B	52	14	4-6-8-10	SA
-1.83 / 1.83	Sand with little silt, brown, saturated	SS-04	B	67	13	5-6-7-7	W
		SS-05	B	38	9	5-5-4-6	
-3.05 / 3.05	Sand with little silt and traces of gravel, grey-brown, saturated	SS-06	B	36	20	6-10-10-11	W
		SS-07	B	43	34	10-15-19-23	
-4.57 / 4.57	Silt and clay with traces of sand, brown, saturated	SS-08	B	100	26	5-9-17-25	
		SS-09	B	85	22	4-8-14-16	HT + Limit
		SS-10	B	100	15	4-7-8-14	
-6.71 / 6.71	End of sampling, beginning of Dynamic Penetration Test End of borehole						

Notes:
Approved by :

Figure 4.2. Example of the results of borehole BH-01

All soil samples recovered were sent to the geotechnical laboratory for visual examination and identification by a geotechnical engineer and some tests were performed on representative samples in order to determine the nature and some physical properties of soils.

Investigations or field drillings carried out at the location of the projected compound have revealed several soil deposits with different properties. Generally and for simplicity, the site is made up of three horizons. On the surface, a thin layer made up of sandy top soil followed by a layer of fine to coarse brown sand of approximately 4.5 meter thickness to finish in a stiff layer of clay and silt. The stratigraphy of the site as encountered in the different boreholes is summarized as follows:

Table 4.1. Stratigraphy of the investigated site

Borehole	***Sandy top soil (Deposit A, m)***	***Brown sand (Deposit B, m)***	***Silt and clay (Deposit C, m)***
BH-01	0.0 – 0.20	0.20 – 4.57	4.57 – > 6.71
BH-02	0.0 – 0.20	0.20 – 4.57	4.57 – > 6.71
BH-03	0.0 – 0.30	0.30 – 3.99	3.99 – > 11.28
BH-04	0.0 – 0.30	0.30 – 4.42	4.42 – > 11.28
BH-05	0.0 – 0.20	0.20 – > 3.05	-
BH-06	0.0 – 0.20	0.20 – > 3.05	-
BH-07	0.0 – 0.20	0.20 – > 3.05	-
BH-08	0.0 – 0.20	0.20 – >3.05	-
BH-09	0.0 -0.20	0.20 – 5.61	5.61 – >6.71
BH-10	0.0 – 0.20	0.20 – > 3.05	-
BH-11	0.0 – 0.20	0.20 – > 3.05	-
BH-12	0.0 – 0.22	0.22 – > 3.05	-
BH-13	0.0 – 0.20	0.20 – 2.60	2.60 - > 3.05
BH-14	0.0 – 0.30	0.30 – 5.33	5.33 – > 6.71

Deposit A: Sandy top soil

At the ground surface, a thin sandy layer with some vegetable roots was identified in almost all the boreholes performed in the area of the project. The thickness of the sampled layer varied between 0.20 and 0.30 m. It is constituted generally by a matrix of brown sand mixed with some organic matter and vegetables roots in different proportions.

Deposit B: Brown sand

A thick deposit of fine to coarse, brown and dry to saturate homogenous sandy soil was encountered below the deposit A in all the boreholes. The thickness of the sampled layer varied between 2.4 m and 5.4 m. The value of the index of the standard penetration test in all layers is situated generally between 2 and 37, indicating that the soil is generally in loose to medium-dense state.

Sieve analyses were performed on samples taken from this layer of brown sand. The results of the particle size distributions indicated that the soil is composed of 0 to 13% of gravel, 60% to 94% of sand, and about 8% to 26% of silt (particles less that 80 μm). Generally, the soil of this layer can be classified as sand with some silt and traces of gravel.

Deposit C: Silt and clay

A thick layer of silt and clay, brown to grey, mainly saturated was encountered in some boreholes (see table 1). The thickness of the sampled layer varied between 0.4 m and 7.3 m. In the exception of boreholes BH-13, the different values of the index of standard penetration test *N* are generally in the range of 12 to 28, which indicate that the deposit C is stiff to very stiff.

Hydrometer tests were carried out on different samples taken from the layer of silt and clay. Generally, the results obtained indicate that the soils encountered in this layer are composed of 4% to 20% of sand, 42% to 58% of silt (particles less than 80 μm) and 30% to 53% of clay (particles less than 2 μm). The soils can be classified as silt and clay with little/traces of sand.

According to the results of consistency tests carried out on samples of silt and clay, the liquid limit of these soils is in the range of 29% and 40%, and the plastic limit is in the range of 15% and 19%. Then, the moisture content values are lower than the liquid limit, which indicate that the silt and clay is not sensitive. Based on these results, the silt and clay soil is classified as soil of type CL2 (low plasticity).

The ground water level was monitored by five (5) piezometer installed in some boreholes (see Table 2). The different water levels measured on the 29th of July are indicated in the following Table:

Table 4.2. Ground water level

Measurement points	***Water level (m)***
BH-01	15.96
BH-02	16.86
BH-07	14.98
BH-09	18.65
BH-14	17.82

3.3 Laboratory works

Five collapsible soil samples (noted A, B, C, D and E) were taken from the site to the laboratory for testing. The soils are composed of a mixture of sand (quartz/angular grains) and clay (kaolinite). The physical and the mechanical properties of these soils are given in Table 1*a*, their mineralogical compositions in Table 1*b*, and the grain-size distributions are shown in Figure ( ).

Soil	**Uniformity coefficient (C_u)**	**Liquid limit (w_L, %)**	**Plastic limit (w_P, %)**	**Plasticity index (*PI*, %)**	**Optimum water content ($w_{opt.}$, %)**	**Maximum unit weight (kN/m^3)**
A	33.4	18.3	11.0	7.3	8.5	20.8
B	45	22.5	13.3	9.2	9.1	20.2
C	57	26.7	16.1	10.6	10.3	19.7
D	78	30.3	18.9	11.4	11.5	19.0
E	110	33.8	20.3	13.5	12.2	18.5

Table 4.3. Physical Properties of soils tested in the project

Clay		**Sand**		**Soils tested**		
Constituents & properties	(%)	Constituents (%)	(%)	Type	SiO_2	Al_2O_3
SiO_2	47	SiO_2	94.0	A	89.7	4.2
Al_2O_3	38	Al_2O_3	0.88	B	87.6	5.9
Grain size ≤ 2 μm	85	Fe_2O_3	0.37	C	86.0	7.2
Specific gravity	2.6	CaO	2.96	D	84.5	8.3
Surface are (BET, m^2/g)	14	MgO	0.11	E	83.2	9.4
pH	5.5					
water-soluble salts content	0.2					

Table 4.4. Mineralogy of the different soils used

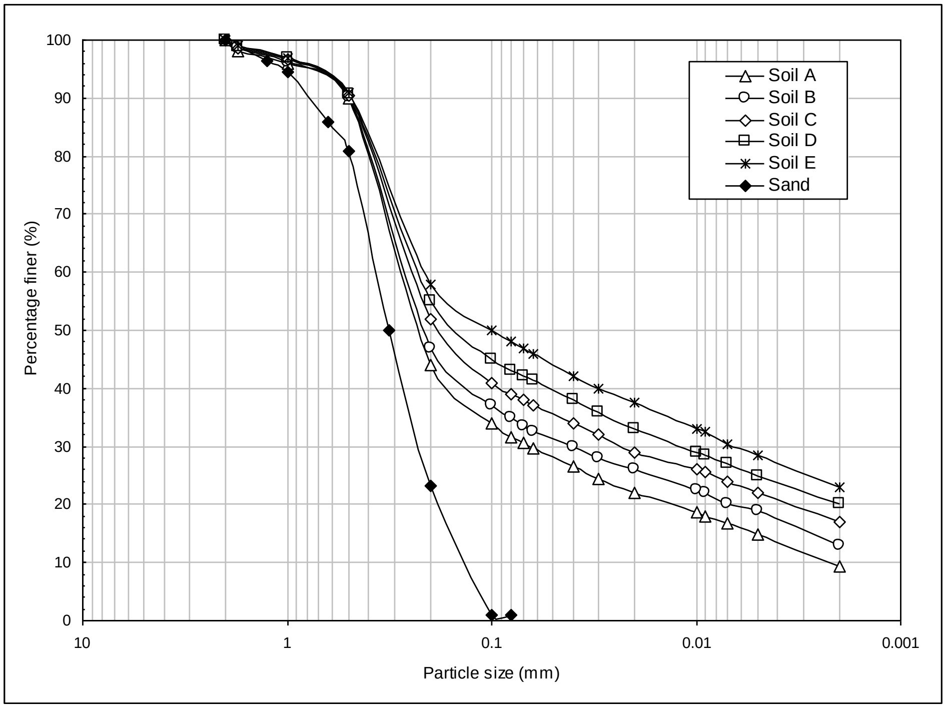


Figure 4.3 . Particle size distribution of the different samples tested

Consolidation-collapse tests were performed on these samples in order to assess their collapsibility. Specimens’ 50.8 mm in diameter and 20 mm in height were taken from these samples and tested in the oedometer apparatus.

Once the soil specimen was placed in the ring, the load was then gradually applied up to 200 kPa, at which point the soil was flooded with distilled water and left for 24 hours. Dial gages were mounted on the topof the specimen for measuring the vertical displacement. The results of the consolidation tests performed are presented in the following figures:

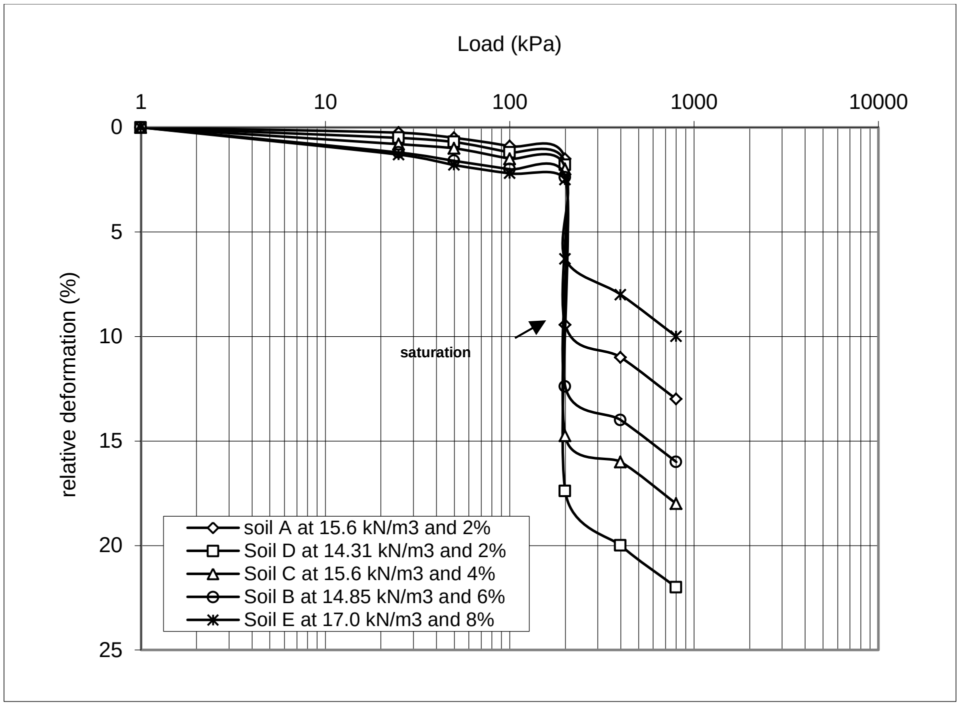


Figure 4.5 . Consolidation test results of the different samples (part I)

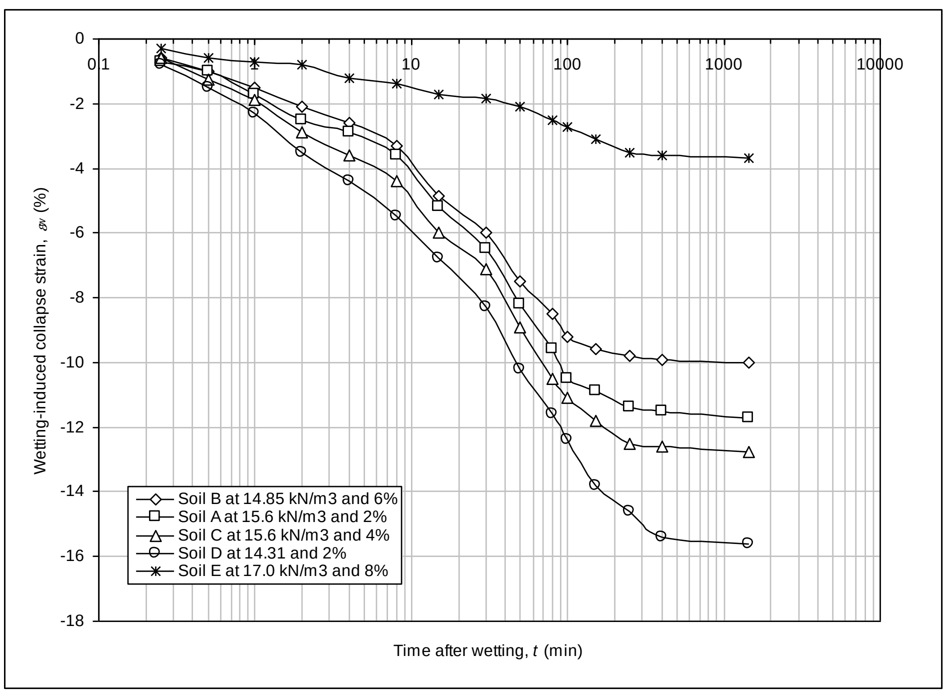


Figure 4.6. Consolidation test results of the different samples (part I)

3.3 Soil Profile

Based on the results of the site and laboratory investigations, the soil profile of the site of the project can be characterized by three different zones as follows (Figure 4.7).

- Zone A (i.e zone nord): In this zone the depth of the collapsible soil was found to be in the order of four (4) meters. In this zone the rock was intercepted at about 4.0 m measured from ground surface.
- Zone B (i.e. zone situated in the middle of the site): The depth of the zone is averaged to be about eight (8) meters,
- Zone C (i.e. zone south): the rock in this zone is located at about ten (10) meters. The soil in this zone is also characterized by a collapsible behavior and its depth is around 10 meters.

