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*Structural and Geotechnical Design of a
Multistory Hotel of Irregular Floor layout
in the Half Moon Area*

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ABSTRACT

Many citizens and expatriate residents in the Saudi capital routinely travel to the Eastern Province with their families on weekends to bond and take a break from daily routine. It is well known that you have not been to Al-khobar if you haven't visited Half Moon Bay. The bay, a crescent of soft sand in Al-khobar, is like a semi-lunar arch with several other attractions, such as entertainment zones and amusement cities. Half Moon Bay is a major recreational area for tourists, who throng the area on weekends. Recently, the Half Moon Bay area is undergoing some remarkable developments. Many construction projects were launched in the area. However, the construction of more touristic complex and hotels is still not receiving full consideration.

The aim of this projects to design a multistory hotel of irregular floor layout (i.e. ellipsoidal layout or shape) in the half-moon area; where the soil is susceptible to liquefaction. The project includes the location of the multistory hotel in the half-moon area, structural design of the different elements of the hotel using appropriate computer software (ETABS), and geotechnical investigation which include the proposed solutions for the foundation systems of the multistory ellipsoidal hotel. Two types of foundation system were investigated, including shallow foundations when the area is dynamically compacted and deep foundations (i.e. driven metallic piles).

LIST OF FIGURES

Chapter 2: Description of the project.....	5-8
Figure 2.1 Example #1 of the results of borehole.....	5
Figure 2.2 Example #2 of the results of borehole.....	6
Figure 2.3 Example #3 of the results of borehole.....	7
Figure 2.4 Example #4 of the results of borehole.....	8
Chapter 3: Structural Design.....	13-67
Figure 3.1 Example of dead load with factor of 1.2	13
Figure 3.2 Example of “live+dead” load combination with a factor of 1.6 & 1.2 respectively.....	14
Figure 3.3 Example of “dead+wind x” load combination with a factor of 0.9 & 1.6 respectively (ETABS).....	15
Figure 3.4 3-D view of the hotel in ETABS.....	17
Figure 3.5 Slab properties.....	18
Figure 3.6 Deformed shape of the slabs – Dead load factor 0.9 (ETABS).....	27
Figure 3.7 Deformed shape of the slabs – Dead load factor 1.2 (ETABS).....	28
Figure 3.8 Deformed shape of the slabs – Dead load factor 1.6 (ETABS).....	29
Figure 3.9 Deformed shape of floor # 8 – Dead load factor 1.2 & Live load factor 1.6 (ETABS).....	30
Figure 3.10 Deformed shape of parking 3 – Dead load factor 1.0 (ETABS).....	31
Figure 3.11 Moments of the different slabs – dead load factor 0.9 & wind x 1.6 (ETABS).....	32
Figure 3.12 Deformed shape of the different slabs – Dead load factor 0.9 & Wind Y 1.6 (ETABS).....	33

Figure 3.13 Shear of different slabs – dead load factor of 0.9 & wind x 1.6 (ETABS).....	34
Figure 3.14 Shear of the different slabs – Dead load of 0.9 & wind y 1.6 (ETABS).....	35
Figure 3.15 Stress in the different slabs – Dead load of 0.9 & Live load 1.6 (ETABS)..	36
Figure 3.16 Gravity load in the different slabs (ETABS).....	37
Figure 3.17 Example of concrete column section properties – parking.....	56
Figure 3.18 Concrete shear wall section properties nearby the elevator.....	57
Figure 3.19 “3-D” Axial force loads on the concrete columns – dead load factor of 1....	58
Figure 3.20 Axial force loads on the concrete columns – dead load factor of 1 (plan view).....	59
Figure 3.21 In-plan moment – dead load with factor of 1.2 & live load with factor of 1.6.....	60
Figure 3.22 Moment 2-2 . Live load with factor of 1.6	61
Figure 3.23 Moment 3-3 . Dead load with factor of 0.9 & wind load in the x direction with factor of 1.6	62
Figure 3.24 Shear force 2-2 . Dead load factor of 1 & live load factor of 1.....	63
Figure 3.25 Shear force 3-3 . Dead load with factor of 1.2 & live load with factor of 1.6.....	64
Figure 3.26 Column torsion . Dead load with a factor of 1.2.....	65
Figure 3.27 Restraint reaction . Dead load with a factor of 1.2 & live load with a factor of 1.6.....	66
Figure 3.28 Concrete design sections.....	67

LIST OF TABLES

Chapter 3: Structure Design.....	16-52
Table 3.1. Hotel Facilities.....	16
Table 3.2. Steel bars Reinforcement options for positive moment design.....	23
Table 3.3 Steel bars reinforcement options for positive moment design.....	25
Table 3.4. Largest slab areas covered by one column.....	39
Table 3.5. Hand calculated dead and live loads per unit area.....	43
Table 3.6. ETABS dead and live loads per unit area.....	44
Table 3.7. Columns dimensions design.....	45
Table 3.8. ETABS load combinations.....	47
Table 3.9. Horizontal grid lines.....	48
Table 3.10. Vertical grid lines.....	50
Table 3.11. Columns and shear walls slopes	52
Chapter 4: Geotechnical Design	68-80
Table 4.1 Summary of the soil profile of the site of the projected hotel.....	68
Tables 4.2 Results of load combination for column #83.....	71
Table 4.3 Results of load combination for column #84.....	71
Table 4.4 Results of load combination for column #85.....	72
Table 4.5 Results of load combination for column #86.....	72
Table 4.6 Results of load combination for column #43.....	73
Table 4.7 Results of load combination for column #58.....	73
Table 4.8 Maximum designing loads.....	74
Table 4.9 Results of bearing capacity of verification for the hotel shallow foundation.....	76
Table 4.10 Summary of the different design options.....	77
Table 4.11 The different characteristics used in the geotechnical design of piles.....	79
Table 4.12 Resistance and shaft capacity of the different piles considered.....	80
Table 4.13 Ultimate and allowable bearing capacity of the different piles.....	80

TABLE OF CONTENT

<i>Acknowledgement</i>	<i>i</i>
<i>Abstract</i>	<i>ii</i>
<i>List of Figures</i>	<i>iii</i>
<i>List of Tables</i>	<i>iv</i>
<i>List of Content</i>	<i>v</i>
Chapter 1: Introduction	1-2
1.1 General	1
1.2 Project Objectives	2
1.3 Scope of the report	2
Chapter 2: Description of the project	2-11
2.1 Introduction	3
2.2 Overall description of the project.....	3
2.3 Soil profile.....	4
2.4 Relevant recommendation.....	10
Chapter 3: Structural Design	12-67
3.1 Introduction	12
3.2 Hotel structure.....	12
3.3 Hotel facilities.....	16
3.4 Structural design	17-67
3.4.1 Concrete slab design.....	17
3.4.2 Concrete column design.....	38

Chapter 4: Geotechnical Design 68-80

4.1 Introduction68
4.2 Proposed solution.....69
 4.2.1 Solution of problem #1.....69
 4.2.2 Solution of problem #2.....69
4.3 Columns axial working loads.....70
4.4 Verification of the bearing capacity.....74
 4.4.1 Isolated Foundation.....74
 4.4.2 Deep foundation.....77-80

Chapter 5: conclusion.....81-82

5.1 Introduction81
5.2 General Conclusions82

List of references.....83

LIST OF FIGURES

Chapter 2: Description of the project.....5-8

Figure 2.1 Example #1 of the results of borehole.....5
Figure 2.2 Example #2 of the results of borehole.....6
Figure 2.3 Example #3 of the results of borehole.....7
Figure 2.4 Example #4 of the results of borehole.....8

Chapter 3: Structural Design.....13-67

Figure 3.1 Example of dead load with factor of 1.213
Figure 3.2 Example of “live+dead” load combination with a factor of 1.6 & 1.2
respectively.....14
Figure 3.3 Example of “dead+wind x” load combination with a factor of 0.9 & 1.6
respectively (ETABS).....15
Figure 3.4 3-D view of the hotel in ETABS.....17
Figure 3.5 Slab properties.....18
Figure 3.6 Deformed shape of the slabs – Dead load factor 0.9 (ETABS).....27
Figure 3.7 Deformed shape of the slabs – Dead load factor 1.2 (ETABS).....28
Figure 3.8 Deformed shape of the slabs – Dead load factor 1.6 (ETABS).....29
Figure 3.9 Deformed shape of floor # 8 – Dead load factor 1.2 & Live load factor 1.6
(ETABS).....30
Figure 3.10 Deformed shape of parking 3 – Dead load factor 1.0 (ETABS).....31
Figure 3.11 Moments of the different slabs – dead load factor 0.9 & wind x 1.6
(ETABS).....32
Figure 3.12 Deformed shape of the different slabs – Dead load factor 0.9 & Wind Y 1.6
(ETABS).....33
Figure 3.13 Shear of different slabs – dead load factor of 0.9 & wind x 1.6
(ETABS).....34
Figure 3.14 Shear of the different slabs – Dead load of 0.9 & wind y 1.6 (ETABS).....35
Figure 3.15 Stress in the different slabs – Dead load of 0.9 & Live load 1.6 (ETABS)..36

Figure 3.16 Gravity load in the different slabs (ETABS).....	37
Figure 3.17 Example of concrete column section properties – parking.....	56
Figure 3.18 Concrete shear wall section properties nearby the elevator.....	57
Figure 3.19 “3-D” Axial force loads on the concrete columns – dead load factor of 1....	58
Figure 3.20 Axial force loads on the concrete columns – dead load factor of 1 (plan view).....	59
Figure 3.21 In-plan moment – dead load with factor of 1.2 & live load with factor of 1.6.....	60
Figure 3.22 Moment 2-2 . Live load with factor of 1.6	61
Figure 3.23 Moment 3-3 . Dead load with factor of 0.9 & wind load in the x direction with factor of 1.6	62
Figure 3.24 Shear force 2-2 . Dead load factor of 1 & live load factor of 1.....	63
Figure 3.25 Shear force 3-3 . Dead load with factor of 1.2 & live load with factor of 1.6.....	64
Figure 3.26 Column torsion . Dead load with a factor of 1.2.....	65
Figure 3.27 Restraint reaction . Dead load with a factor of 1.2 & live load with a factor of 1.6.....	66
Figure 3.28 Concrete design sections.....	67

LIST OF TABLES

Chapter 3: Structural Design.....	10-21
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Table 3.1. Details of drilled boreholes.....	11
Table 3.2. Standard codes of Laboratory Tests	12
Table 3.3. Summary of Unconfined Compressive Strength (UCS) Test Results.....	13
Table 3.4. Chemical Analysis Results	13
Table 3.5. Subsurface Stratification	14
Table 3.6. Subsurface Stratification.....	15
Table 3.7. Ground Water Level.....	15
Table 3.8. Soil Design Parameters	16
Table 3.9. Coefficient of Earth Pressure.....	17
Table 3.10. Depth of loose layer of sand.....	18
Table 3.11. Fill height at the proposed constructions	19
Table 3.12. Expected Settlement	19
Table 3.13. Recommended NABC & Recommended Footing Type.....	20
Table 3.14. Recommended Net allowable Safe Bearing Capacity.....	21
Chapter 5: Geotechnical Design	35-51
Table 5.2. Bearing Capacity Factors	44
Tables <i>B1</i> to <i>B3</i> Appendix B Stability assessment	76-85
Tables <i>C1</i> to <i>C5</i> : Appendix C(Determination of Foundation Bearing Capacity).....	86-91

TABLE OF CONTENT

<i>Acknowledgement</i>	<i>i</i>
<i>Abstract</i>	<i>ii</i>

<i>List of Figures</i>	<i>iii</i>
<i>List of Tables</i>	<i>iv</i>
<i>List of Content</i>	<i>v</i>

Chapter 1: Introduction.....1-2

1.2 General	1
1.2 Project Objectives	2
1.3 Scope of the report	2

Chapter 2: Description of the project2-11

2.1 Introduction	3
2.2 Overall description of the project.....	3
2.3 Soil profile.....	4
2.4 Relevant recommendation.....	10

Chapter 3: Structural Design 12-67

3.1 Introduction	12
3.2 Hotel structure.....	12
3.3 Hotel facilities.....	16
3.4 Structural design	17-67
3.4.1 Concrete slab design.....	17
3.4.2 Concrete column design.....	38

Chapter 4: Geotechnical Design 68-80

4.1 Introduction	68
4.2 Proposed solution.....	69
4.2.1 Solution of problem #1.....	69
4.2.2 Solution of problem #2.....	69
4.3 Columns axial working loads.....	70
4.4 Verification of the bearing capacity.....	74
4.4.1 Isolated Foundation.....	74
4.4.2 Deep foundation.....	77-80

Chapter 5: conclusion.....	81-82
5.1 Introduction	81
5.2 General Conclusions	82
List of references.....	83

CHAPTER 1: INTRODUCTION

1.1. General

Half Moon Bay is a major recreational area for tourists, who throng the area on weekends. Recently, the Half Moon Bay area is undergoing some remarkable developments. Many construction projects were launched in the area. It is worthwhile to note that the soil of the area is sandy saturated soil susceptible to liquefaction.

Soil liquefaction is a phenomenal condition where the soil would behave as liquid. The idea behind this phenomenon is that when the soil reacts to any form of stress, usually earthquake load, the particles of the soil start shaking; therefore the soil becomes less dense. This phenomenon is usually observed in places where the soil is saturated, loose and sandy. Loose soil and especially sandy loose soil compress easily when a load is applied on it. In an area below water table or sea level, it becomes very easy for the water to penetrate and fill the gaps between the soil particles. After a large volume of water has penetrated between the soil particles, the pressure of the soil will be very high. In response the water will try to escape to an area of lower pressure. However, if the stress was applied continuously in the same location, the soil will continue shaking, therefore becoming less dense again and allowing more quantities of water to penetrate in the same spot. In this case, the only way for the water to escape the high-pressure zone is to push vertically against the structure, which will force the structure to be pushed away from the soil and perhaps even fall down eventually

Once the liquefaction manifests, the following problem may arise loss of bearing strength: the ground can liquefy and lose its ability to support structures, Lateral spreading: the ground can slide down very gentle slopes or toward stream banks riding on a buried liquefied layer., sand boils: sand-laden water can be ejected from a buried liquefied layer and erupt at the surface to

form sand volcanoes; the surrounding ground often fractures and settles, and flow failures: earth moves down steep slope with large displacement and much internal disruption of material, etc.

The most well-known treatment methods for the liquefaction can be vibro-compaction (process of densification by vibration and compaction of sandy and gravely backfill material), compaction piles, dynamic compaction, drains, grouting (process of injection and mixing of a stabilizer with soil, e.g. cement, lime), etc.

1.2. Project Objectives

The main objectives of this project are to design a multistory hotel of irregular floor layout (i.e. ellipsoidal layout) to be constructed in the half-moon area where the soils are granular and saturated (i.e. susceptible to liquefaction). Moreover, the level of the water table in this case is almost on the surface. The project includes structural and geotechnical design of the multistory hotel of ellipsoidal shape.

The project is composed by the following stages or parts:

- 1- Development of the different architectural plans of the multistory hotel (including top and side views plans, and perspectives plans).
- 2- Structural design of the multistory hotel of irregular floor layout.
- 3- Geotechnical design of the different foundation system: shallow foundations on dynamically compacted soil and piles.

1.3. Scope of the report

The present report is composed of five chapters. A detailed description of the project and its components (i.e. the multistory hotel and components) is presented in chapter 2. Soil report is also introduced in chapter 2 including site investigation, laboratory works and soil profile. Following that, the structural and geotechnical design of the different elements of the hotel-apartment are given in chapters 3 and 4, 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 of the multistory hotel. The hotel, which has an irregular floor layout (i.e. ellipsoidal form or shape), is composed of twelve floors including three levels of underground car parking. It is constituted of about 520 rooms distributed in six different floors (i.e. about 86 rooms by floor). The mechanical room is located in the seventh floor; while, the swimming pool and the fitness room are located in the roof of the hotel. Moreover, the complex is surrounded by many car parking and two local roads.

The chapter will start by describing the overall component of the multistory hotel (i.e. the different parts of the complex). Following that a brief description of the soil profile is included. 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 is used to design the foundation systems for multistory hotel. Finally, the chapter is achieved by some information concerning the different cars parking and the access roads.

2.2 Overall description of the project

The site of the hotel extends over an area of 70m x 34m. 70 and 34 are the major and minor diameters respectively. The area of the ellipse equals $(\pi \times a \times b)$, where “a” and “b” are the major and minor radiuses, equaling 35m and 17m respectively. Thus, the total area = $(\pi \times 35 \times 17) = 1870 \text{ m}^2$. The hotel is located fictively in the Half Moon area in the eastern province of the Kingdom of Saudi Arabia. The hotel is delimited by two different streets. One main street located on the front of the hotel units and parallel to the beach, and another secondary street, situated beside of the hotel.

The following parts compose the project:

- A multistory hotel of ellipsoidal shape composed of twelve floors, including three underground levels of car parking. The seventh floor accommodates the mechanical room, and the swimming pool and the fitness room are situated in the roof of the hotel. The hotel is composed of 520 rooms distributed on six floors. The type of frame used in the hotel structure is reinforced concrete composed mainly by reinforced concrete slabs, columns, and shear walls.
- Two different roads (main and secondary roads) and many cars parking are planned to be in the back of the hotel unit.

The view of the hotel is shown in the following Figures:



Figure 2.1 Day view of the Hotel (www.yamsafer.com)



Figure 2.2 Evening view of the Hotel (www.yamsafer.com)

2.3 Soil Profile

A geotechnical investigation was carried out by the company Geotechnical & Environmental Co. Ltd. in a site nearby the site of the actual project (i.e. the site of the new constructed bridge near PMU campus). The purpose of the geotechnical study was to investigate the upper 100 feet of subsurface soil at selected location within the site for evaluation of physical and engineering properties. From this geotechnical investigation, site conditions were deduced. The scope of the geotechnical investigation consisted of the following: i)-Field exploration and in-situ testing of the site soils at selected locations and depths, ii)- Laboratory testing for physical and /or chemical properties of selected samples, iii)- Engineering analysis and evaluation of the data collected, and iv)- Recommendations for the geotechnical aspects of the project design and construction.

Subsurface exploration was performed on December 22 and 23, 2013 using 2R Drilling of Ontario California to advance to depths of 30 meter below existing ground surface. The borings were advanced with a truck-mounted, B-53 M drill rig using 8-inch diameter, hollow stem, and continuous-flight augers. An example of the subsurface logs (prototype) is presented herein below. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata.

However, the transition from one stratum to another may be gradual over some range of depth.

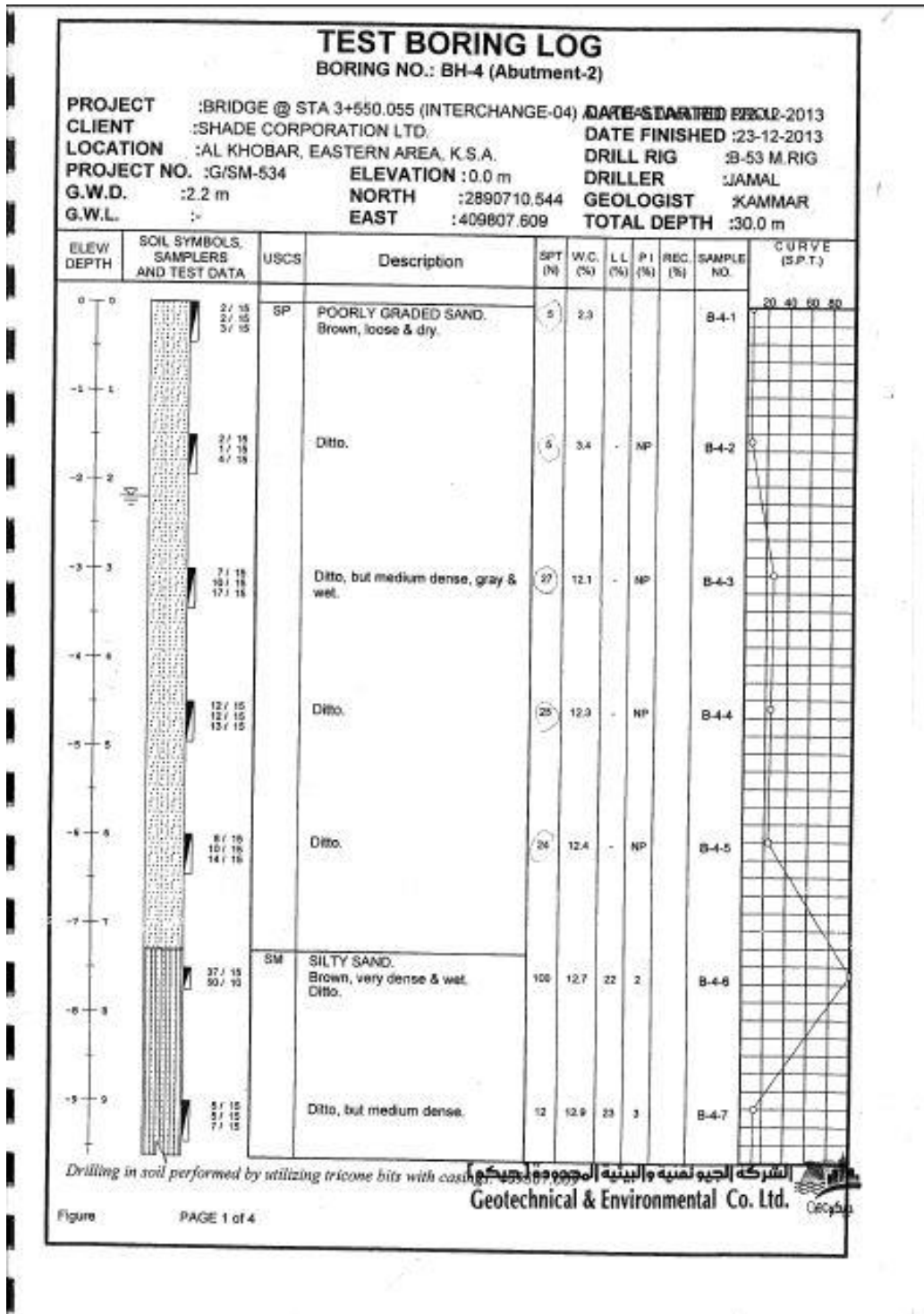


Figure 2.3. Example of field boreholes results – Part I (Geotechnical & Environmental Co Ltd)

