



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

The Structural and Geotechnical Design of a Steel Frame Multistory Carpark at PMU

Senior Design Project
Spring 2019

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Last but not least, we dedicate this project to our parents' unimaginable efforts in supporting us and our aspirations. You will forever have our unbridled love and utmost devotion.

Abstract

The student body at Prince Mohammed bin Fahd University (PMU) is growing at a pace that has overpopulated the current parking facilities on campus. A need has risen for an effective and reliable solution, that can handle the growing number of students in an effective and efficient manner.

This report deals with the structural and geotechnical design of a multi-story carparking facility at PMU. The main frame of the building is to be built of structural steel, while the supporting elements of the facility will be constructed from reinforced cement concrete.

The report will also outline the modelling and analysis of the structure, calculate and design reinforcement for the concrete elements, estimate the material cost of the project and attempt to provide a suitable methodology for which one may design such a facility.

This senior design project is in fulfillment of the requirements for completing the Senior Design Capstone Project, of the Civil Engineering Degree at Prince Mohammad bin Fahd University (PMU). Successful completion of this project in association with the fulfillment of the Civil Engineering degree plan proves the eligibility of the involved students to obtain a Bachelor of Sciences in Civil Engineering, as outlined in the criteria for successful commencement of engineering degrees set by the governing program bodies at PMU and the American Board of Engineering and Technology Accreditation (ABET).

We have attempted, through our chosen method of writing this report to clearly state and explain, to the best of our ability, the methods and techniques used to design this structure. The motive behind this explanatory approach, is to clarify the process in which one would approach designing such a structure. We also hope through the writing of this report, to offer as much assistance for as many generations of Civil Engineering students to come.

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Chapter 1

Introduction

1. Introduction

1.1 Problem Statement

Prince Mohammad bin Fahd University (PMU), is a private, higher-education institution, located on the eastern coastline of the Kingdom of Saudi Arabia. As of the writing of this report, the university has 4500+ students enrolled in its programs, with an almost even ratio of male to female students. According to the official enrollment statistics published by the university, annual student growth is at an average of 12% [1]. All classes and functions are currently held at the university's one and only campus, shown in the *Figure 1*.



Figure 1: PMU's Location in KSA; Image retrieved from Google Earth

As per the laws governing the conduct of educational institutions in Saudi Arabia, the university and all of its functions are gender segregated into male and female campuses [2]. Both campuses have their designated buildings and support facilities.

The male campus, includes an outdoor parking area with the capacity of 900+ cars. The parking area is utilized by the university's administration, faculty, staff and male students. After subtracting the specified areas for admin and faculty, students are left with a little over 700 spaces available to them at any given time. Important to note, is the fact that public transportation in the Eastern Province of Saudi Arabia is not at a level that enables students to depend on it for trips to and from PMU. With the exception of a university bus system offered exclusively for female students, the vast majority of male students commute to PMU using private vehicles. Thus, for the purpose of this project, we will be exclusively focusing on the male student population and their vehicular parking needs.

Regularly, during peak attendance hours (10 am - 2:00 pm), the on-campus parking reaches its capacity, and students are forced to park on the sides of an exterior road, spanning the western perimeter of the university. This poses both an inconvenience for the students and a risk to their safety, due to the lack of designated pedestrian crossings on the aforementioned road. Given the obvious safety issues concerning the usage of on-street parking, coupled with a steady rise in student enrollment, the need has risen for an extension to the current parking facilities at PMU.

We propose, in this theoretical exercise, an extension in the form of a multistory-building, situated on an empty lot, adjacent to the male students' entrance.

This recommendation stems from our desire to keep the visual appeal of the current building profiles at PMU and maintain the amount of green landscapes on campus.



Figure 2: View of male campus main entrance; Image retrieved from PMU website.

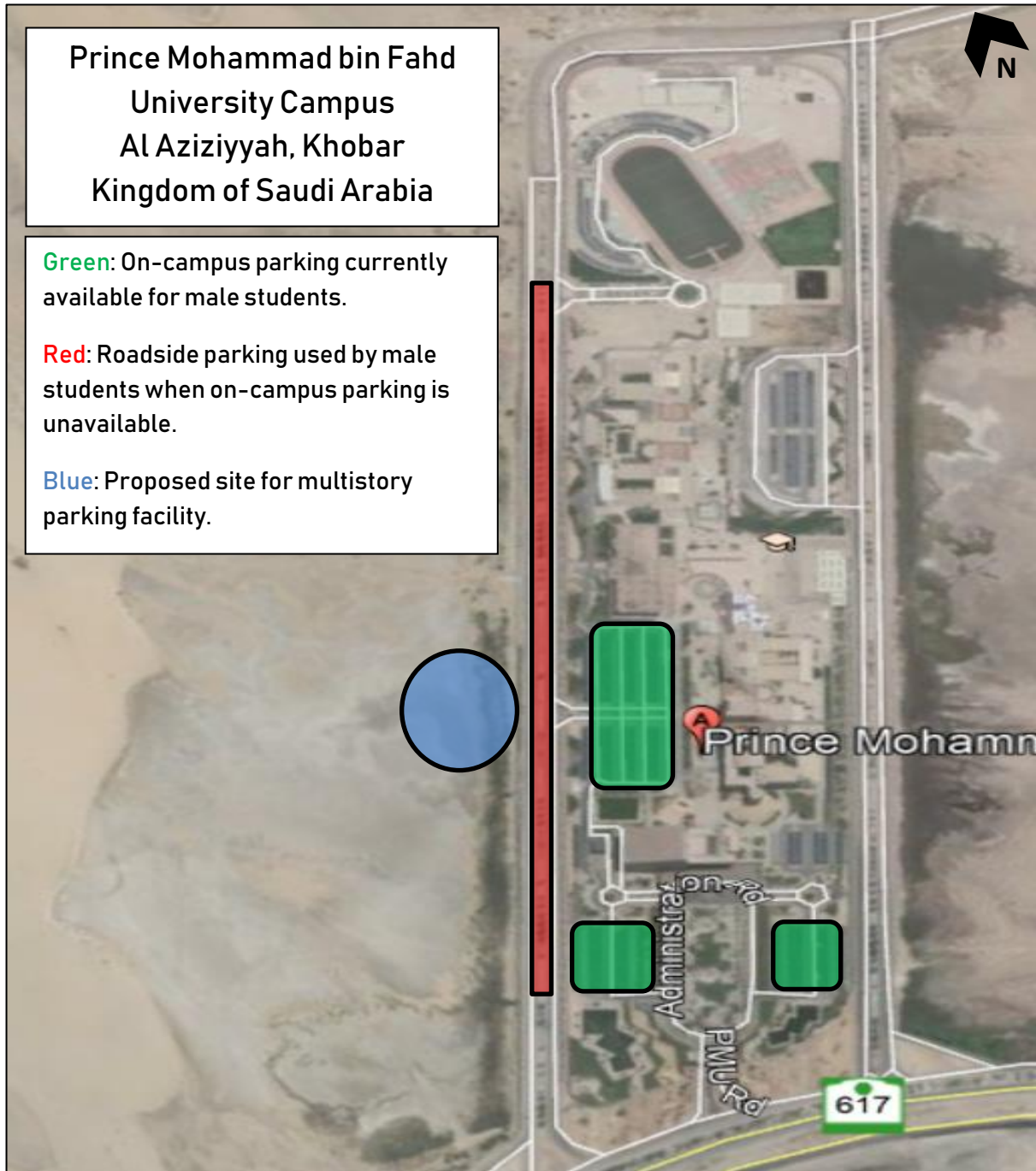


Figure 3: Diagram showing PMU campus parking areas, roadside parking and proposed project site.

1.2 Project Objective

The structural and geotechnical design of a multistory, steel-frame carparking facility, to be used for the male student population of Prince Mohammad bin Fahd University, located adjacent to the western border of the university's campus, which shall safely accommodate a minimum of 1000 vehicles.

1.3 Project Scope

The main body of this report is found in the following chapters:

Chapter 2: Project Guidelines and Constraints

Chapter 3: Project Description

Chapter 4: Structural Design

Chapter 5: Geotechnical Design

Chapter 6: Reinforcement

Chapter 7: Structural Modelling

Chapter 8: Cost Estimation

Chapter 2

Project Guidelines and Constraints

2. Project Guidelines and Constraints

2.1 Geotechnical

- Exemplify an appropriate soil property investigation and profile the findings in a suitable manner.
- Study, explain and set appropriate recommendations for soil remediation, temporary or\and permanent dewatering process\system (if found to be required).
- Provide a detailed design of suitable foundations, using universally accepted methods of calculation, that adequately support the structure, meet the bearing capacity of the underlying soil, meet allowable settlement values and provide long term support against likely problems.

2.2 Material

- The project must be designed for the implementation of a structural steel skeleton. Students must clearly state the mathematical and computational reasoning behind choosing the utilized sections.
- The flooring system “slabs” must be designed for the use of typical reinforced concrete. An explanation of the chosen slab thickness(es) and reinforcement detailing must also be provided.
- The design of any shear walls elements of the building must be detailed, with the calculations, modelling and reinforcement clearly demonstrated.

2.3 Environmental

- Design and construction of the building must consider the minimization of land disturbance, pollution of soil and groundwater, and must not lead to the obstruction of any waterways in the proximity of the site or downstream from it.
- The management plan outline must consider the preservation of local flora and fauna, and abide by national and international regulations for construction activities.

2.4 Structural and Design

- The structure must meet all general and specific code requirements set by the regulations of the Saudi Arabian MOMRA ¹, SBC ², AISC ³ and ACI ⁴. The structure must prove structural integrity, free of excessive slab deflection and sufficiently braced against excessive displacement or sway, resulting from seismic activity or wind drift.

¹ Saudi Ministry of Municipal and Rural Affairs [3].

² Saudi Building Code [4].

³ American Institute for Steel Construction [5].

⁴ Building code requirements; American Concrete Institute [6] .

- The structure must accommodate a minimum of one-thousand (1000) vehicles, as per project requirements, within a predefined building height restriction of six (6) stories.
- The manual calculations for the structural design must be accompanied by modelling the structure. This is both to affirm the accuracy of the manually calculated values and to exemplify knowledge of using CAD⁵ and analysis software for structural design purposes. The programs used for this purpose must be clearly stated and referenced herein.

2.5 Cost

- The design must adhere to economical and cost-effective methods of design and construction.
- A general approximation of the project's cost must be estimated, using local industry prices for materials.

2.6 Safety

- Propose a safe method to facilitate pedestrian movement to and from the structure, in accordance with local and international laws governing these traffic additions.

⁵ Computer Aided Design Software; AutoCAD.

Chapter 3

Project Description



Figure 4: Hamburg Airport's "Rotunda" Parking Building. Courtesy of Arch20.com.

3 Project Description

3.1 Original Layout

We've chosen to base our project on an existing multistory carparking facility, owned and operated by the Hamburg International Airport, in Hamburg, Germany. Designed by the German Architectural firm GMP Architekten GMBH [7], this cylindrically shaped building, sits 92 meters in diameter, stands 29 m in height, and is comprised of 10 circular floors, centered around a helical ramp. Total parking capacity is 2,115 cars and gross floor area is approximately 62,000 m² [7].

The main incentive behind our choice of this particular design is the high stability and resistance to physical and mechanical forces, circular buildings offer [8]. Furthermore, the structure utilizes space elegantly and efficiently, allowing for an organic yet orderly transition between various rows of parking spaces and floor levels. Lastly, the centrally placed ramp, comprised of two semi-circular shear walls, should protect against excessive lateral displacement due to wind or seismic forces, adding structural rigidity and integrity to the building.