CHAPTER 4: STRUCTURE DESIGN

4.1 Introduction

This chapter covers the structural design of the different habitation elements of the compound. A computer program namely SAP2000 (Structural Analysis Program 2000) is used for this purpose. The design will be performed in the following order: apartment units, the Gym and the mosque, and then the high-rise reservoir. The design is concerned mainly by determining the appropriate steel reinforcements for the different concrete elements and the solicitations which will be transmitted to the foundation systems.

4.2 Apartment Units

As indicated in chapter 2, the apartment units are composed of five floors or stories. Each unit will cover an area of 21m×32m. The dead and live load considered for the design area taken from the Saudi Building Code (SBC). The live load is taken to be 5 KN/m^2. However, a factor of safety of 1.6 is applied on the live load; while, a factor of 1.2 is applied on the dead load. In addition, all safety factors are calculated automatically in SAP2000.

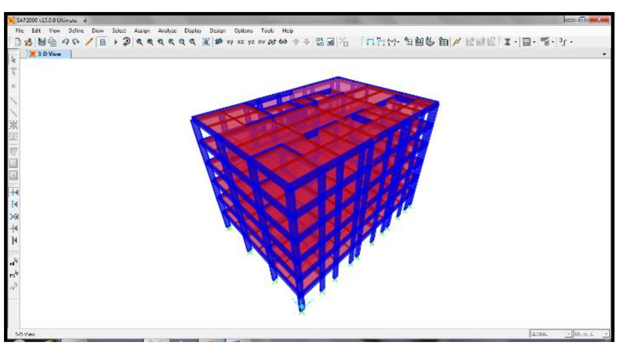


Figure 4.1: Design of apartment unit (SAP2000).

All columns are considered to be connected (from the ground to the fifth floor). The following date are used in the SAP design: i)- The columns are identical and have a cross section of 80 cm×30 cm, ii)- 7 #18 are considered in the longest side and 3 #18 in the shorter side, iii)- The concrete is of the type 4000 psi, iv)- The tie reinforcement is taken #4 every 15 cm, and v)- The reinforcement of the columns is considered the same in the vertical direction from ground to fifth floor

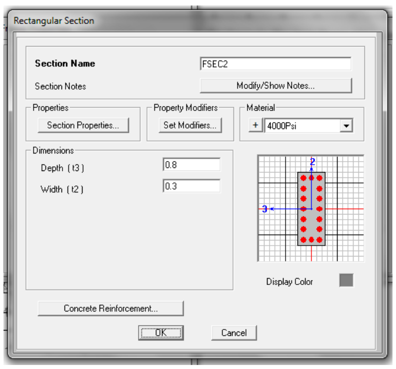


Figure 4.2: Reinforcement of the columns.

All the area is designed in plane-stress with a concrete of the type 4000 psi concrete, a thickness of 28 cm, and the load is considered acting or distributed over the entire area.

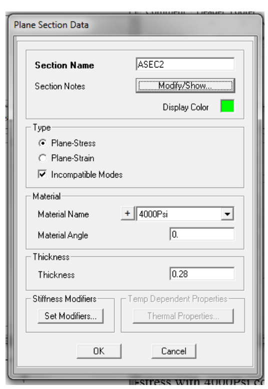


Figure 4.3: Plane section data.

The results obtained from the software SAP2000 are represented as follows:

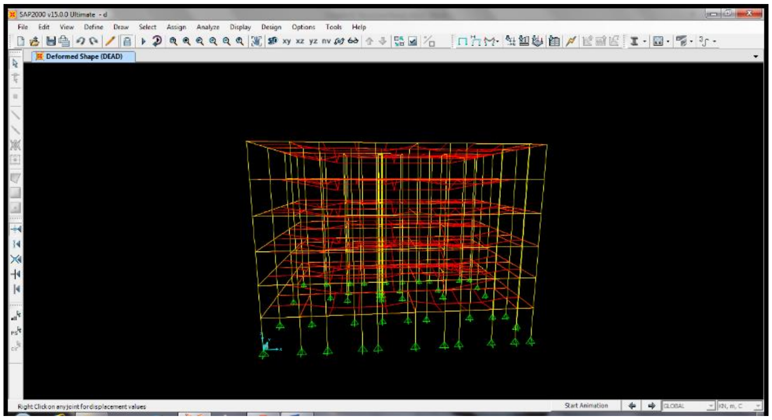


Figure 4.3: Deformation Shape (SAP2000)

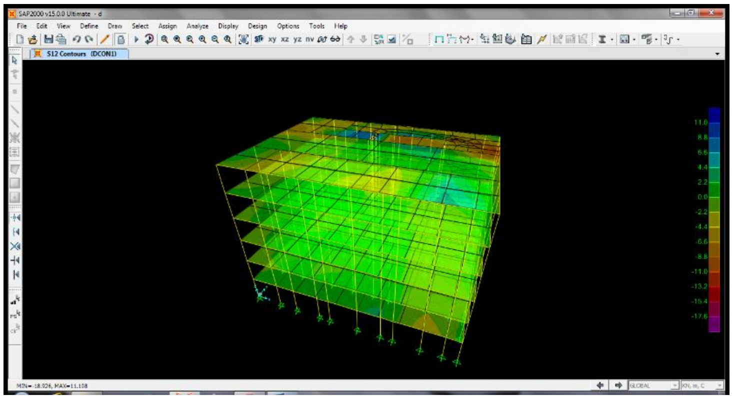


Figure 4.4: Stress in the Slabs (SAP2000)

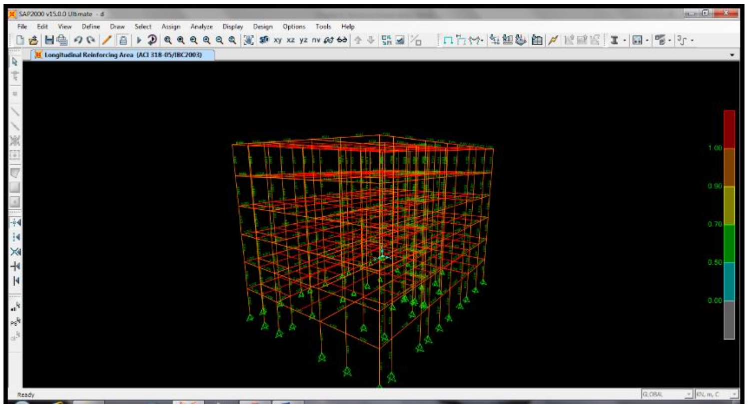


Figure 4.5: Verification of the columns (SAP2000).

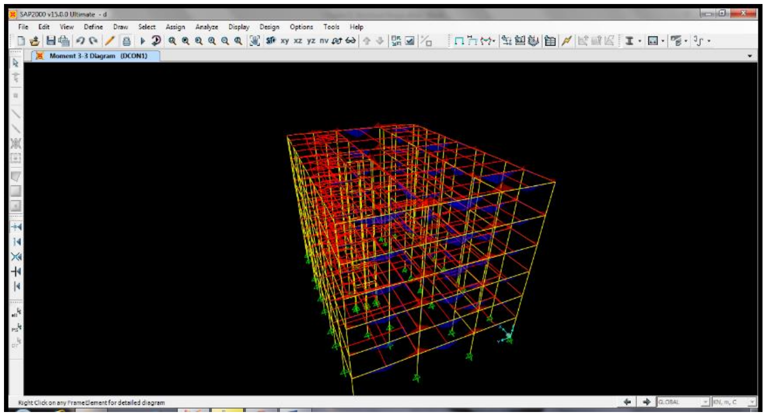


Figure 4.6: Moment in the Beams (SAP2000)

4.3 Design of the Gym

The gym is a two floor (stories) structure. It is covering an area of about 560 m^2. The dead load on the Gym is set automatically by the computer software SAP2000; whereas, the live load is taken to be 4 KN/m^2 and considered distributed over the area. Obviously, all the loads are transferred to the beams and then to the columns.

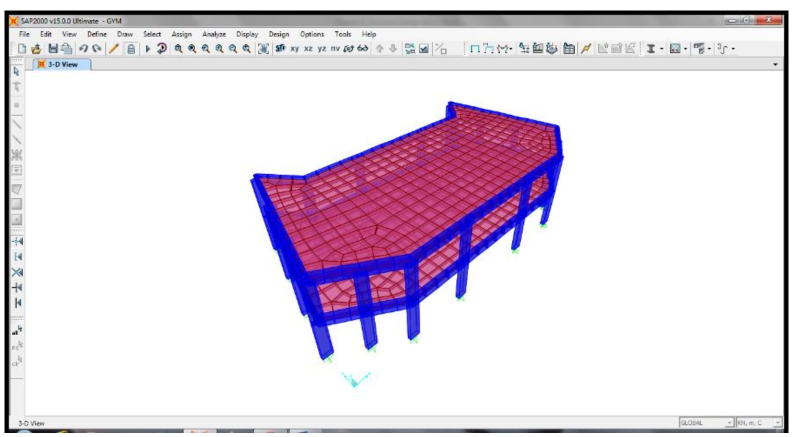


Figure 4.7: Design of Gym #1 (SAP2000)

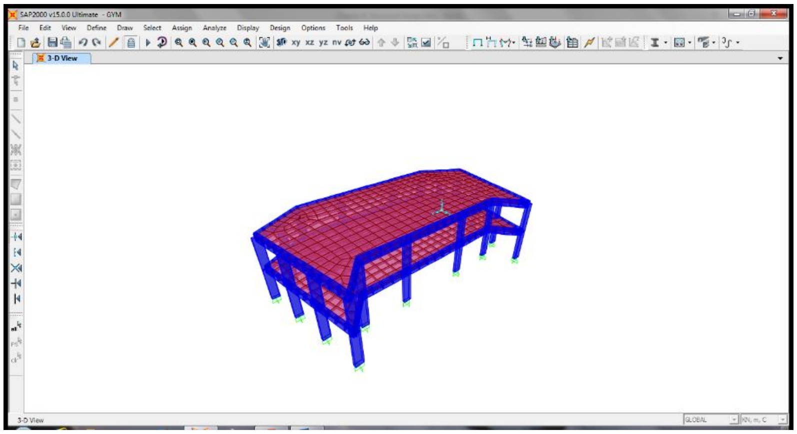


Figure 4.8: Design of the Gym #2 (SAP2000).

Similarly to the apartment units, the columns, in the ground and first floor, are designed to have a cross section area of 80 cm×30 cm. They are taken to have 4 #11 in the longest side and 3 #11 in the shortest side. The tie reinforcement is considered to be #4 every 15 cm.

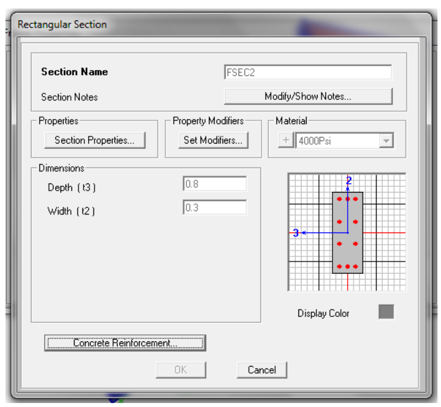


Figure 4.9: Gym's Columns design (SAP2000).

The thickness of the Gym slabs is 25 cm and is made of a concrete corresponding to the type 4000 psi. The live load is considered to be 4 kN/m^2 distributed over the entire area.

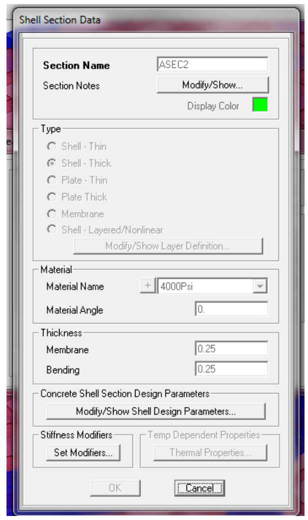


Figure 4.10: Gym Shell Section Data (SAP2000).

The following figure shows the results obtained from SAP2000 corresponding to the assessment of collapse failure of the Gym structure. It is clear from this figure (based on the orange color) that all the columns are designed with an adequate safety and cost. Moreover, it is indicated that all the beams can handle the applied loads which are safely transferred to the slabs.

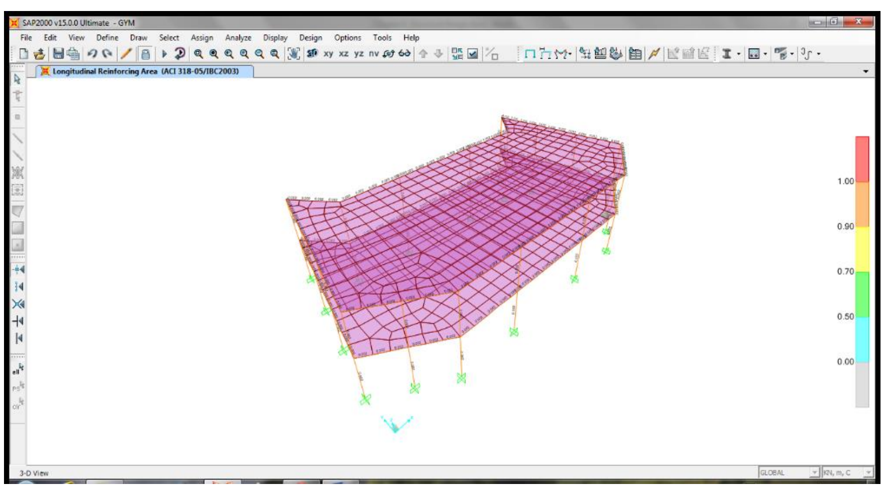


Figure 4.11: Results of the assessment of stability of Gym columns and Beams (SAP2000)