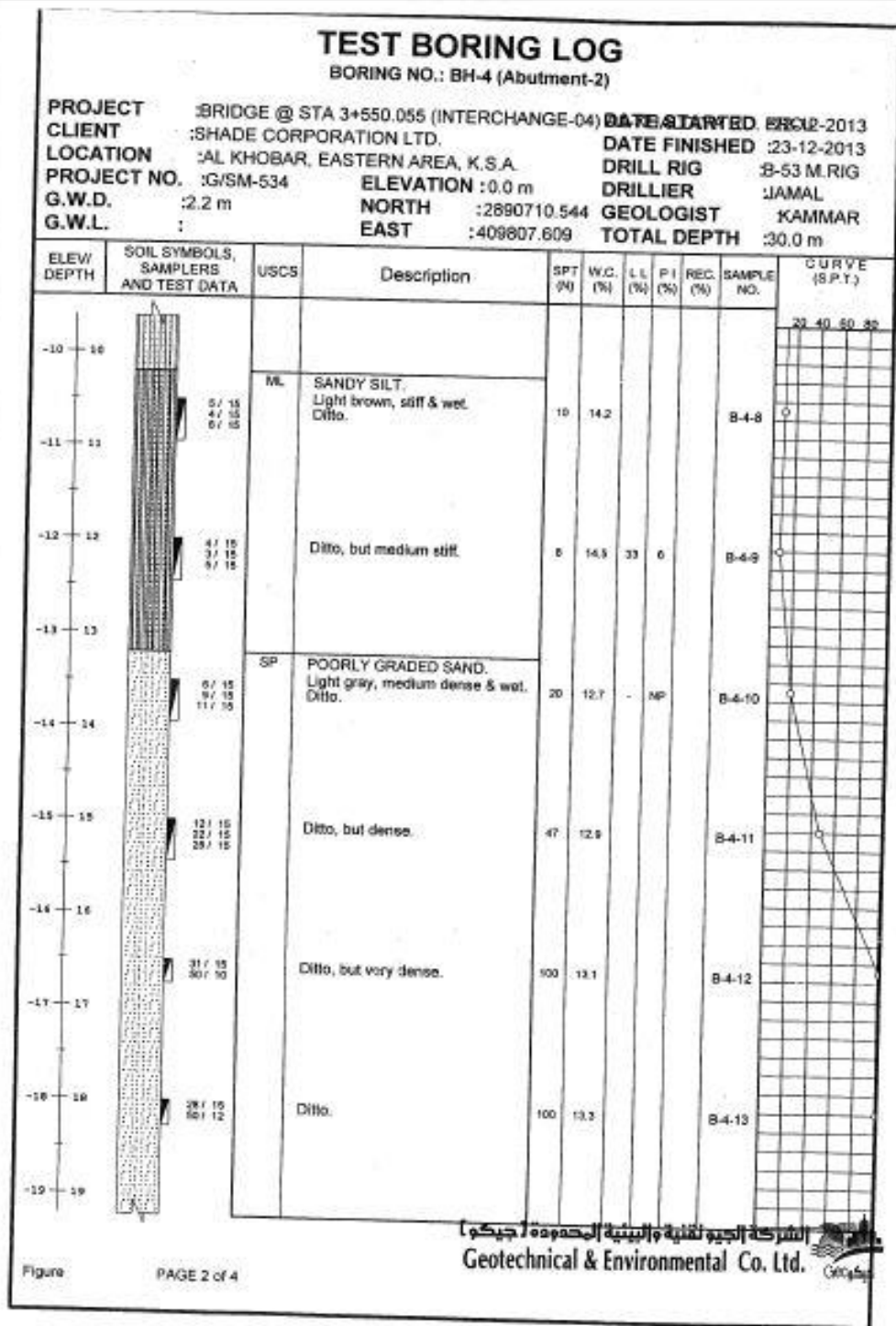


Figure 2.4 Example of field boreholes results – Part II (Geotechnical & Environmental Co Ltd)

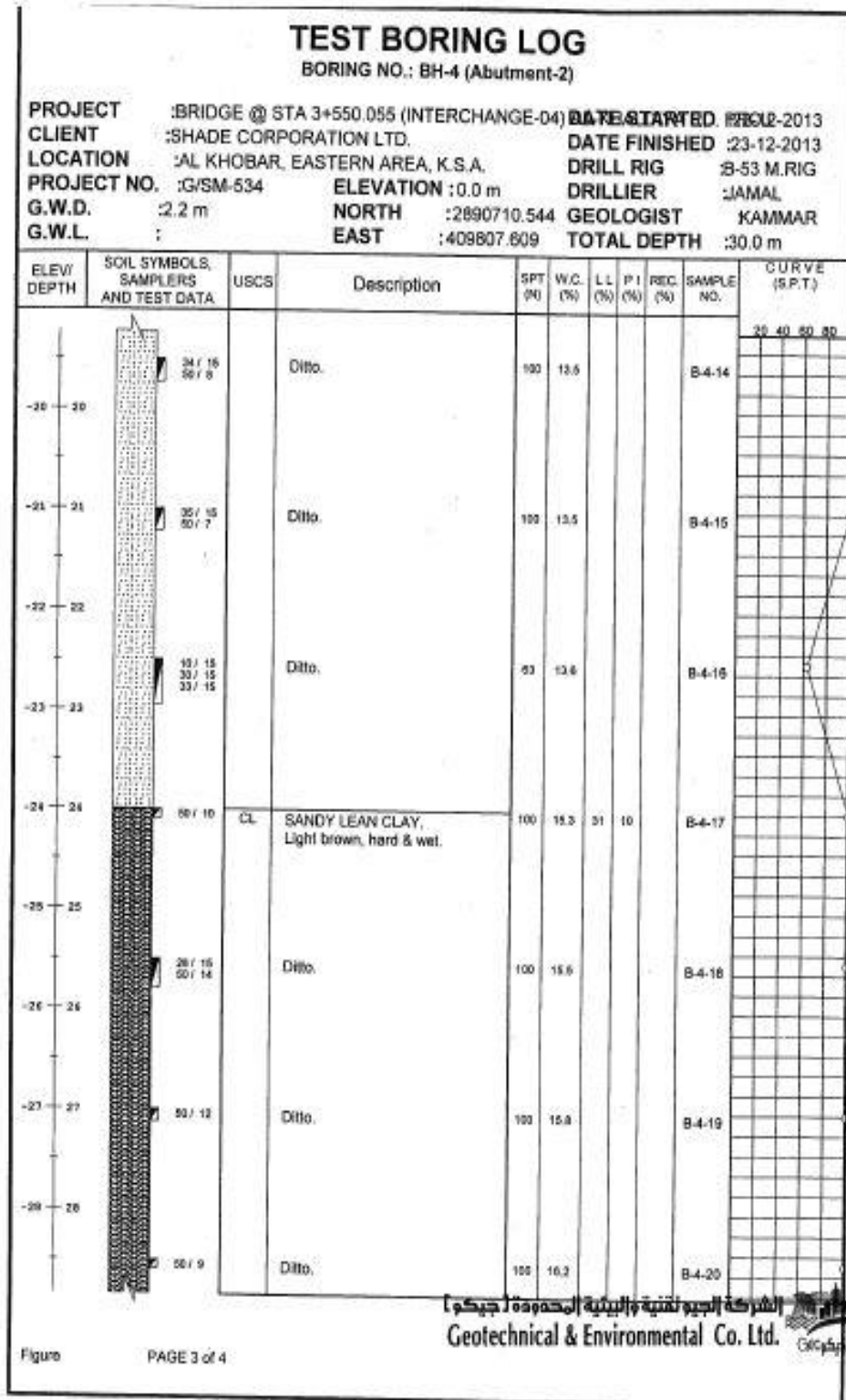


Figure 2.5 Example of field boreholes results – Part III (Geotechnical & Environmental Co Ltd)

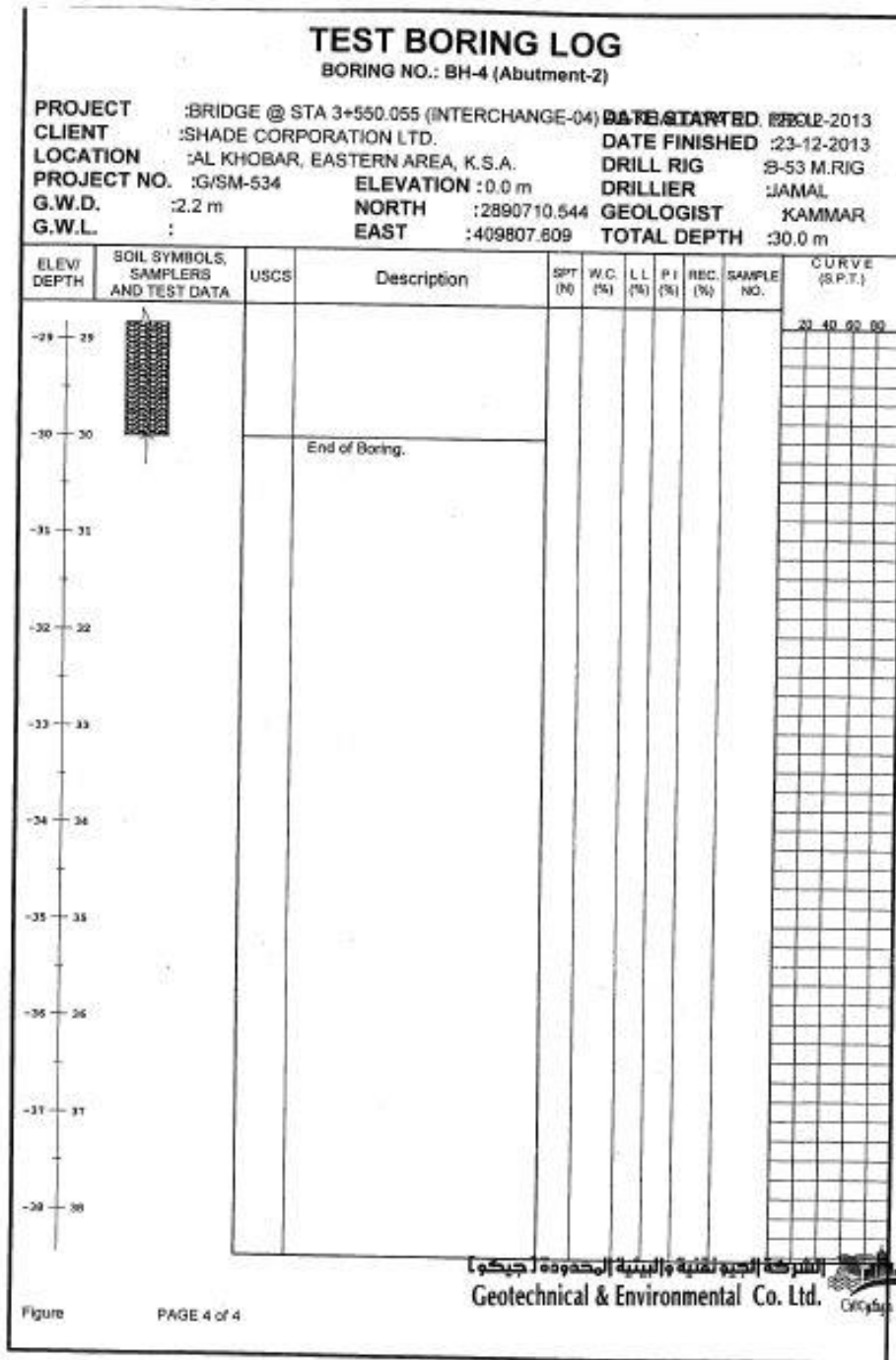


Figure 2.6 Example of field boreholes results – Part IV (Geotechnical & Environmental Co Ltd)

Laboratory tests were conducted on selected bulk (auger cuttings) and relatively undisturbed soil samples obtained from the soil boring to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory-testing program consisted of the following tests :i)- Plasticity index (ASTM D4 318) – used for soil clarification, ii)- Particle size Analyses (ASTM D 422) – used for soil classification and liquefaction evaluation, ii)- Unit dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216)- used for inset soil parameters, iii)- Direct shear (ASTM D3080) –used for soil strength determination, iv)- Unconfined Compression (ASTM D2166) – used for soil strength estimates, v)- and Chemical Analyses (soluble sulfates & chlorides, pH and resistivity) (Caltrans Methods) –used for concrete mix proportions and corrosion protection requirements. Some of the results of the laboratory test results are presented on the previous example of the subsurface logs.

The project site is located near a beach in an active area from a seismology point of view. The project site lies at elevation of approximately 1.2 meter above to 0.6 meter below mean sea level. The surrounding properties lie on terrain, which is flat (planar). Annual rainfall in this semi-arid region is less than 8 cm per year with eight months of average summertime temperatures above 35 C°. Winter temperatures are mild, seldom reaching freezing during the night.

The project site is located in the seismically active area and is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. The proposed site structures should be designed in accordance with the Saudi Building Code for ‘Maximum Considered Earthquake’ (MCE) and with the appropriate site coefficients. The MCE is defined as the ground motion having a 2 percent probability.

Subsurface soils encountered during the field exploration conducted on December 22 and 23, 2013 consist of dominantly of sands with interbedded silty sand in the northwestern portion of the site and interbedded sandy clay in the southeastern portion of the site. The examples of the subsurface logs (Figures 1 to 4) depict the stratigraphic relationships of the various soil types.

Liquefaction occurs when granular soil below the water table is subjected to vibratory motions, such as produced by earthquakes. With strong ground shaking, an increase in pore water pressure develops as the soil tends to reduce in volume. If the increase in pore water pressure is sufficient to reduce the vertical effective stress (suspending the soil particles in water), the soil strength decreases and the soil behaves as a liquid (similar to quicksand). Liquefaction can produce excessive settlement, ground rupture, lateral spreading, or failure of shallow bearing foundations. As stated previously, four conditions are generally required for liquefaction to occur: i)-The soil must be saturated (relatively shallow groundwater), ii)- The soil must be loosely packed (low to medium relative density), iii)- The soil must be relatively cohesionless (not clayey), and iv)- Groundshaking of sufficient intensity must occur to function as a trigger mechanism.

All of these conditions exist to some extent at this site. Liquefaction potential at the project site was evaluated using the 1997 NCEER Liquefaction Workshop methods. The 1997 NCEER methods utilize direct SPT blow counts or CPT cone readings from site exploration and earthquake magnitude PGA estimates from the seismic hazard analysis.

2.4 Relevant Recommendations

If the site of the project is treated by dynamic compaction, shallow spread footings are suitable to support the new multistory hotel, because the ground is adequately compacted. Footings shall be founded on adequately compacted soils. All exterior footings should be embedded a minimum of 18 inches below the building support pad or lowest adjacent final grade, whichever is deeper. Embedment depth of interior footings should be a minimum of 12 inches deep. Interior footing embedment depths shall be determined by the structural engineer/designer and should be sufficient to limit differential movement to 1.0 inch or less. The footings should have a minimum dimension of 24 inches and should be structurally tied to perimeter footings or grade beams. Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.

The use of driven steel piles requires special provisions for corrosion protection due to the corrosive nature of the subsurface soils. Precast, prestressed concrete piles are often used in the

corrosive soil environments of the site. Selection of pile type may be based on drivability and cost comparisons. Total settlements of less than 1 inch, and differential movement of about two-thirds of total movement for single piles designed according to the preceding recommendations. If pile spacing is at least 2.5 pile diameters center-to center, no reduction in axial load capacity is considered necessary for a group effect.

Concrete floor slabs shall be monolithically placed with the footings (no cold joints). The concrete slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. The vapor retarder should be properly lapped and continuously sealed and extend a minimum of 12 inches into the footing excavations. The vapor retarder should be overlain by 2 inches of clean sand (Sand Equivalent SE>30). Concrete slabs may be placed without a sand cover directly over a 15-mil vapor retarder (Stego-Wrap or equivalent). Concrete slab and flatwork reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 3 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential cracking. Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 3-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator). The construction joint between the foundation and any mow strips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint.

Pavements should be designed according to AASHTO or other acceptable methods. Traffic indices were not provided by the project engineer or owner. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. The site is dominated by surficial sands. Pavement structural section is provided for such soil.

CHAPTER 3: STRUCTURAL DESIGN

3.1 Introduction

This chapter covers the structural design of the multistory hotel. A computer program software named ETABS is used for this purpose. 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.

3.2 Hotel Structure

As indicated in chapter 2, the hotel is composed of twelve floors or stories. Each floor will cover an area around 1870 m^2 (i.e. $\pi \times 35 \times 17$). The dead and live load considered for the design area were taken from the Saudi Building Code (SBC). The maximum live load is taken to be 6.4 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 ETABS (Figures 2.1 to 2.7).

The structural design can be affected by two main loads: i)- gravitational loads (live and dead loads), and ii)- Lateral loads due to wind. However, there is a third type of load which can be included noted as seismic load (load due to earthquakes). This load will not be taken into consideration in this project because it is usually only effective in the case of high-rise buildings, or in areas of highly sensitive plates against earthquakes. Usually, after the tenth story the lateral drift will be the governing criteria in this structure. The interior structural system of this hotel consists of: i)- Moment Resisting Frames (MRF), and ii)- Reinforced Concrete Shear Walls. The first type, which is the MRF, consists of slabs and vertical columns connected rigidly. This type resists the load through the flexural stiffness of the members. The second type, which is

reinforced concrete shear walls, which is the popular type, resists the lateral load primarily through the axial stiffness of its members.

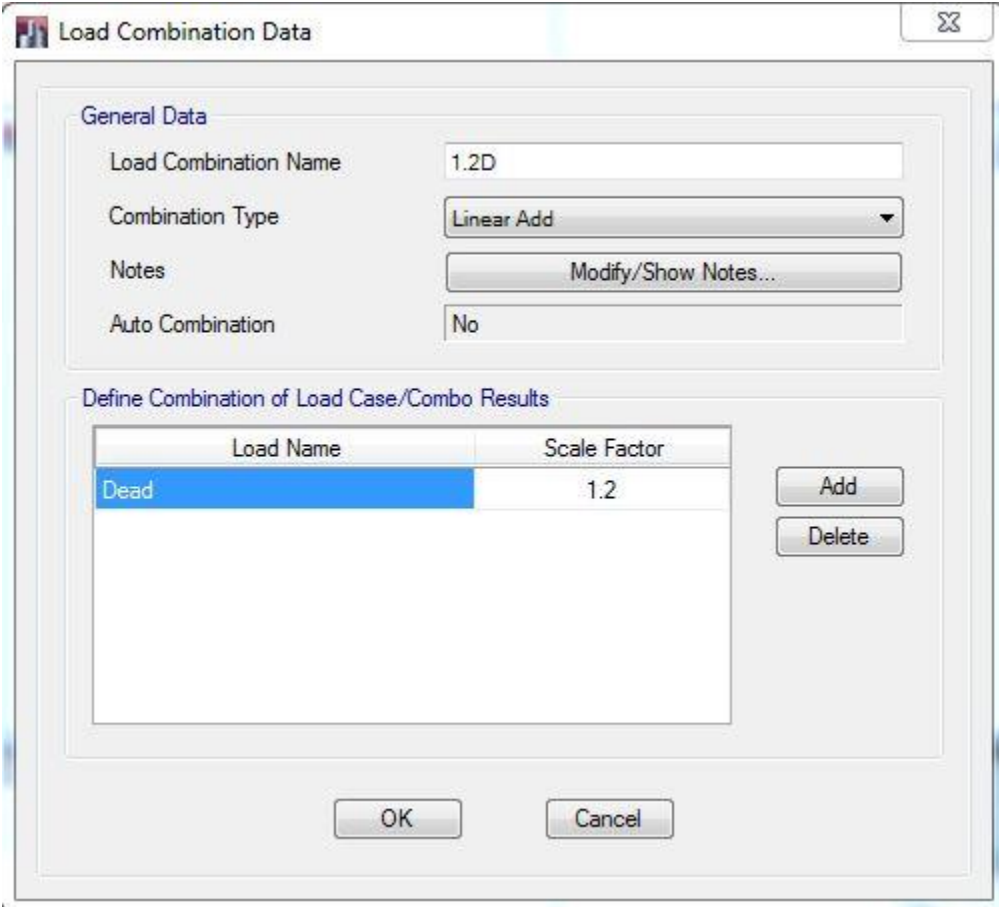


Figure 3.1 Example of "Dead" load with factor of 1.2 (ETABS)

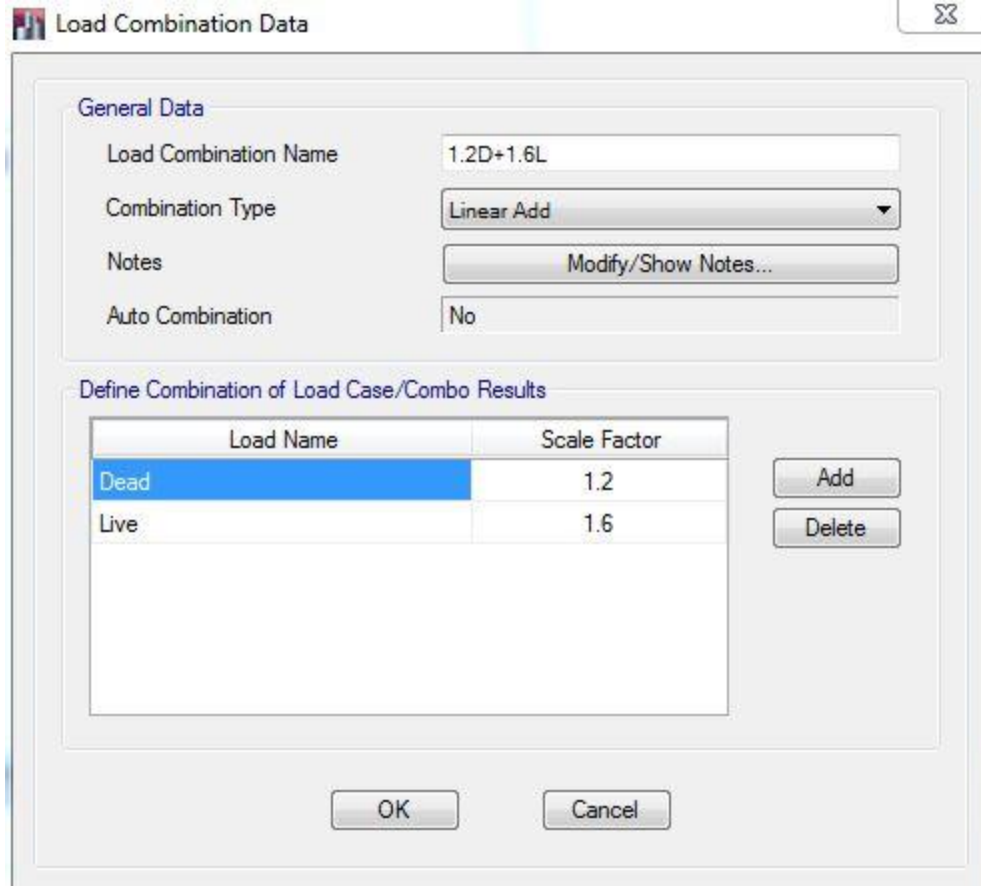


Figure 3.2 Example of “Live + Dead” load combination with a factor of 1.6 and 1.2 respectively (ETABS)