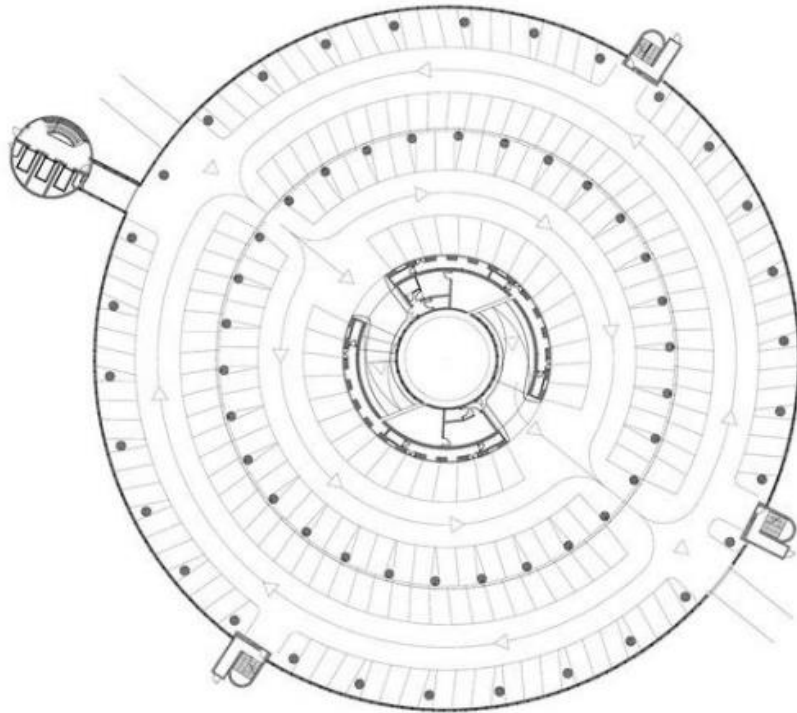


Figure 6: Hamburg Airport's "Rotunda" parking; Typical Floor Layout

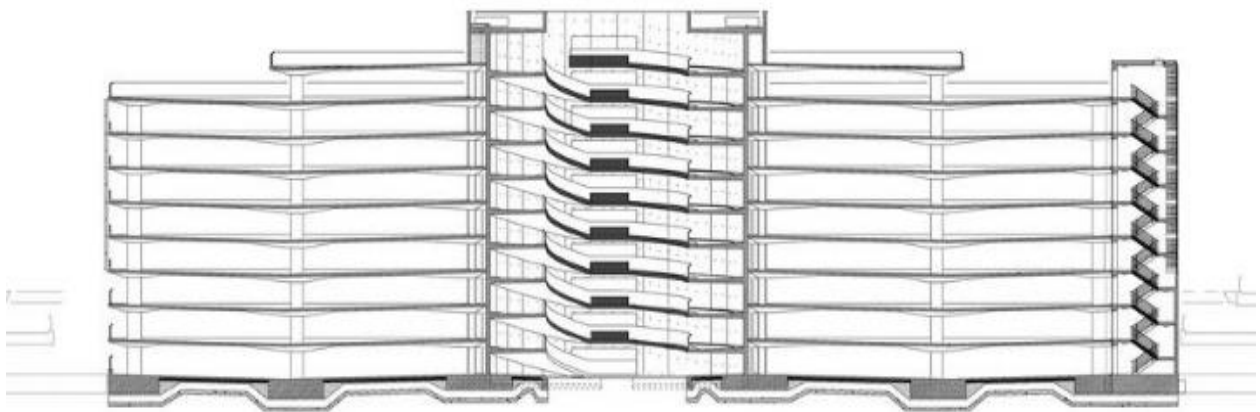


Figure 5: Hamburg Airport's "Rotunda" parking; Elevation

3.2 Modified Layout

Due to the initial design and construction of the chosen layout having abided to the European Standard for Building and Civil Works (Eurocode), it was necessary to modify the layout in conformity to the local codes and design standards, implemented in Saudi Arabia at the time being. More specifically, the Saudi Arabian Ministry of Municipalities and Rural Affairs' (MOMRA) [3] and the American Association of State Highway Transportation Officials' (AASHTO) standards and requirements for designing parking facilities. However, considering the codes specified in MOMRA are largely based on the AASHTO standards, we will suffice by planning our structure using MOMRA only, for the parking requirements part of this project.

Moreover, the base project we've chosen to modify has been designed as a column-slab structure, to be built of reinforced concrete. But, seeing as how our material of choice is structural steel, the design must be modified to a column-beam-slab structure, in order to achieve full structural rigidity using steel members and avoid issues with slab integrity, such as punching shear failure ⁶.

The modifications to the layout's dimensions have resulted in the following alterations to the base design:

- Individual parking stalls extended to 2.6 m in width, 5.5 m in length.
- Lane width extended to 7 m.
- Central ramp widened to 3.6 m.
- Story height raised to 3.5 m; to accommodate the minimum 2.4 m of free-standing clearance.
- Diameter of main structure extended to \approx 94 m.
- Single floor usable space extended to 6940 m².
- Based on the requirement of 5 stories, gross built area is approximately 35,500 m².

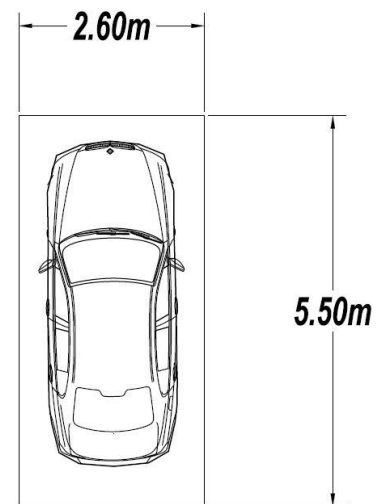


Figure 7: Modified parking stall.

These modifications offer us a total parking capacity of 1,004 cars, in line with our set target.

Worthy to note, is the omission of any accessibility parking spaces in our parking structure. This was due to the ample space that can be repurposed in the existing campus parking area in much closer proximity to the buildings. This negates the need to designate accessibility spaces in a relatively farther, off-campus parking facility.

⁶ **Punching shear** is a type of failure of reinforced concrete slabs subjected to high localized forces. In flat slab structures, this occurs at column support points [9].

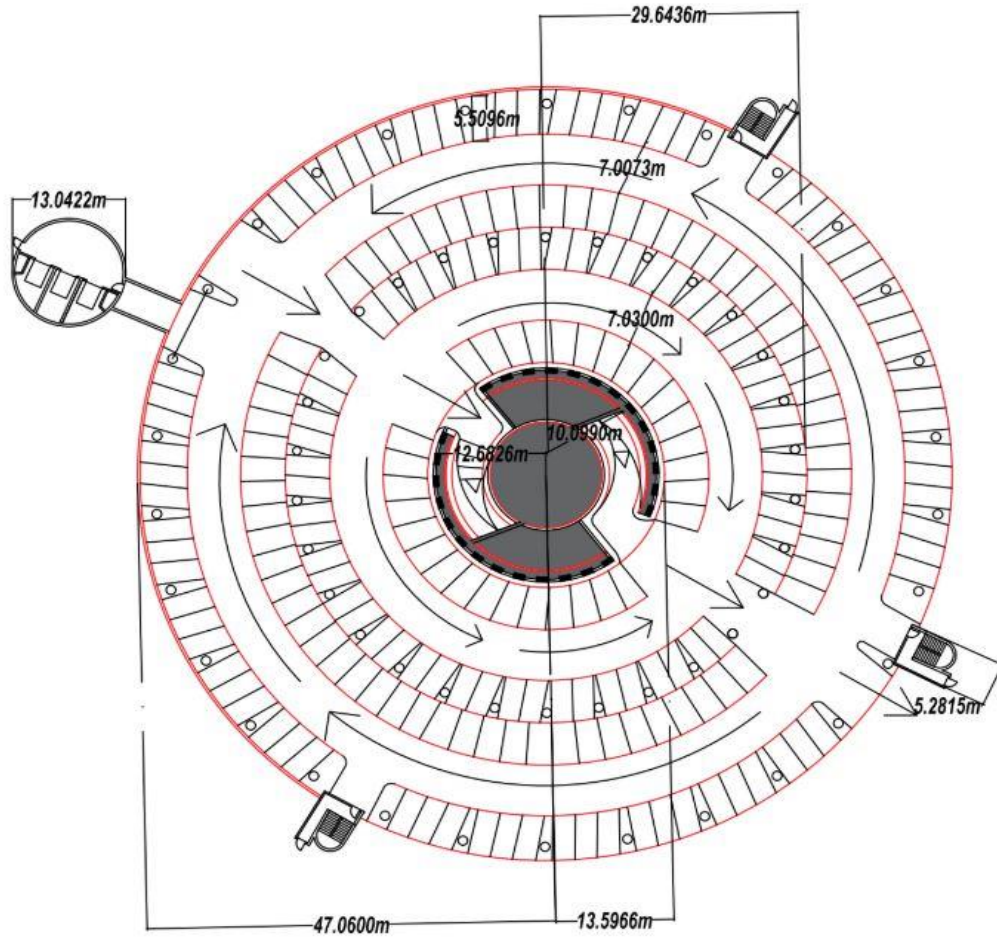


Figure 9: Modified elevation details

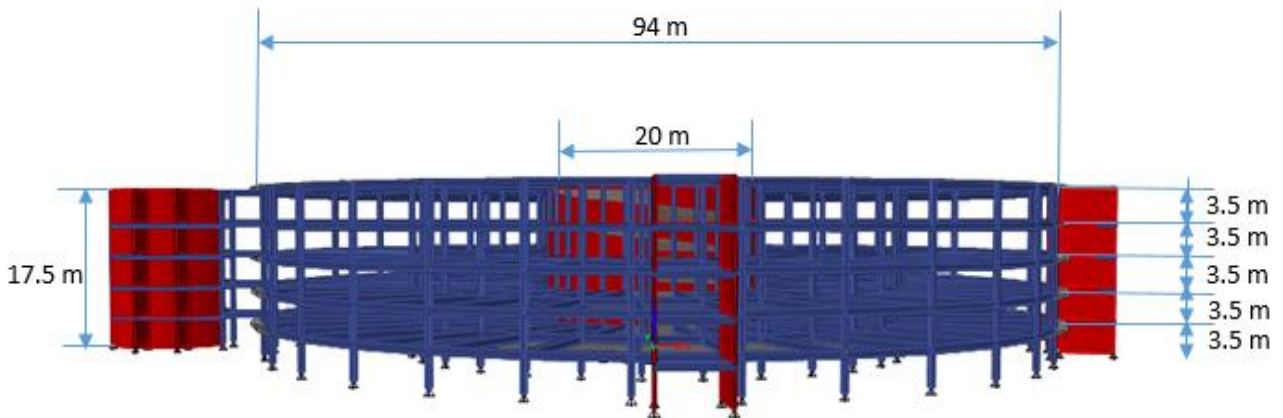


Figure 8: Modified layout dimensions

3.3 Preliminary Soil Survey

3.3.1 Soil Profile

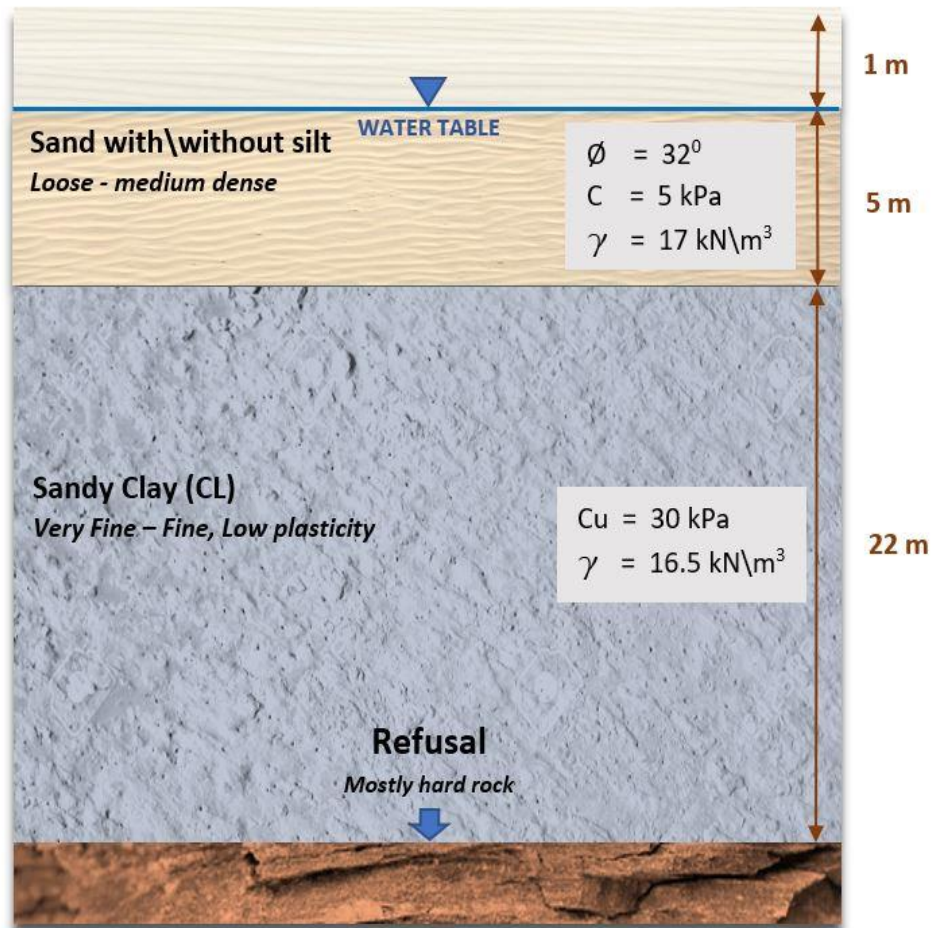


Figure 10: Soil profile at project site [10] .