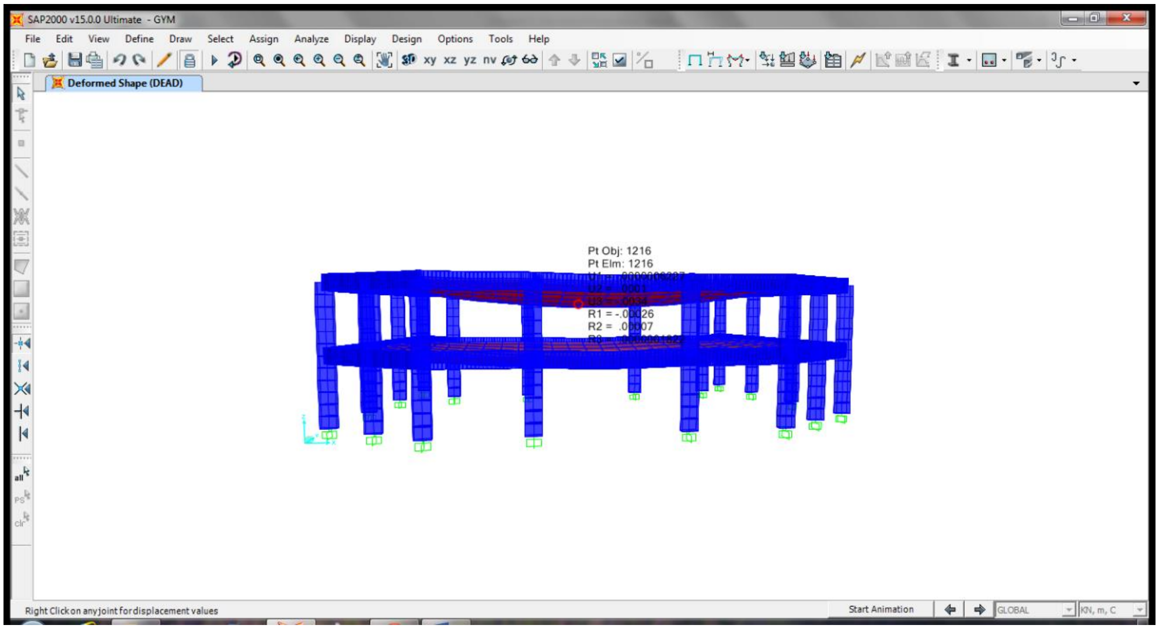


Figure 4.12: Results of deformation shape (SAP2000)

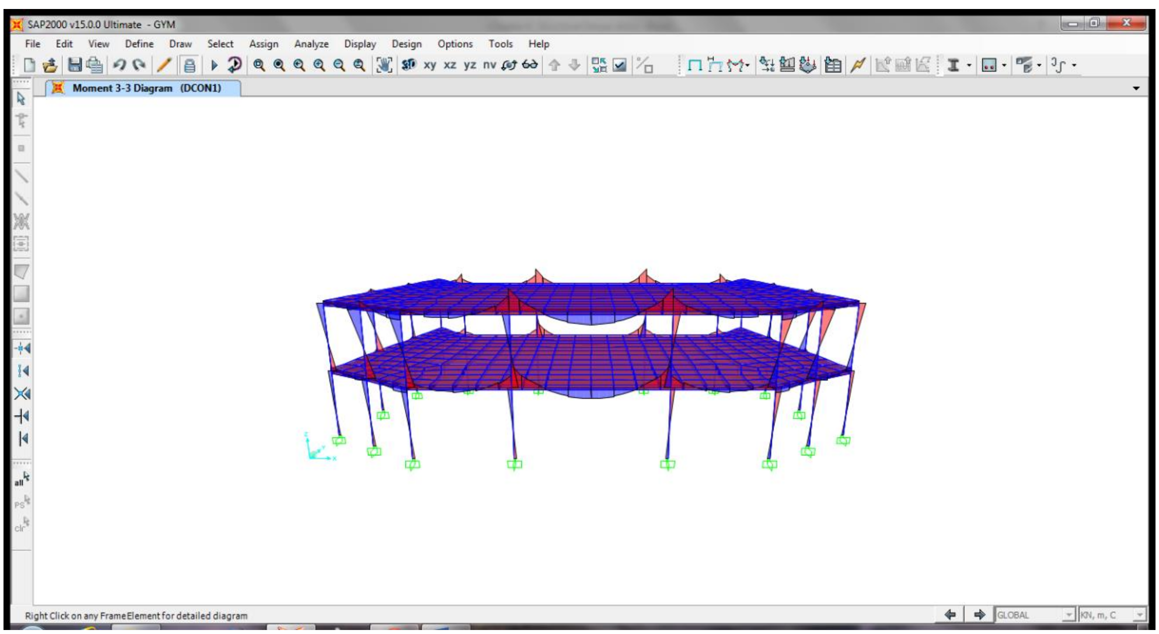


Figure 4.13: Distribution of the moment in the Gym beams (SAP2000).

4.4 Design of the mosque

Similar to the Gym, the mosque is design as a two stories structure extending over an area of 550 m^2. Figure 4.14 shows the configuration of this structure. The same set data, used for the Gym, is also used in designing the mosque. The live load is taken to be 5 kN/m^2 as required by the SBC, whereas, the dead load is introduced by the software SAP2000. The load are considered to be distributed over the entire area.

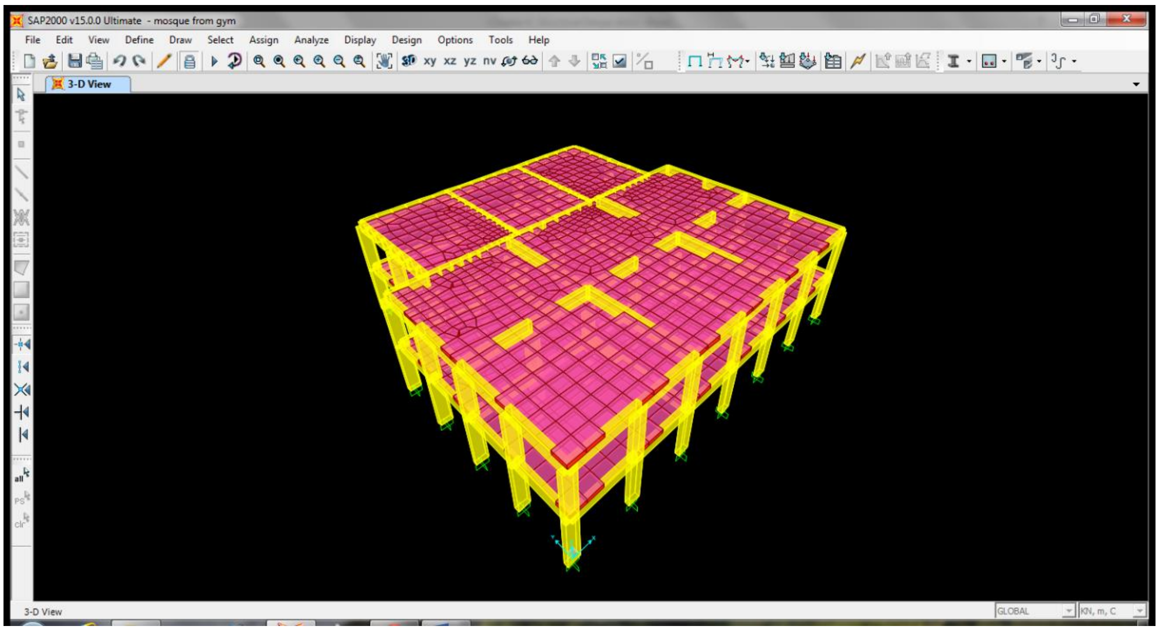


Figure 4.14: Mosque overview #1 (SAP2000).

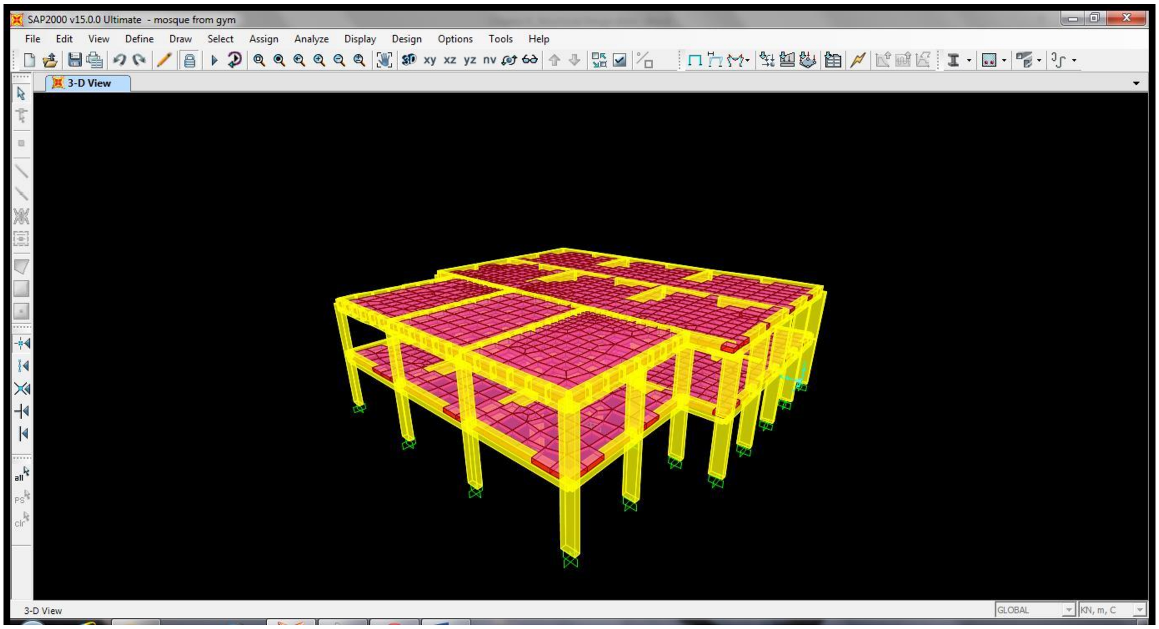


Figure 4.15: Mosque overview #2 (SAP2000).

The cross section of the columns are also taken to be 80cm×30cm and has a reinforcement of the type 4 #11 in the longest side and 3 #11 in the shortest side. The tie reinforcement is taken to be #4 every 15 cm long. For the slabs, the same type of concrete is used in this case a concrete of 4000 psi with a thickness of 25 cm. The dead load distributed over the entire area is considered to be 4 kN/m^2.

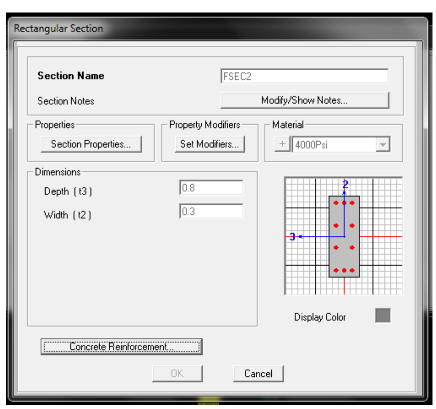


Figure 4.16: Reinforcement of the Mosque (SAP2000).

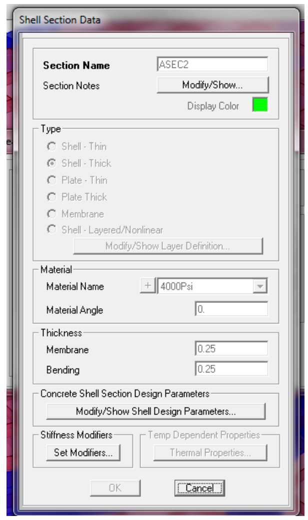


Figure 4.17: Mosque Shell Section Data (SAP2000).

Verifications using SAP2000 were carried out concerning the mosque. At the beginning, the results obtained showed some discrepancies because the loads were huge and there were not a sufficient number of columns to handle these loads (even with columns having large cross section size). However, by increasing the number of columns, notably in the middle area of the mosque, adequate results were obtained. The following figures show some of these results.

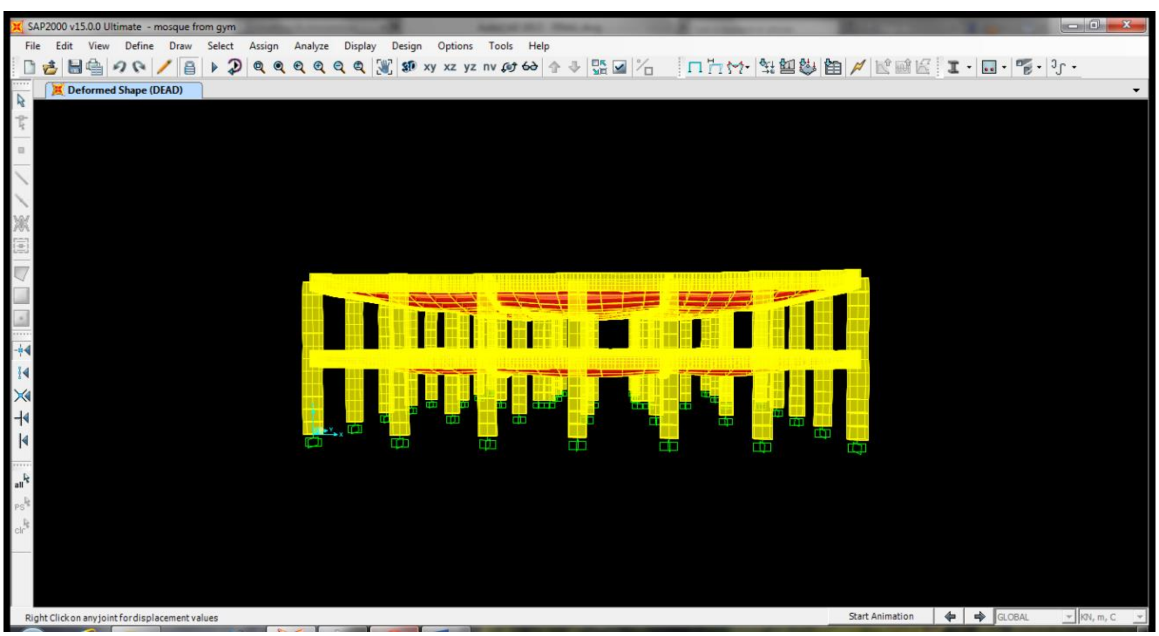


Figure 4.18: Deformation shape of the Mosque (SAP2000).