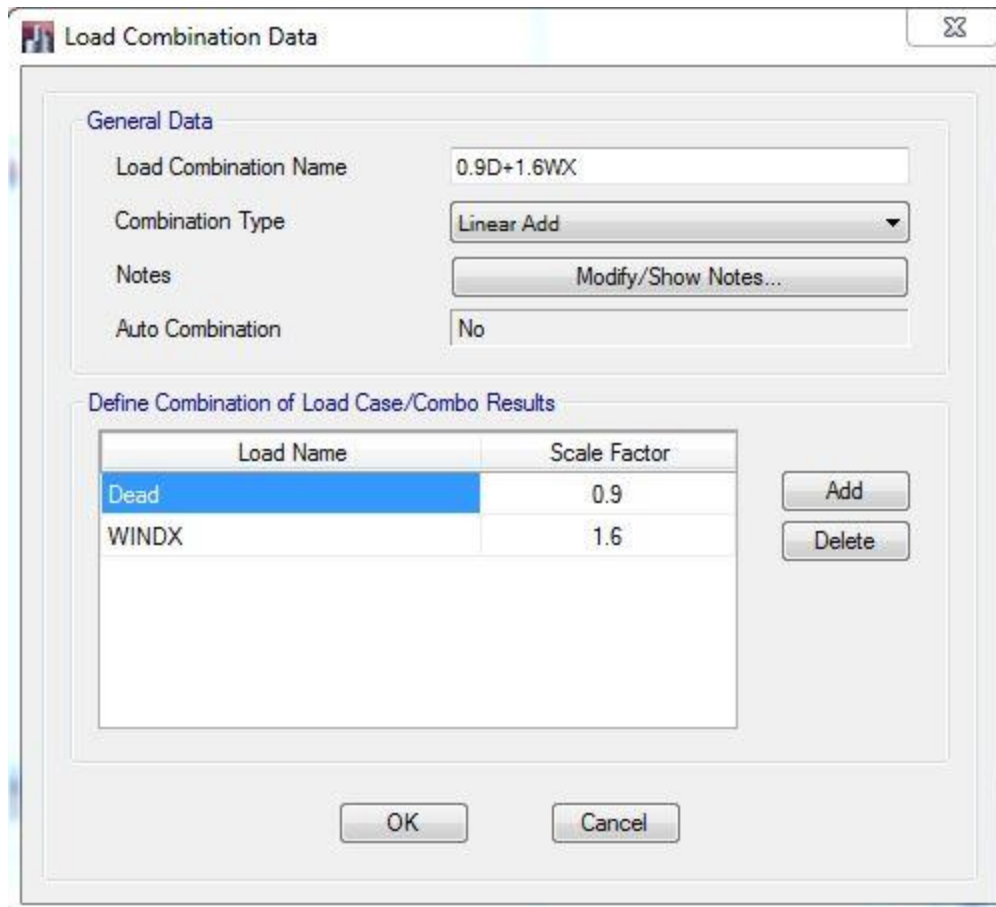


Figure 3.3 Example of “Dead + Wind x” load combination with a factor of 0.9 and 1.6 respectively (ETABS)

Four different column sizes have been used in the design for the projected hotel. It is well known that stiffer columns should produce less lateral drift when speaking about the static loads of the structure. However, in terms of real life practice, it might cost more to use one column size for the whole structure instead of using different column sizes. It is always more appropriate to use certain damping mechanisms to dissipate and absorb the vibration caused by the dynamic load of the wind. A damping mechanism is required to dissipate and absorb the vibration that is caused by the dynamic load of the wind. There are two types of damping systems; one is passive system, and the second is active system. However, these systems will not be discussed in this report because the wind factor will be mostly effective in high-rise buildings.

3.3 Hotel Facilities

The following table summarize the different hotel facilities for the twelve floors:

Floor #	Facilities
8th floor	Swimmingpool&gymnasium
7th floor	Elevator electrical machine room+ air conditioning+ elevator mechanical machine room+ fan room
6th floor	Regular bedrooms
5th floor	Regular bedrooms
4th floor	Regular bedrooms
3rd floor	Regular bedrooms
2nd floor	Regular bedrooms
1st floor	Regular bedrooms
Ground floor	Lobby+ public rooms+ dining room+ private kitchen+ Hotel laundry+ public toilets
Parking (-1)	Car parking
Parking (-2)	Car parking
Parking (-3)	Car parking

Table 3.1Hotel facilities

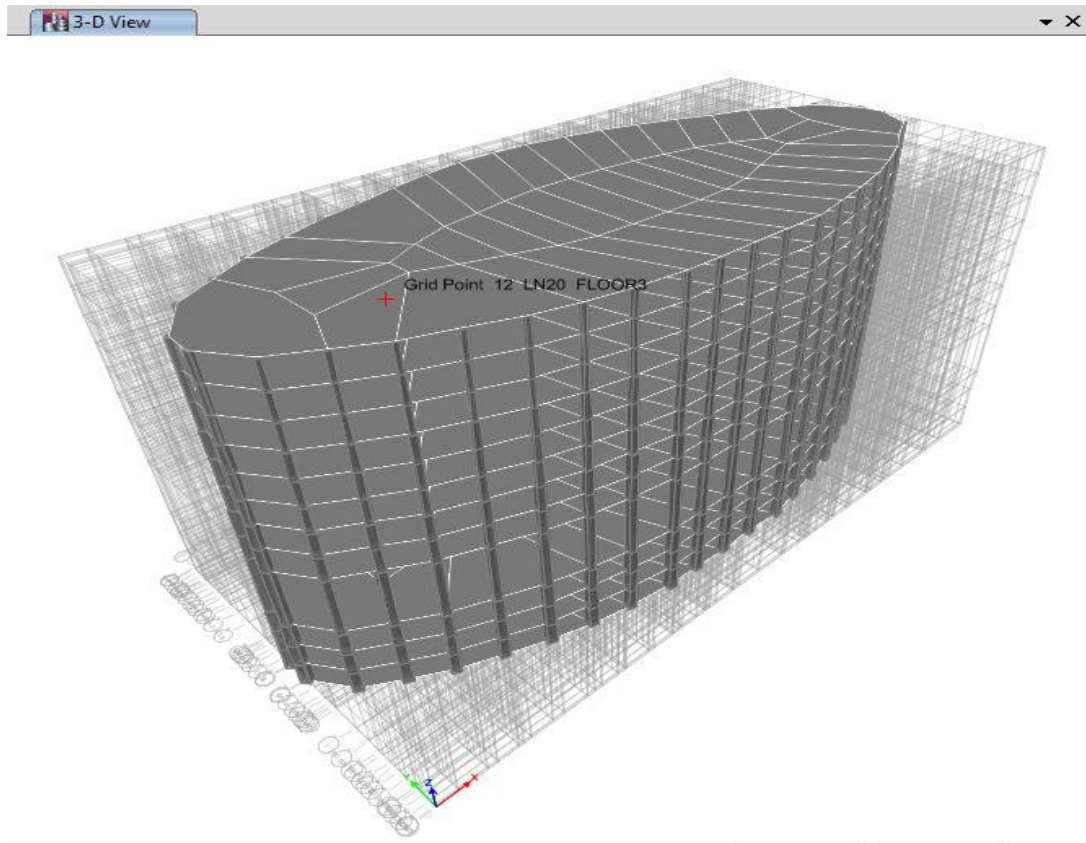


Figure 3.4“3-D View” of the hotel in ETABS

3.4 Structural Design

3.4.1 Concrete Slab Design:

The first stage of the structural design in a building is the Slab. In this structure the slab is a two-way slab without girders, beams, or sub-beams. Using a column-slab design system has its advantages and disadvantages. One of the advantages is that it requires less hand calculations and less modeling in the ETABS software. One of the disadvantages is that the slabs will have two-way moments; deflection along the x-axis and another one along the y-axis. Another disadvantage is that the loads will be transferred directly from the slab to the column, which might cause deflection in the slab more than the allowed maximum of 2.54 cm. Excessive torsion and moment could also occur in the columns.

The first step in designing the concrete slab is to check the minimum slab thickness. According to the American Concrete Institute (ACI) Code (Table 9.5-c, page 130), to design the slab thickness, $(L_n/30)$ and $(L_n/33)$ formulas must be used to check the slab thickness for exterior and interior panels consecutively. “ L_n ” corresponds to any vertical or horizontal distance between the tip of two columns. In this case, the maximum slab thickness achieved was found in the exterior panel by dividing 10m over 30 to get 0.3m. The next step is to check the shear punch strength of the slabs and columns. The purpose of this procedure is to make sure that the slab strength (ΦV_c) is greater than the column strength (V_u). If V_u is less than ΦV_c , the column may punch through the slab, which will cause the slab to fail. The following calculations are done for different floors of the hotel to show that the slab thickness will be safe.

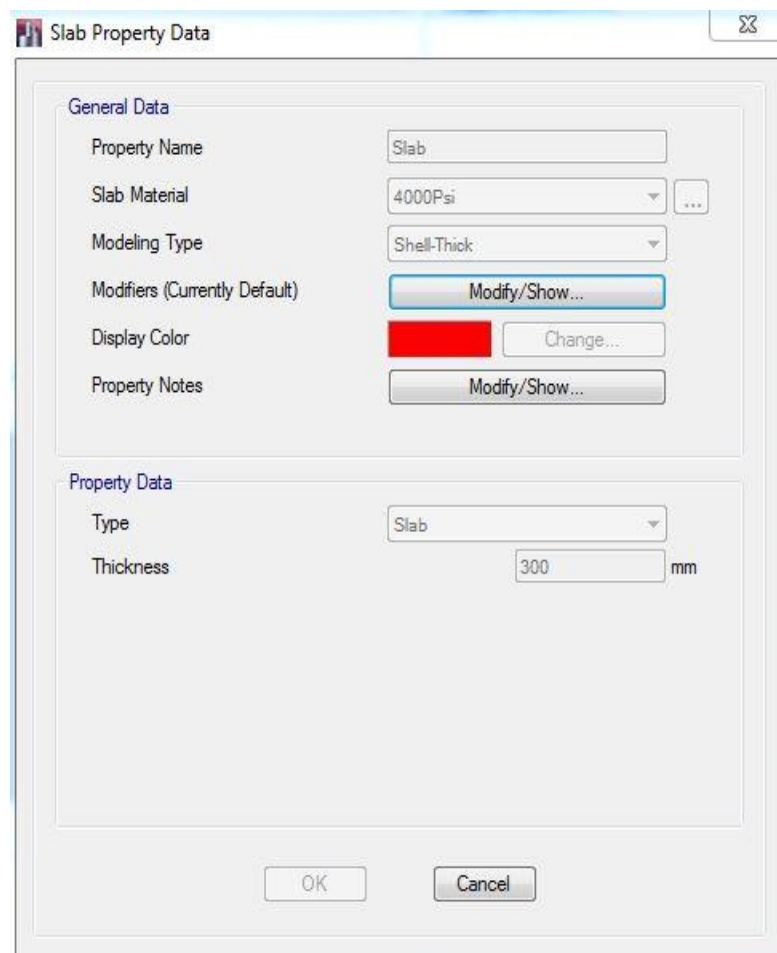


Figure 3.5 Slab properties

3.4.1.1 Punching Shear Check

The first punching shear calculations are performed for the parking floors:

Length of column (depth of column) = 0.88 m

Density of concrete = 23.5 KN/m³

Thickness of concrete slab = 0.3 m

Largest slab area resting on the column = 60 m²

Self-weight of slab = 23.5 × 0.3 × 60 = 423 (total dead load)

Live load = 4.9 KN/m²

Total live load = 4.9 KN/m² × 60 = 294 KN

Using 1.2 factor for dead load and 1.6 factor for live load:

Total load (P_u) = 1.2 × (423) + 1.6 × (294) = 978 KN

To find the critical shear parameter, “d” must be calculated and added to the original length of the column. For 50.8 cm column length, d = 15.494 cm (Reference: Reinforced Concrete Mechanics and Design sixth edition Textbook-Page 759)

50.8 cm = 15.494 cm

88 cm = “d” cm → d = 26.84 cm = 0.2684 m

So, critical shear parameter = 0.88 + 0.2684 = 1.1484 m

b₀ = 2 × (1.1484 + 1.1484) = 4.5936 m

Dividing the total load over the area to get the load per unit area:

978/60 = 16.3 KN/m²

Column strength (V_u) = 16.3 × [60 - (1.1484 × 1.1484)] = 16.3 × [57.7032] = 940.56 KN

Now, converting “d” and “b₀” to inches to obtain the slab strength (ΦV_c):

D = 26.84 cm = 10.57 inches

b₀ = 459.36 cm = 180.85 inches

Taking Φ = 0.75, λ = 1, f_c' = 4000 (Reinforced Concrete Mechanics and Design sixth edition Textbook-Page 760):

Slab strength (ΦV_c) = Φ × 4 × λ × √f_c' × b₀ × d = 0.75 × 4 × 1 × √4000 × 180.85 × 10.57 = 362697.6576

lbs = 1613.36 KN

Since V_u is less than ΦV_c , then the slab thickness condition is good (i.e. the column will not punch through the slab).

The second punching shear calculations are performed for the lobby and reception area (ground floor):

Length of column= depth of column= 0.74 m

Density of concrete= 23.5 KN/m³

Thickness of concrete slab= 0.3m

Dead load of ceramic tile= density of ceramic tile x thickness of ceramic tile x largest slab area resting on the column= $60\text{m}^2 = 23.5 \times 0.005 \times 60 = 7 \text{ KN}$

Dead load of concrete = 423 KN

Live load = 6.4 KN/m²

Total live load = 6.4 x 60 = 384 KN

Total dead load = 430 KN

$P_u = 1.2 \times (430) + 1.6(384) = 1130.4 \text{ KN}$

For 50.8 cm column length, $d = 15.494 \text{ cm}$:

50.8 cm= 15.494 cm

74 cm= "d" cm $\rightarrow d = 22.57 \text{ cm} = 0.2257 \text{ m}$

So, critical shear parameter= $0.74 + 0.2257 = 0.9657 \text{ m}$

$b_0 = 2(0.9657 + 0.9657) = 3.8628 \text{ m}$

Dividing the total load over the area yields:

$1130.4/60 = 18.84 \text{ KN/m}^2$

Column strength (V_u)= $18.84 \times [60 - (0.9657 \times 0.9657)] = 18.84 \times [59.0674] = 1112.8298 \text{ KN}$

$D = 22.57 \text{ cm} = 8.89 \text{ inches}$

$b_0 = 386.28 \text{ cm} = 152.08 \text{ inches}$

Slab strength (ΦV_c)= $\Phi \times 4 \times \lambda \times \sqrt{f_c} \times b_0 \times d = 0.75 \times 4 \times 1 \times \sqrt{4000} \times 152.08 \times 8.89 = 256522.2941 \text{ lbs} = 1141.068 \text{ KN}$

Since V_u is less than ΦV_c , then the slab thickness condition is good (i.e. the column will not punch through the slab).

Now, checking the punching shear for bedroom floors located in the 3rd, 4th, and 5th floors of the hotel:

Column length = column depth = 0.6 m

Deal load of ceramic tile = density of ceramic x ceramic thickness x area = $23.5 \times 0.005 \times 60 = 7$ kN

Self-weight of slab = slab thickness x concrete density x area = $0.3 \times 23.5 \times 60 = 423$ kN

Total dead load = $423 + 7 = 430$ kN

Live load = 2 kN/m

Total live load = $2 \times 60 = 120$ kN

$P_u = 1.2 \times (430) + 1.6 \times (120) = 708$ kN

For 50.8 cm column length, $d = 15.494$ cm:

50.8 cm = 15.494 cm

60 cm = "d" cm $\rightarrow d = 18.3$ cm = 0.183 m

So, critical shear parameter = $0.6 + 0.183 = 0.783$ m

$b_0 = 2 \times (0.783 + 0.783) = 3.132$ m

Dividing the total load over the area yields:

$708/60 = 11.8$ kN/m²

Column strength (V_u) = $11.8 \times [60 - (0.783 \times 0.783)] = 11.8 \times [59.39] = 700.802$ kN

$D = 18.3$ cm = 7.20 in

$b_0 = 313.2$ cm = 123.31 in

Slab strength (ΦV_c) = $\Phi \times 4 \times \lambda \times \sqrt{f'_c} \times b_0 \times d = 0.75 \times 4 \times 1 \times \sqrt{4000} \times 123.31 \times 7.20 = 168454.278$ lbs = 749.32 kN

Since V_u is less than ΦV_c , then the slab thickness condition is good (i.e. the column will not punch through the slab).

3.4.1.2 Steel Bars Reinforcement

After checking that the slab thickness is safe for different floors in the hotel, the next step is to design the slab reinforcement. Instead of designing the reinforcement for each floor separately, a more efficient method time wise is to reinforce all the slabs based on the maximum

positive, and maximum negative moments. These moment values are taken from the ETAB Software after running the analysis. It was found that both positive and negative slab moments were occurring in the slab of the first floor. The maximum positive value was 252 kN.m/m, and the maximum negative value was -225 kN.m/m. after obtaining these values, the reinforcement design of the positive and negative moments have to be performed separately as follows:

Positive moment design calculations:

Ultimate moment (M_u)= 252 KN.m = 185.9Kips.ft (take as 186 Kips.ft)

Slab thickness (h)= 0.3 m= 30 cm= 11.8 in (take as 12 in)

Design width (b)= 1 m= 100 cm= 39.4 in (take as 40 in)

Estimated depth (d)= slab thickness (h) – 3= 12 – 3= 9 in

Required Coefficient of resistance (K)= (M_u)/($\Phi b d^2$)

Where,

M_u is in Kips.in, and Φ = 0.9

$K = 186(12)/(0.9)(40)(9)^2 = 0.7654 \text{ Ksi}$

Using compressive strength of concrete (f'_c) and yield strength of steel (f_y) as 4,000 psi and 60,000 psi consecutively.

From coefficient of resistance (K) versus Reinforcement Ratio (ρ) table, and by interpolation, the value of $\rho = 0.0146$ (required reinforcement ratio)

(Reference: Table A-8 of American Concrete Institute (ACI 318-11) Manual)

Required area of steel (A_s)_{req}= (ρ)_{req} x b x d = (0.0146) x (40) x (9)= 5.256 in²

Balanced area of steel (A_s)_{bal}= (ρ)_{bal} x b x d = (0.0120)x(40)x(9)= 4.32 in²

Minimum area of steel (A_s)_{min} = (200/ f_y) x b x d = (200/60,000) x 40 x 9= 1.2 in²

Maximum area of steel (A_s)_{max} = (ρ)_{max} x b x d = (0.00214) x (40) x (9) = 7.704 in²

The required area of steel has to be in the range between the minimum and the maximum areas of steel in order to continue the design procedure. When designing the slab reinforcement, the design must be performed for 1 foot of slab (b= 1ft= 12in):

$$\text{So, required } A_s = \rho \times b \times d = (0.0146) \times (12) \times (9) = 1.5768 \text{ in}^2$$

According to the “cross-sectional area of reinforcing bars per foot of slab” table in ACI 318-11, the reinforcement options are as follows:

Bar #	Number of bars	Total area of bars (in ²)	Cross sectional area of one bar (in ²)	Bar-to-bar spacing (in)
5	6	1.86	0.31	To be calculated
6	4	1.76	0.44	To be calculated

Table 3.2 Steel bars reinforcement options for positive moment design

Bar #5 option:

$$\text{Area of one bar} = 0.31 \text{ in}^2 \rightarrow \text{radius} = 0.314 \text{ in} \rightarrow \text{diameter} = 0.628 \text{ in}$$

$$\text{Width of slab} = (\# \text{ of bars} \times \text{diameter}) + (\#3 \text{ stirrup diameter} \times 2) + (\text{concrete cover} \times 2) + \text{spacing} = 12 \text{ in}$$

$$(6 \times 0.628) + (0.38 \times 2) + (1.5 \times 2) + S = 12 \text{ in}$$

$3.768 + 0.76 + 3 + S = 12 \rightarrow 7.528 + S = 12 \rightarrow S = 4.472 \text{ in}$, this is the total spacing distance, to get the spacing distance between 2 bars (bar-to-bar spacing), the total spacing distance is divided by the number of distances and subtracted by one. In this case, $6 - 1 = 5$, thus, the bar-to-bar spacing = $4.472 / 5 = 0.8944 \text{ in}$.

Bar#6 option:

Area of one bar = $0.44 \text{ in}^2 \rightarrow$ radius = $0.374 \text{ in} \rightarrow$ diameter = 0.748 in

Width of slab = $(4 \times 0.748) + (0.38 \times 2) + (1.5 \times 2) + S = 12 \rightarrow 6.752 + S = 12 \rightarrow S = 5.248 \text{ in}$

Thus, the bar-to-bar spacing = $5.248/3 = 1.7493 \text{ in}$

Bar #5 actual depth (d') = slab thickness (h) – cover - #3 stirrup diameter – (diameter of bar)/2 =
 $12 - 1.5 - 0.38 - (0.628/2) = 9.806 \text{ in}$

Bar #6 actual depth (d') = $12 - 1.5 - 0.38 - (0.748/2) = 9.746 \text{ in}$

Actual depth in both bar #5 and bar #6 is greater than the estimated depth (d), which is 9 in.