The soil profile found in **Figure 10**, yields the following results;

- A topsoil layer of Sand with traces of Silt, loose to medium in density. Known to be problematic with relatively low bearing capacity (locally referred to as Sabkha soil⁷).
- A water table at the depth of 1 m.
- 22 m of low plasticity, Very Fine to Fine Sandy Clay.
- A bottom layer which refused penetration, assumed to be of hard bedrock in composition.

⁷ **Sabkha** is an Arabic word meaning salt flat and is applicable to both coastal and interior salt flats. The term can refer to one of two types of soils, based on level of saturation; Sandy and Muddy [11].

Notes on soil profile:

The relatively high water table ensues the need to provide a temporary dewatering system for the construction phase, and the subsequent design of a permanent dewatering system for further protection, for the remainder of the structure's design life.

Furthermore, the profile indicates a need for both shallow and deep foundations, in order to sufficiently support the structure. Considering the topsoil is comprised of a low bearing capacity material, we recommend the design of a mat (raft) foundation, resting on a pile system, which in turn sits on the hard bedrock for anchorage. The detailed design of the foundation solution shall be further discussed in the geotechnical chapter of project.

3.3.2 Chemical Analysis of Topsoil

The survey included a chemical analysis of the topsoil layer, in order to ascertain the risks on the shallow foundation. The analysis yielded the following results:

Table 1: Chemical analysis of the site's soil [10]

Sample No.	Depth (m)	Chlorides %	Sulphates %
1	1.00	0.381	0.391
2	1.00	0.321	0.400
3	3.00	0.088	0.375
Average		0.24	0.38
		Max permissible [12]	Max permissible [12]
		value > 0.05 %	value > 0.3 %

Notes on the chemical analysis:

As can be attained from the chemical analysis of the site's soil, it is both contaminated with Sulphates and Chlorides, indicating a need for adequate protection for the structure, from Chloride permeation or Sulphate attack⁸. This can be solved by modifying the w\c ratio in the

⁸ **Sulphate attack** on concrete is a chemical breakdown mechanism where Sulphate reacts with the C₃A (Tricalcium Aluminate) and gypsum present in the cement, forming expansive compounds that crack the concrete [13].

concrete mix to offer lower permeability, by using Type V cement ⁹ or by applying hydrophobic coating for the concrete elements in contact with the soil.

Recommendations for the design of the foundation system and the recommendations for the stabilization or remediation of this problematic soil will be further discussed in the geotechnical chapter.

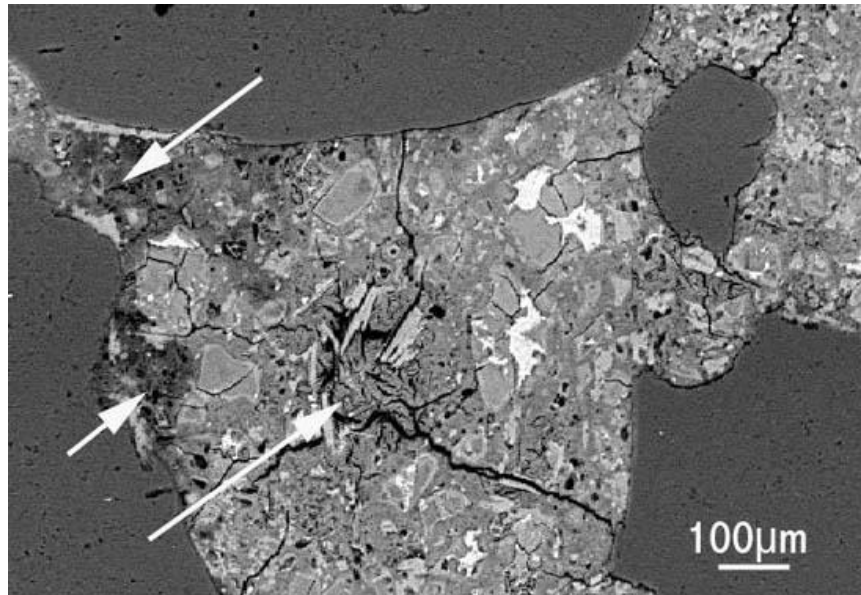


Figure 11: Electron microscopy of a Sulphate attack on concrete ⁱ

3.4 Benchmark Vehicle

We've opted throughout this project, wherever required or applicable, to test our design against the operation of a "benchmark vehicle".

The vehicle we've chosen is a late model, large sized SUV, that is a fairly common vehicle on Saudi roads. This choice is also attributable to the fact that using a larger vehicle for such a purpose, allows for a more comfortable design, and subsequently a more usable experience for vehicles of the same size or smaller.

⁹ **Type V Cement** also referred to as Sulphate Resistant Cement is a low C_3A content cement, used to lessen the occurrence of Sulphate Attack [13].

ⁱ Image courtesy of understanding-cement.com



Figure 12: Benchmark Vehicle: 2018 Toyota Land Cruiser ©

Table 2: Benchmark vehicle technical specifications ¹⁰

Dimension	Imperial	Metric
Length	194.9 in	495.046 cm
Width	77.95 in	197.993 cm
Height	74 in	187.96 cm
Wheelbase	112.2 in	284.48 cm
Turning circle diameter	38.7 ft	11.79576 m
Vehicle curb weight ¹¹	5815 lb.	2637.639 kg
Gross vehicle weight rating ¹²	7385 lb.	3553.896 kg
Weight on each tire (gross)	1846.25 lb.	837.445 kg

¹⁰ Technical specifications of a 2018 Toyota Land Cruiser; Retrieved from toyota.com.us.

¹¹ A vehicle's **Curb Weight** is the sum of all weights of vehicle components, excluding passengers or cargo.

¹² **The Gross Vehicle Weight Rating** is the maximum weight achievable when at full passenger and cargo capacity.

Chapter 3

Structural Design

4 Structural Design

4.1 Preface

4.1.1 *General methodology of design*

The design of any structural element or system of this project, was subjected to a certain process, to ensure the safety of the design, and conformity to the chosen codes and regulations;

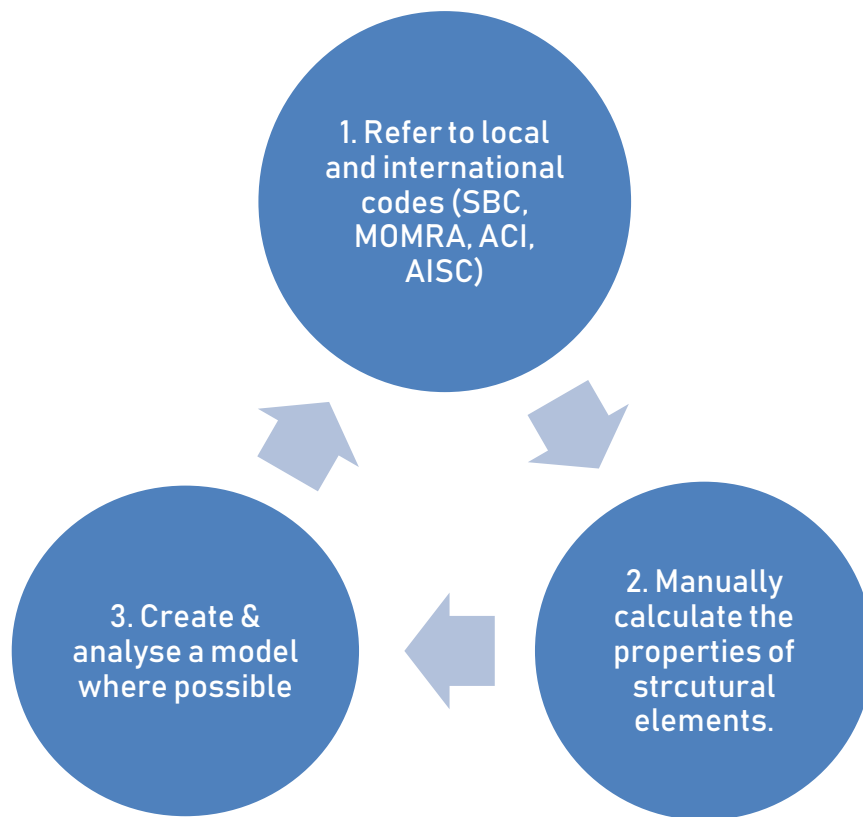


Figure 13: General design practice.

4.1.2 Loads, Forces and Stresses Acting on the Structure

As is the norm for any structural design process, the forces and stresses exerted on the structure must be factored into the design of the load bearing elements. The forces acting on our structure can be categorized into:

- **Dead loads;** this includes all of the non-dynamic forces that exert stress on the structure or element intended for analysis. These loads act on the structure throughout its design life and will generally stay constant.
- **Live loads;** this includes the stress exerted on the element from vehicular traffic, pedestrian activity and service loads (Natural forces; Wind and Precipitation). These loads are variable and subject to change depending on the conditions of operation.

The equation used for calculating the strength of a member under ultimate stress conditions is as follows¹³:

$$U = W_u = 1.2 (L_D) + 1.6 (L_L) = kN/m^2 \text{ or } kips/ft^2$$

Equation 1: Factored Load Combination

Notes on Equation for Load Combination:

The term **0.5 (L_r or S or R)**, which is present in the original form of the equation has not been implemented in this instance. This is due to a separate calculation of the roof's service load (**L_r**), a negligible precipitation value (**R**) and a climatic improbability of a snow service load (**S**).