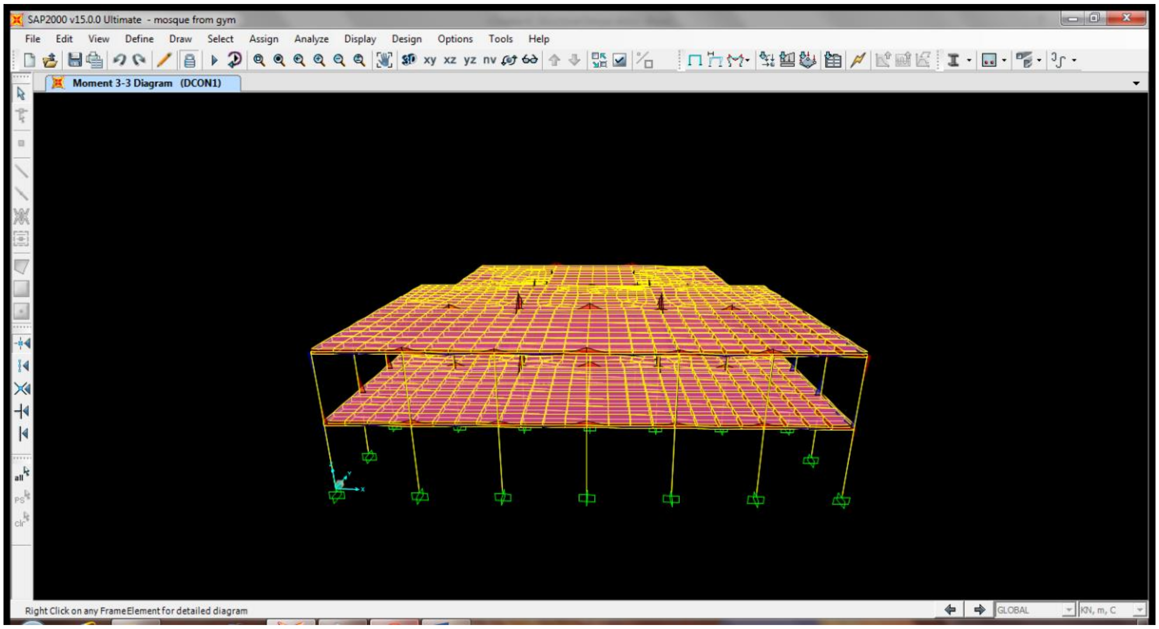


Figure 4.19: Diagram of moment in the different beams of the mosque (SAP2000).

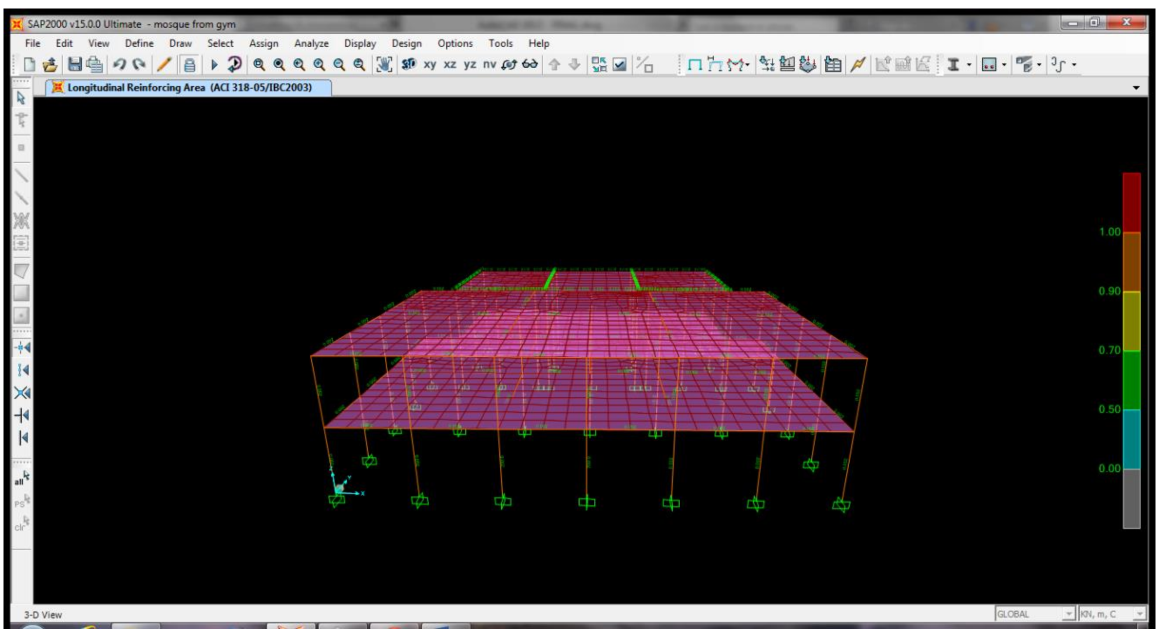


Figure 4.20: Verification of the mosque (SAP2000).

4.5 Design of the high-rise reservoir

As indicated in the previous chapters, the diameter of the high-rise reservoir is about 8.0 m. The loads of the reservoir are supported by eight columns. Moreover the columns of the reservoir are divided into three parts separated by a distance of about 4.0 m. They (i.e. the columns) are connected to each other by two circular beams every 4.0 m. The height of the water tank is about 3 m.

The tip of all the columns is connected to the pile cap at a depth of 1.5 m. The load is considered to be distributed over the entire area of the water tank. Then, the load of the concrete and reinforcement of the water tank is calculated and added to the water load. Following that the load is distributed on the different beams, then to the columns to reach finally the foundation system.

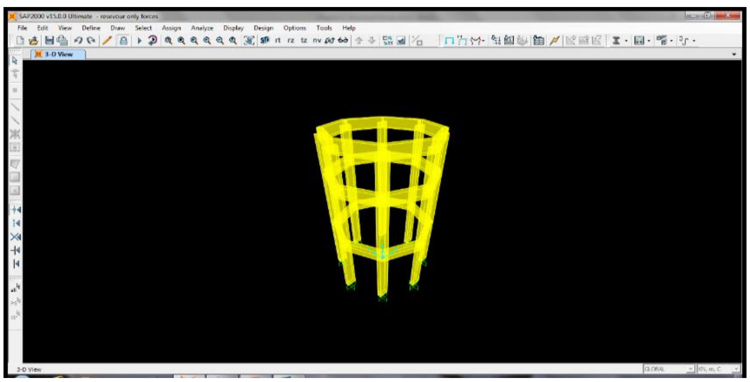


Figure 4.21: Design of the High-rise Reservoir (SAP2000).

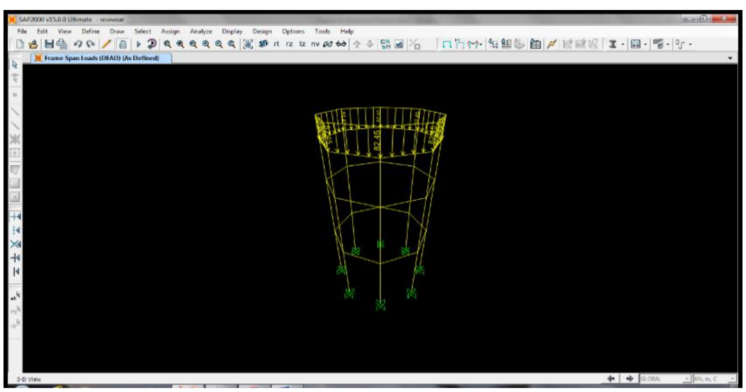


Figure 4.22: Forces Assigned to the op beams (SAP2000).

The cross section of the columns is taken to be 30cm×80cm with a reinforcement of 3 #11 in both directions; while, the tie reinforcement is taken to be will be #4 every 15 cm. The cross section of the beams is considered to be 80cm×40cm with a reinforcement of 5 #11 in the longest side and 3 #11 in the shortest side.

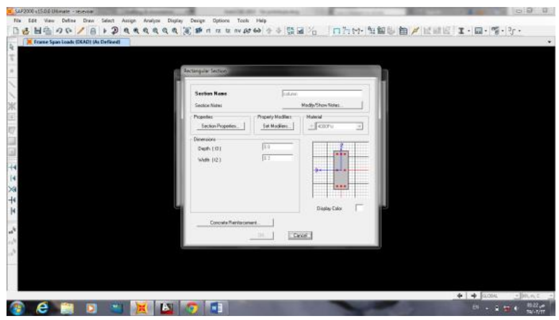


Figure 4.23: Design of the columns of the reservoir (SAP2000)

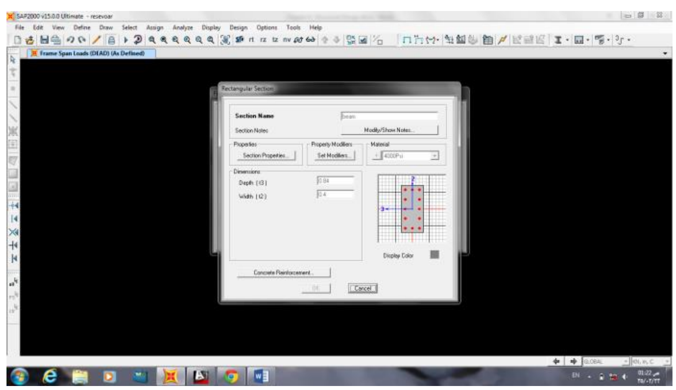


Figure 4.24: Design of the beams of the reservoir (SAP2000).

The results of the verification using SAP2000 are shown in the following figure:

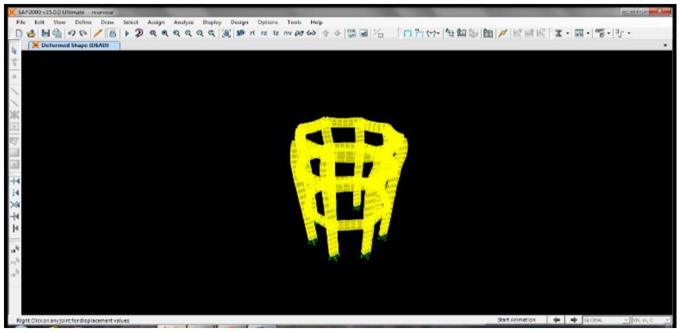


Figure 4.25: Deformation shape of the reservoir (SAP2000).

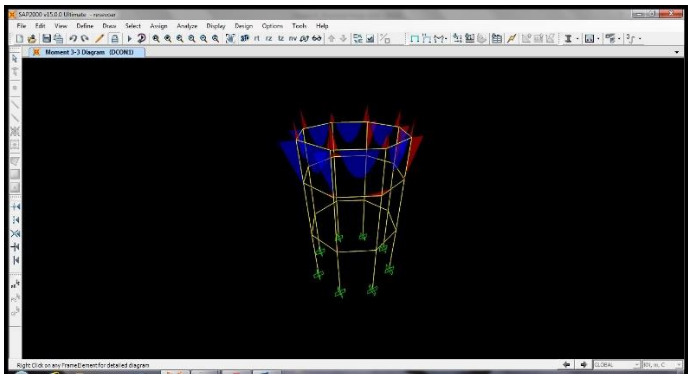


Figure 4.26: Diagram of moment of the beams of the reservoir (SAP2000)

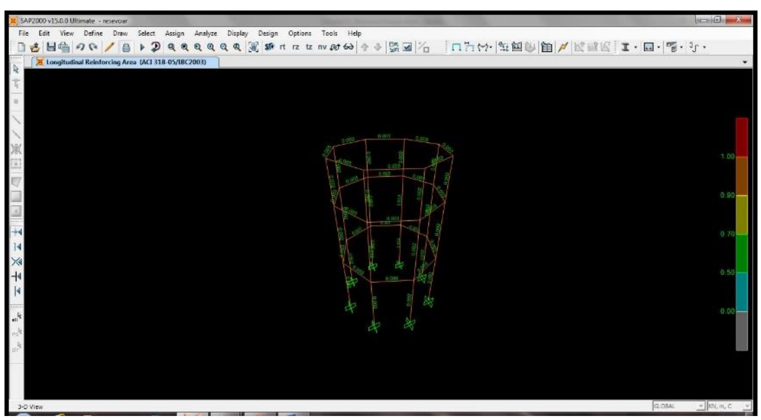


Figure 4.27: Verification of the reservoir (SAP2000)

CHAPTER 5: GEOTECHNICAL DESIGN

5.1. Introduction

Based on the fact that the soil deposits intercepted on the site is potentially collapsible, the loads of the new structure must be transmitted to the ground as follows as follows:

1- Shallow foundations placed on the area subjected to soils replacement (i.e. area of the mosque and the Gym → area of zone A).
2- Raft foundations for the different apartment units founded on the ground subjected to dynamic compaction (i.e. zone situated in the middle having 8.0 m depth → zone B).
3- Deep foundations for the reservoir placed in zone C (i.e. the zone where the depth of the collapsible soil is about 10.0 m)

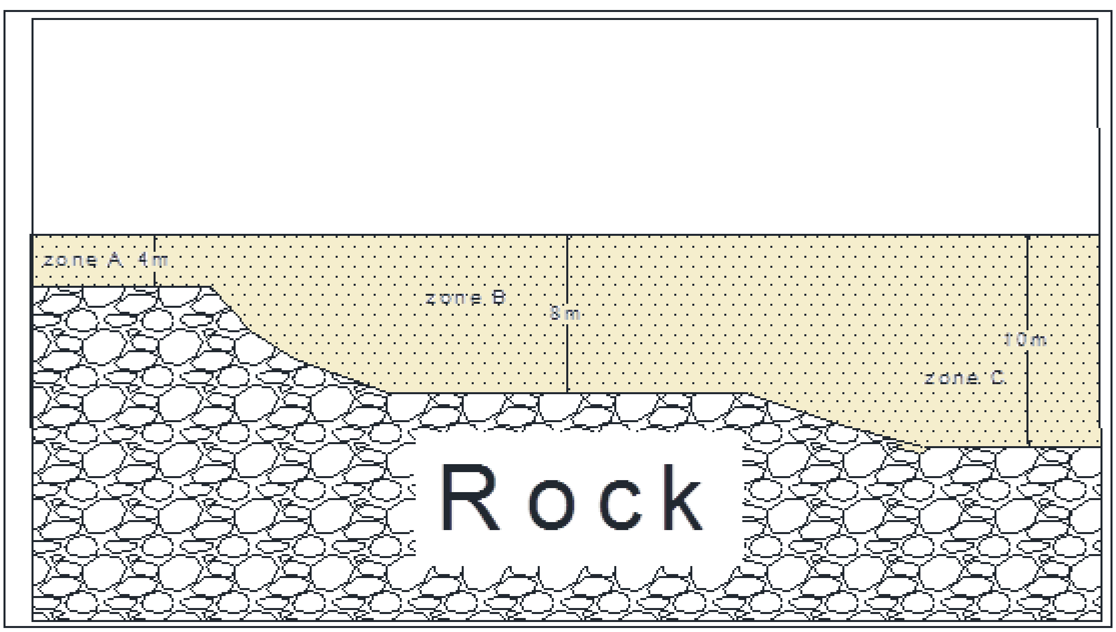


Figure 5.1. Soil profile of the site of the project.

This chapter is concerned by the geotechnical design of the different elements of the compound. The design will include the selection, calculation and verification of the appropriate foundation system for the different components.