Thus, any of the two options is safe for the positive moment design of this slab.

The final step of designing the slab reinforcement is the negative moment design. The procedure is exactly the same as the positive moment design. However, since the maximum negative moment value is different from the maximum positive moment value, the coefficient of resistance (K), the required reinforcement ratio, and the required area of steel will be different. The negative moment design calculations are as below:

Negative Moment Design Calculations:

Ultimate moment (M_u) = $225 \text{ KN.m} = 166 \text{ Kips.ft}$

Slab thickness (h) = $0.3 \text{ m} = 30 \text{ cm} = 11.8 \text{ in}$ (take as 12 in)

Design width (b) = $1 \text{ m} = 100 \text{ cm} = 39.4 \text{ in}$ (take as 40 in)

Estimated depth (d) = slab thickness (h) – 3 = $12 - 3 = 9 \text{ in}$

Required $K = 166(12)/(0.9)(40)(9)^2 = 0.6832 \text{ Ksi}$

From coefficient of resistance (K) versus reinforcement ratio (ρ) table, and by interpolation:

Required reinforcement ratio (ρ)_{req} = 0.0129

Required A_s = (0.0129) x (40) x (9) = 4.644 in²

Balanced area of steel (A_s)_{bal} = 4.32 in²

Maximum area of steel (A_s)_{max} = 7.704 in²

Minimum area of steel (A_s)_{min} = 1.2 in²

The balanced, maximum, and minimum areas of steel values are equal in the negative and positive moment design because they are taken from the same table. Designing for 1 foot of slab (b = 1ft = 12in):

Required area of steel (A_s)_{Req} = (0.0129) x (12) x (9) = 1.3932 in²

The steel bar that will be chosen for the negative moment design has to have a minimum cross sectional area of 1.3932 in². Based on this required area of steel value, the steel bar reinforcement options are as follows:

Bar#	Number of bars	Total area of bars (in ²)	Cross sectional area of one bar (in ²)	Bar-to-barspacing (in)
5	6	1.86	0.31	0.8944 (calculated in the positive moment design)
5	To be calculated	1.49	To be calculated	2.5
6	4	1.76	0.44	1.7493 (calculated in the positive moment design)
6	To be	1.51	To be	3.5

	calculated		calculated	
--	------------	--	------------	--

Table 3.3 Steel bars reinforcement options for negative moment design

Bar #5 (area= 1.86 in²):

Was calculated in the positive moment design:

Bar-to-bar spacing = 0.8944 in

Bar #6 (area= 1.76 in²):

Was calculated in the positive moment design:

Bar-to-bar spacing = 1.7493 in

Bar #5 (area= 1.49 in²):

Let # of bars= n, bar diameter= 0.628 in

Width of slab= (n x 0.628) + (0.38x2) + (1.5x2) + (n-1)(2.5) = 12

→ 0.628n + 3.76 + 2.5n – 2.5 = 12 → 3.128n + 1.26 = 12

→ n = 3.434, since “n” has to be a counting number, the number of bars will be taken as 4. In this case, the spacing has to be recalculated:

Let spacing = S

Width of slab = (4x0.628) + (0.38x2) + (1.5x2) + S = 12 in

→ 6.272 + S = 12 → S = 5.728 in, thus, bar-to-bar spacing = 5.728/3 = 1.9093 in.

Bar #6 (area = 1.51 in²):

Let # of bars= n, bar diameter = 0.748 in

Width of slab = (n x 0.748) + (0.38x2) + (1.5x2) + (n-1)(3.5) = 12 in

$$\rightarrow 0.748n + 3.76 + 3.5n - 3.5 = 12 \text{ in} \rightarrow 4.248n + 0.26 = 12 \text{ in}$$

$\rightarrow n=2.764$, take $n=3$, and recalculate spacing:

Let spacing = S

$$\text{Width of slab} = (3 \times 0.748) + (0.38 \times 2) + (1.5 \times 2) + S = 12$$

$$\rightarrow 6.004 + S = 12 \rightarrow S = 5.996 \text{ in, bar-to-bar spacing} = 5.996/2 = 2.998 \text{ in.}$$

Bar #5 actual depth (d') = 9.806 in (calculated previously in the positive moment design).

Bar #6 actual depth (d') = 9.746 (calculated previously in the positive moment design).

All the four options mentioned above have an actual depth greater than the estimated depth (d), which is 9 in. Thus, they are all safe for the negative moment design of this slab.

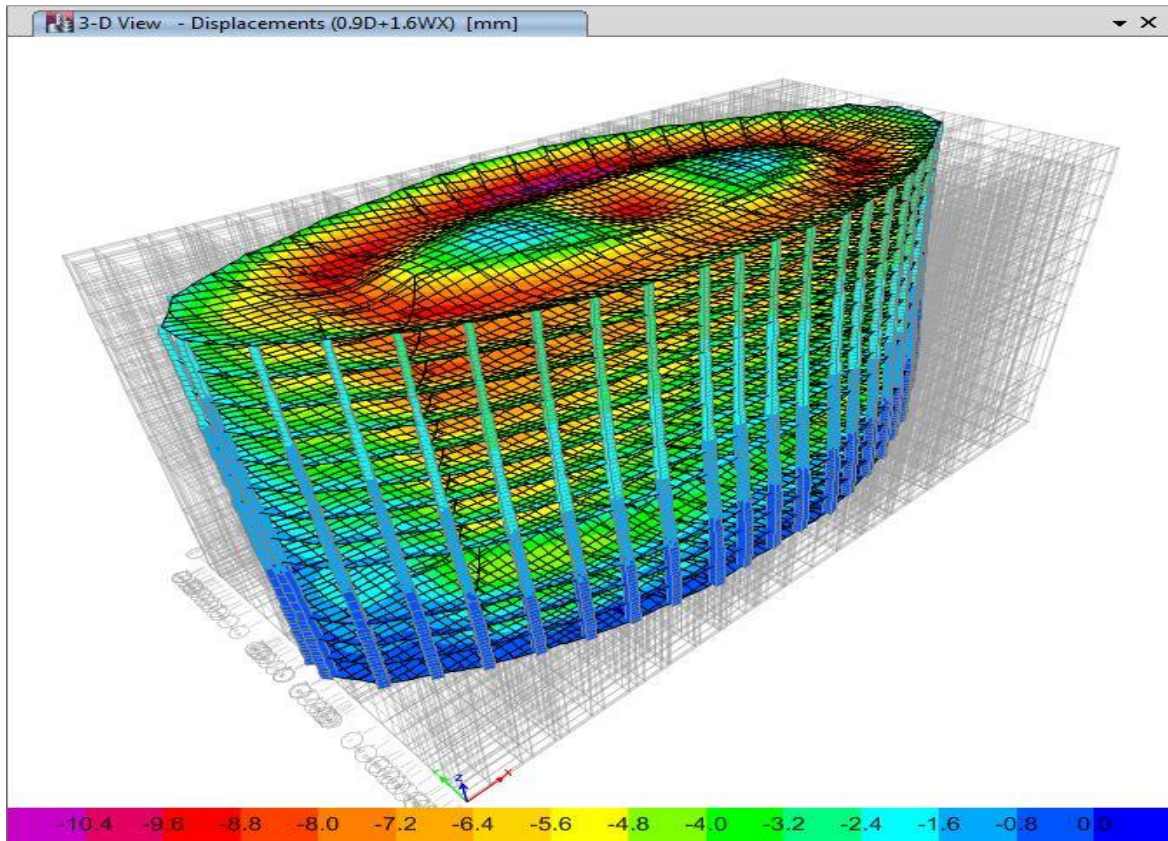


Figure 3.6 Deformed shape of the slabs – Dead load factor 0.9 (ETABS)

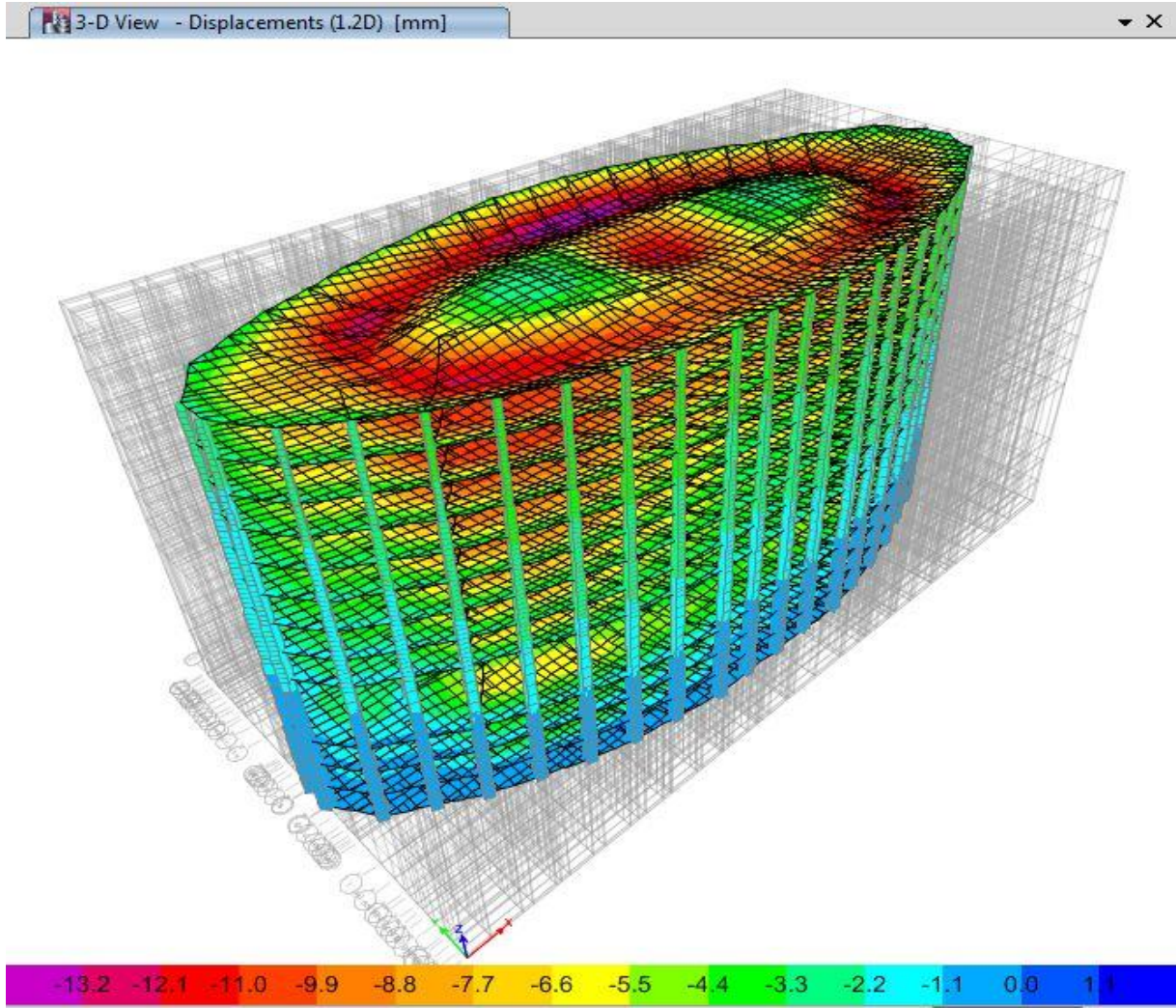


Figure 3.7 Deformed shape of slabs – Dead load factor 1.2 (ETABS)

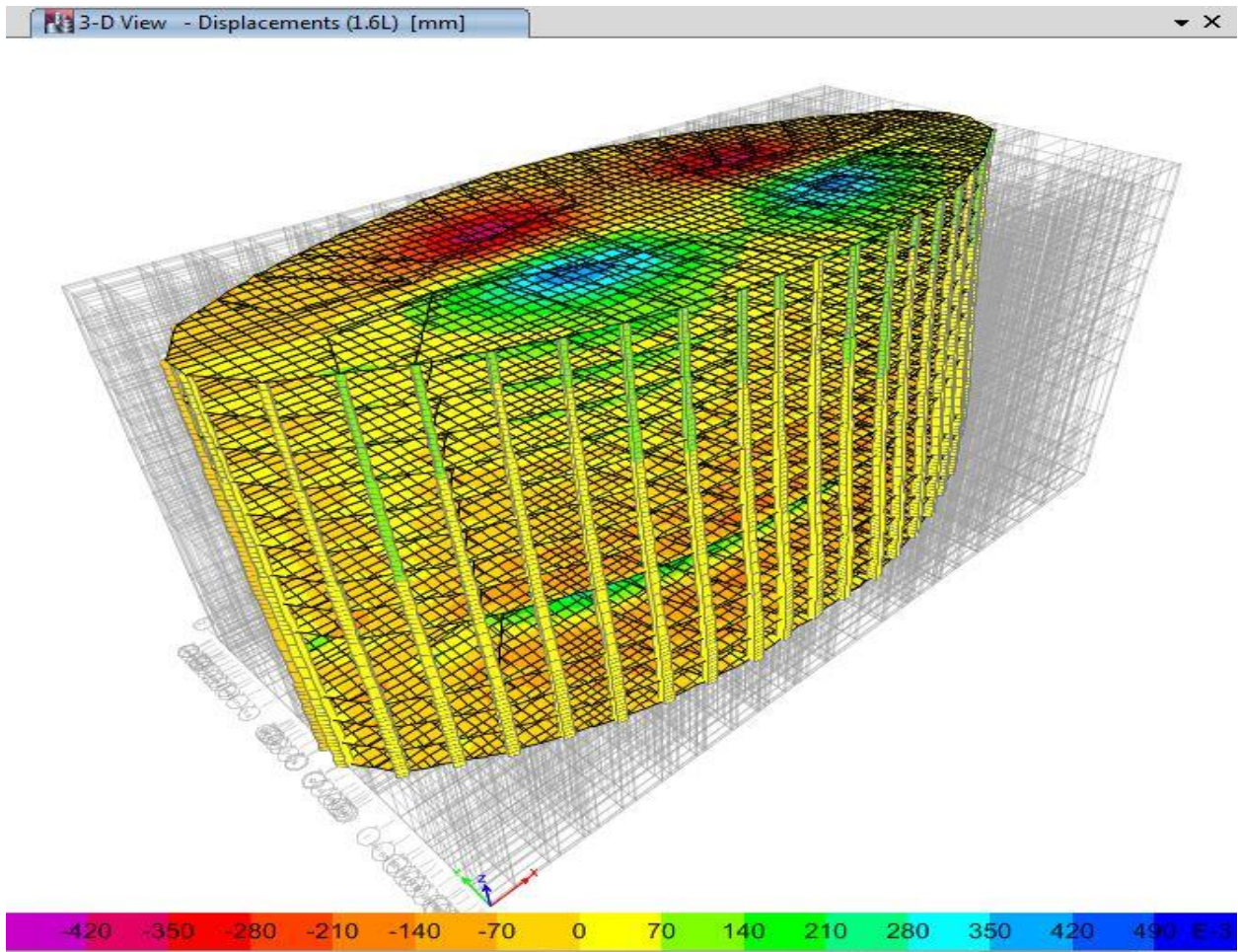


Figure 3.8 Deformed shape of slabs – dead load factor 1.6 (ETABS)

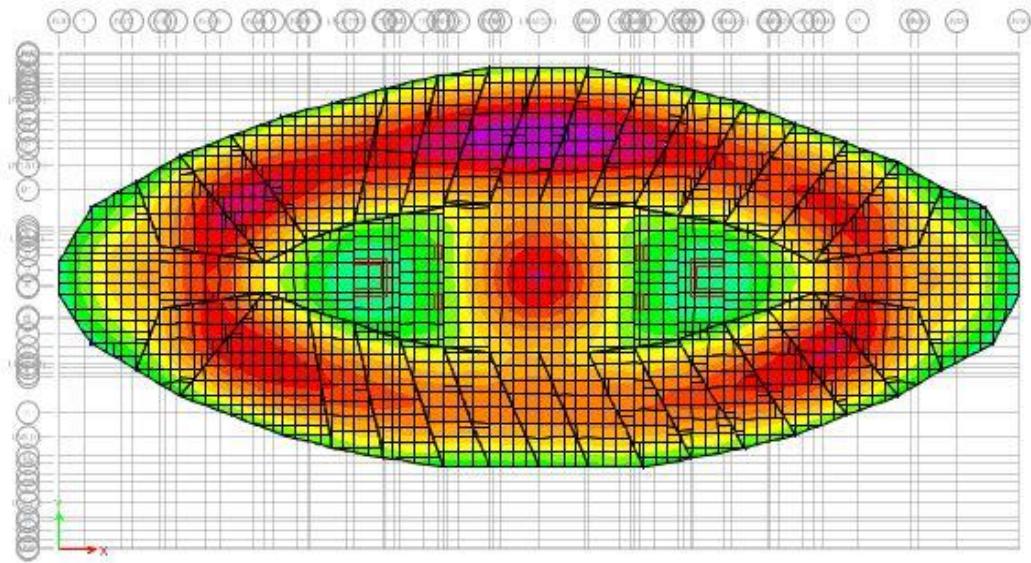


Figure 3.9 Deformed shape of the slab of floor #8 – Dead load factor 1.2 & Live load factor 1.6 (ETABS)

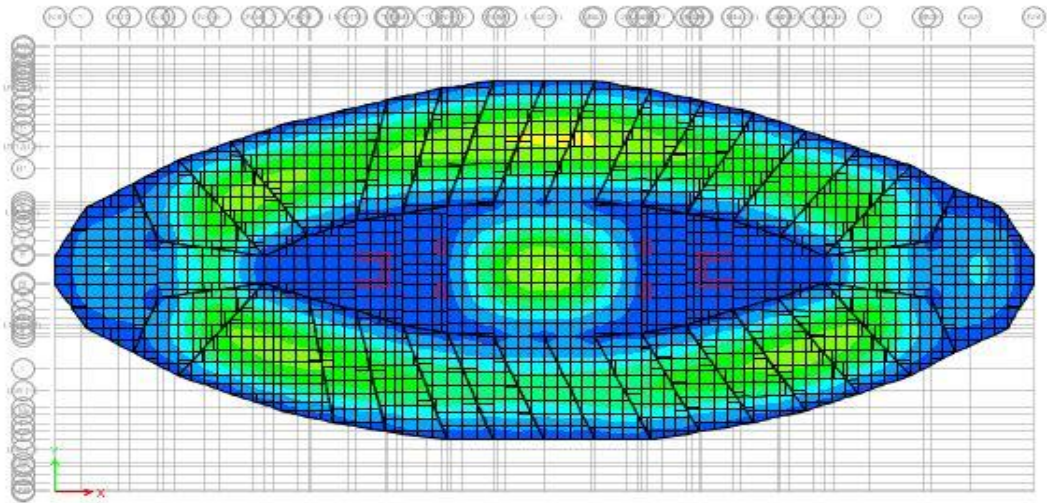


Figure 3.10 Deformed shape of the slab of parking 3 – Dead load factor 1.0 (ETABS)

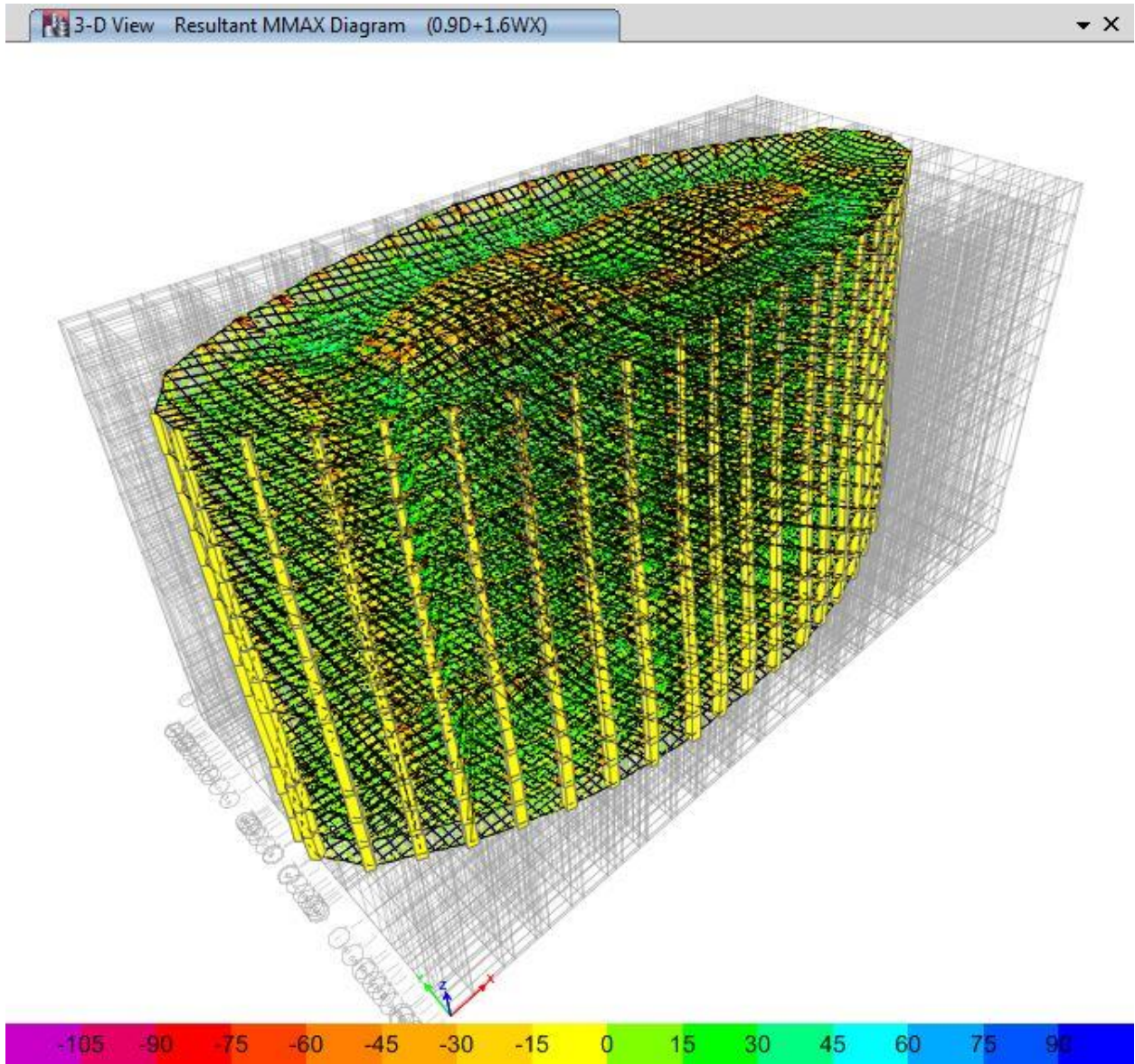


Figure 3.11 Moments of the different slabs – Dead load factor 0.9 & Wind X 1.6 (ETABS)