¹³ Equation 5.3.1b; ACI [6]

Breakdown of the parking facility's structures:

The parking facility is comprised of 4 types of structures: -

- a) *Main parking building.*
- b) *Elevator building.*
- c) *Pedestrian bridges, linking elevator building to main structure.*
- d) *Staircases and fire exits.*
- e) *Central ramp*

Legend

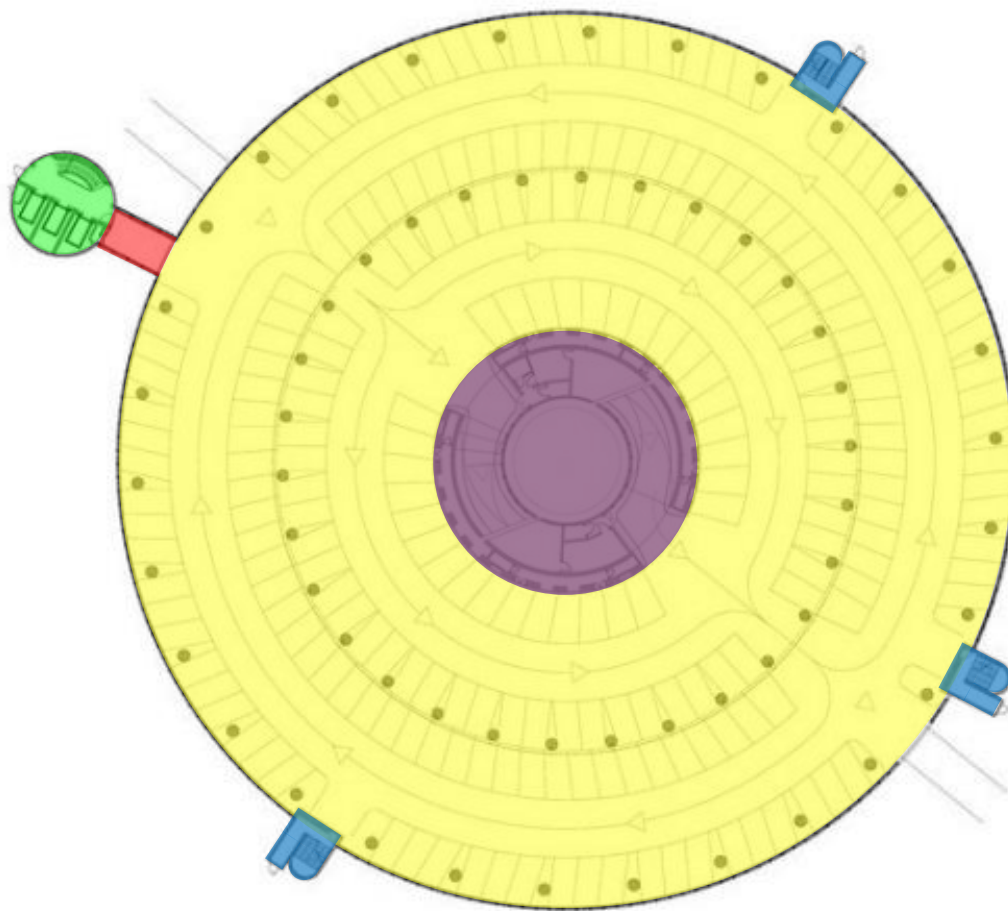


Figure 14: Breakdown of Parking Facility Structures

Each of these structures possesses a unique set of load combinations, imposed on the slabs of each floor. The usage of a nominal slab thickness or steel section may not be feasible for all types of structures. As a result, they may or may not utilize the same structural elements as each other. But, whenever possible, a nominal depth or section will be chosen, to facilitate construction and erection.

Table 3: Summary of Loads on the Parking Facility Structures ¹⁴

Structure	kN/m^2		$kips/ft^2$	
	Dead	Live ¹⁵	Dead	Live
<i>Main</i>	9.058	2	≈ 0.189	≈ 0.0577
<i>Elevators</i>	10.733	2	≈ 0.224	≈ 0.0577
<i>Staircase</i>	8.833	2	≈ 0.184	≈ 0.0577
<i>Pedestrian Bridges</i>	10.733	3	≈ 0.224	≈ 0.0865
<i>Roof</i>	9.258	0.676	≈ 0.193	≈ 0.0195

Note: The roof value shown, has been calculated for the main structure's roof respectively, under the highest possible critical condition. Seeing as how it would provide adequate strength for the other structures, the same value will be applied nominally for all roofs.

¹⁴ Refer to Appendix A: section 10.1.1 for specific breakdown and calculations of dead loads.

¹⁵ SBC 301; Table 4-1, 4-2 [4]

4.2 Slabs

4.2.1 Slab Design Method

1. Classification of slab behavior type:

In order to use the correct method of determining the thickness of a slab, we must first determine the slab's behavior under loading, which is more commonly referred to as "Type".

This is classified by finding the ratio of the slab's longer span to its shorter one, as can be attained from **Equation 2**¹⁶:

$$\text{Ratio} = \frac{\text{Long Span}}{\text{Short Span}}$$

Equation 2: Method of slab type classification

There are two distinct types of slabs, as defined by the ACI¹⁷ criteria;

- One-way \longrightarrow $\text{Ratio} \geq 2$
- Two-way \longrightarrow $\text{Ratio} < 2$

This distinct classification entails the usage of different formulas to calculate the dimensions and properties of the slabs. For the design of concrete elements in this project, an $f'_c = 4000$ psi is considered.

2. Determining slab thickness:

After classifying the slab's behavior type, we can find the minimum slab thickness required to safely accommodate the loads imposed. Worthy to note, is the fact that the ACI code for determining the minimum thickness of a two-way slab requires knowledge of the dimensions of beams that support it. However, considering the beams have not yet been designed at this stage, a calculation using the ACI recommended equation was not possible. Thus, we have referred to another method for determining slab thicknesses, as is demonstrated by **Equation 3**¹⁸:

$$\text{Slab thickness} = \frac{\text{perimeter}}{180}$$

Equation 3: Minimum slab thickness

¹⁶ Based on interpretations of ACI R7, R8 [6].

¹⁷ Refer to ACI [6] chapter R7, R8 for further details.

¹⁸ Equation retrieved from reference [16].

4.2.2 Slab Design Example; Main Structure

The tributary area for the slabs of the main structure, have been taken as trapezoidal segments with dimensions as shown in **Figure 15**.

Our longer span ≈ 16.4 m, and the shorter span will be considered to ≈ 9.7 m. We've taken the shorter span to be the longer of the two edges, to ensure higher safety in design.

Calculating the ratio for this segment yields:

$$\text{Ratio} = \frac{\text{Long Span}}{\text{Short Span}} = \frac{16.4017\text{m}}{9.6778\text{m}} = 1.695 < 2$$

Considering the Ratio is less than 2, the slab can be classified as two-way.

Now, to calculate the minimum thickness:

$$\text{Slab thickness} = h_{min} = \frac{\text{parameter}}{180} = \frac{2(16.407)\text{m} + 6.0313\text{m} + 9.6778\text{m}}{180} = 0.269\text{m}$$

\therefore The obtained slab thickness $h_{min} = \underline{0.269\text{ m}}$.

By rounding this value up, we would gain ample space for cover and reinforcement.

Thus, $h_{min} = 0.280\text{ m} = 280\text{ mm}$ (11.024 inches).

4.2.3 Slab Design Example; Pedestrian Bridge

Using the same method in the previous slab calculation, we attain the following results:

- Using **Equation 2**, to find ratio;

$$\text{Ratio} = \frac{\text{Long Span}}{\text{Short Span}} = \frac{6.9626\text{ m}}{4.2\text{ m}} = 1.657 < 2$$

\therefore The slab is \longrightarrow two-way.

- Using **Equation 3**, to find thickness:

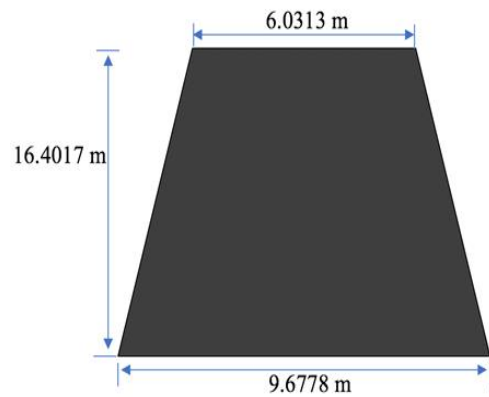


Figure 15: Dimensions of main structure slab segment

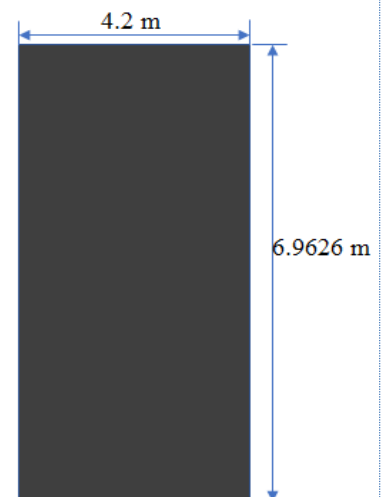


Figure 16: Dimensions of pedestrian bridge slab

$$h_{min} = \frac{parameter}{180} = \frac{2(6.9626)m + 2(4.2)m}{180} = 0.124 m \approx 4.88 \text{ inches}$$

The obtained value of $h_{min} = 4.88$ inches, is below the minimum allowable thickness of a non-drop panel, two-way slab, specified by the ACI code ¹⁹. Thus, the thickness of the slab shall be raised to 9 inches (229 mm).

4.2.4 Slab Design Example; Staircase Structure

The actual effective area on which the load combination acts is as shown in **Figure 17**.



Figure 17: Dimensions of staircase slab

- Using Equation 2, to find ratio;

$$Ratio = \frac{Long\ Span}{Short\ Span} = \frac{6\ m}{5\ m} = 1.2 < 2$$

∴ The slab is \longrightarrow two-way.

- Using Equation 3, to find thickness:

$$Slab\ thickness = \frac{parameter}{180} = \frac{2(6)m + 2(5)m}{180} = 0.122 m \approx 4.80 \text{ inches}$$

Similarly, the obtained value is less than the required minimum, as well. Thus, the thickness we be considered as the minimum recommended depth of 6 inches (153 mm).

4.2.5 Slab Design Example; Elevator Building

As shown in **Figure 18**, the tributary area for the elevator building's slabs, are designed as semicircles with a diameter of 13 m. Thus, the slab thickness is:

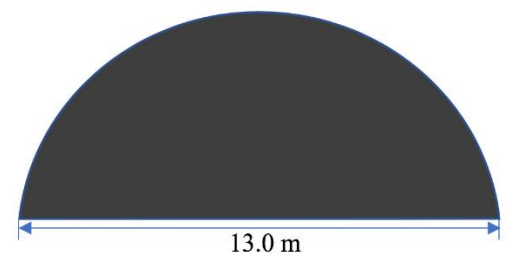


Figure 18: Dimensions of elevator building slabs.

$$Slab\ thickness = \frac{parameter}{180} = \frac{\pi + 13\ m}{180} = 0.226 m$$

¹⁹ ACI 8.2.4 [6]

Despite the result generated above being satisfactory, we will increase the depth of the slab to 9 inches (229 mm). This recommendation stems from the perceived benefit of unifying slab depths, on the construction process.

4.2.6 Summary of Slab Thicknesses

Table 4: Summary of slab thicknesses

Units	mm	in
<i>Main structure</i>	280	11
<i>Stairs structure</i>	153	6
<i>Pedestrian bridge structure</i>	229	9
<i>Elevator structure</i>	229	9

4.3 Horizontal supports; Beams and Girders

4.3.1 Girder and Beam Design Method

The integrity of any structure acting as a unit is gauged by the individual integrity of each of its constituting elements. Thus, it is imperative in the design of any structure that possesses horizontal support elements (girders or beams), to account for safe accommodation of the imposed stresses or loads. To do so, one must first determine the tributary area on which the imposed stresses act and resolve them into a UDL.

All horizontal and vertical support elements in our project will comprised of industry-standard, structural-grade, steel sections, listed in the AISC manual²⁰. The method of design implemented in determining what section are chosen, can be summarized as follows:

1. Convert the stress (trapezoidal or otherwise) into a UDL, to simplify the analysis of forces acting on the element, and choice of a suitable section. The formula for transforming trapezoidal stress into a UDL acting on a girder is shown in **Equation 4**. Alternatively, the formula for transforming the same area to a UDL for a beam is shown in **Equation 5**.
2. Calculate the element's maximum allowable deflection. Details of this are shown in **Equation 6**.
3. Calculate the nominal moment acting on the element, as detailed in **Equation 7**.
4. Use the obtained values to find the minimum moment of inertia the section must possess, in order to safely counteract the imposed loads. Details in **Equation 8**.
5. Choose a section from the manual that satisfies all the above criteria. Give due regard to the section's proportions. A larger section may be able to bear a larger load. But, may not be practical when attempting to connect it to the columns.
6. Lastly, perform a design check using the properties of the chosen section. Recalculate the section's nominal moment and shear bearing (**Equation 9**) ensure the deflection is less than the allowable limit. If the results are satisfactory, apply.