5.2. Proposed Solutions (Appropriate Foundation Systems)

As mentioned in the previous chapters, the main objective of this project is designing a habitation compound in a difficult soil such as sabkha soils. In addition to the structural design, a geotechnical design of the different elements of a compound, such as apartment units, sport center, mosque, high rise water reservoir and local roads & parking are also required.

5.2.1. Excavation and replacement of collapsible soil

In the case where the top layer of the ground is composed of 4.0 m or less of collapsible soil, the ground is completely excavated and replaced by granular soil which could be have a spread particle size distribution. The excavation should be performed within the first 4.0 to 5.0 m until reaching the rock. The surface of the rock layer will have to be as horizontal as possible, and a slope of 10% will be accepted as maximum limit. The granular fill should be placed in layers of 300 mm thickness and compacted to at least 95% of the maximum dry density of the material measured in laboratory by means of the modified Proctor test (P.M.). The compacted fill should be extended under the foundation with a slope of 1.0V; 1.0H up to the base of the excavation.

Worthwhile to note that, it is required to embed the foundations to a minimum depth of 1.5 m. If these conditions are respected, a bearing capacity of 300 kPa could be used for design of the footing. These values included a safety factor of 3 against any failure and limit the total settlement to 25 mm and the relative settlement to 20 mm.

5.2.2. Ground improvement by dynamic compaction

For the middle part, where the layer depth is about 8.0 m, this ground improvement technique can be used as an alternative for stabilizing the site of the project. By considering the nature and grading of the brown sand layer, and the level of groundwater table, we could consider dynamic compaction technique, described herein, to eliminate or to reduce, to an acceptable extend, the collapsible potential of this layer.

The collapsible sand layer could be treated in place by dynamic compaction which improves its geotechnical properties. The method consists in ramming the ground surface by a series of impacts of strong magnitude or intensity generated by means of concrete or steel mass dropped in freefall (Canadian Handbook of Engineering Foundations, 1994).

Figure 5.2. Process of dynamic compaction (#1)

Figure 5.3. Process of dynamic compaction (#2)

The bearing capacity should be evaluated in place before and after dynamic compaction to verify the efficiency of the treatment by means of dynamic penetration tests or any other in situ evaluation method of soil bearing capacity. It should be noted that it is necessary to guarantee a bearing capacity of 200 kPa for the projected structures. Moreover, it is necessary to guarantee total settlement less than 25 mm under working loads.

By considering the vibrations caused by dynamic compaction, we recommend that, adjacent constructions, if any, are inspected before and after the beginning of work to monitor any cracks.

5.2.3. Piles foundations

The foundation of the reservoir can be supported by driven piles embedded in the bed of stiff to very stiff silt and clay up to the rock in order to give the required bearing capacity.

However, the structural loads applied on the piles must be increased by the load induced by negative skin friction. Negative skin friction will induced by collapse of the ground and will be caused by excessive settlement of soils situated above the water table. Preliminary calculations showed that the value of this friction is around 30 KN for a pile of 0.3 m in diameter. Nevertheless, this negative friction could be reduced to an acceptable level if the piles are encapsulated in smooth casing or covered by a layer of bitumen.

5.2.4. Stability of excavated area

If the excavated area is adequately drained, any excavation projected up to 3.0 m depth underground surface, should be performed under a slope of about 1.0H: 1.0V profiled in the layer of brown sand. Beyond this depth and up to 5.0 m, a slope of 2.0H : 1.0V is recommended. An excavation in steps is also a possibility. Nevertheless, if the extent of the excavated area causes constraints for installations or if the state of the ground in place is problematic, it is advised to use retaining structures (temporary retaining walls such as Berliner walls or sheet piles) for excavation works.

Slopes of the excavated area should be sheltered by fabrics of polythene or mattresses, firmly anchored in the ground in order to avoid soil erosion by rainwater. One should also avoid circulating or storing material on these slopes throughout all construction stages.

It is important to mention that slopes given above will have to be adjusted according to the nature of the ground and local conditions of water table and according to any sign of instability which could be detected during excavation works.

5.3 Foundation system for apartment units

As mentioned previously, the foundation system of the apartment units is a raft foundation. The different solicitations applied on the raft foundation were determined using the computer software SAP2000. Worthwhile to note at this stage that, the dynamic compaction of would guaranteed a bearing capacity of 200 KPa.

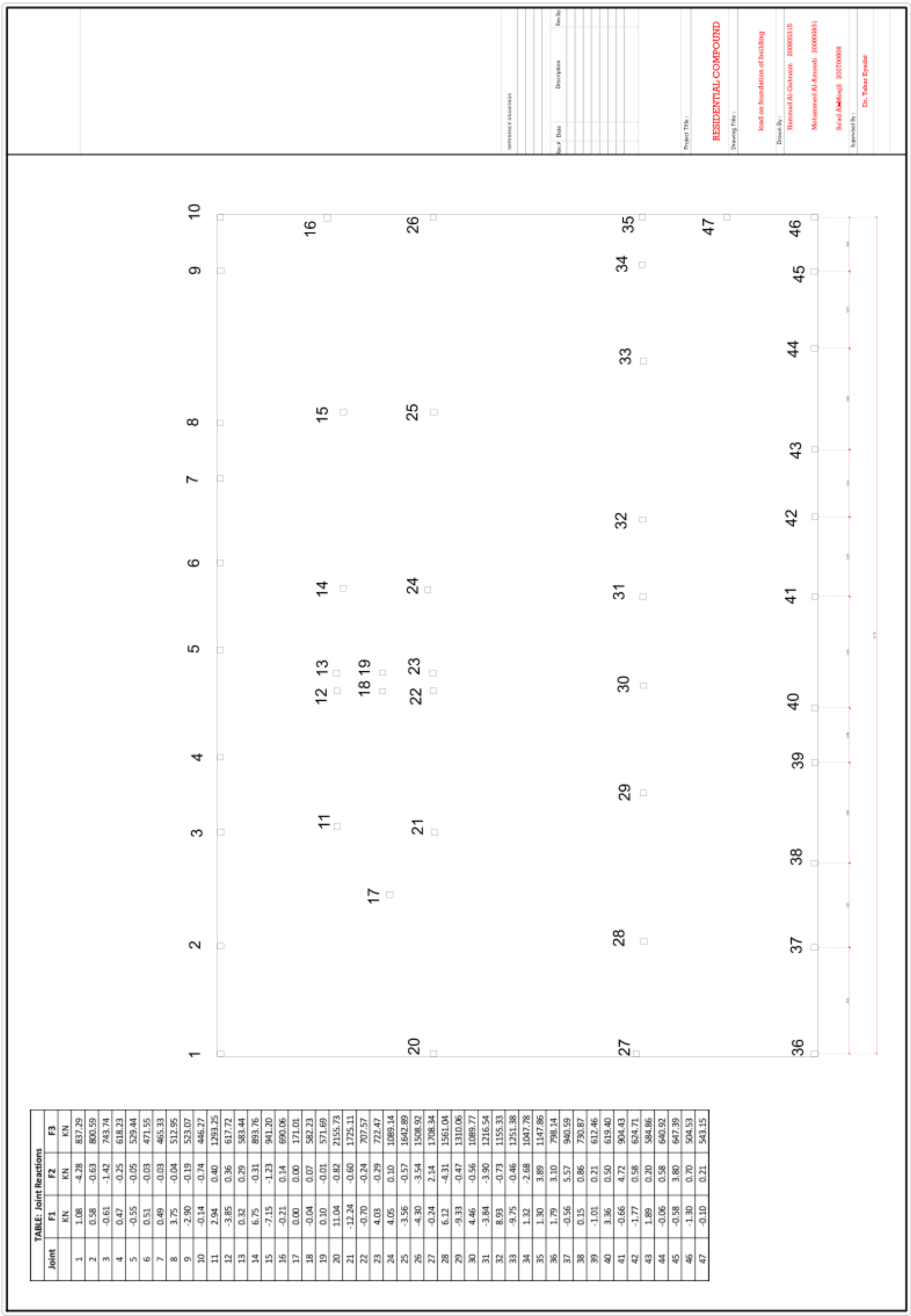


TABLE: Joint Reactions			
Joint	F1	F2	F3
	KN	KN	KN
1	1.08	-4.28	837.29
2	0.58	-0.63	800.59
3	-0.61	-1.42	743.74
4	0.47	-0.25	618.23
5	-0.55	-0.05	529.44
6	0.51	-0.03	471.55
7	0.49	-0.03	465.33
8	3.75	-0.04	512.95
9	-2.90	-0.19	523.07
10	-0.14	-0.74	446.27
11	2.94	0.40	1293.25
12	-3.85	0.36	617.72
13	0.32	0.29	583.44
14	6.75	-0.31	893.76
15	-7.15	-1.23	941.20
16	-0.21	0.14	690.06
17	0.00	0.00	171.01
18	-0.04	0.07	582.23
19	0.10	-0.01	571.69
20	11.04	-0.82	2155.73
21	-12.24	-0.60	1725.11
22	-0.70	-0.24	707.57
23	4.03	-0.29	722.47
24	4.05	0.10	1089.14
25	-3.56	-0.57	1642.89
26	-4.30	-3.54	1508.92
27	-0.24	2.14	1708.34
28	6.12	-4.31	1561.04
29	-9.33	-0.47	1310.06
30	4.46	-0.56	1089.77
31	-3.84	-3.90	1216.54
32	8.93	-0.73	1155.33
33	-9.75	-0.46	1251.38
34	1.32	-2.68	1047.78
35	1.30	3.89	1147.86
36	1.79	3.10	798.14
37	-0.56	5.57	940.59
38	0.15	0.86	730.87
39	-1.01	0.21	612.46
40	3.36	0.50	619.40
41	-0.66	4.72	904.43
42	-1.77	0.58	624.71
43	1.89	0.20	584.86
44	-0.06	0.58	640.92
45	-0.58	3.80	647.39
46	-1.30	0.70	504.53
47	-0.10	0.21	543.15

Figure 5.4. Plan of columns’ repartition on the raft foundation (calculated forces).

For calculating the soil pressure at the corner of the raft foundation:

- $q = \frac{Q}{A} \mp \frac{M_y}{I_y} x \mp \frac{M_x}{I_x} y$
- $Area\ of\ the\ mat = B\ x\ L$
- $I_{x\ ,y} = \frac{1}{12} B L^3$
- $M_y = Q\ e_x \qquad where \quad e_x = x' - \frac{B}{2}$

  From table 5.2, it can be deduced that:

- $q = 61.73\ \pm\ 0.34\ x\ \pm 0.14\ y$

Table 5.1. Total column load on this strip.

	B	***L***	***A***	***Q_{tot}***	***I_x***	***I_y***	**X'**	**e_x**	***M_y***	***Y'***	***e_y***	***M_x***
unit	M	m	m^2	KN	m^2	m^2	m	m	KN.m		m	KN.m
	21	32	672	41484.2	57344	24696	10.7	0.2	8296.84	15.98	0.02	829.684

Table 5.2. Total column load on this strip.

							q_{tot}	**check**
#	*x*	*y*	*q* =	61.73	0.34 x	0.014 y		
A	10.5	16	q_A =	61.73	3.53	0.23	65.492	ok
B	-10.5	16	q_B =	61.73	-3.53	0.23	58.436	ok
C	10.5	-16	q_C =	61.73	3.53	-0.23	65.029	ok
D	-10.5	-16	q*D* =	61.73	-3.53	-0.23	57.973	ok

Designing a raft foundation means determining two important parameters, in this case thickness and reinforcement. Starting with finding the highest joint reaction in the x or y direction (i.e from column 36 to column 46).

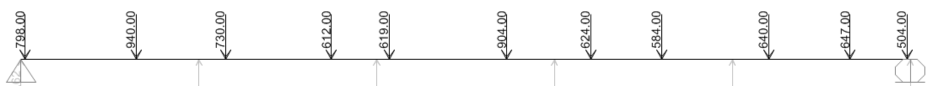


Figure 5.5. Reaction forces from columns 36 to 46.

- Averag soil pressure $= q_{av} = \frac{58.436+57.973}{2} = 58.2\ {KN}/{m^2}$
- $Total\ soil\ reaction =\ q_{av}\text{BL} = (58.2)(3.33)(32) =\ 6201.8\ KN$

Table 5.3. Total column load on this strip.

	F1	**F2**	**F3**
36	1.79	3.10	798.14
37	-0.56	5.57	940.59
38	0.15	0.86	730.87
39	-1.01	0.21	612.46
40	3.36	0.50	619.40
41	-0.66	4.72	904.43
42	-1.77	0.58	624.71
43	1.89	0.20	584.86
44	-0.06	0.58	640.92
45	-0.58	3.80	647.39
46	-1.30	0.70	504.53
		Total =	7608.28

- $Average\ load = \frac{(total\ soil\ reaction + column\ loads)}{2} = \frac{6201.8 + 7608.28}{2} = 6905.04\ KN$
- $Modified\ average\ soil\ pressure =$
- $q_{av(modified)} = q_{av}\left(\frac{6905.04}{6201.8}\right) = 64.8\ ^{KN}/_{m^2}$
- $B_1 q_{av(modified)} = 3.33\ x\ 64.8 = 215.78\ ^{KN}/_{m^2}$

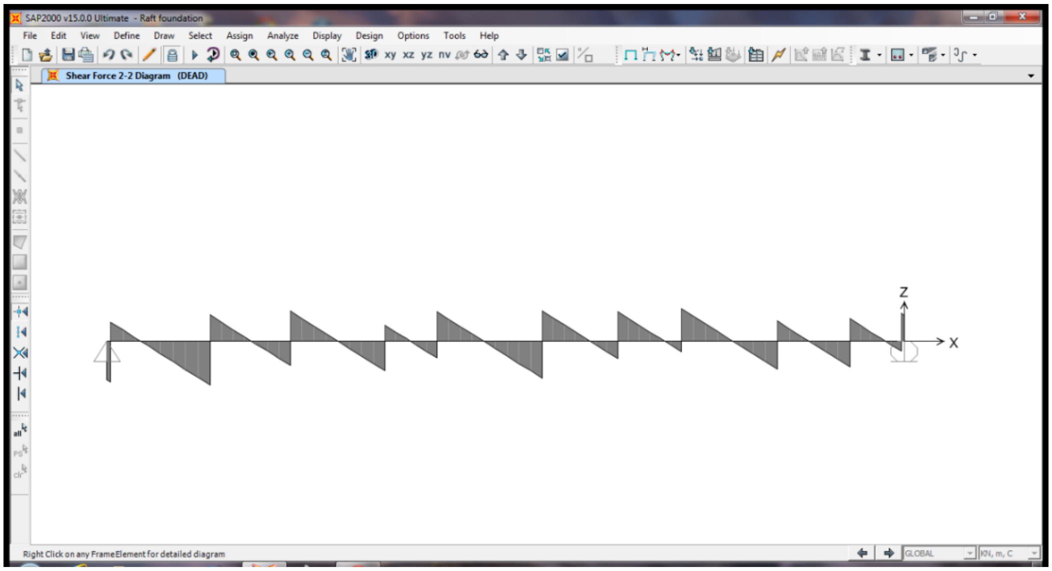


Figure 5.5. Shear diagram of the raft foundation.