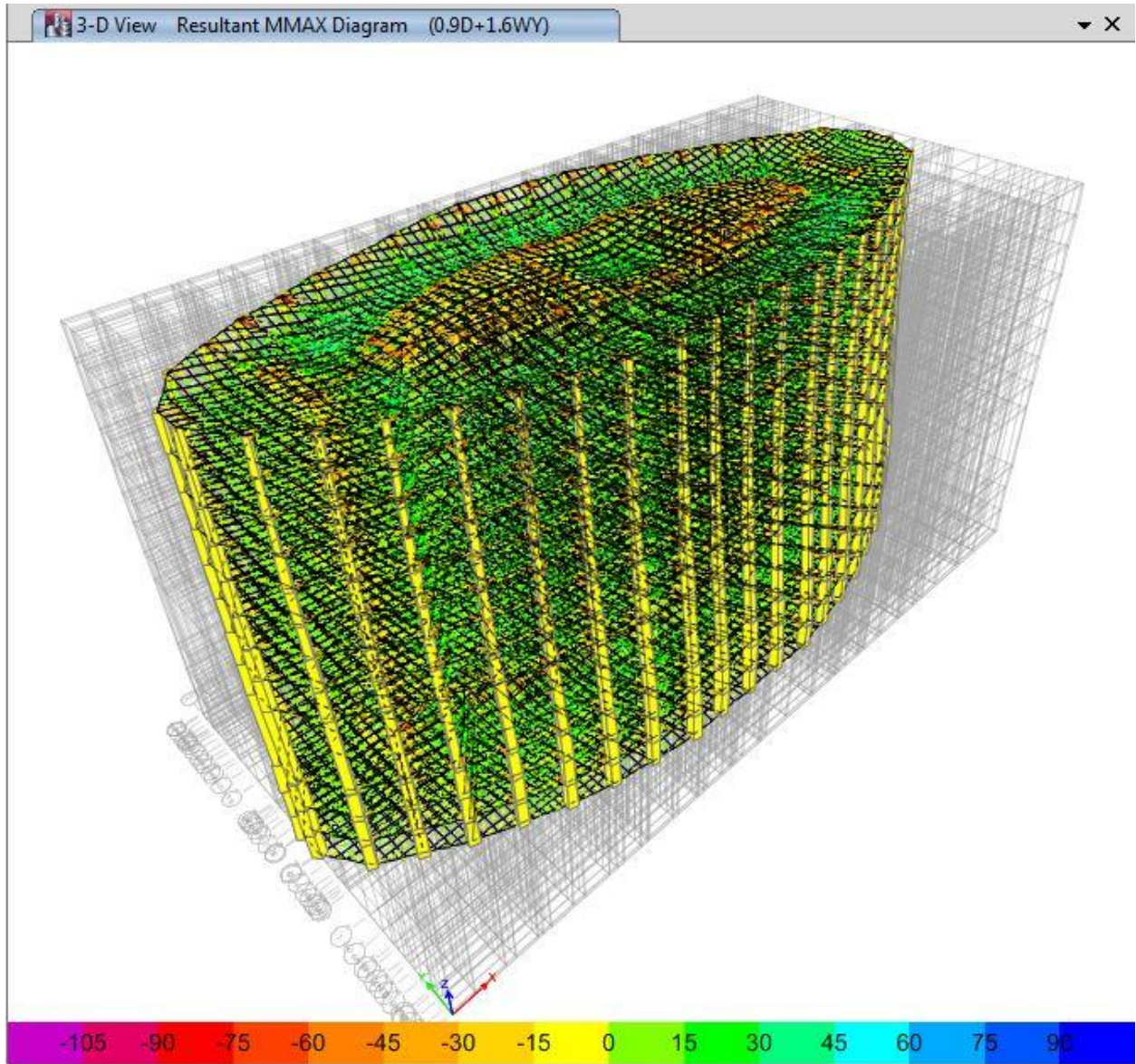


Figure 3.12 Deformed shape of the different slabs – Dead load factor 0.9 & Wind Y 1.6 (ETABS)

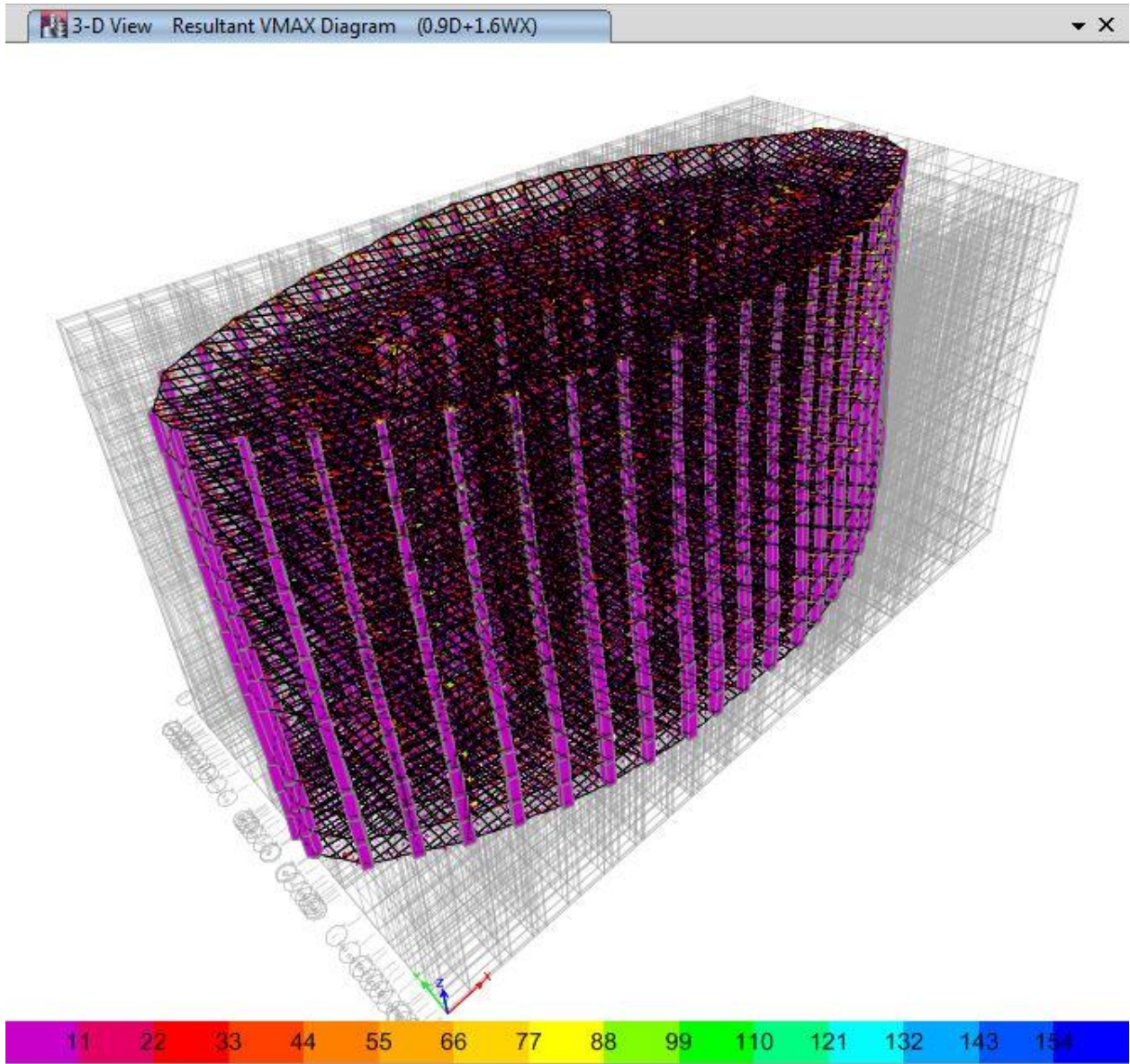


Figure 3.13 Shear of the different slabs – Dead load factor 0.9 & Wind X 1.6 (ETABS)

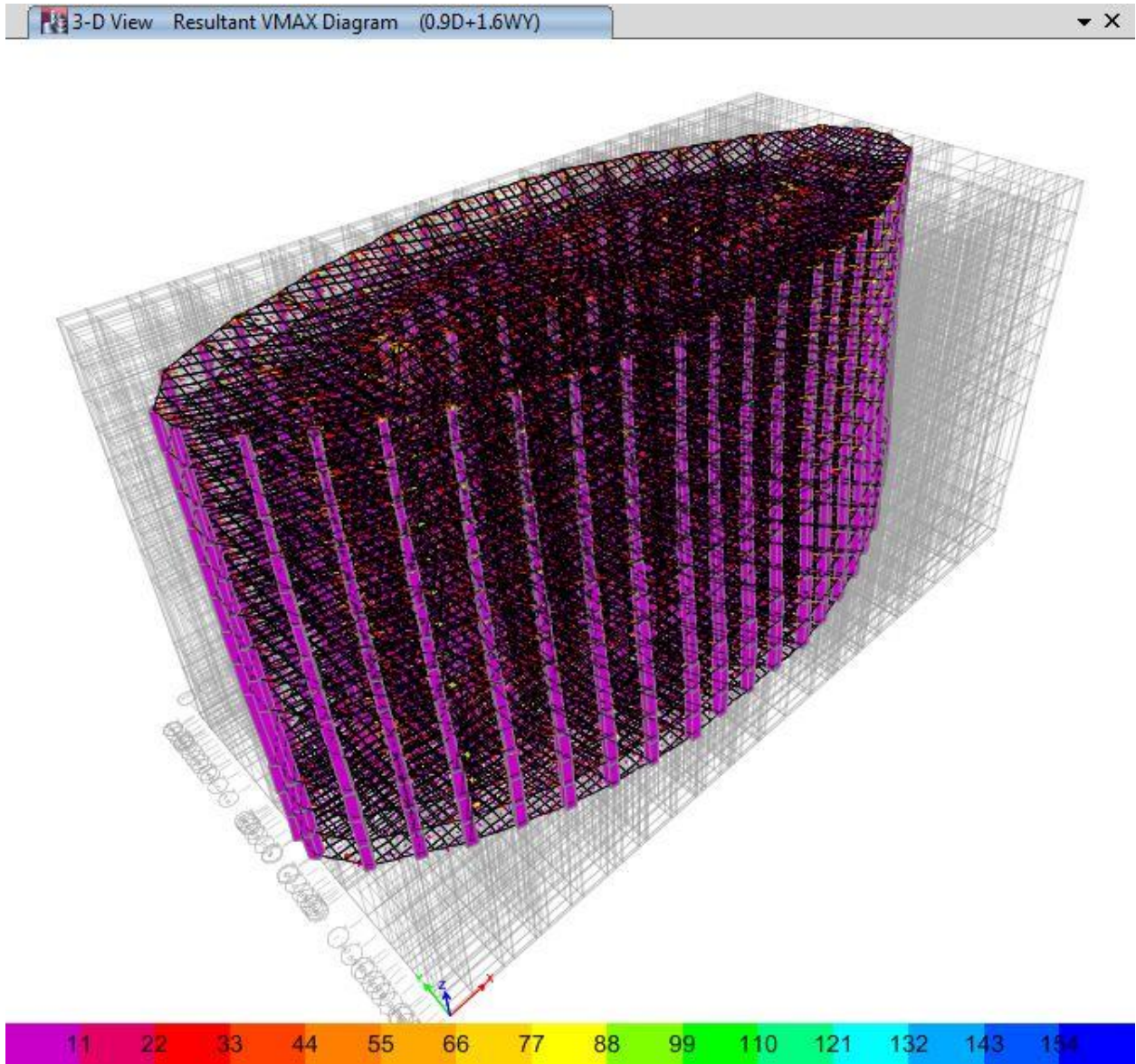


Figure 3.14 Shear of the different slabs – Dead load factor 0.9 & Wind Y 1.6 (ETABS)

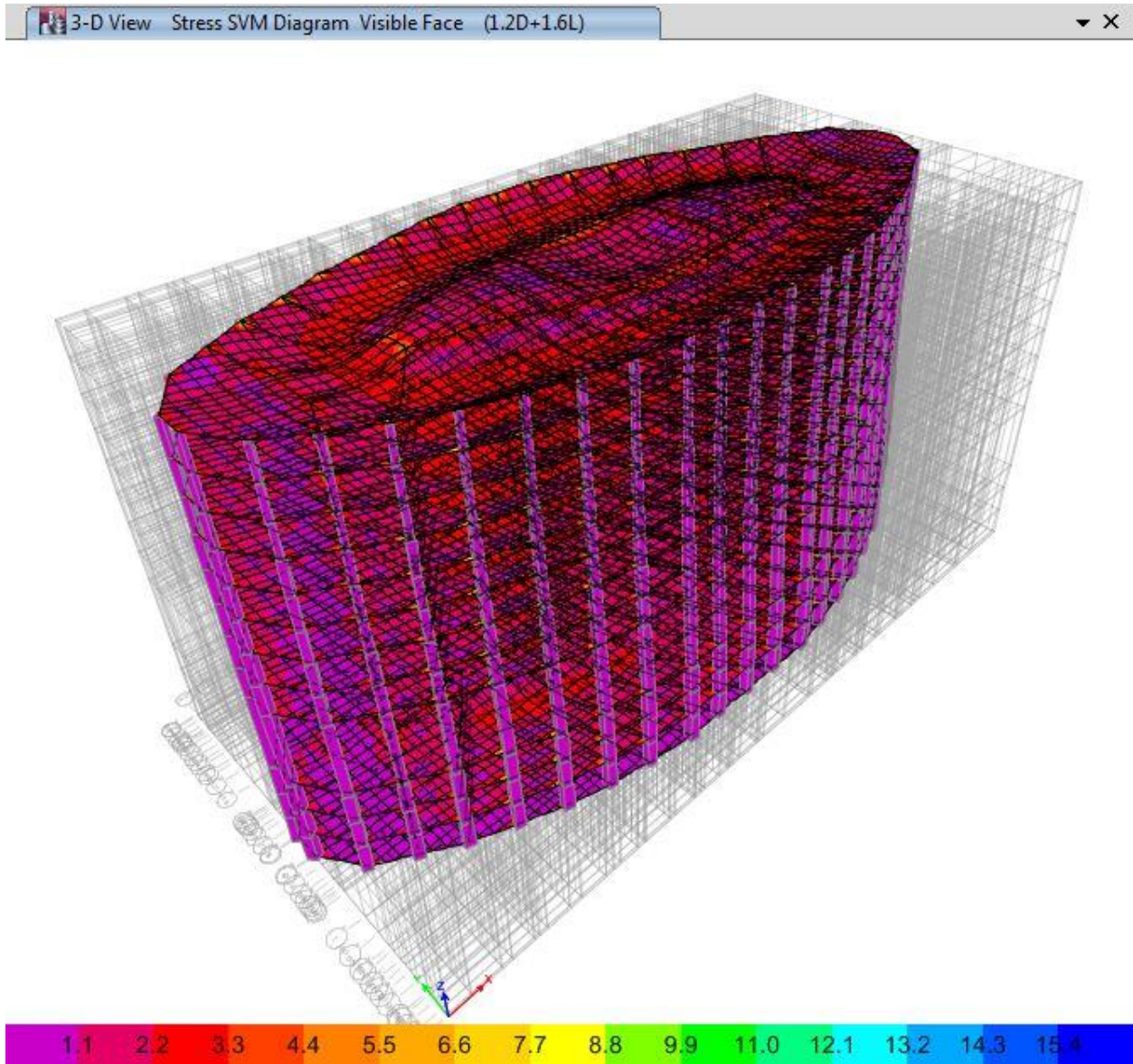


Figure 3.15 Stress in the different slabs – Dead load factor 1.2 & live load 1.6 (ETABS)

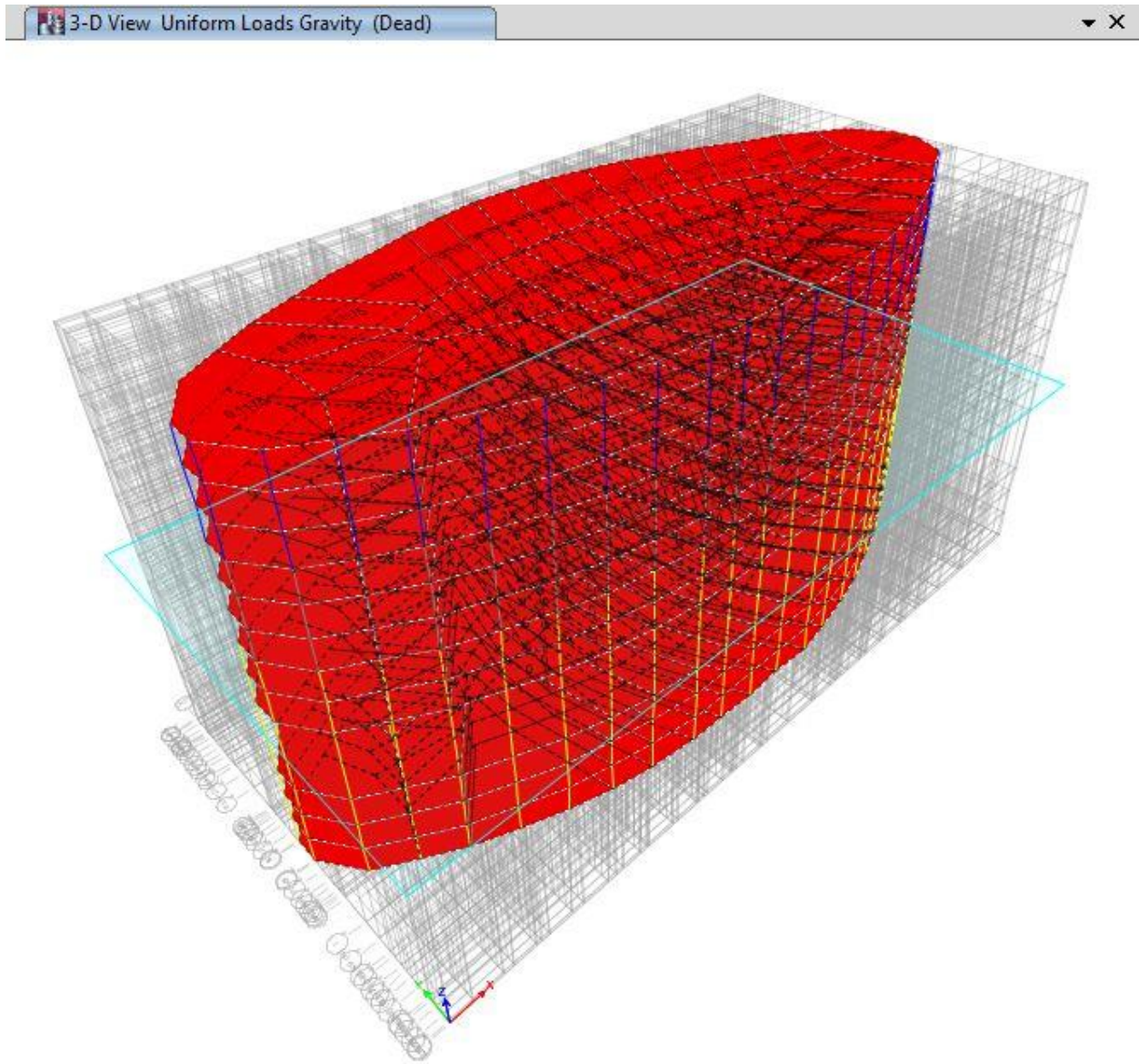


Figure 3.16 Gravity load in the different slabs (ETABS)

3.4.2 Concrete Column Design:

3.4.2.1 Column Design:

One of the most important stages in structural design of a building is the column. It is the stage that comes after the slab design and before the foundation design. Columns or compression members as some books say are designed to resist the compression forces that are acting on it. In this project, there are three sources of loads that are acting on the columns. They are: 1- Live load. 2- Dead load. 3- Slab self-weight. Simply, the summation of these loads will give the total compression force that is acting on the column as a point load. In addition, to design the column dimensions, it is important to know the area covered by each column. In this project the largest area covered by a single column is 60 m². As a result, this column will have the largest dimensions. In order to make the process of designing the columns smoother, these dimensions will be applied to all the columns in the same group. Column groups will join the columns with the same column dimensions together. Each 3 floors will have the same column dimensions. In total, there were four groups. The purpose of this step is to avoid designing column dimensions for each individual floor separately in order to consume time efficiently through out this semester. The dimensions for the columns will decrease when going up and will increase when going down in the structure. This means that the largest column dimensions will be in the three parking floors and the smallest dimensions will be in the 6th, 7th, and 8th floors. In this project, the design will be in compliance with the Saudi Building Code (SBC). The following table shows eight different areas taken from the architectural plan of the hotel where individual columns are covered by the largest slab areas:

Name	Area (cm ²)	Area (m ²)
Area 1	10.58	26.22

Area 2	11.67	28.92
Area 3	9.03	22.38
Area 4	14.4045	35.69
Area 5	23.9525	59.35
Area 6	10.4575	25.91
Area 7	11.12625	27.57
Area 8	10.63875	26.36

Table 3.4 Largest slab areas covered by one column

The area shaded in grey above represents the largest slab area covered by one column. In order to figure out the column dimensions, the live load and dead load on each floor has to be calculated. The following calculations show all the specific details of these loads:

Roof (swimming pool +gymnasium):

density of ceramic tile=23.5kn/m³

thickness of ceramic tile=0.005m

dead load of ceramic tile=23.5x0.005=0.1175kn/m²

self-weight of slab:

thickness of slab= 30cm=0.3m

s5 area=60m²

density of concrete=2396.3kg/m³=23.5 KN/m³

Live load=5kn/m²

total live load=5x60=300

total dead load of ceramic tile=0.1175x60 =7 kN

total dead load of concrete slab=23.5x0.3x60=423kn

Pu=1.2(430)+1.6(300)=996

Mechanical floor (floor#7):

live load=2.68kn/m²

total live load=2.68x60=160.8 kN

self weight of slab=423kn

$$P_u=1.2(423)+1.6(161)=766\text{kn}$$

Floor#6:

dead load of ceramic=7kn

self weight of slab=423kn

total live load=2x60=120kn

total dead load=430kn

$$P_u=1.2(430)+1.6(120)=708\text{kn}$$

Floor #5:

$$P_u=708\text{kn}$$

Floor#4:

$$P_u=708\text{ kn}$$

Floor #3:

$$P_u=708\text{ kn}$$

Floor#2:

$$P_u=708\text{ kn}$$

Floor #1:

$$P_u= 708\text{ Kn}$$

Ground Floor:

dead load of ceramic=7kn

self weight of slab=423kn

live load= 6.4kn/m²

total live load=6.4x60=384

$$P_u=1.2(430)+1.6(384)=1131$$

Parking (-1 floor):

self weight of slab=423kn

live load =4.9kn/m²

total dead load=423KN

$$P_u=1.2(423)+1.6(294)=978KN$$

Parking (-2floor):

$$P_u=978kn$$

Parking (-3floor):

$$P_u=978kn$$

When entering the floor dead and live loads in the ETABS Software, the load values are entered in terms of unit area (KN/m²). The live load values in the hand calculations are equal to the values entered in the ETABS Software. However, the dead load values in the hand calculations are different from the values entered in the ETABS Software. The difference is in the dead load of the slab. Once the slab thickness is calculated and entered in the ETABS, the dead load of the slab is automatically calculated and added to the total dead load value entered by the user. The following tables shows a comparison between the

hand calculated load values, and the values entered in the ETABS:

Hand Calculated Load Values		
Floor	Dead load (kN/m ²)	Live load (kN/m ²)
Swimming Pool+gymnasium (8)	7.1175	5
Mechanical room (7)	7	2.68
6	7.1175	2
5	7.1175	2
4	7.1175	2
3	7.1175	2
2	7.1175	2
1	7.1175	2
G	7.1175	6.4
Parking (1)	7	4.9
Parking (2)	7	4.9
Parking (3)	7	4.9

Table 3.5 Hand calculated dead and live loads per unit area

ETABS load values		
Floor	Dead load (kN/m ²)	Live load (kN/m ²)
Swimming pool+ Gymnasium (8)	0.1175	5
mechanical room (7)	0	2.68
6	0.1175	2
5	0.1175	2
4	0.1175	2
3	0.1175	2
2	0.1175	2
1	0.1175	2

G	0.1175	6.4
Parking (1)	0	4.9
Parking (2)	0	4.9
Parking (3)	0	4.9

Table 3.6 ETABS dead and live loads per unit area

The floors from one through the sixth floor have the same loads. The three parking floors have the same loads. In order to design the columns dimensions, the load per unit area has to be converted to point load by multiplying it with Area 5 mentioned in the table above (59.35 m²). In this project, square columns will be used, meaning that the columns will have the same width and depth. To design the column dimensions, only one parameter needs to be known, and that is the cumulative load on the column (Pu). The following equation and table describes this process:

Floor #	Loads (kN)	Cumulative loads (kN)	Gross area (m ²)	Width(m)	Depth(m)	Width (mm)	Depth(mm)
Roof (floor#8)	996	996	0.077	0.277	0.277	280	280
Floor#7	766	1762	0.135	0.367	0.367	370	370
fFloor#6	708	2470	0.189	0.434	0.434	440	440

Floor#5	708	3178	0.243	0.492	0.493	500	500
Floor#4	708	3886	0.297	0.544	0.548	550	550
Floor#3	708	4594	0.351	0.592	0.5924525 3	600	600
Floor#2	708	5302	0.406	0.637	0.637	640	640
Floor#1	708	6010	0.46	0.678	0.678	680	680
Ground floor	1131	7141	0.546	0.738	0.739	740	740
Parking (-1 floor)	978	8119	0.621	0.788	0.788	790	790
Parking (-2 floor)	978	9097	0.695	0.833	0.833	840	840
Parking (-3 floor)	978	10075	0.77	0.877	0.877	880	880

Table 3.7 Columns dimensions design

$$P_u = 0.8 \times 0.7 \times 0.85 \times 27500 \times A_g$$

Where,

P_u : cumulative load on column (KN)

A_g : gross area of column (m^2)

3.4.2.2 ETABS Modeling:

After obtaining the preliminary design of slabs and columns, it was important to verify that the design complies with SBC and ACI. The program used for the analysis is (ETABS) as mentioned earlier in the first chapter. The ETABS allows us to check slab deflection, members passing or not, and story drift. The first step was defining the materials that will be used in the modeling. The concrete used for columns and shear walls is 4000psi concrete. The dimensions of all structural members were used as per the preliminary design. Slab thickness is 0.3 m, shear wall thickness is 0.55 m. The second step was modeling the floor with all the columns and shear walls. Third step was applying loads on the structure. The loads include dead, live, and wind loads. Dead and live load were applied to the slab, while wind loads were assigned to the frame. Wind loads were defined as linear static using lateral load definition of ASCE 7-10 and according to the Saudi Building Code (SBC 301). Since the wind load does not act only on one axis, they will be defined separately in X and Y directions. The last step was defining the load cases and combinations. The load cases are according to the SBC 301. By the end of this step, the model was ready to check the performance of the building against the different load combinations. The job of this analysis is to check the structural performance, to make sure that the structure is safe. All the members have passed. The following table shows the load combinations that have been used in the ETAB model:

Load combination name	Load name	Scale factor
0.9D+1.6WX	DEAD	0.9
	WINDX	1.6
0.9D+1.6WY	DEAD	0.9
	WINDY	1.6
1.2D	DEAD	1.2
1.2D+1.6L	DEAD	1.2
	LIVE	1.6
1.6L	LIVE	1.6
DAED1	DAED	1

LIVE+DEAD	DEAD	1
	LIVE	1

Table 3.8ETABS load combinations

Grid Lines:

Before entering the columns in the ETABS Software, a grid system has to be made. This grid system consists of horizontal and vertical lines. There are 76, and 10 shear walls in each floor of the hotel. Each column and shear wall is represented by a horizontal and vertical intersection. The following table shows these grid lines and the distances between them:

i) Horizontal Grid Lines:

Horizontal Lines	Distance (cm)	Distance (m)
A-B	0.2	0.3148
B-C	0.2	0.3148
C-D	0.3	0.4722
D-E	0.4	0.6296
E-F	0.6	0.9444
F-G	0.7	1.1018
G-H	0.8	1.2592
H-I	1	1.574

I-J	1.1	1.7314
J-K	0.2	0.3148
K-L	0.2	0.3148
L-M	0.3	0.4722
M-N	0.3	0.4722
N-O	0.4	0.6296
O-P	0.7	1.1018
P-Q	0.6	0.9444
Q-R	0.1	0.1574
R-S	1.5	2.361
S-T	0.1	0.1574
T-U	0.6	0.9444
U-V	0.7	1.1018
V-W	0.4	0.6296
W-X	0.3	0.4722
X-Y	0.3	0.4722
Y-Z	0.2	0.3148
Z-A1	0.2	0.3148
A1-B1	1.1	1.7314
B1-C1	1	1.574
C1-D1	0.8	1.2592
D1-E1	0.7	1.1018
E1-F1	0.6	0.9444
F1-G1	0.4	0.6296
G1-H1	0.3	0.4722
H1-I1	0.2	0.3148
I1-J1	0.2	0.3148

Table 3.9 Horizontal grid lines

ii) Vertical Grid Lines:

Vertical Lines (horizontal distances)	Distance (cm)	Distance (m)
line1-2	2.4	3.7776

line2-3	1.7	2.6758
line3-4	0.4	0.6296
line4-5	2.1	3.3054
line5-6	2.2	3.4628
line6-7	0.6	0.9444
line7-8	1.6	2.5184
line8-9	0.1	0.1574
line9-10	2	3.148
line10-11	0.3	0.4722
line11-12	1.5	2.361
line12-13	0.1	0.1574
line13-14	0.6	0.9444
line14-15	1.2	1.8888
line15-16	0.7	1.1018
line16-17	0.4	0.6296
line17-18	0.6	0.9444
line18-19	1.7	2.6758
line19-20	0.4	0.6296
line20-21	1.8	2.8332
line21-22	1.9	2.9906
line22-23	0.4	0.6296
line23-24	1.7	2.6758
line24-25	0.7	1.1018
line25-26	0.5	0.787
line26-27	0.7	1.1018
line27-28	1	1.574
line28-29	0.7	1.1018
line29-30	0.2	0.3148
line30-31	1.4	2.2036
line31-32	0.4	0.6296
line32-33	1.8	2.8332
line33-34	0.2	0.3148
line34-35	1.5	2.361
line35-36	0.8	1.2592
line36-37	2.1	3.3054

Table 3.10 Vertical grid lines

The “cm” grid lines distances are taken from the architectural drawing. To obtain the “m” grid lines distances, we multiplied by a factor of 1.574. This factor was decided after performing many calculations and finding the appropriate relationship between the grid lines geometry based on the architectural drawing.

Column and Shear Walls Slopes:

According to the architectural plan, the columns are drawn with different slopes. However, each two symmetrical columns will have the same slope. Furthermore, all the columns lying above each other in the different floors will have the same slope. All the shear walls are either vertical or horizontal, meaning they all have slopes of 90 degree or 0 degree. These slopes are shown in the following Table:

Column #	Slope (degrees)
1	90
2	87
3	84
4	80
5	77
6	73
7	72
8	68
9	64
10	61
11	33
12	147
13	119
14	116
15	112
16	108
17	107
18	103
19	100
20	96
21	93
22	90
23	87
24	84
25	80
26	77
27	73
28	72
29	68
30	64
31	61
32	33
33	147
34	119
35	116
36	112
37	108
38	107
39	103
40	100
41	96
42	93
43	90

44	87
45	84
46	80
47	77
48	73
49	72
50	68
51	112
52	108
53	107
54	103
55	100
56	96
57	93
58	90
59	87
60	84
61	80
62	77
63	73
64	72
65	68
66	112
67	108
68	107
69	103
70	100
71	96
72	93
73 (shear wall)	0
74 (shear wall)	0
75 (shear wall)	90
76 (shear wall)	90
77 (shear wall)	90
78 (shear wall)	90
79 (shear wall)	90
80 (shear wall)	0
81 (shear wall)	0
82 (shear wall)	90
83	61
84	119
85	119
86	61

Table 3.11 Columns and shear walls slopes

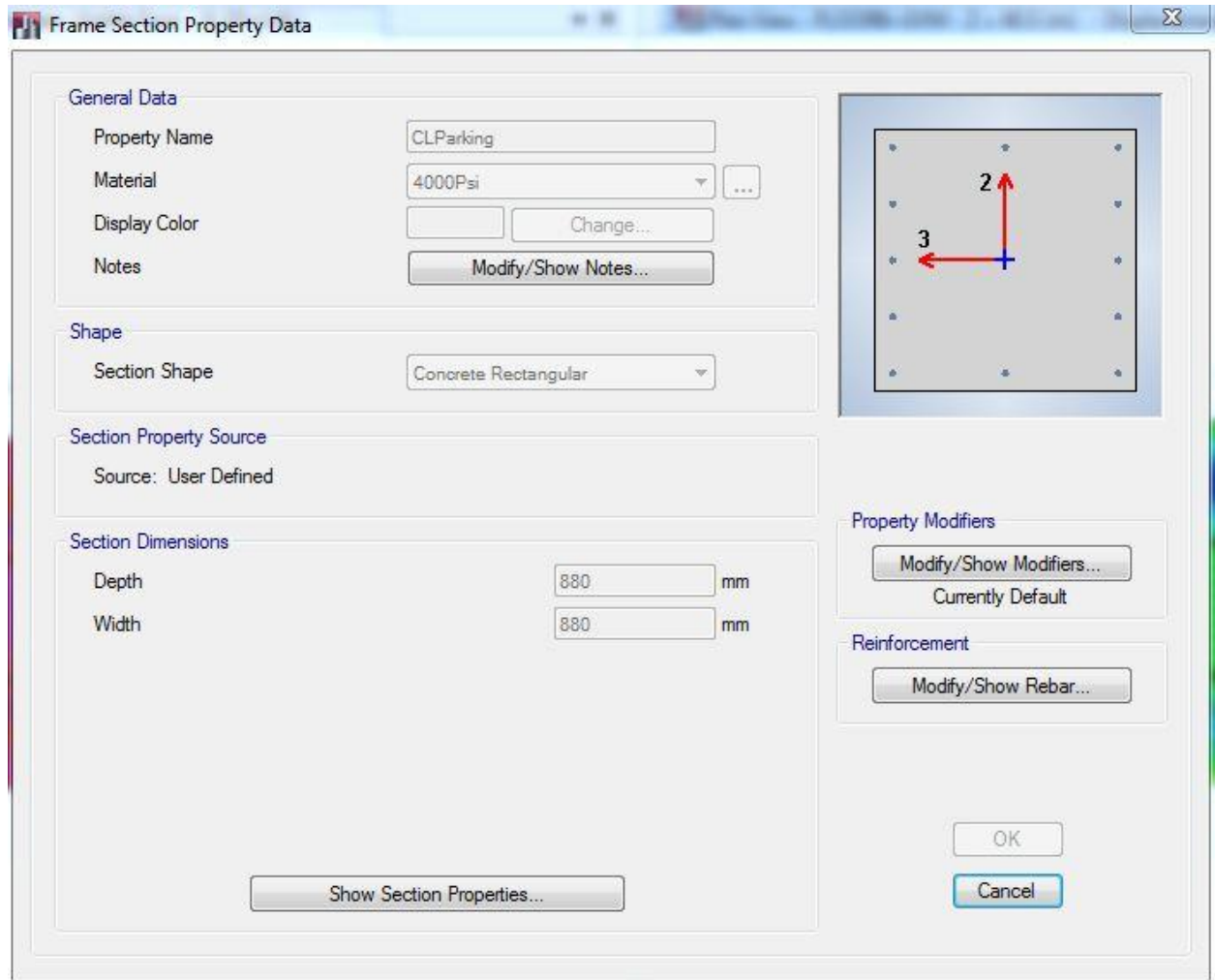


Figure 3.17 Example of Concrete column section properties - Parking

Wall Property Data ✕

General Data

Property Name	<input type="text" value="Wall1"/>
Property Type	<input type="text" value="Specified"/>
Wall Material	<input type="text" value="4000Psi"/> ...
Modeling Type	<input type="text" value="Shell-Thick"/>
Modifiers (Currently Default)	<input type="button" value="Modify/Show..."/>
Display Color	<input type="color" value="#FFFF00"/> <input type="button" value="Change..."/>
Property Notes	<input type="button" value="Modify/Show..."/>

Property Data

Thickness	<input type="text" value="550"/> mm
-----------	-------------------------------------

Figure 3.18 Concrete shear wall section properties nearby the elevator -

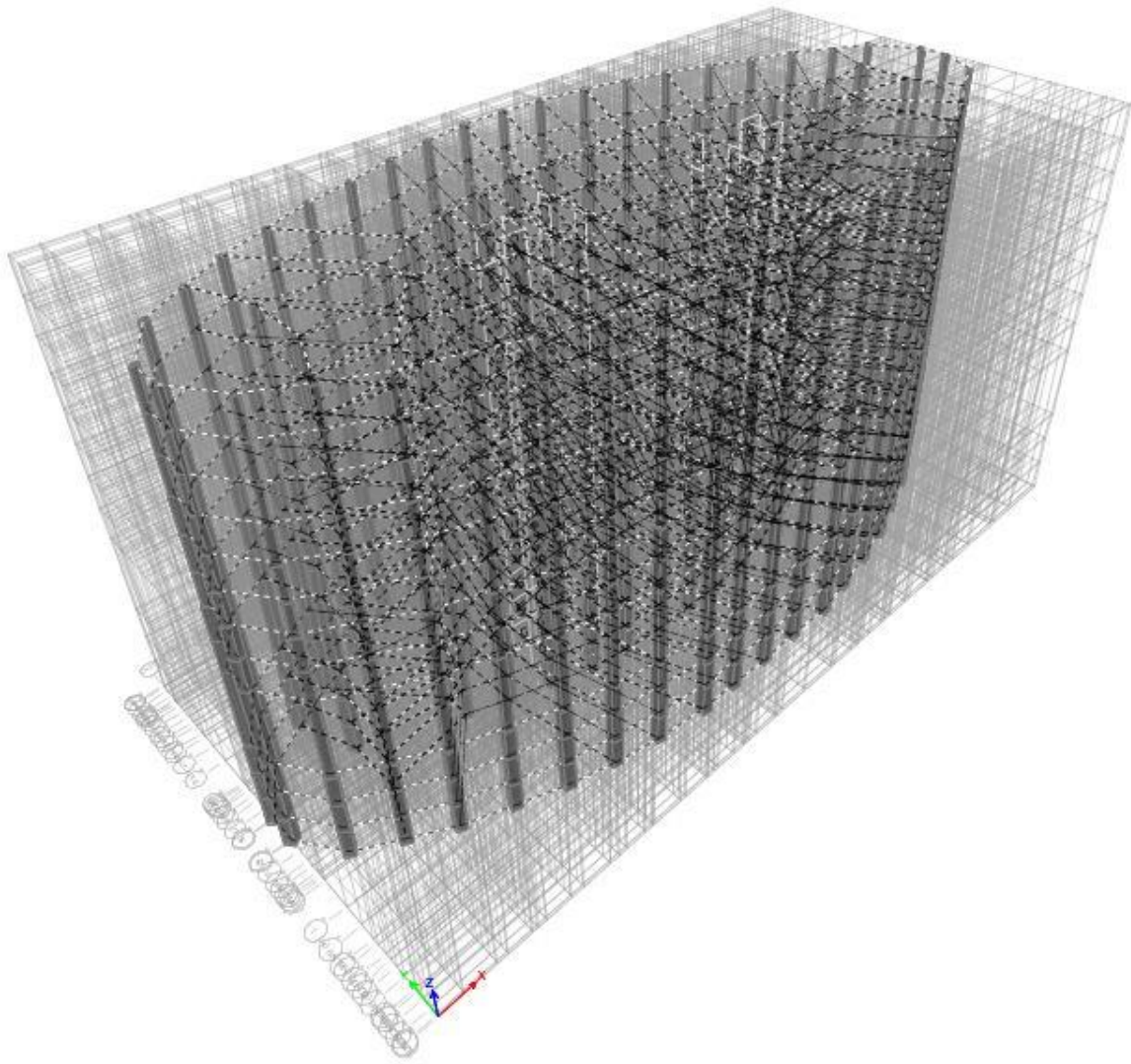


Figure 3.19 Axial force loads on the concrete columns- Dead Load factor of 1 (ETABS)