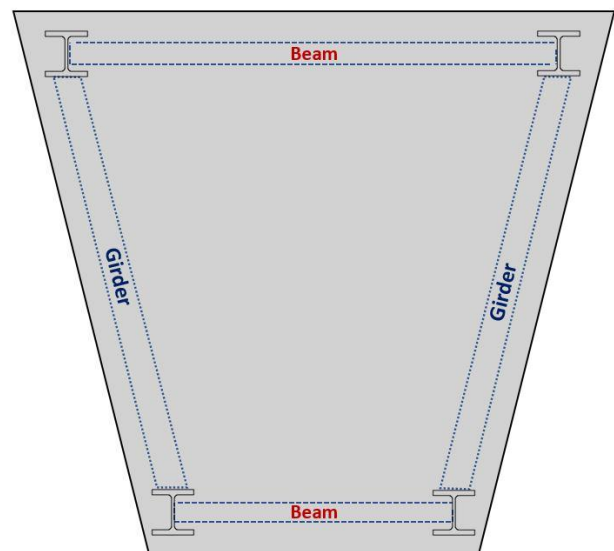


Figure 19: Distribution of horizontal support elements; main structure.

²⁰ AISC Steel Construction Manual [5].

Equations used:

$$I_y = \frac{W_u l_x}{6} \left[3 - \left(\frac{l_x}{l_y} \right)^2 \right]$$

*Equation 4: Transformation of trapezoidal stress into a UDL for a girder*²¹

Where:

- I_y transformed trapezoidal stress, in the form of a UDL (kN/m or kips/ft)
- W_u factored load combination (kN/m² or kips/ft²)²².
- l_x length of the trapezoidal area, on the x-axis (m or ft).
- l_y length of the trapezoidal area, on the y-axis (m or ft).

$$I_y = \frac{W_u L_x}{3}$$

Equation 5: Transformation of trapezoidal stress into a UDL for a beam.

Where:

- I_y transformed trapezoidal stress, in the form of a UDL (kN/m or kips/ft)
- W_u factored load combination (kN/m² or kips/ft²)²³.
- L_x length of the trapezoidal area, on the x-axis (m or ft).

$$\delta_{all.} = \frac{l}{240}$$

*Equation 6: Maximum deflection limit for horizontal structural elements*²⁴.

Where:

- $\delta_{all.}$ maximum allowable deflection (inches).
- l length of the horizontal element (inches).

²¹ Equation 3.5; Al-Mulhim, et al. (2018) [17].

²² See *Equation 1*.

²³ See *Equation 1*.

²⁴ SBC; Table 1.4-1 [4].

$$M = \frac{w (L)^2}{8}$$

Equation 7: Moment calculation ²⁵.

Where:

- M nominal moment (**kips. Ft**).
- W uniformly distributed load, UDL (**kips/ft**).
- L length of the horizontal element (**ft**).

$$\delta = \frac{M (L)^2}{C_1 I_x}$$

Equation 8: Deflection calculation for a simply supported, steel beam ²⁰.

Where:

- δ deflection (**inches**).
- L length of the horizontal element (**ft**).
- M nominal moment (**kips. Ft**)
- I_x moment of inertia about the x-axis (**inches⁴**)
- C_1 modification factor, also referred to as a loading constant (**unitless**)

Note on Equation 8:

The modification factor $C_1 = 161$ for a uniformly distributed load, acting on a simply supported beam ²⁶.

$$V = \frac{w \cdot l}{2}$$

Equation 9: Maximum shear force from a UDL ²²

Where:

- V shear force (**kips**).
- M nominal moment (**kips. Ft**)
- L length of the horizontal element (**ft**).

²⁵ AISC [5] ; Table 3-23: 1

²⁶ AISC [5] ; Fig. 3-2.

All calculations for the design of girders and beams will use **Equation 1**²⁷ to determine the value of W_u , relying on the values found in **Table 2**²⁸.

4.3.2 Girder Design Example; Main Structure Roof

- $W_u = 12.311 \frac{kN}{m^2} = 0.257 \frac{Kips}{ft^2}$
- $L_x = 31.651 \text{ ft} ; L_y = 53.811 \text{ ft}$

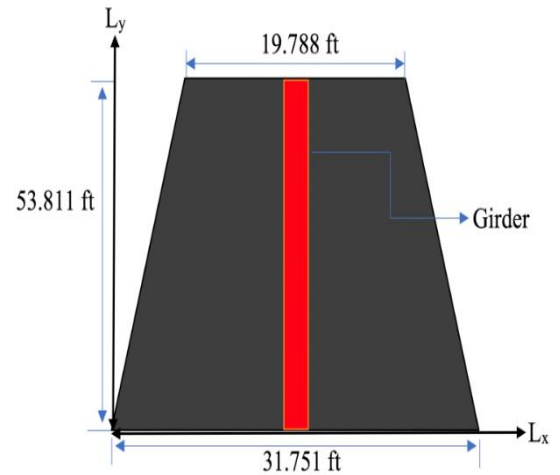


Figure 20: Roof girder design; main structure.

$$I_y = \frac{0.257 \frac{Kip}{ft^2} (31.751 \text{ ft})}{6} \left[3 - \left(\frac{31.7515 \text{ ft}}{53.811 \text{ ft}} \right)^2 \right] = 3.608 \frac{Kips}{ft}$$

$$\therefore I_y = \underline{3.608} \text{ kips/ft}$$

$$\delta_{all.} = \frac{53.811 \text{ ft} \left(12 \frac{in}{ft} \right)}{240} = 2.69 \text{ in}$$

$$\therefore \delta_{all.} = \underline{2.69} \text{ in.}$$

$$M = \frac{3.608 \frac{Kip}{ft} (53.811 \text{ ft})^2}{8} = 1306.182 \text{ kips} \cdot \text{ft}$$

$$\therefore M = \underline{1306.182} \text{ kips} \cdot \text{ft}$$

Plugging the obtained values into **Equation 8**, to attain the desired section's moment of inertia:

$$2.69 \text{ in} = \frac{1306.182 \text{ kip} \cdot \text{ft} (53.811 \text{ ft})^2}{161 I_x}$$

$$\text{Solving for } I_x \longrightarrow \therefore I_x = \underline{8731.786} \text{ in}^4$$

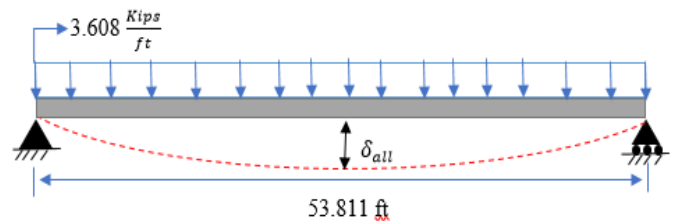


Figure 21: Beam deflection representation; roof girder of main structure.

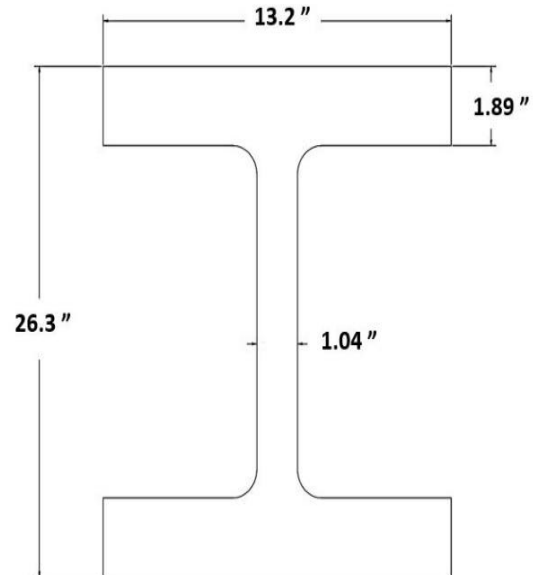
²⁷ Equation 1; Page 24.

²⁸ Table 2; Page 26.

- From the calculations above, we can find a section that satisfies the required criteria:

Table 5: Roof girder section properties; main structure

W 24 x 250	
Depth (in)	24
I_x (in⁴)	9600
$\phi_n V_x$ (Kips)	929
$\phi_n M_x$ (Kips.ft)	3130
Weight $\frac{Kips}{ft}$	0.250



- Performing a design check, to confirm the section's suitability;

Checking the moment, shear force, and deflection limit of the girder including its own weight:

$$W_u = 1.2 (0.279) + 3.608 = 3.943 \frac{Kip}{ft}$$

$$M = \frac{3.943 \frac{Kip}{ft} (53.811 ft)^2}{8} = 1427.378 Kip \cdot ft < 3130 Kip \cdot ft$$

∴ Since the nominal moment the section can bear is bigger than the moment caused by the specified load combination, we can consider the section safe in regards to deforming under the force of moment.

$$\delta = \frac{1427.378 kips \cdot ft (53.811 ft)^2}{161 (9600 in^4)} = 2.67 in < 2.69 in$$

∴ Considering the amount of deflection calculated using the properties of this section is less than the allowable limit, the section will not experience intolerable deflection when subjected to its specified load combination.

$$V = \frac{w l}{2} = \frac{3.943 \frac{\text{kips}}{\text{ft}} (53.811 \text{ft})}{2} = 106.097 \text{ Kip} < 929 \text{ Kip}$$

∴ Considering the section's resistance to deformation under shear force is larger than that of which the load combination would impose, we can consider the section as safe in this regard as well.

- ***Taking into consideration all the above calculations and results, we can safely conclude that the section will adequately provide structural integrity and safeguard against the failure of the structure it is intended to support.***

Therefore, our choice for a section to use as the roof girder is safe to implement.

4.3.3 Beam Design Example; Typical Floor

The design method for a beam is similar to that of a girder. The sole difference lies in transforming the trapezoidal stress, resulting from the slab segment's shape and the loads acting on it, into a UDL. The following is an example of the design method applied, for a typical floor's beam ²⁹:

1. Using **Equation 5**, for the transformation of the load combination to a UDL:

- $W_u = 14.069 \frac{kN}{m^2} = 0.249 \frac{kips}{ft^2}$
- $L_x = 31.751 \text{ ft}$

$$I_y = \frac{0.249 \frac{kips}{ft^2} (31.751 \text{ ft})}{3} = 3.11 \frac{kips}{ft}$$

2. Checking for allowable deflection:

$$\delta_{allowable} = \frac{L \text{ (in)}}{240} = \frac{31.751 \text{ ft} \left(12 \frac{\text{in}}{\text{ft}}\right)}{240} = 1.587 \text{ in}$$

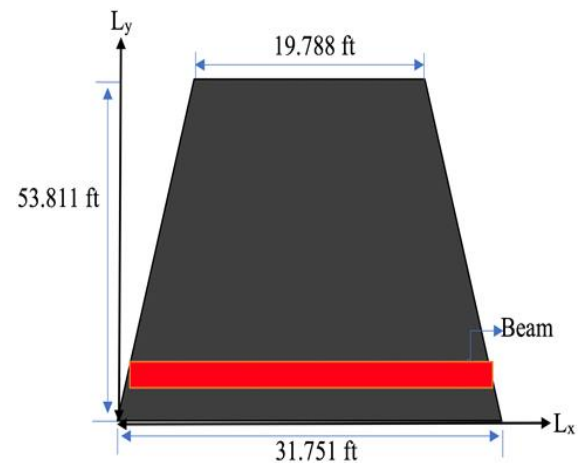


Figure 22: Typical-floor beam design; main structure

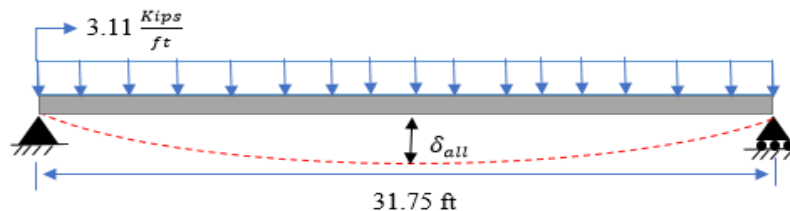


Figure 23: Beam deflection representation; typical floor beam of main structure.

3. Calculating the nominal moment:

$$M = \frac{w(L)^2}{8} = \frac{3.11 \frac{kips}{ft} (31.751 \text{ ft})^2}{8} = 391.962 \text{ kips.ft}$$

²⁹ The **typical floors** in the main structure are the 1st (ground), 2nd, 3rd and 4th floors.