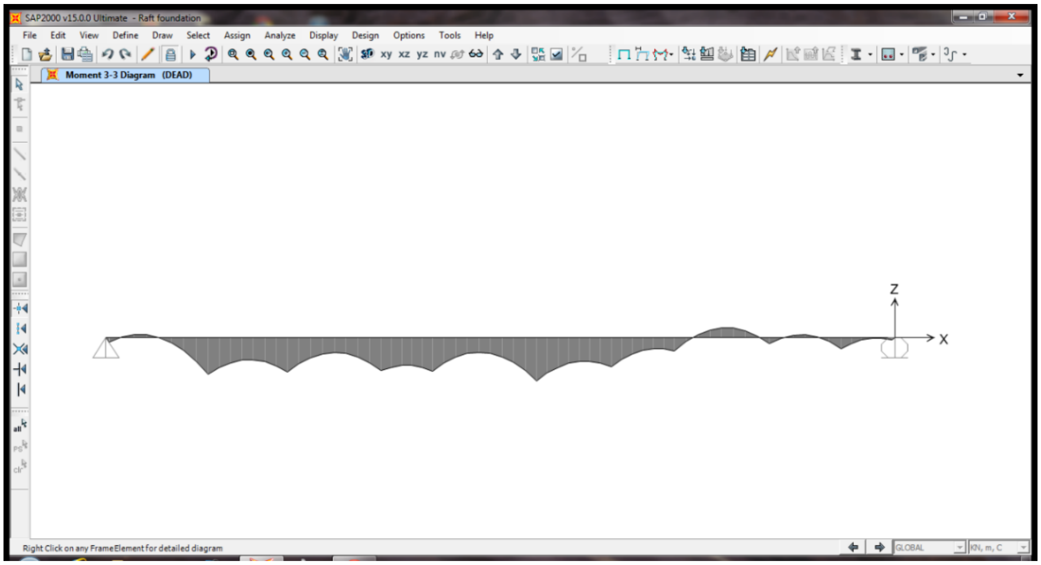


Figure 5.6. Moment diagram of the raft foundation.

- $b_0 = 1.5 + 2d$
- $U = (b_0 d)\left[(\emptyset)(0.34)\sqrt{f'_c}\right]$ $\quad where \quad U = 1.7\ x\ 940 = 1.6\ MN$
- $1.6 = (1.5 + 2d)(d)(0.85)(0.34)\left(\sqrt{20.7}\right) \rightarrow \quad d = 0.5m$
- Minimum cover 76mm and steel bars to be used are 25mm in diameter
- $h = 0.5 + 0.076 + 0.025 = 0.6\ m$
- $M_{max} = 872.6\ KN - m\ at\ x = 17.48m$ $\quad Given\ from\ SAP2000$
- $M' = \frac{872.6}{3.33} = 262.04\ {KN - m}/{m}$
- $Mu = (M')(load\ factor) = \emptyset A_s f_y \left(d - \frac{a}{2}\right)$

For positive moment

- $M_u = 262.04\ x\ 1.7 = 0.9(A_s)(413.7x1000)\left(0.5 - \frac{a}{2}\right)$
- $a = 23.51A_s$ $\quad A_s = 0.0425a$
- $445.468 = 0.9\ (0.04259)(413700)\left(0.5 - \frac{a}{2}\right)$
- $a = 0.059m$
- $A_s = 0.0425x0.059 = 0.0025\ {mm^2}/{m}$

As a result we can use: 25 mm diameter bars at 175 mm center to center

For negative moment

- $M' = \frac{203}{3.33} = 61\ {KN - m}/{m}$
- $M_u = 61x\ 1.7 = 103.6\ {KN.m}/{m}$
- $103.6 = \emptyset A_s(413.7x10^3)\left(0.5 - \frac{a}{2}\right)$
- $A_s = 0.0425a$

- $103.6 = 0.9(0.0425a)(413700)\left(0.5 - \frac{a}{2}\right)$
- $a = 0.014$m
- $A_s = 603\,{}^{mm^2}/_{m}$

As a result, we can use: 16 mm diameter bars at 400 mm center to center

5.3 Foundation system for the gym and the mosque

Sallow foundations were adopted as a foundation system for the mosque and the gym. Once the collapsible soil is replaced in zone A with granular soil compacted to 95% of proctor optimum, a bearing capacity of the order of 300 KPa can be reached. In design the shallow foundation, we have to check σ_{max}, σ_{min} and the eccentricity, *e*.

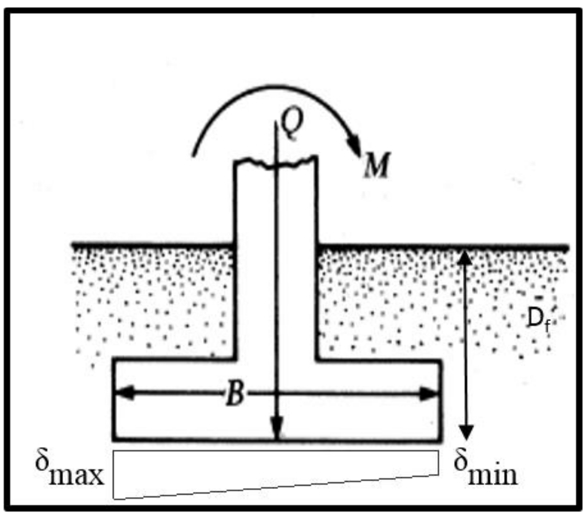


Figure 5.7. General Shallow Foundation.

- $\sigma \max = \frac{N}{A} + \frac{M}{S} \leq \sigma all = 300$ Where $S = \frac{1}{12}BH^2$
- $\sigma \min = \frac{N}{A} - \frac{M}{S} \geq 0$
- $e = \frac{M}{N} \leq e = \frac{B}{6}$

4.4.1. Foundation design for Gym

Table 5.4. Columns' reactions (or joints) in the gym.

Joint	***N***	M_{max}
Text	KN	KN.m
1	120.76	4.46
2	98.26	1.37
3	195.76	3.14
4	195.77	3.14
5	98.25	1.37
6	120.77	-0.01
7	115.96	-0.01
8	256.57	4.95
9	256.56	0.02
10	251.99	8.98
11	251.99	8.98
12	156.60	-0.01
13	156.61	6.45
14	182.64	1.16
15	143.33	1.08
16	143.33	2.40
17	182.65	6.02
18	115.96	6.43

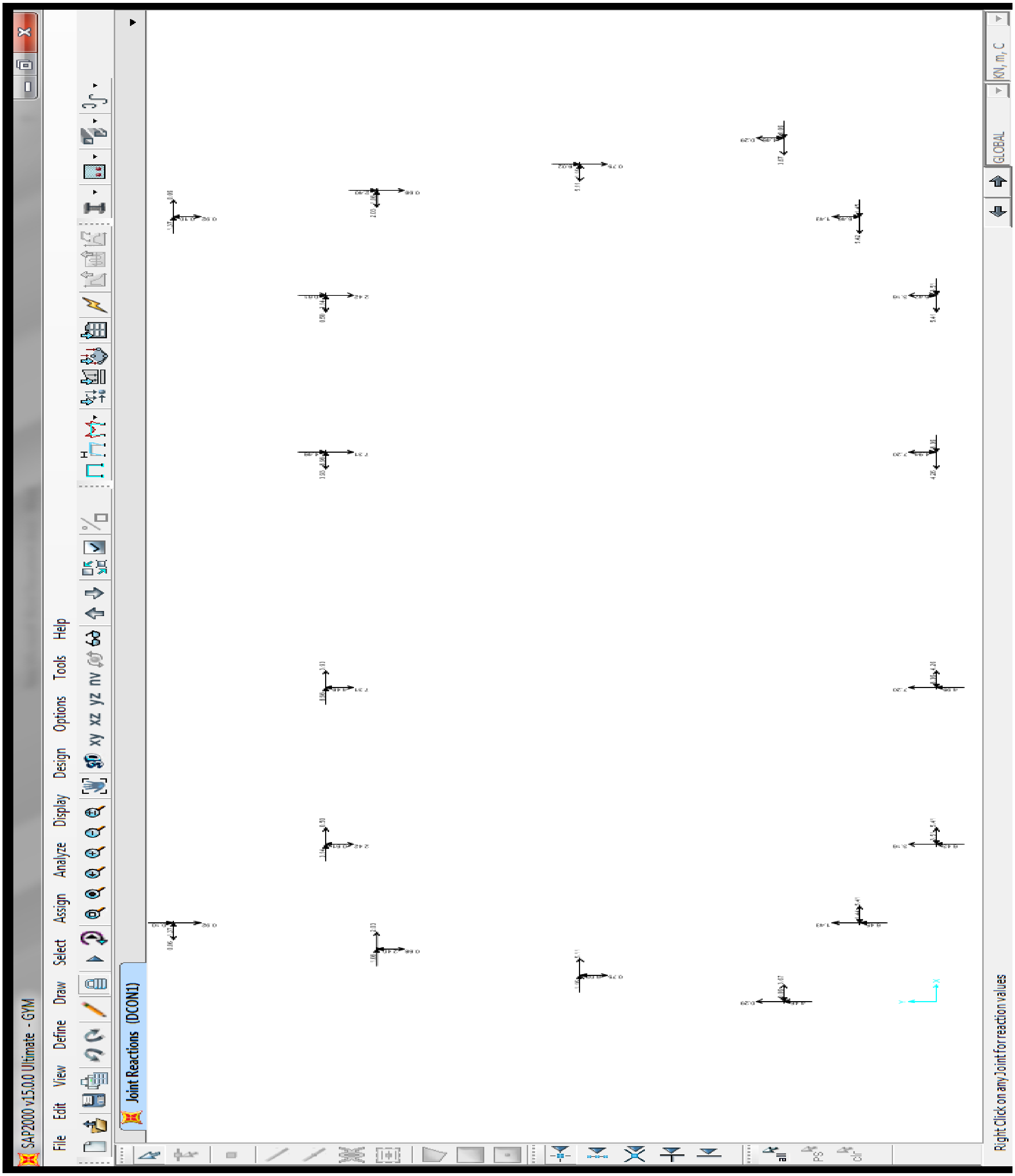


Figure 5.8. Columns' reaction (or joints) in the gym.

Table 5.5. Foundation design of the gym.

Joint	N	M_{max}	B	Area	H	S	σ_{max}	check	σ_{min}	check	e	e	check
Text	KN	KN.m	m	m^2	m	m^3	KPa		KPa		**M/N**	**B/6**	
1	120.76	4.46	1.00	1.00	0.70	0.04	229.93	ok	11.58	ok	0.04	0.17	ok
2	98.26	1.37	1.00	1.00	0.55	0.03	152.52	ok	44.00	ok	0.01	0.17	ok
3	195.76	3.14	1.20	1.44	0.55	0.03	239.70	ok	32.19	ok	0.02	0.20	ok
4	195.77	3.14	1.20	1.44	0.60	0.04	223.08	ok	48.81	ok	0.02	0.20	ok
5	98.25	1.37	1.00	1.00	0.55	0.03	152.47	ok	44.04	ok	0.01	0.17	ok
6	120.77	-0.01	1.00	1.00	0.40	0.01	119.75	ok	121.80	ok	0.00	0.17	ok
7	115.96	-0.01	1.00	1.00	0.40	0.01	114.98	ok	116.93	ok	0.00	0.17	ok
8	256.57	4.95	1.20	1.44	0.65	0.04	295.32	ok	61.02	ok	0.02	0.20	ok
9	256.56	0.02	1.00	1.00	0.40	0.01	257.84	ok	255.28	ok	0.00	0.17	ok
10	251.99	8.98	1.55	2.40	0.83	0.09	205.81	ok	3.97	ok	0.04	0.26	ok
11	251.99	8.98	1.55	2.40	0.83	0.09	205.80	ok	3.97	ok	0.04	0.26	ok
12	156.60	-0.01	1.00	1.00	0.40	0.01	155.76	ok	157.44	ok	0.00	0.17	ok
13	156.61	6.45	1.20	1.44	0.78	0.06	214.77	ok	2.74	ok	0.04	0.20	ok
14	182.64	1.16	1.50	2.25	0.40	0.02	139.27	ok	23.08	ok	0.01	0.25	ok
15	143.33	1.08	1.50	2.25	0.40	0.02	117.88	ok	9.53	ok	0.01	0.25	ok
16	143.33	2.40	1.50	2.25	0.60	0.05	117.03	ok	10.38	ok	0.02	0.25	ok
17	182.65	6.02	1.20	1.44	0.70	0.05	249.68	ok	3.99	ok	0.03	0.20	ok
18	115.96	6.43	1.11	1.23	0.87	0.07	185.89	ok	2.34	ok	0.06	0.19	ok

Based on the results showed in the previous table, it can be deduced that:

- The average dimension of the foundation width is situated between *B* = 1 m and *B* = 1.55 m.
- However, two categories can be adopted:
  - Category A: B = 1.25m taken for all foundation between 1 m and 1.25 m.
  - Category B: B = 1.5m taken for all foundation between 1.25m and 1.55 m.
- For the height, we consider two different heights: 0.5 m for category A and 0.8 m for category B.

5.4.2. Foundation design for mosque

Table 5.6. Columns' reactions (or joint) in the mosque.