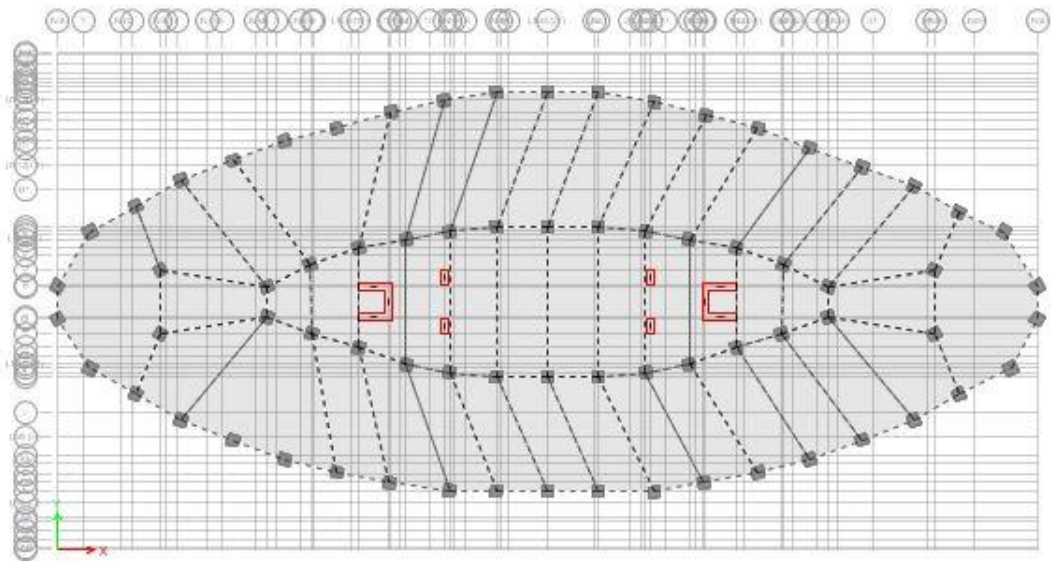


Figure 3.20 Axial force on the concrete columns- Dead Load factor of 1- ETABS

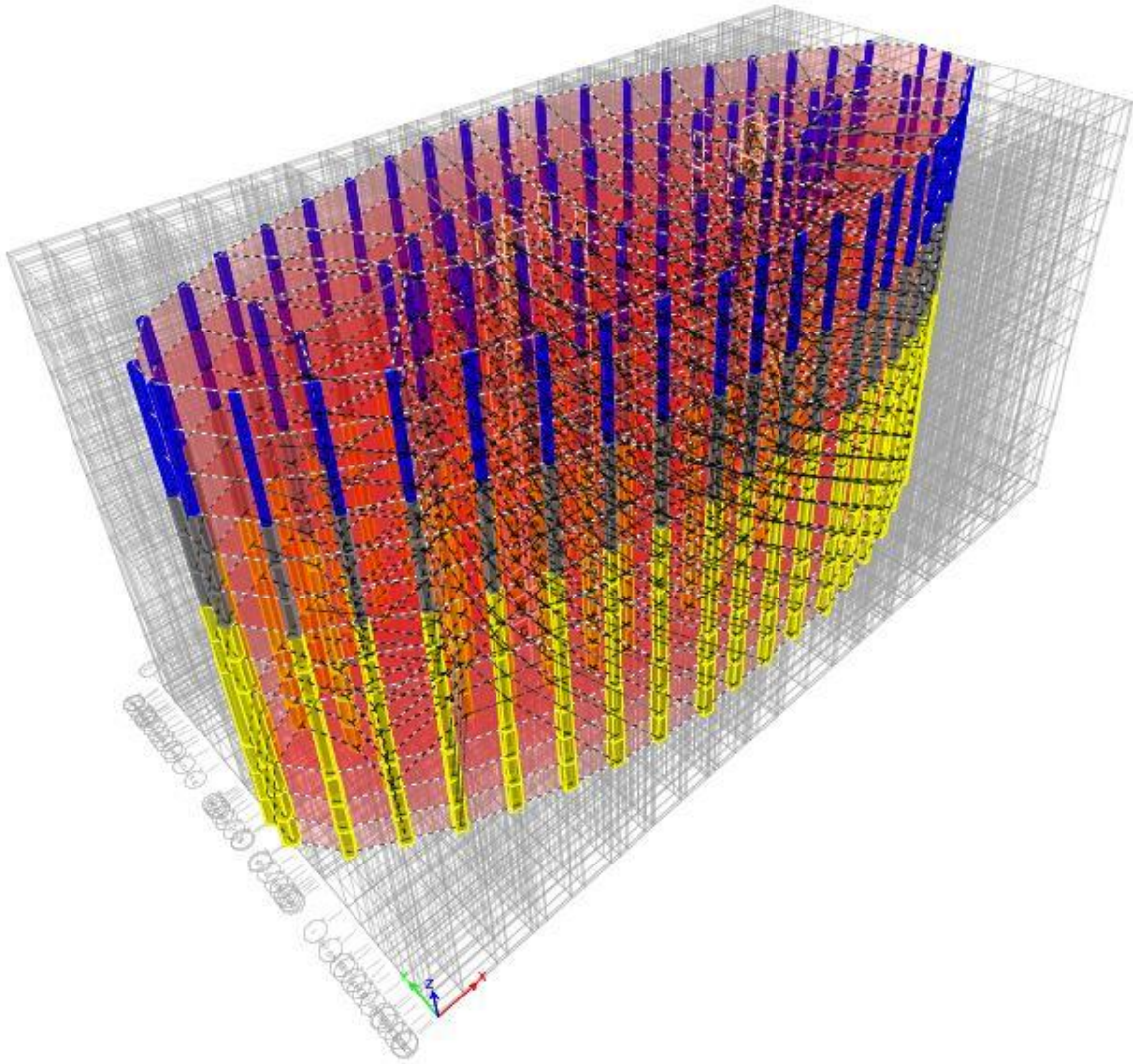


Figure 3.21 In-plane Moment- Dead Load with factor of 1.2 & Live load with factor of 1.6- (ETABS)

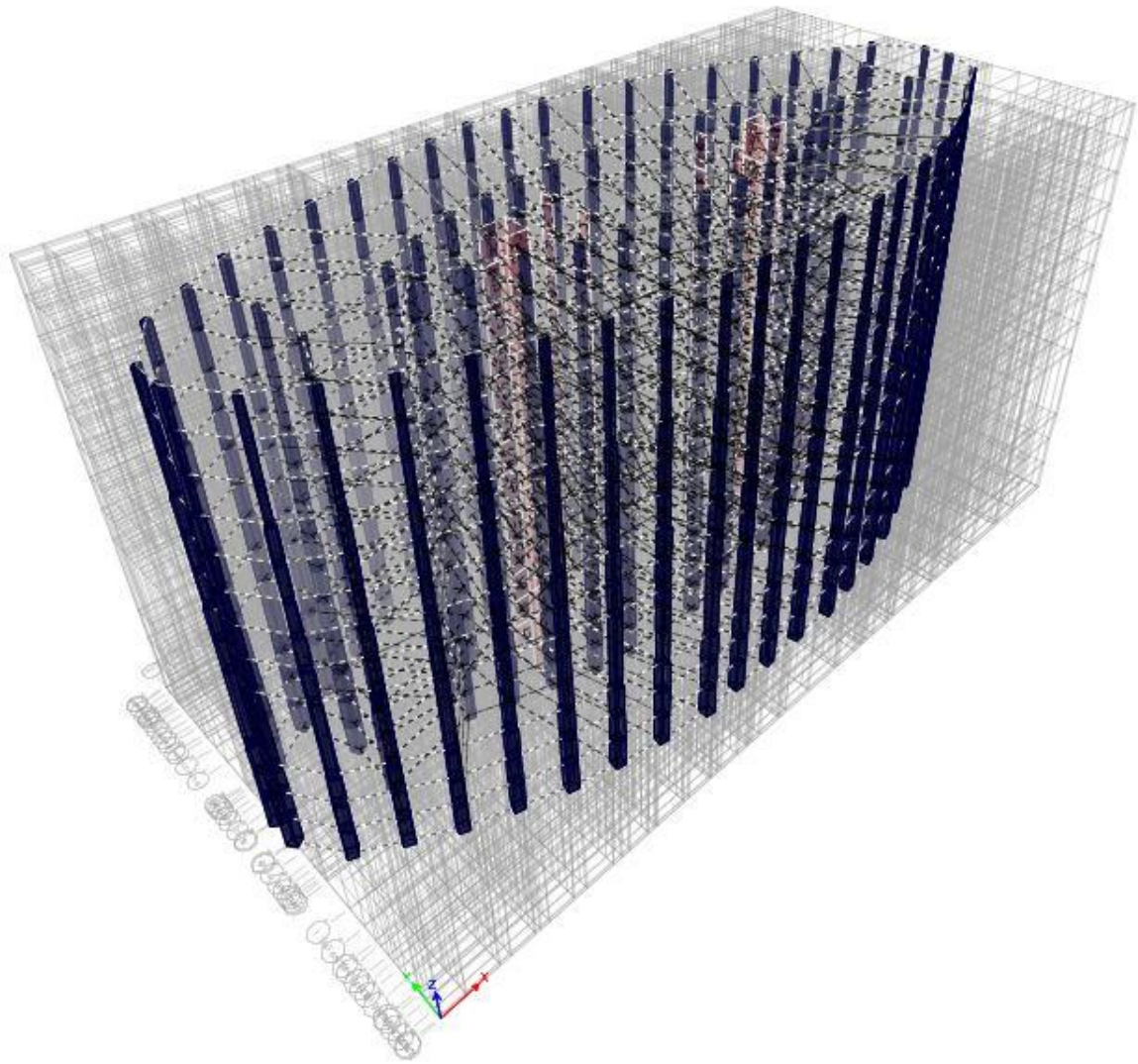


Figure 3.22 Moment 2-2. Live Load with factor of 1.6- ETABS

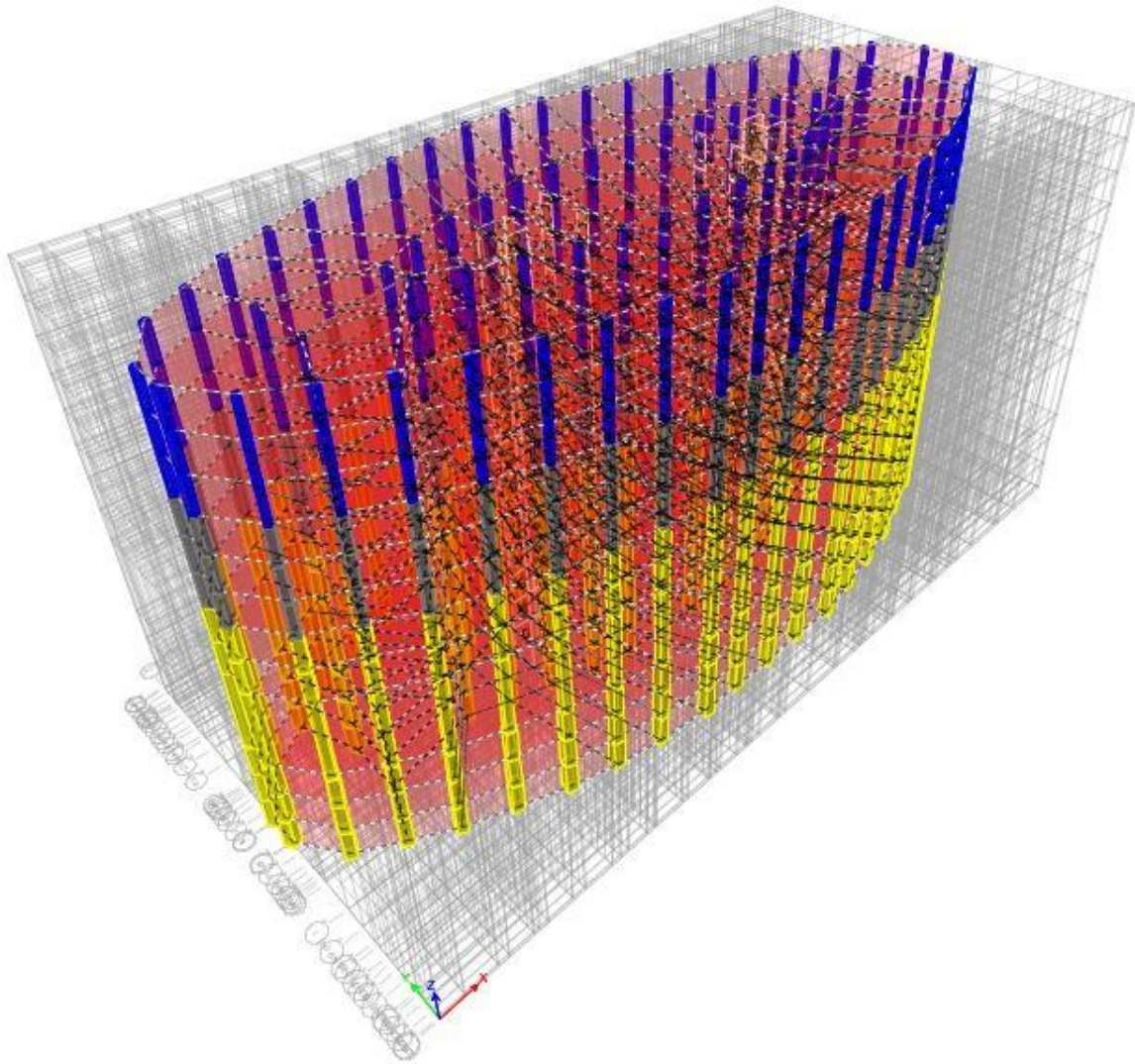


Figure 3.23- Moment 3-3. Dead load with factor of 0.9 and wind load in the X direction with factor of 1.6- (ETABS)

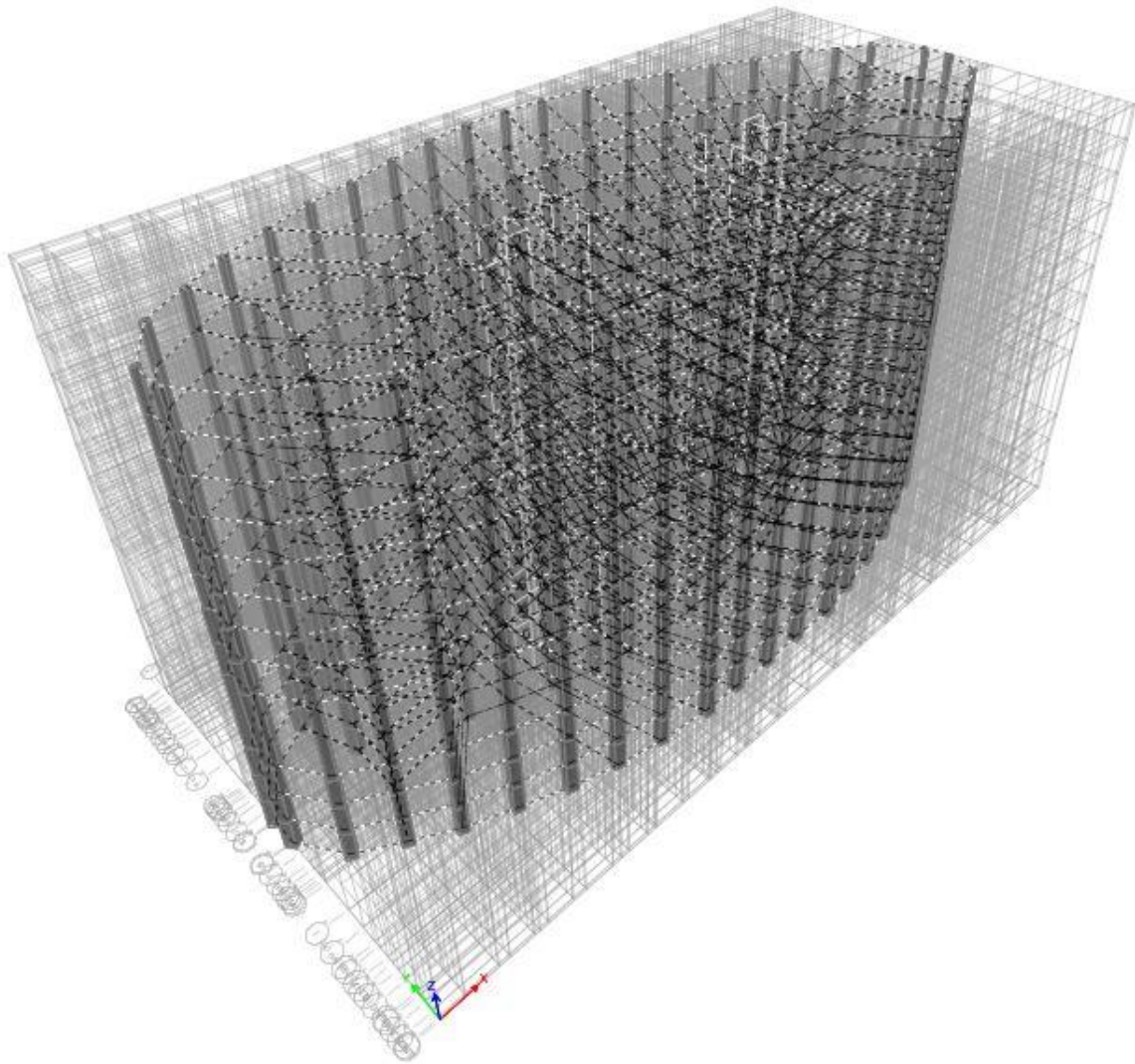


Figure 3.24 Shear force 2-2. Dead load with factor of 1 and Live load with factor of 1- (ETABS)

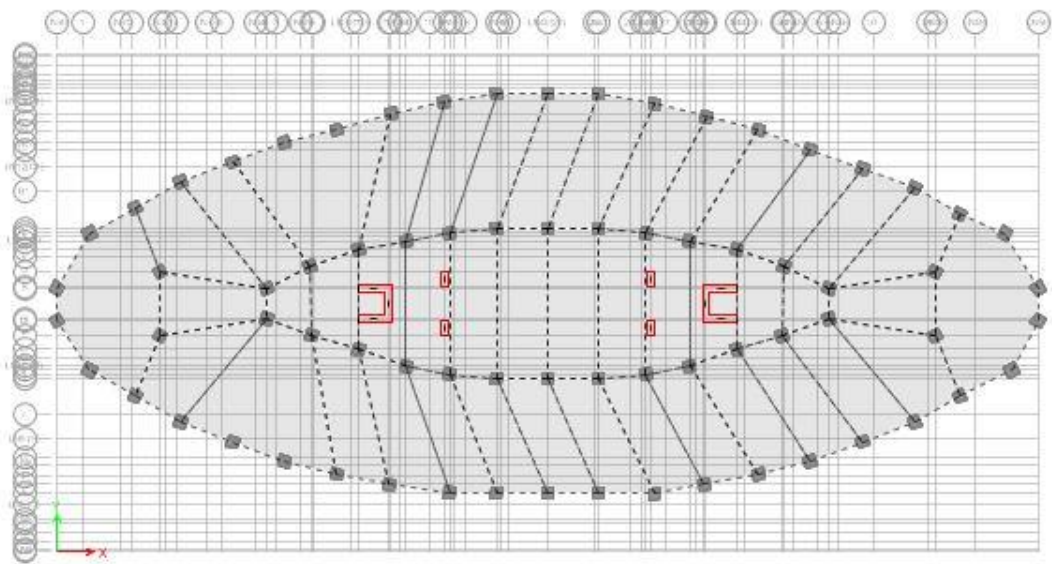


Figure 3.25- Shear force 3-3. Dead Load with factor of 1.2 & Live Load with factor of 1.6- (ETABS)

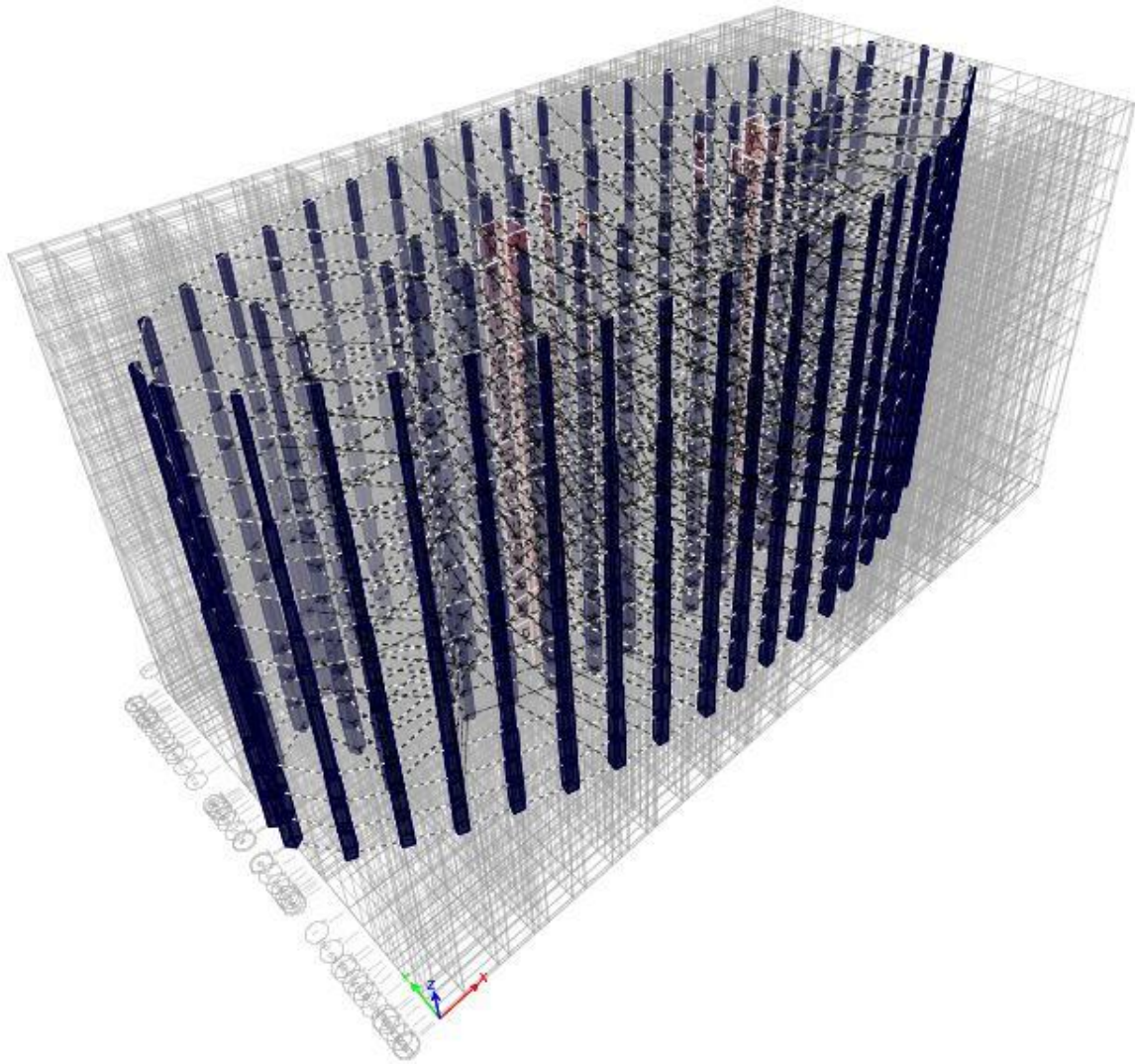


Figure 3.26- Column Torsion. Dead Load with factor of 1.2- (ETABS)