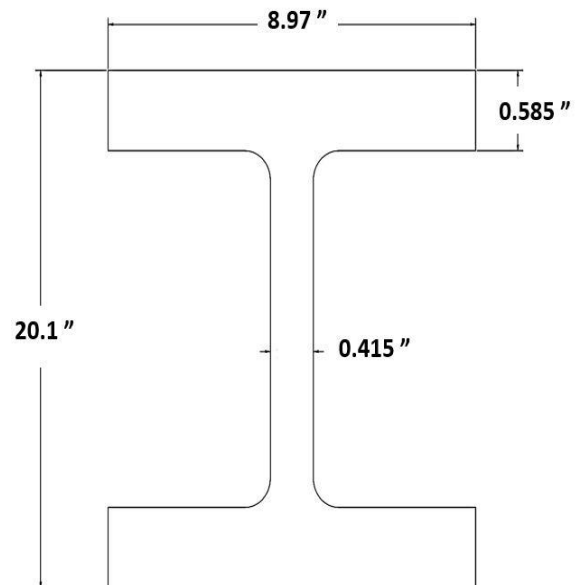
4. Using the above values to attain the minimum moment of inertia required:

$$1.587 \text{ in} = \frac{391.962 \text{ kip.ft} (31.751 \text{ ft})^2}{161 I_x}$$

5. When solving for I_x in the above equation, we get an $I_x = \underline{1546.078 \text{ in}^4}$. Using the calculated properties, we choose a section and test it:

Table 6: Typical-floor beam properties; main structure

W 24 x 68	
Depth (in)	24
$I_x (\text{in}^4)$	1830
$\phi_n V_x (\text{Kips})$	295
$\phi_n M_x (\text{Kips.ft})$	664
Weight $\frac{\text{Kips}}{\text{ft}}$	0.068



6. Performing a design check; Shear, Moment, Deflection:

$$V = \frac{w l}{2} = \frac{3.192 \frac{\text{kips}}{\text{ft}} (31.75 \text{ ft})}{2} = 85.878 \text{ kips} < 295 \text{ kips}$$

$$Wu = 1.2 (0.068) + 3.11 = 3.192 \frac{\text{kips}}{\text{ft}}$$

$$M = \frac{3.192 \frac{\text{kips}}{\text{ft}} (31.75 \text{ ft})^2}{8} = 402.246 \text{ kips.ft} < 664 \text{ kips.ft}$$

$$\delta = \frac{664 \text{ kips.ft} (31.75 \text{ ft})^2}{161 (1830 \text{ in}^4)} = 1.377 \text{ in} < 1.587 \text{ in}$$

- **According to the results above, the section has passed all the required design checks and exceeds the limit values. Thus, it is safe to utilize in our structure.**

4.3.4 Summary of Horizontal Sections Chosen

The design of all the beams and girders of this project, has been conducted using the previously explained methods. For further details on the design of the remaining elements, refer to **Appendix B**.

Table 7: Summary of sections chosen as horizontal support elements

<i>Element Location</i>	<i>Type</i>	<i>Section Used</i>
Main Structure; Roof	Girder	W 24 x 250
Main Structure; Roof	Beam	W 24 x 94
Main Structure; Typical floor	Girder	W 24 x 279
Main Structure; Typical floor	Beam	W 24 x 68
Elevator Building	Beam	W 24 x 279
Pedestrian bridges	Beam	W 14 x 38

4.4 Vertical Supports; Columns

4.4.1 Column Design Method³⁰

In order to ensure the safety of our design and select a column section that can sustain the structural loads imposed, we utilize to the following design method:

1. First, transform and sum all UDLs imposed from horizontal elements, connected to the column your wished to design. Factor in their self-weights and transform them into reaction forces (point loads). This is achieved using **Equation 8** from the previous section.
2. Calculate the summation of the reaction forces found in *step 1*, as well as all the cumulative loads acting on the column. These forces will be resolved and will act as a single, downward resultant, centrally loaded on the column, as is shown in **Figure 24**.
3. Afterwards, use the nominal yield strength of the grade of steel utilized in the design and substitute the same value for the slenderness ratio found in **Equation 10**. This allows us to find a value for the column's Critical Compression Stress value, from table 4-22³¹.
4. Substitute the newfound critical stress into **Equation 11**, to find the gross area of the section on which the forces will act.
5. Use the gross area to select a suitable column section from Table 6-1³¹. Apply a factor of safety to the section's required properties, to account for future amendments to the design. If the section has proven excessively safe during modelling, reverse engineer to find a more balanced choice.
6. Finally, using the properties of the selected section, solve for the actual nominal capacity by applying **Equation 11**. If the section proves calculated safety, apply.

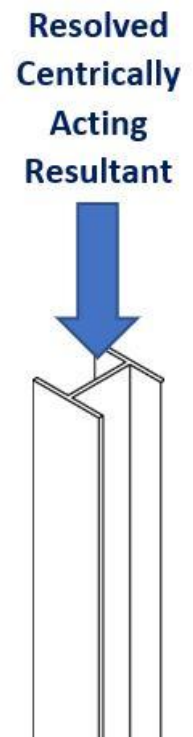


Figure 24: Reaction forces resolved into a centric resultant

³⁰ Steps for steel column design method based on recommendations from [18].

³¹ AISC [5]

Equations used:

$$\lambda = \frac{kl}{r}$$

Equation 10: Ratio of slenderness for a steel column

Where:

- λ slenderness ratio (**unitless**)
- kl effective length (**inches**)
- r radius of gyration (**inches**)

$$\phi P_n = A_g \phi F_{cr}$$

Equation 11: Factored nominal capacity of a steel section

Where:

- ϕP_n factored nominal capacity for the section (**kips**)
- A_g gross area of the section (**inches²**)
- ϕF_{cr} factored flexural buckling stress of the column (**ksi**)

4.4.2 Critical Column Design Example; 1st floor of main structure

1. Transforming the UDLs to reaction forces:

$$R_{W24 \times 250} = \frac{wl}{2} = \frac{4.491 \frac{\text{kips}}{\text{ft}} (53.841 \text{ ft})}{2} = 120.9 \text{ kips}$$

$$R_{W24 \times 68} = \frac{4.491 \frac{\text{kips}}{\text{ft}} (31.753 \text{ ft})}{2} = 50.678 \text{ kips}$$

2. Resolving the summation of forces into a resultant. (Cumulative load \approx 1330 kips):

$$P_u = 2 (120.9) + 2 (50.678) + 1330.988 = 1674.143 \text{ kips}$$

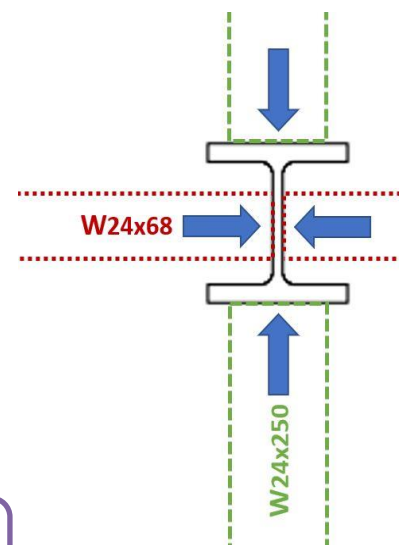


Figure 25: Resolving beams and girders as reaction forces

3. Using an $F_y = 50 \text{ ksi}$ and a value of $\frac{KL}{r} = 50$ to find F_{cr} :

Table 4-22 $\longrightarrow F_{cr} = \underline{37.5 \text{ ksi}}$

4. Substituting F_{cr} to find the minimum required gross area of the section:

$$1674.143 \text{ kip} = A_g (37.5 \text{ ksi}) \longrightarrow A_g = 44.64 \text{ in}^2$$

5. Applying a factor of safety ≈ 2 for the gross area required, we select a section for the design of the column:

Table 8: Properties of the section chosen for critical columns

W 14 x 283	
Gross Area A_g (in^2)	83.3
r_y (in)	4.17

6. Solving for $\frac{KL}{r}$ to find F_{cr} :

Note: The height of the column is $3.5 \text{ m} = 11.482 \text{ ft} = 137.784 \text{ in}$.

$$\frac{KL}{r} = \frac{137.784 \text{ in}}{4.17 \text{ in}} = 33.04 \longrightarrow F_{cr} = 29.8 \text{ ksi}$$

$$\phi P_n = A_g \phi F_{cr} = 83.3 \text{ in}^2 (0.9)(29.8 \text{ ksi}) = 2234.106 \text{ kips} > 1674.143 \text{ kips}$$

According to the verification above, the section is safe and will suffice.

4.4.3 Summary of Vertical Support Sections Chosen

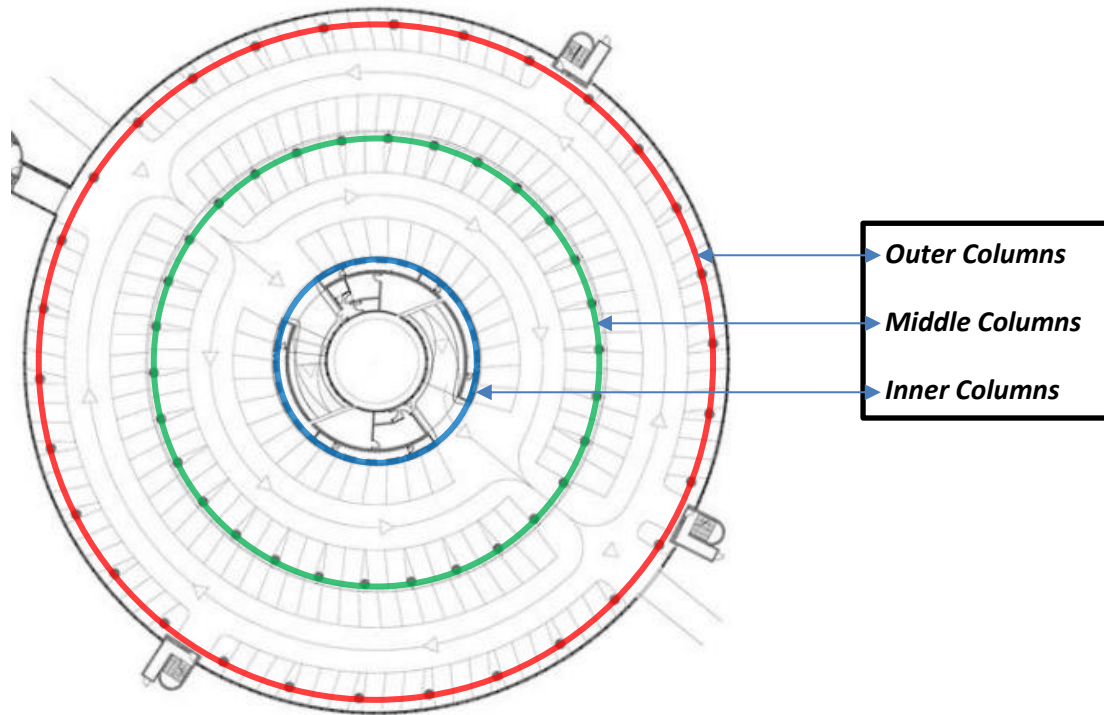


Figure 26: Location of column groups in main structure layout

Table 9: Summary of columns in the main structure and their locations ³²

Story No.	Inner columns	Middle columns	Outer columns
5	W12x26	W12x58	W12x45
4	W12x65	W14x176	W12x120
3	W12x65	W14x176	W12x120
2	W12x106	W14x283	W12x190
1	W12x106	W14x283	W12x190

³² Refer to **Appendix B: 10.2.2.1** for a detailed breakdown of column sections and their locations.

4.5 Shear Walls

Shear walls are structural elements, that bear loads and provide resistance to lateral forces acting on a given structure³³. This can significantly reduce lateral sway and thereby lessen the extent of damage to the structure and its contents, under exhaustive loading scenarios. There are various types of shear walls, as categorized by the SBC³⁴;

- **Ordinary reinforced concrete structural wall.**
- **Special reinforced concrete structural wall.**
- **Intermediate precast structural wall.**
- **Special precast structural wall.**

For our intended usage, ordinary reinforced concrete walls should provide sufficient structural rigidity and effectively bear the loads we wish to impose on them.