Joint	**F3**	M_{max}
Text	KN	KN-m
1	159.81	6.976
2	159.83	6.997
3	140.43	5.035
4	140.57	5.12
5	315.76	6.601
6	249.33	14.82
7	249.23	14.81
8	323.73	7.023
9	437.91	7.233
10	437.91	7.232
11	352.65	3.487
12	351.06	3.602
13	407.94	15.19
14	407.92	15.19

15	358.84	8.027
16	352.57	8.123
17	351.32	13.05
18	313.43	11.97
19	278.72	12.42
20	279.46	8.899
21	279.68	8.916
22	278.65	12.42
23	313.4	11.96
24	325.16	10.13
25	339.54	12.68
26	325.15	10.12
27	594.04	0.826
28	594.13	0.826
29	350.95	13.06
30	605.99	4.732
31	865.43	3.2
32	649.74	2.413
33	864.67	3.223
34	606.25	4.73
35	399.85	2.85
36	427.02	3.375
37	399.73	2.916

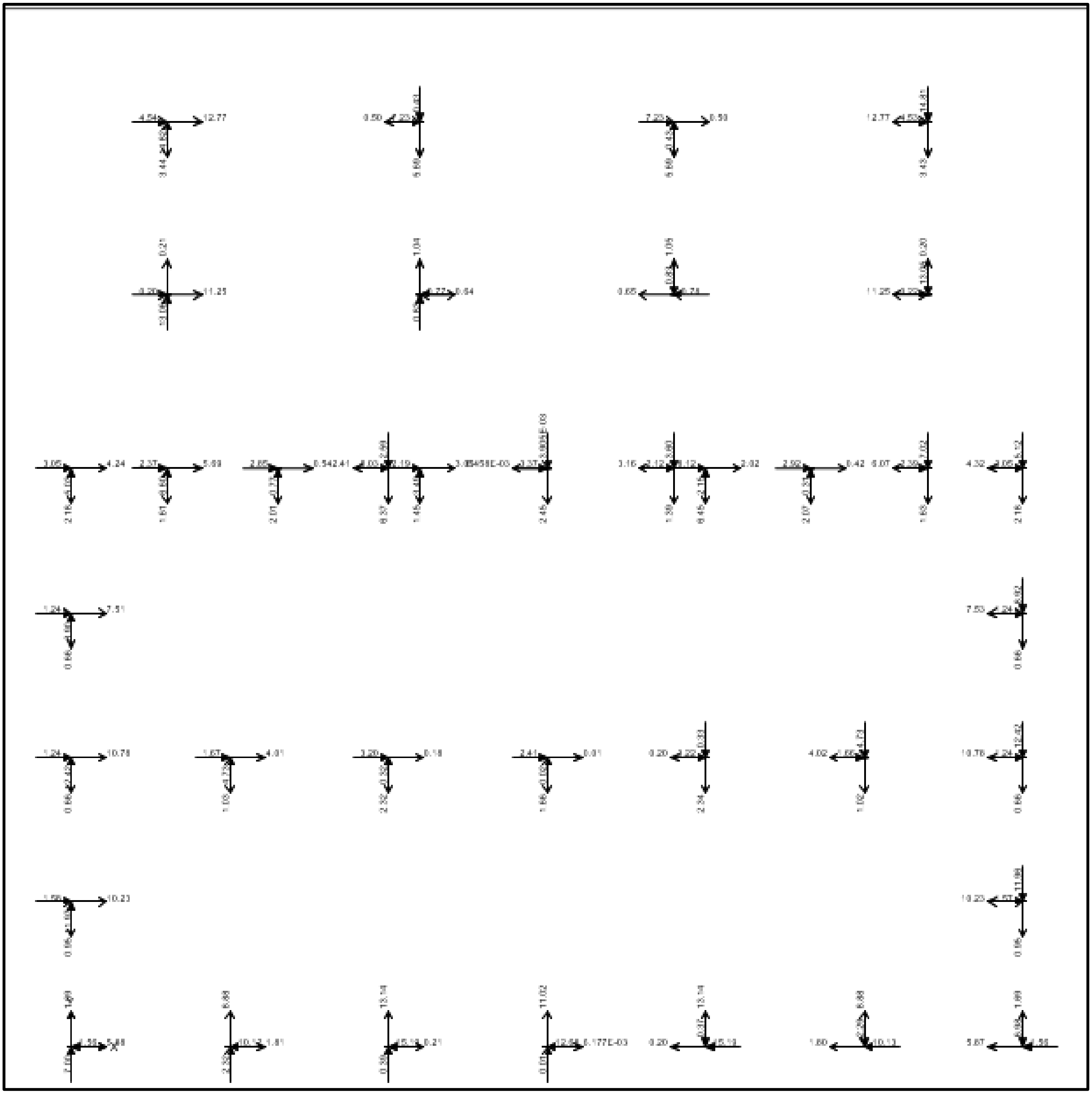


Figure 5.9. Plan of columns' joint in the mosque.

Table 5.7. Foundation design of the mosque.

Joint	***F3***	***M_{max}***	***B***	***Area***	***H***	***S***	***σ_{max}***	***check***	***σ_{min}***	***check2***	***e***	***e***	***Check3***
#	KN	KN.m	m	m^2	m	m^3	KPa		KPa		**M/N**	**B/6**	
1	159.81	6.98	1.10	1.21	0.76	0.05	263.83	**ok**	0.32	**ok**	0.04	0.18	**ok**
2	159.83	7.00	1.35	1.82	0.85	0.08	173.78	**ok**	1.62	**ok**	0.04	0.23	**ok**
3	140.43	5.03	1.20	1.44	0.75	0.06	187.03	**ok**	8.01	**ok**	0.04	0.20	**ok**
4	140.57	5.12	1.20	1.44	0.75	0.06	188.65	**ok**	6.59	**ok**	0.04	0.20	**ok**
5	315.76	6.60	1.60	2.56	0.70	0.07	224.38	**ok**	22.31	**ok**	0.02	0.27	**ok**
6	249.33	14.82	1.30	1.69	1.00	0.11	284.29	**ok**	10.77	**ok**	0.06	0.22	**ok**
7	249.23	14.81	1.30	1.69	1.00	0.11	284.20	**ok**	10.75	**ok**	0.06	0.22	**ok**
8	323.73	7.02	1.50	2.25	0.65	0.05	276.85	**ok**	10.91	**ok**	0.02	0.25	**ok**
9	437.91	7.23	2.42	5.86	0.70	0.10	147.97	**ok**	1.57	**ok**	0.02	0.40	**ok**
10	437.91	7.23	1.55	2.40	0.83	0.09	263.55	**ok**	101.00	**ok**	0.02	0.26	**ok**
11	352.65	3.49	1.55	2.40	0.83	0.09	185.98	**ok**	107.60	**ok**	0.01	0.26	**ok**
12	351.06	3.60	1.70	2.89	0.60	0.05	192.10	**ok**	50.85	**ok**	0.01	0.28	**ok**
13	407.94	15.19	1.95	3.80	0.95	0.15	210.89	**ok**	3.68	**ok**	0.04	0.33	**ok**
14	407.92	15.19	1.95	3.80	0.95	0.15	210.85	**ok**	3.70	**ok**	0.04	0.33	**ok**
15	358.84	8.03	1.95	3.80	0.95	0.15	149.11	**ok**	39.63	**ok**	0.02	0.33	**ok**
16	352.57	8.12	1.50	2.25	0.70	0.06	289.32	**ok**	24.07	**ok**	0.02	0.25	**ok**
17	351.32	13.05	1.95	3.80	1.00	0.16	172.70	**ok**	12.08	**ok**	0.04	0.33	**ok**
18	313.43	11.97	1.95	3.80	0.95	0.15	164.04	**ok**	0.82	**ok**	0.04	0.33	**ok**
19	278.72	12.42	1.80	3.24	1.00	0.15	168.84	**ok**	3.20	**ok**	0.04	0.30	**ok**
20	279.46	8.90	1.50	2.25	0.77	0.07	244.28	**ok**	4.13	**ok**	0.03	0.25	**ok**
21	279.68	8.92	1.95	3.80	0.95	0.15	134.35	**ok**	12.76	**ok**	0.03	0.33	**ok**
22	278.65	12.42	1.70	2.89	1.00	0.14	184.06	**ok**	8.77	**ok**	0.04	0.28	**ok**
23	313.40	11.96	1.70	2.89	0.90	0.11	212.64	**ok**	4.25	**ok**	0.04	0.28	**ok**
24	325.16	10.13	1.95	3.80	1.00	0.16	147.83	**ok**	23.20	**ok**	0.03	0.33	**ok**
25	339.54	12.68	1.70	2.89	0.90	0.11	228.02	**ok**	6.95	**ok**	0.04	0.28	**ok**
26	325.15	10.12	2.00	4.00	1.00	0.17	142.04	**ok**	20.54	**ok**	0.03	0.33	**ok**

27	594.04	0.83	2.00	4.00	0.70	0.08	158.62	**ok**	138.40	**ok**	0.00	0.33	**ok**
28	594.13	0.83	1.50	2.25	0.70	0.06	277.54	**ok**	250.58	**ok**	0.00	0.25	**ok**
29	350.95	13.06	1.95	3.80	1.00	0.16	172.65	**ok**	11.94	**ok**	0.04	0.33	**ok**
30	605.99	4.73	2.00	4.00	0.50	0.04	265.08	**ok**	37.92	**ok**	0.01	0.33	**ok**
31	865.43	3.20	2.00	4.00	0.50	0.04	293.15	**ok**	139.56	**ok**	0.00	0.33	**ok**
32	649.74	2.41	2.00	4.00	0.50	0.04	220.35	**ok**	104.52	**ok**	0.00	0.33	**ok**
33	864.67	3.22	2.00	4.00	0.50	0.04	293.52	**ok**	138.81	**ok**	0.00	0.33	**ok**
34	606.25	4.73	2.00	4.00	0.50	0.04	265.07	**ok**	38.05	**ok**	0.01	0.33	**ok**
35	399.85	2.85	2.00	4.00	0.50	0.04	168.36	**ok**	31.56	**ok**	0.01	0.33	**ok**
36	427.02	3.37	2.00	4.00	0.50	0.04	187.75	**ok**	25.76	**ok**	0.01	0.33	**ok**
37	399.73	2.92	2.00	4.00	0.50	0.04	169.92	**ok**	29.94	**ok**	0.01	0.33	**ok**

Based on the results of the previous In the previous table, we check all the foundation dimensions and thickness. The average dimension are between B = 1m to B = 2m. It is better to have the foundation in different categories, so we divide the categories to category A (B = 1.5m) for foundation between 1 m and 1.5 m. Category B (B = 2m) for foundation between 1.5m and 2 m. while the height of the foundation are between 0.5m to 1m height, so it divide into two categories A and B. category A (B = 0.75m) for foundation height between 0.5m to 0.8m. Category B (B = 1m) for foundation between 0.8m to 1m.

The foundations will be in 1.5 m depth and with a 4000 psi concrete.

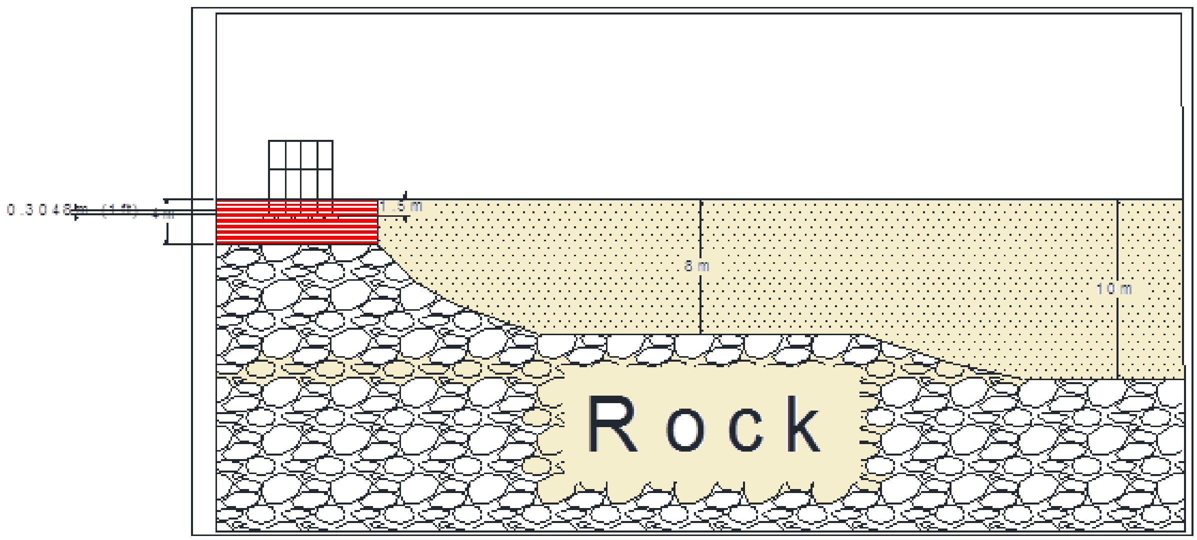


Figure 5.10. Cross view of the foundation in zone A (replaced ground).

5.3 Foundation system for the high-rise reservoir.

The reservoir located in zone C with 10 m depth of collapsible soil. The pile cap will be in 1.5 m depth, so the total pile depth will be 8.5 m. The foundation system that can handle the load and the lateral load due to height of the reservoir is a pile system. To design a pile system we have to check if the pile design as group or single system by checking the distance between the piles and compare to 10 multiply by the diameter of the pile. The load from the reservoir are distributed over 8 columns. If the design of the pile as one pile under each columns, so we have 8 piles.

- $S \leq 10B \rightarrow group\ pile$
- $S > 10B \rightarrow single\ pile$

Table 5.8. Pile system verification.

	Diameter of the foundation	Parameter	Diameter of the pile	Radius of the pile	S	10*B	check	Pile length
Units	**m**	M	m	m	m	m		m
	8	25.12	0.3	0.15	2.99	3	group	8.5

For calculating the allowable bearing capacity for single:

- $Q_{all} = \frac{Q_p+Q_s}{F.S.}$ $\quad where \quad F.S. = 3$
- $Q_p = A_p q_p = A_p\left(cN_C^* + qN_q^*\right)$
- $where\ \left(N_C^* = 9\ and\ N_q^* = 1\right) \quad because\ (\emptyset_{rock} = 0)$

$$q' = \gamma L$$

- $Q_s = \sum p\Delta Lf = \sum \alpha c_u p\Delta L$

Table5.9. Point bearing of piles Q_p calculation

	A_p	C_u	γ	q	Q_p
Unit	m^2	kpa	KN/m^2	kpa	Kpa
	0.07065	250	17	144.5	169.17

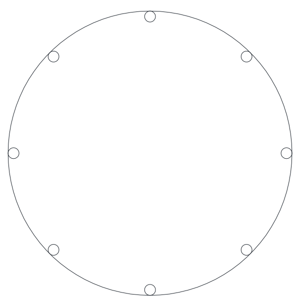
Table 5.10. Pile frictional resistance calculation Q_s

	f	Pile Parameter	Q_s
unit	kpa	M	kpa
	20	0.942	160.14

For calculating the allowable bearing capacity for group piles:

- $Q_{g(u)} = \eta \sum Qu$
- $\eta = 1 - \left[\frac{(n_1 - 1)n_2 + (n_2 - 1)n_1}{90 n_1 n_2}\right] \theta \ where\ \theta = 5.71$

For calculating n_1 and n_2:



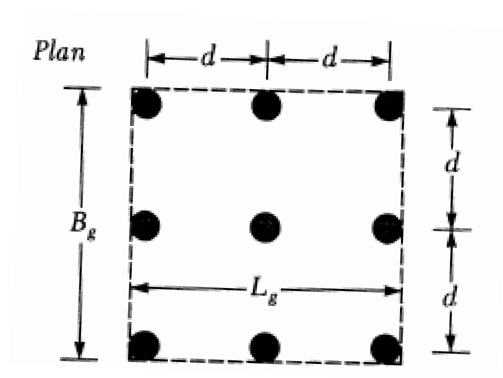


Figure 5.11. Piles' distribution.