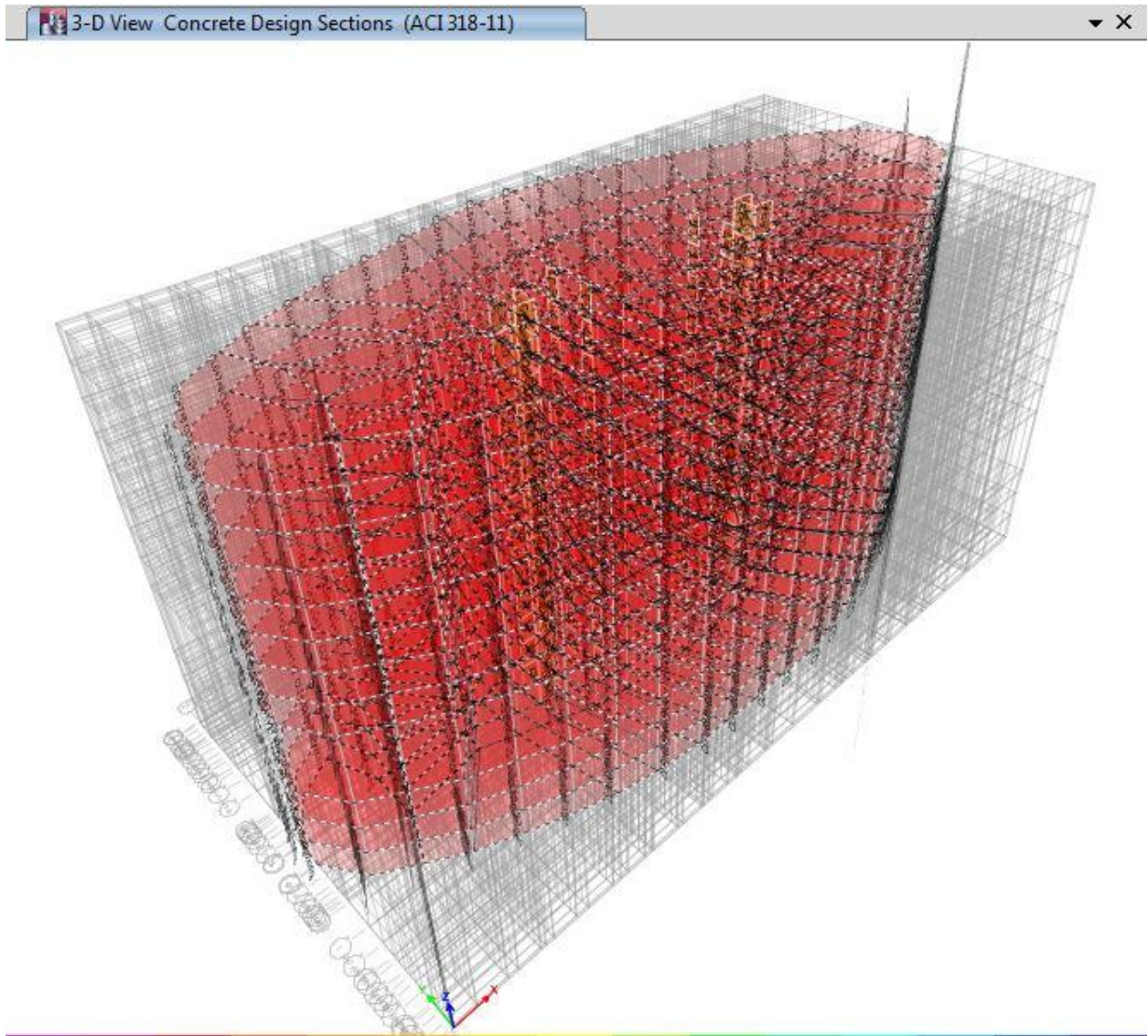


Figure 3.28- Concrete design sections- (Etabs)

CHAPTER 4: GEOTECHNICAL DESIGN

4.1 Introduction

As mentioned in Chapter 2, the geotechnical investigation carried out on the site of the new constructed bridge, located nearby PMU campus, is used in the project. The following table summarizes the soil profile of the site project (i.e. the site of the projected hotel):

Layers	Depth (m)	Description	Number of blows (N)	Soil state	Height of layer (H)	f (unit skin friction)
Layer #1	0 – 7.2	Poorly graded brown sand	5 to 27	Loose to medium dense	-	-
Layer #2	7.2 – 10.2	Silty brown sand	12 to 100	Medium to very dense	-	-
Layer #3	10.2 – 13.2	Sandy silt light brown	8 to 10	Medium	3	20
Layer #4	13.2 – 24	Poorly graded sand	20 to 100	Medium to very dense	11	30
Layer #5	24 – 30 (end of boring)	Sandy lean clay light brown	100	Hard	6	50

Table 4.1 Summary of the soil profile of the site of the projected hotel

This chapter is concerned by the geotechnical design of the multistory hotel in the Half Moon area. In addition to proposing solutions to the problem of liquefaction described previously. The design will include the selection, calculation and verification of the appropriate foundation system for the different elements of the hotel.

4.2 Proposed solutions

The problems encountered in this project are summarized as follows:

- 1- The water table at the half Moon area is located in the ground surface.
- 2- The state/conditions of the site (i.e. the sand is loose and saturated so susceptible to liquefaction phenomenon). Bearing in mind that soil liquefaction substantially loses [strength](#) and [stiffness](#) in response to an applied earthquake [stresses](#).

4.2.1 Solution of problem #1:

In this solution the ground is elevated from the actual surface by 2.0 m (i.e. above the ground surface) by a controlled compacted fill. The compaction of the fill is performed on a granular soil of good quality using either the method of sand pumping or method of mechanical compaction.

4.2.2 Solution of problem #2:

As stated previously, the problem in this case is related to the phenomenon of liquefaction. The following solutions can be adapted to overcome this problem, such as: i)- Dynamic compaction, or Deep foundations (i.e. piles). In case 1 of this solution, the system of foundations adopted is isolated square foundations (Figure 5.6). However, in case 2, it is recommended to use metallic driven piles.

The dynamic compaction technique can be used as an alternative for stabilizing the site of the project. By considering the nature and grading of soil of the project site, and the level of groundwater table, we could consider dynamic compaction technique, described herein, to eliminate or to reduce, to an acceptable extend, the liquefaction potential of this layer.

The liquefiable brown 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.

In the case of dynamic compaction, it is recommended to measure the permeability of the soil before the compaction process using wells of observation. This action is needed in order to check that the ground can be effectively drained and that dynamic compaction will not generate excessive interstitial pore water pressure which would harm uniform densification of compacted layers.

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. Once the soil is densified and after confirmation of the requested soil bearing capacity by means of dynamic penetration test or any in situ method, the structures could be built on conventional footings sited directly on the improved soil.

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.

4.3 Columns Axial Working Loads

In order to design the foundation, the maximum axial load transmitted from the hotel columns has to be known. We got seven load combinations applied on 6 columns that have the potential of having the maximum load of all the 1032 columns and shear walls

in the hotel. We also obtained the values of the torsion, shear (x and y direction), and moment (x and y direction) for these columns. After that, the foundation system is designed based on the maximum load value from these 6 values. The tables below show the mentioned calculations:

column#	load combination	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment(M2) (KN-m)
83	Dead 1	3723.4330 (negative)	0.0539	0.2028	25.8554	3.5046 (negative)	14.7027 (negative)
	Live+dead	3729.7115 (negative)	0.0666 (negative)	3.5328	37.4634	6.3321	15.8109
	1.2D+1.6L	4478.1652 (negative)	0.1282 (negative)	5.5714	49.5993	11.5332	31.1785
	1.2D	4468.1197 (negative)	0.0647 (negative)	0.2433	31.0265	4.2055 (negative)	17.6432 (negative)
	1.6L	10.0456 (negative)	0.1928 (negative)	5.3281	18.5729	15.7387	48.8217
	0.9D+1.6WX	3361.1353 (negative)	0.1443 (negative)	5.5106	41.8427	12.5845	35.5893
	0.9D+1.6WY	3361.1353 (negative)	0.1443 (negative)	5.5106	41.8427	12.5845	35.5893

Table 4.2 Results of Load Combinations for Columns #83

column#	load combination	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment(M2) (KN-m)
84	Dead 1	3632.5501 (negative)	0.2609 (negative)	2.5095	14.2506	3.1793 (negative)	33.7521 (negative)
	live+dead	3631.3687 (negative)	0.4345 (negative)	15.0972	51.2967	1.6205	23.5571 (negative)
	1.2D+1.6L	4357.1698 (negative)	0.5909 (negative)	23.1517	76.3745	3.8646	28.0057 (negative)
	1.2D	4359.0602 (negative)	0.3130 (negative)	3.0114	17.1007	3.8152 (negative)	40.5025 (negative)
	1.6L	1.8904	0.2778 (negative)	20.1403	59.2738	7.6797	24.0905
	0.9D+1.6WX	3267.4048 (negative)	0.5126 (negative)	22.3988	72.0993	4.8184	20.7415 (negative)
	0.9D+1.6WY	3267.4048 (negative)	0.5126 (negative)	22.3988	72.0993	4.8184	20.7415 (negative)

Table 4.3 Results of Load Combinations for Columns #84

column#	load combination	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment(M2) (KN-m)
85	Dead 1	3559.1018 (negative)	0.0977 (negative)	2.2010 (negative)	14.5840 (negative)	2.4047	25.959
	live+dead	3556.5517 (negative)	0.0927	8.4125	17.49	5.9692	37.8438
	1.2D+1.6L	4266.8421 (negative)	0.1874	14.3404	33.8177	8.5889	50.1664
	1.2D	4270.9221 (negative)	0.1173 (negative)	2.6412 (negative)	17.5008 (negative)	2.8857	31.1508
	1.6L	4.08	0.3047	16.9816	51.3185	5.7032	19.0156
	0.9D+1.6WX	3199.1116 (negative)	0.2167	15.0007	38.1929	7.8675	42.3787
	0.9D+1.6WY	3199.1116 (negative)	0.2167	15.0007	38.1929	7.8675	42.3787

Table 4.4 Results of Load Combinations for Columns #85

column#	load combination	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment(M2) (KN-m)
86	Dead 1	3671.6750 (negative)	0.2374	4.5791 (negative)	35.1777 (negative)	2.9191	14.9922
	live+dead	3668.0480 (negative)	0.5126	1.0076	21.3918 (negative)	16.0698	53.1269
	1.2D+1.6L	4400.2068 (negative)	0.7252	3.4439	25.6507 (negative)	24.5441	79.0062
	1.2D	4406.0099 (negative)	0.2848	5.4949 (negative)	42.2132 (negative)	3.5029	17.9906
	1.6L	5.8031	0.4404	8.9387	26.894	21.0412	61.0156
	0.9D+1.6WX	3298.7043 (negative)	0.654	4.8176	19.2186 (negative)	23.6684	74.5085
	0.9D+1.6WY	3298.7043 (negative)	0.654	4.8176	19.2186 (negative)	23.6684	74.5085

Table 4.5 Results of Load Combinations for Columns #86

column#	load combination	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment(M2) (KN-m)
43	Dead 1	3937.2455 (negative)	0.0255 (negative)	1.4831	2.5262 (negative)	1.6173 (negative)	5.2237
	live+dead	3950.8810 (negative)	0.0147	2.2296	3.9032	1.753	26.7002
	1.2D+1.6L	4746.5114 (negative)	0.0338	2.9741	5.4759	3.4517	42.5716
	1.2D	4724.6946 (negative)	0.0306 (negative)	1.7798	3.0314 (negative)	1.9408 (negative)	6.2685
	1.6L	21.8168 (negative)	0.0644	1.1944	3.168	5.3925	42.1254
	0.9D+1.6WX	3565.3377 (negative)	0.0414	2.5292	4.8989	3.9369	42.4601
	0.9D+1.6WY	3565.3377 (negative)	0.0414	2.5292	4.8989	3.9369	42.4601

Table 4.6 Results of Load Combinations for Columns #43

column#	load combination	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment(M2) (KN-m)
58	Dead 1	3641.1870 (negative)	0.0194 (negative)	7.1785 (negative)	18.2532	0.6965 (negative)	1.1132 (negative)
	live+dead	3634.4700 (negative)	0.0223	4.0643 (negative)	34.3182	0.023	0.7231
	1.2D+1.6L	4358.6773 (negative)	0.0435	3.6314 (negative)	47.6078	0.3154	1.6023
	1.2D	4369.4244 (negative)	0.0233 (negative)	8.6142 (negative)	21.9038	0.8358 (negative)	1.3359 (negative)
	1.6L	10.7471	0.0668	4.9828	40.6525	1.1512	2.9382
	0.9D+1.6WX	3266.3212 (negative)	0.0493	1.4779 (negative)	42.1319	0.5244	1.9363
	0.9D+1.6WY	3266.3212 (negative)	0.0493	1.4779 (negative)	42.1319	0.5244	1.9363

Table 4.7 Results of Load Combinations for Columns #58

Load	axial force(P) (KN)	Torsion (T) (KN-m)	shear (V2) (KN)	Moment (M3) (KN-m)	Shear (V3) (KN)	Moment (M2) (KN-m)
highest positive value	10.7471	0.7252	23.1517	76.3745	24.5441	79.0062
highest negative value	4746.5114	0.5909	8.6142	42.2132	4.2055	40.5025

Table 4.8 Maximum Designing Loads

4.4 Verification of the bearing capacity

4.4.1 Isolated foundation

Based on the results of the ETABS (Chapter 3), the foundation should be designed using a positive load $Q_1 = 10.75$ kN and negative load $Q_2 = 4749.52$ kN. (Table 4.2 to 4.8). The value of this moment is very small which can be neglected. The results of the verification are grouped in Table 4.1. It is worthy to mention that:

The bearing capacity of the shallow foundation and the stress transmitted to the foundation are calculated as follows (Das, 20002):

$$q_u = 1.3cN_c + qN_q + 0.4BN_\gamma$$

Where:

γ = unit weight of soil,

c = cohesion of soil,

$q = \gamma D_f$

Table 3.1 Terzaghi's Bearing Capacity Factors—Eqs. (3.4), (3.5), and (3.6)

ϕ'	N_c	N_q	N_γ^a	ϕ'	N_c	N_q	N_γ^a
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

^aFrom Kumbhojkar (1993)

In many cases, foundations are subjected to moments in addition to vertical load (e.g. base of retaining wall). The distribution of the pressure by the foundation on the soil is not uniform:

$$q_{\max} = \frac{Q}{BL} + \frac{6M}{B^2L} \qquad q_{\min} = \frac{Q}{BL} - \frac{6M}{B^2L}$$

Where:

Q = total vertical load,

M = moment on the foundation

The eccentricity is defined by: $e = \frac{M}{Q}$

The principle of bearing capacity is verified if:

$$q_{max} \leq q_{all} \text{ (allowable stress)}$$

The results obtained from the application of the bearing capacity formulate are summarized in the following Table:

D (m)	Z (m)	B (m)	L (m)	γ (Kn/m ³)	C (kPa)	Nq	$N\gamma$	q_u (kPa)	q_{allow} (kPa)
1.5	11.5	1.5	1.5	10	0	134.88	271.76	16619	5539
2	12	2	2	10	0	134.88	271.76	17026	5675
2.5	12.5	2.5	2.5	10	0	134.88	271.76	17434	5811
3	13	3	3	10	0	134.88	271.76	17842	5947
1.5	11.5	1.5	1.5	10	0	134.88	271.76	17288	5762
2	12	2	2	10	0	134.88	271.76	17696	5898
2.5	12.5	2.5	2.5	10	0	134.88	271.76	18103	6034
3	13	3	3	10	0	134.88	271.76	18511	6170
1.5	11.5	1.5	1.5	10	0	134.88	271.76	17957	5985
2	12	2	2	10	0	134.88	271.76	18365	6121
2.5	12.5	2.5	2.5	10	0	134.88	271.76	18773	6257
3	13	3	3	10	0	134.88	271.76	19180	6393
1.5	11.5	1.5	1.5	10	0	134.88	271.76	18627	6209
2	12	2	2	10	0	134.88	271.76	19034	6344
2.5	12.5	2.5	2.5	10	0	134.88	271.76	19442	6480
3	13	3	3	10	0	134.88	271.76	19850	6616

Table 4.9 Results of bearing capacity verification for hotel shallow foundation

The following table shows the different design options:

D (m)	Z (m)	q _{allow} (Kpa)	q _{column} (Kpa)	B (m)
1.5	11.5	5947	6130	Not verified (5947<6130)
2	12	6170	6130	B = 3m
2.5	12.5	6257	6130	B ≥ 2.5m
3	13	6209	6130	B ≥ 1.5m

Table 4.10 Summary of the different design options

Based on the results of Table 4.10, the following minimal dimensions were retained for the design of the foundation: $D = 2.0$ m and $B = 3.0$ m.

Where:

D = embedment depth.

B = width of the foundation.

4.4.2 Deep foundations (piles)

In this case, it is suggested to install under each column (through a cap) a driven pile (Figure 4.3). The pile will be designed to penetrate the whole depth of the liquefiable layer (i.e. to penetrate from level – 9.0 m (bottom end of the underground car parking) to the rock situated at - 30 m depth. The total depth of each pile will be around 20 m. The numbers of piles, if any and their diameters is determined according to the principle of bearing capacity.

A pile derives its load-carrying capacity by friction or adhesion along the pile shaft with surrounding soil and/or by compressive resistance at the contact of the pile base with underlying soil (Figure). The load Q_u at the head of a single pile can be expressed as the sum of the load Q_p carried at the base and the load Q_s carried by the shaft of the pile (Das, 2002):

$$Q_{all} = \frac{Q_u}{FS} \qquad Q_u = Q_p + Q_s$$

$FS = 2.5$ to 4

Therefore, the point bearing of piles is



Where:

A_p = area of pile tip,

c = cohesion of the soil supporting the pile tip,

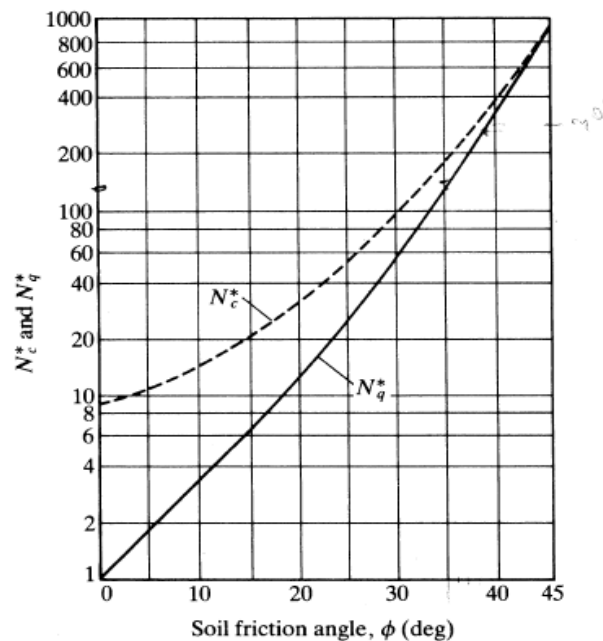
q_p = unit point resistance,

q' = effective vertical stress at the level of the pile tip,

And

N_c^* & N_q^* = the bearing capacity factors

Estimation of the bearing factors using Meyerhof's method:



The frictional or skin resistance of pile may be written as

$$Q_s = \sum p \Delta L f$$

Where:

p = perimeter of the pile section,

ΔL = incremental pile length over which p and f are taken constant,

f = unit friction resistance at any depth z ,

q' = effective vertical stress at the level of the pile tip,

Based on the soil profile summarized in section 4.1, the following soil characteristics were adopted for the design of the deep foundations:

Soil type	Depth (m)	F (Unit skin friction in kPa)	Cu (kPa)	Bearing capacity factor
Sandy silt	3	20	-	-
Poorly graded sand	11	30	-	-
Sandy clay	6	50	200*	$N_q^* = 1$ & $N_c^* = 9$ $\Phi = 0$ (from Meyerhof chart)

Table 4.11 The different characteristics used in the geotechnical design of piles

The different bearing capacity of pile foundations obtained are summarized as follows:

	D=0.325 m		D=0.4 m		D=0.5m		D=0.6m	
Soiltype	Qs (KN)	Qp (KN)	Qs (KN)	Qp (KN)	Qs (KN)	Qp (KN)	Qs (KN)	Qp (KN)
Sandy silt	61.3	-	75.4	-	94.2	-	113.1	-
Poorly graded sand	336.9	-	414.7	-	518.3	-	622	-
Sandy clay	306.3	180.9	377	274	471.2	428.1	565.5	616.5
Sum	704.6	180.9	867.1	274	1083.8	428.1	1300.6	616.5

Table 4.12. Resistance and shaft capacity of the different piles considered

	Qu(KN)	Q _{allow} (KN)	Ap(m ²)	qu (kPa)	Q _{allow} (kPa)
D=0.325m	885.4	295.1	0.083	10667.46	3555.42
D=0.4m	1141.1	380.3	0.1257	9077.96	3025.45
D=0.5m	1511.9	503.9	0.197	7674.61	2557.86
D=0.6m	1917.1	639	0.283	6774.20	2257.95

Table 4.13. Ultimate and allowable bearing capacity of the different piles

Based on the results summarized in Tables 4.12 to 4.13, it can be deduced that pile having diameter $B=325\text{mm}$ is the most appropriate deep foundation for the apartments hotel units. However, the driven pile should have a cap composed by a shallow foundation of $1.2\text{m}\times 1.2\text{m}$.

CHAPTER 5: CONCLUSION

5.1 Introduction

Many residents of the Saudi capital routinely travel to the Eastern Province, notably to the Half Moon bay, with their families on weekends to bond and take a break from daily routine. The bay, a crescent of soft sand in Al-khobar, is like a semi-lunar arch with several other attractions, such as entertainment zones and amusement cities. Half Moon Bay is a major recreational area for tourists, who throng the area on weekends. The soil area is mainly loose and saturated sandy soil susceptible to liquefaction.

The Liquefaction is a phenomenon in which the shear strength of a soil is reduced by shaking caused by an earthquake or another fast loading. The liquefaction occurs generally in saturated granular soils. The presence of water within the soil mass causes (during shaking) an increase of the pore water pressure on the soil particles; leading to a reduction of the actual effective lateral pressure in the soil. This leads to a decrease of the resistance to shearing of the soil; which at the ultimate status could become zero. The liquefaction of the soil is a serious problem that can cause damage, often irreversible, both to existing or new constructions

In this chapter, some relevant conclusions, drawn from this investigation, are summarized. These general conclusions are concerned by the structural and geotechnical design of the multistory hotel of irregular floor layout.

5.2 General Conclusions

The main objectives of this project are to design a multistory hotel of ellipsoidal shape to be constructed in the Half Moon where the soils are granular and saturated (i.e. susceptible to liquefaction). Moreover, the level of the water table in this case is almost on the surface. The project included:

- The design and repartition of the different components of the multistory hotel.
- Structural design of the different elements of the hotel using appropriate computer software, in this case ETABS.
- Geotechnical design which include the solutions proposed for the foundation systems for the hotel structure.

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

- A multistory hotel was developed and designed in a liquefiable soil. The site of the project extends over an area of about 1870 m². It is composed of twelve floors including three underground car parking.
- A structural design using computer software (ETABS) was performed, on the different components of the hotel, 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.
- Two types of foundation system were adopted for this project, including shallow foundations when the area subjected is subjected to dynamic compaction and deep foundations (i.e. driven piles).