As can be shown from the figures below, there are three regions within our structure that incorporate shear walls in their design (**Shown in Red**):

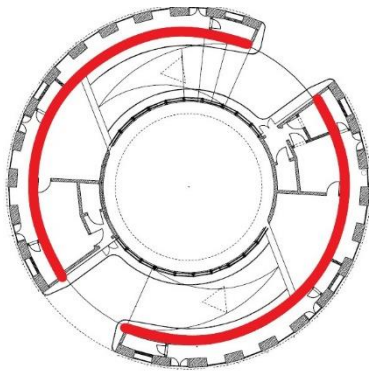


Figure 28: Ramp Shear Wall

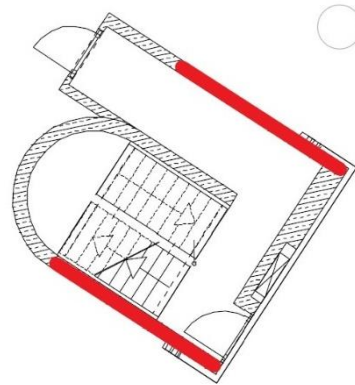


Figure 29: Staircase shear wall

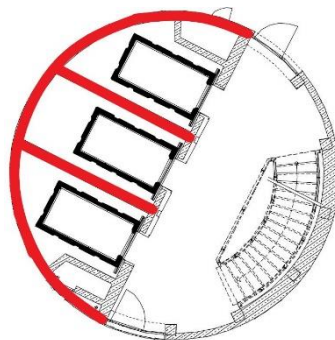


Figure 27: Elevator shear wall

³³ Interpreted from SBC: 304 [4].

³⁴ SBC: 304 [4]

4.5.1 Shear Wall Design Method

The method of designing a shear wall is identical to that of a reinforced concrete column. In order to obtain an approximation for the thickness of a shear wall, we can simply divide the gross area the wall occupies, by the desired length. However, the empirical method shown in the equation below has proven more precise and practical when solving for the gross area of a vertical, reinforced concrete element ³⁵;

$$\phi P_u = \phi r [0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$$

Equation 12: Formula for the Design of an Axially Loaded Reinforced Concrete Element

Where:

- ϕP_u axial load acting on the element (in **kN** or **kips**).
- ϕ safety factor for a rectangular element ≈ 0.65 (**unitless**).
- r safety factor for a rectangular column ≈ 0.80 (**unitless**).
- f'_c compressive strength of the concrete (using **24 mPa**).
- f_y yield strength of the concrete (using **240 mPa**)
- A_g gross area of the concrete element.
- A_{st} area of steel required ³⁶.

4.5.2 Shear Wall Design Example; Staircase structure

The slab of the staircase structure has been simplified as a full rectangle, neglecting the area required to accommodate the stairs. Seeing as how this negation should not negatively affect the design or behavior of the structure's shear walls,

The tributary area in this structure, considered to be acting on each wall segment would be half of the slab's area; 15 m².

Therefore, the design of this shear wall shall be as follows:

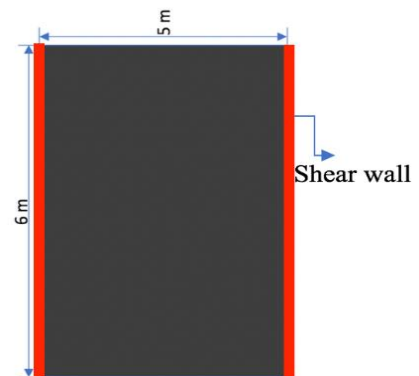


Figure 30: Staircase slab area and shear wall location

³⁵ As per (Ahmed, 2017) [19].

³⁶ The area of steel (A_s) is usually considered to be in the range of 1-6% of the gross area of the element (A_g); ACI: 18.7.4.1 [6].

- Using **Equation 1**, we calculate the factored stress acting on the tributary area ³⁷:

$$w_u = 1.2 \left(8.8327 \frac{kN}{m^2} \right) + 1.6 \left(2 \frac{kN}{m^2} \right) = 13.699 \frac{kN}{m^2}$$

- We convert the stress into a point load, to find the axial force acting on the wall segment (p_u).

$$p_u = 13.699 \frac{kN}{m^2} (15 m^2) = 206.985 kN = 23.363 kips$$

- Seeing as how the above value is representative of only a single floor's load, we must multiply the value by the number of floors in the structure (5 floors), to obtain a cumulative value that satisfies an ultimate loading condition at the lowest floor.

$$\therefore p_u(\text{cumulative}) = 23.363 kips (5) = 116.815 kips$$

- Next, we need to find the gross area of the reinforced concrete element. This can be achieved by solving for the gross area of the concrete element (A_g) in **Equation 12**. We've already found the ultimate (cumulative) load that will act shear element from the calculation in the previous step. All that's left as an unknown variable is the area of steel required to sufficiently reinforce the shear wall (A_{st}). However, considering the area of the steel reinforcement is usually in the range of 1% - 6% of the concrete element's gross area, we shall express (A_{st}) as $(0.04 A_g)$ ³⁸. Resulting in the equation:

$$116.815 kips = 0.65 (0.80) [0.85 (4 kip) (A_g - 0.04 A_g) + (60 kip) (0.04 A_g)]$$

$$\rightarrow \therefore A_g = 49.57 in^2$$

- By finding the gross area required for the shear wall, we can now simply divide it by the length of the segment, to find the thickness required.

$$\text{Length of wall segment} = 6 m = 236.22 in$$

$$\therefore \text{Thickness of wall segment} = \frac{49.57 in^2}{236.22 in} = 0.21 in$$

³⁷ Dead loads and Live loads have been calculated and obtained using the same methods as in previous sections.

³⁸ ACI: 18.7.4.1 [6]

- Finally, considering the thickness found for the shear wall segment (0.21 in) is far less than the minimum thickness of a cast-in place, reinforced structural wall, specified by the designated codes and standards³⁹, we shall revert to using the minimum value of 4 inches.

4.5.3 Summary of Shear Wall Thicknesses:

Table 10: Summary of Shear Wall Thicknesses⁴⁰

<i>Element</i>	<i>Thickness (mm)</i>	<i>Thickness (in)</i>
<i>Stairs</i>	<i>100</i>	<i>4</i>
<i>Elevator</i>	<i>203</i>	<i>8</i>
<i>Ramp</i>	<i>152</i>	<i>6</i>

³⁹ As per (Ahmed, 2017) [19].

⁴⁰ Refer to Appendix 12.2.3 for remaining shear wall thickness calculations.

Chapter 5
Geotechnical Design

5 Geotechnical Design

5.1 Introduction

“It is not the beauty of a building you should look at; it’s the construction of the foundation that will stand the test of time”

David Allan Coe

The reliability and longevity of any structure relies heavily on the quality of its foundation. In technical terms a foundation system would be described as; the interfacing element between the superstructure and the underlying soil or rock ⁴¹. A well-designed foundation is one that can effectively bear the weight of the structure and the imposed stresses of occupation, in a cost-effective and reliable manner. It is imperative that the loads transmitted by the foundation do not cause shear failure of the underlying soil, or damaging settlement of the super-structure. Thus, there are different types of foundations, in accordance with the amount of stress it will bear and the condition of the soil it rests upon.

Foundations are generally divided into two main types:

- **Shallow Foundations:** a type of building foundation that transfers building loads to the earth, very near to the surface. Shallow foundations are predominantly made of reinforced concrete, due to the high compressive strength and flexibility in design the material possesses. Types of Shallow Foundations include:
 - Individual (isolated) footing.
 - Combined footing.
 - Strip foundation.
 - Raft or mat foundation.
- **Deep Foundations:** a type of foundation that transfers building loads to the earth farther down from the surface than a shallow foundation does, to a subsurface layer or a range of depths. Usually, deep foundations refer to the usage of “Piles”, either in tandem with the installation of a shallow foundation, or if the piles can sufficiently bear the load, they simply support a small isolated footing (commonly referred to as a pier cap in this case). A pile can be made out of various materials; Steel, Reinforced Concrete, Prestressed Concrete or Timber.

⁴¹ As per (Ayadat, 2019) [20]

For the purpose of designing a suitable foundation system for this project we shall first investigate the site's soil properties, then ascertain the characteristics and type of the foundation solution.

5.2 Soil Findings

As discussed in **Section 3.3**, a preliminary soil survey has been conducted on the site designated for the project. The findings of the survey indicate two major issues with native soil:

1. A contaminated topsoil layer, unfit to be in contact with the reinforced concrete foundation, and with low bearing capacity, as can be found in **Figure 39**.
2. A large layer of clay that most probably will subdue the structure to differential settlement throughout its design life.

Concerning the contaminated and weak topsoil, we are opting to completely exchange and backfill the existing sand layer (6 meters in depth) with a purified, granular soil as can be seen in **Figure 40**.

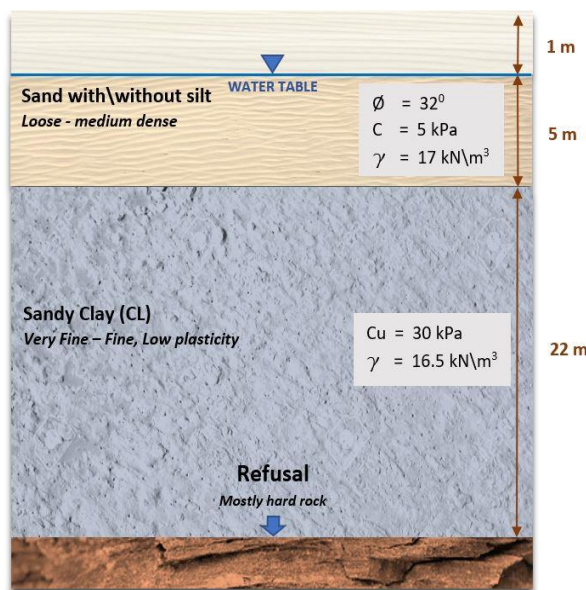


Figure 31: Native Soil Profile

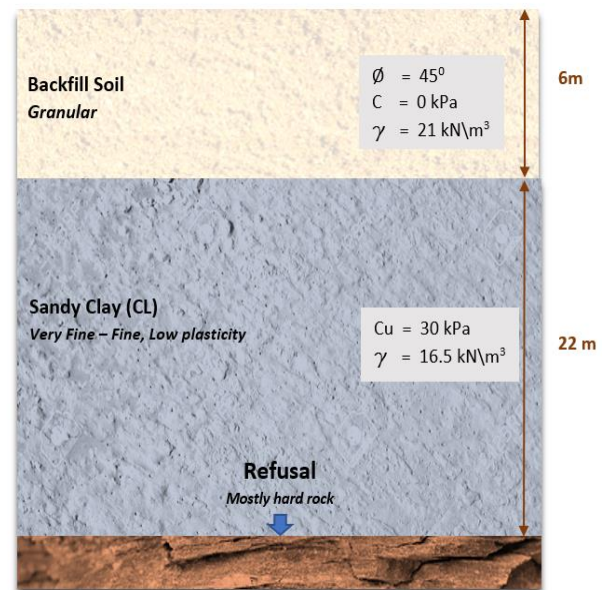


Figure 32: Backfilled Soil Profile

This relatively inexpensive procedure will lend the underlying soil greater bearing capacity, while negating the need for excessive protection for the concrete from Sulphate attacks or Chloride permeation.

As for the deep layer of clay found under the structure's site, our main concern is the occurrence of differential settlement. But, considering a hard layer of rock lies underneath the clay, a suitable solution would be the utilization of a deep foundation system (piles), that

rest on the rock layer, adding to the bearing capacity of the combined system while diminishing any damaging settlement that may occur.

5.3 Dewatering

Dewatering is a process in which we relieve the groundwater from the desired site. This process may be implemented temporarily; in order to facilitate proper concreting of the foundations during construction. Or, as a permanent solution; to alleviate chronic hydrostatic pressure on the foundation system, and eliminate the possibility of seepage or water accumulation from underneath the structure.