$n_1 = 3$

$n_2 = 3$

Table 5.11. Calculation of the allowable bearing capacity of the group piles.

	n1	n2			
	3	3			
	Qp	Qs	Qall	η	Qg
unit	kpa	kpa	kpa		Kpa
	169.17	160.14	109.77	0.92	803.88

Table 5.12. Reaction force from SAP2000 of the high-rise reservoir.

	F1	F2	F3	M1	M2	M3
unit	KN	KN	KN	KN-m	KN-m	KN-m
1	-0.96	-0.14	559.17	0.18	-1.78	-0.16
2	0.96	-0.14	559.17	0.18	1.78	0.16
3	1.23	0	570.3	0	3	0
4	0	-0.29	543.81	0.35	0	0
5	-1.23	0	570.3	0	-3	0
6	-0.96	0.14	559.17	-0.18	-1.78	0.16
7	0	0.29	543.81	-0.35	0	0
8	0.96	0.14	559.17	-0.18	1.78	-0.16

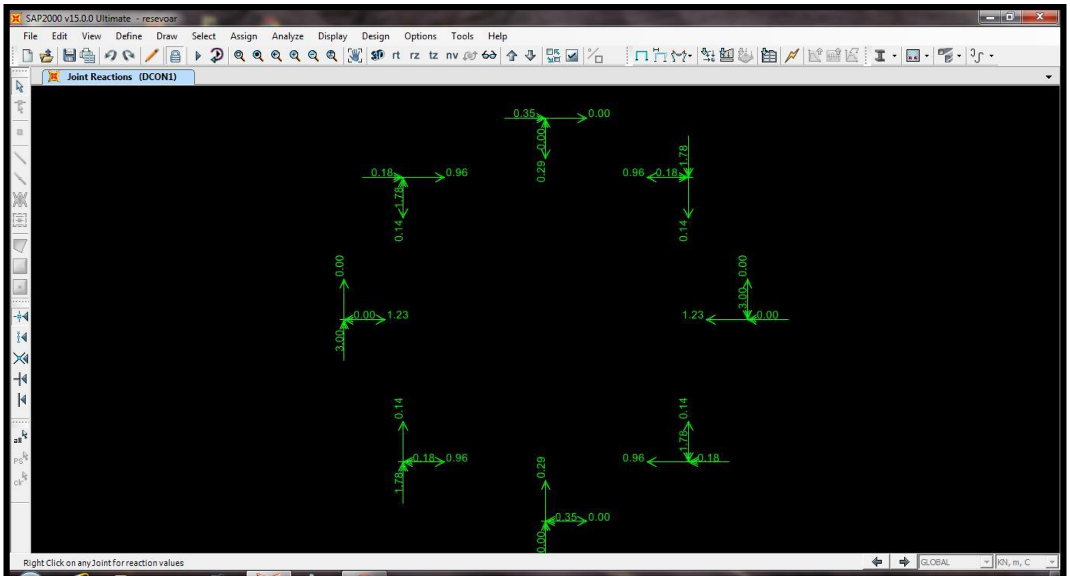


Figure 5.12. Reactions force in high-rise reservoir.

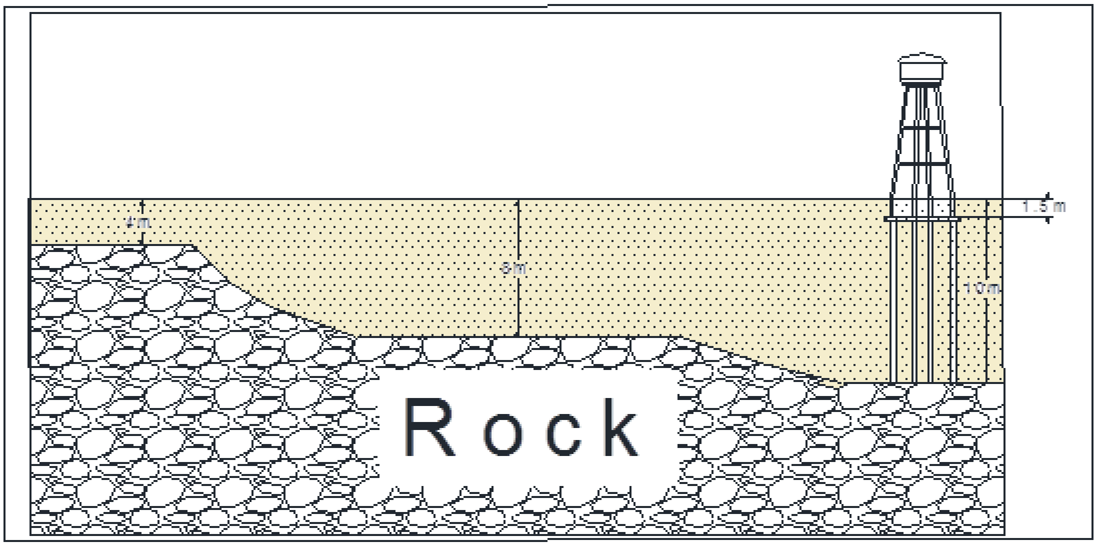


Figure 5.13. Pile foundation of the high-rise reservoir.

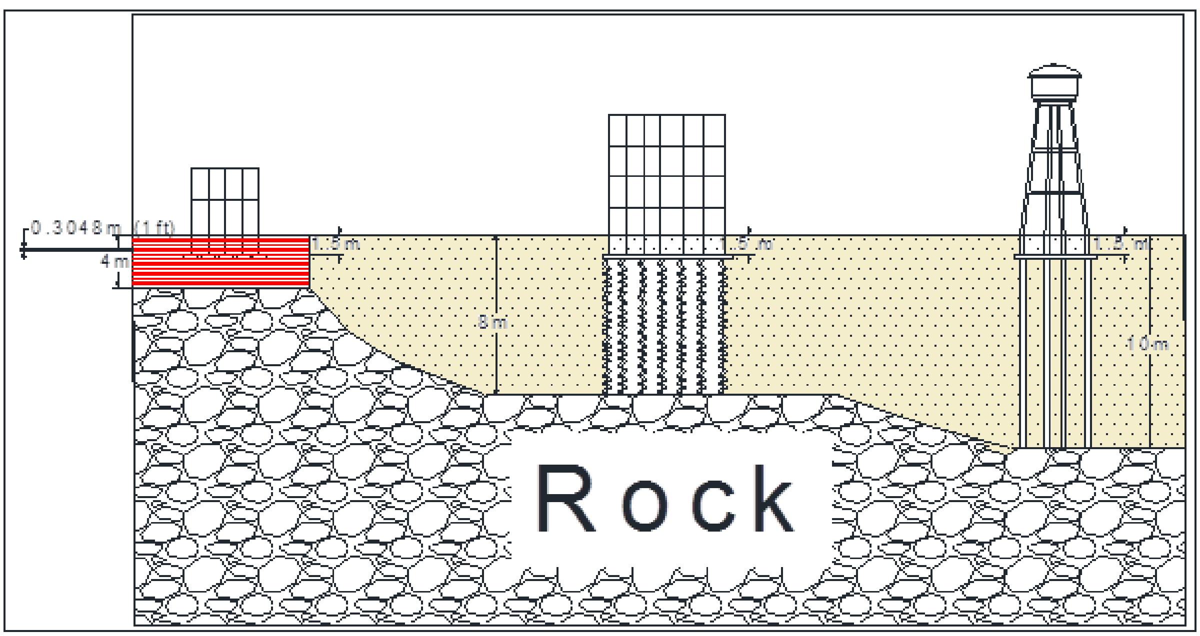


Figure 5.14. Recap. drawing of the different foundation systems.

5.6. Pavement Design (Parking & Access Roads)

From a geotechnical point of view, the intercepted top sandy soil (fill) cannot be used as sub-base material for the access road and the parking areas according to the existing rules. However, the construction of the access road and the parking areas could be performed following the usual procedures by excavating the soil to a minimum depth of 0.6 m and removing any organic materials if locally intercepted, in accordance with the practicing rules described below. In this case, we will have to expect some defects in the pavement such as cracks and depressions, in short or long term.

However, if the ground in place will be densified (compacted) within the areas of the access road and the parking spaces, no early defect of paving is anticipated.

Also, it might be necessary to envisage a peripheral drainage around the paved surfaces in order to prevent accumulations of water under pavement coming from the adjacent unpaved zones.

In order to ensure a stable foundation to the pavement, we recommend the following methods for the preparation of the ground:

- If the fill layer is not compacted, fill material and organic matter, disturbed or frozen materials if any, should be excavated to a minimum thickness of 0.6 m;
- Proceed with some surface rolling in order to check the presence of any soft or flexible zones. All flexible zones detected should be excavated and replaced by granular materials of good quality, adequately compacted;
- Granular foundation for the parking areas and the access road are to be made of 300 mm sub-base layer of granular soil 0-112 mm compacted at least 95% P.M. followed on the top by a layer of 250 mm of crushed stone 0-20 mm, compacted to at least 98% P.M.;
- Setting up bituminous pavement following the usual procedures. The following data can be used:

- Parking for light traffic: a layer of a standard mixture EB-14 (or equivalent) of 70 mm thickness;
- Parking for heavy traffic and access road: a base course of a mixture EB-20 (or equivalent) of 80 mm thickness, followed by a surfacing of a mixture EB-10S (or equivalent) of 40 mm thickness,

CHAPTER 6: CONCLUSION

6.1. Introduction

Many parts of the arid regions around the world are plagued with collapsible soils (sabkha soils). Collapsible soil is characterized by the sudden decrease in volume that occurs when subjected to inundation under constant stress. This volume change will be materialized in the form of drastic and uncalculated foundations settlement, which may further leads to catastrophic failure of the supported structures. Collapse settlement is the term applied to the additional settlement of a foundation due to the wetting of the overlain soils.

Due to the increasing use of water for irrigation, industrial projects and domestic purposes, foundations built on these types of soil (collapsible) have experienced uncalculated settlement, and accordingly significant damage to these structures. Construction on such a kind of soil is one of the prominent problems in geotechnical engineering.

In this chapter, some relevant conclusions, drawn from this investigation, are summarized. These general conclusions are concerned by the design and repartition of the different components of the compound, the structural and geotechnical design and the prototype.

6.2. General Conclusions

The goals of this investigation or this project were to design a habitation compound in a difficult or problematic collapsible soil, such as sabkha soil. The project included:

- The design and repartition of the different components of the compound (such as apartment units, sport center, mosque, high rise water reservoir and local roads & parking).
- Development and design of a prototype for the compound.

- Structural design of the different elements of the compound using appropriate computer software, in this case SAP2000.
- Geotechnical design which include the solution proposed for the foundation systems for the different elements of the compound.

The general conclusion drawn from this project can be summarized as follows:

- A compound was developed and designed in a difficult soil (sabkha soil). The site of the project extends over an area of about 33000 m^2. It is composed of thirteen apartment units, a two floor mosque and its own car parking, a sport center (gym), and a fifteen meters high rise reservoir. Furthermore, many access roads and different cars parking were designed inside the habitation area.
- A prototype of the compound was developed. The prototype shows the different components of the compound, as well as, a side view (taken as a cut) which show the different solutions proposed and adopted for the foundation systems of the different compound elements.
- A structural design using computer software (SAP2000) was performed, on the different components of the compound, in order to determine the appropriate reinforcements and the different solicitations transmitted to the foundations systems.
- A geotechnical investigation was carried out in order to select the appropriate solutions for the problem. Moreover, a geotechnical design was performed on the different foundation system proposed.
- Three types of foundation system were adopted for this project, including shallow foundations for area subjected to soils replacement (i.e. area of the mosque and the Gym), raft foundations for the different apartment units founded on the ground subjected to dynamic compaction, and deep foundations for the reservoir.

CHAPTER 6: CONCLUSION

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Due to the increasing use of water for irrigation, industrial projects and domestic purposes, foundations built on these types of soil (collapsible) have experienced uncalculated settlement, and accordingly significant damage to these structures. Construction on such a kind of soil is one of the prominent problems in geotechnical engineering.

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- The design and repartition of the different components of the compound (such as apartment units, sport center, mosque, high rise water reservoir and local roads & parking).
- Development and design of a prototype for the compound.

- Structural design of the different elements of the compound using appropriate computer software, in this case SAP2000.
- Geotechnical design which include the solution proposed for the foundation systems for the different elements of the compound.

The general conclusion drawn from this project can be summarized as follows:

- A compound was developed and designed in a difficult soil (sabkha soil). The site of the project extends over an area of about 33000 m^2. It is composed of thirteen apartment units, a two floor mosque and its own car parking, a sport center (gym), and a fifteen meters high rise reservoir. Furthermore, many access roads and different cars parking were designed inside the habitation area.
- A prototype of the compound was developed. The prototype shows the different components of the compound, as well as, a side view (taken as a cut) which show the different solutions proposed and adopted for the foundation systems of the different compound elements.
- A structural design using computer software (SAP2000) was performed, on the different components of the compound, in order to determine the appropriate reinforcements and the different solicitations transmitted to the foundations systems.
- A geotechnical investigation was carried out in order to select the appropriate solutions for the problem. Moreover, a geotechnical design was performed on the different foundation system proposed.
- Three types of foundation system were adopted for this project, including shallow foundations for area subjected to soils replacement (i.e. area of the mosque and the Gym), raft foundations for the different apartment units founded on the ground subjected to dynamic compaction, and deep foundations for the reservoir.

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