5.3.1 Temporary Dewatering

Temporary dewatering can be achieved using multiple methods, such as; sumps, trenches or wellpoints. It is important to understand the water table conditions not only at the site but its surroundings as well. Poorly planned construction dewatering can achieve inadequate levels of water transport, potentially exposing the structure to future instability or endangering the work site conditions. Point wells are generally more encouraged for construction sites with similar conditions to ours, as they have the ability to siphon large volumes of water relatively quickly, and swiftly stabilize the conditions for setting the foundation system.

Since the water table has been recorded at a nominal depth of 1 meter, we recommend the installation of a single-stage well point system to temporarily dewater the site. Wellpoint systems are considered the most reliable and economical choice for dewatering construction sites. Especially where the required lowering of ground water level is approximately 6 m (20 feet) or less [21] [22].

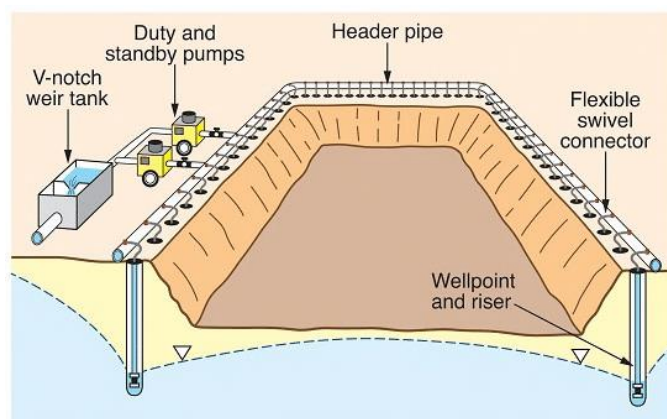


Figure 33: Wellpoint temporary dewatering for a shallow excavation. Retrieved from [23]

5.3.2 Permanent Dewatering

Permanent dewatering systems are rarer, as they are often technically difficult to design and install. They usually involve submersible pumps and permanent drainage systems with complex piping mechanisms. Furthermore, a permanent dewatering solution for our specific project may not be feasible due to the naturally high water table and close proximity to the shoreline. Thus, for this specific project we shall suffice with a temporary dewatering of the construction site and the installation of an impermeable boundary (membrane) around the foundations to inhibit future issues.

5.4 Deep Foundation; Piles

Piles, the most common form of deep foundations are long, slender elements, embedded into the ground to support a structure's foundations. They can be made of concrete, steel, timber, or of a specific type of polymer. They are generally regarded as a reliable method to ⁴²:

- Support structural loads.
- Transfer structural loads to deeper rock, or firm soil layers at sites where soft clay or loose sands exist in shallow depths.
- Absorb tensile and lateral loads.
- Reduce the settlement of a mat foundation.

There are two distinct types of piles; **Friction Piles** and **End Bearing Piles**. A Friction Pile relies on the frictional resistance of the soil working against the shaft of the pile to carry the loads it supports. Whereas an End Bearing Pile primarily carries the load by resting its tip on a hard layer beneath the soft soil. An end bearing pile uses both the frictional force acting against its shaft and the force of the hard layer it rests on to offer a higher bearing capacity. Thus, end bearing piles are favorable when the conditions allow their utilization.

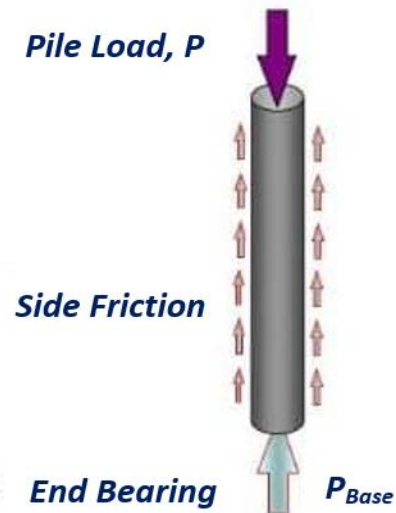


Figure 34: Forces acting on a pile [24]

Furthermore, piles may be considered to act as a single element or as a group. They are classified as one or the other depending on the spacing between each individual pile ⁴³. However, there is an observed reduction in the effective bearing capacity of piles working as a group, due to a loss in each individual pile's efficiency when placed in close proximity to others. Most piles are driven into the ground by means of hammers or vibratory devices (driven piles). Other methods include Auguring (drilling) into the soil and embedding the piles within.

Considering the deep layer of clay our site's soil possesses, an excessive settlement would be inevitable. Thus, we are opting to design **End Bearing Piles** that extend through the clay layer and rest on the layer of hard rock below. They will assist the shallow foundations in bearing the loads of the structure and diminish any settlement that may arise. Also, in continuity with the primary structural material used for this project, we will be designing the deep foundation to be of driven, epoxy coated ⁴⁴, steel pipe-piles as our choice of material and method of installation.

⁴² See reference [20].

⁴³ Piles are considered to act as singular elements when the spacing between them is equal to or greater than 3 times their diameter [20].

⁴⁴ Anti-corrosive, Epoxy-Based coating.

5.4.1 Single Pile Design Method

Equations used:

$$Q_u = Q_p + Q_s$$

Equation 13: General equation for the bearing capacity of a single pile

Where:

- Q_u load carried at the head of a single pile (kN)
- Q_p load carried at the base of the single pile (kN)
- Q_s load carried by the shaft of the pile (skin friction) (kN)

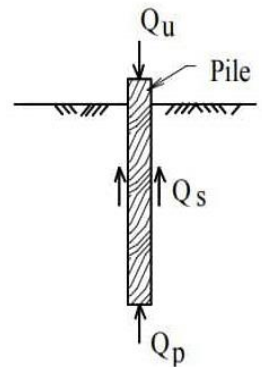


Figure 35: Representation of Forces Acting on a Single Pile [24]

$$Q_{all} = \frac{Q_u}{FS}$$

Equation 14: Allowable load on a single pile

Where:

- Q_{all} allowable load that a single pile can carry (kN)
- FS safety factor (unitless, taken from 2.5 – 4)

$$Q_{p(rock)} = A_p q_p$$

Equation 15: Load carried at the base of a single pile, when resting on a layer of Rock

Where:

- Q_p load carried at the base of the single pile (kN)
- A_p area of pile's base (m²)
- q_p unit point resistance of the underlying rock layer ($\frac{kN}{m^2}$)

$$q_p (\text{submerged rock}) = \frac{0.8 R_c}{FS}$$

Equation 16: Unit Point Resistance of a Submerged Rock Layer

Where:

- q_p unit point resistance of the rock ($\frac{kN}{m^2}$)
- R_c bearing capacity of the rock layer ($\frac{kN}{m^2}$)
- FS factor of safety (**unitless, taken here as 10**)

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

Equation 17: Frictional resistance on a pile's shaft, in sand

Where:

- Q_s frictional resistance in sand (**kN**)
- p is the perimeter of the pile section (**m**)
- ΔL incremental pile length, over which P and f are taken constant (**m**)
- f unit friction resistance at any depth z

$$f = K \sigma'_v \tan \delta$$

Equation 18: Unit friction resistance at depth z

Where:

- K effective earth coefficient (**unitless**)
- δ soil-pile friction angle (**degrees, taken in the range 0.5ϕ to 0.8ϕ**)
- σ'_v effective vertical stress at the depth under consideration ($\frac{kN}{m^2}$)

$$\sigma'_v = \gamma z$$

$$K = 1 - \sin \phi$$

Equation 19: Effective vertical stress

Equation 20: Effective earth coefficient

Where:

- z depth (m, to be taken as $15 D$).

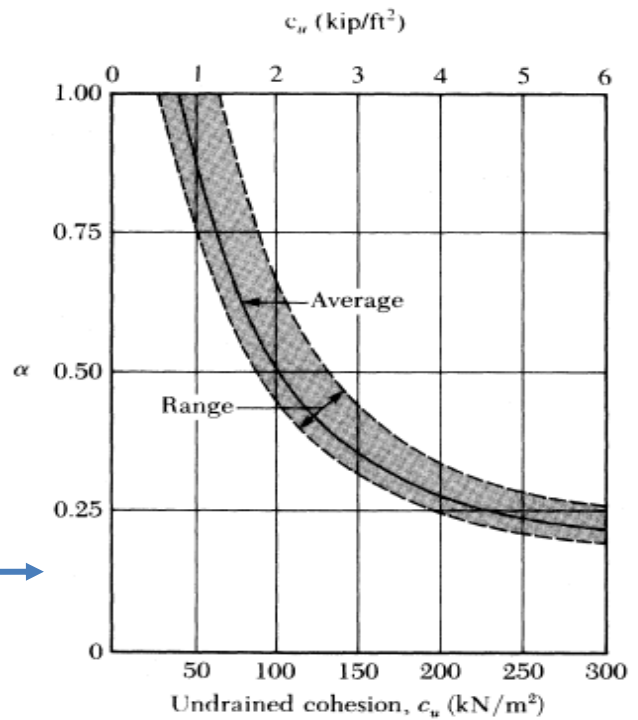
$$Q_s = \sum \alpha C_u p \Delta L$$

Equation 21: Frictional (skin) resistance, in a Clay layer

Where:

- Q_s load carried by the shaft (kN).
- α empirical adhesion factor (unitless)
- C_u undrained cohesion ($\frac{kN}{m^2}$)

The empirical adhesion factor α can be determined using Chart 1



Graph 1: Empirical adhesion factor (α) [20]

5.4.2 Single Pile Design Example; Main Structure

1. Finding the bearing capacity at the base of the single pile Q_p :

In order to calculate the load carried at the base of the pile, the diameter of our steel pipe-pile will be 0.5 m ⁴⁵, and the bearing capacity of the rock shall be taken as 11 MPa ($11,000 \frac{\text{kN}}{\text{m}^2}$)⁴⁶.

$$Q_{p(\text{rock})} = A_p q_p$$

$$A_p = \frac{\pi (0.5 \text{ m})^2}{4} = 0.1963 \text{ m}^2$$

$$q_p = \frac{0.8 \left(11000 \frac{\text{kN}}{\text{m}^2} \right)}{10} = 880 \frac{\text{kN}}{\text{m}^2}$$

$$\therefore Q_p = \frac{(0.1963 \text{ m}^2) 880 \frac{\text{kN}}{\text{m}^2}}{10} = 172.788 \text{ KN}$$

2. Finding the Frictional (Skin) Resistance Q_s :

Since the soil consists of two distinct layers, our frictional resistance shall be the summation of the backfilled sand and the clay layer resistances:

$$Q_s = Q_{s(\text{sand})} + Q_{s(\text{clay})}$$

➤ Backfilled Sand Layer:

$$\sigma'_v = 21 \frac{\text{kN}}{\text{m}^3} (7.5 \text{ m}) = 157.5 \frac{\text{kN}}{\text{m}^2}$$

$$K = 1 - \sin 45^\circ \approx 0.3$$

$$f = 0.3(157.5)(\tan 36^\circ) \approx 34.3 \frac{\text{kN}}{\text{m}^2}$$

Sand Characteristics

- $P = \pi (0.5 \text{ m}) = 1.57 \text{ m}$
- $\Delta L (\text{sand}) = 6 \text{ m}$
- $\phi = 45^\circ$
- $\gamma_{\text{sand}} = 21 \frac{\text{kN}}{\text{m}^3}$
- $z = 15 (0.5 \text{ m}) = 7.5 \text{ m}$
- $\delta = 0.8 (45^\circ) = 36^\circ$

⁴⁵ Refer to Appendix D; 12.4.1, for other pile diameter calculations and results.

⁴⁶ Assumed value, in correlation with values gathered from surrounding sites in the area [10].

$$Q_s = \sum p\Delta Lf$$

$$\therefore Q_{s(sand)} = 1.57 \text{ m} (6\text{m}) \left(36.04 \frac{\text{kN}}{\text{m}^2} \right) \approx \mathbf{323.1 \text{ kN}}$$

➤ **Clay Layer:**

Clay Characteristics

- $\Delta L \text{ (clay)} = 22 \text{ m}$
- $C_u = 30 \frac{\text{kN}}{\text{m}^2}$
- $\alpha = 1$ (using C_u in Graph 1)

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

$$\therefore Q_{s(clay)} = (1) \left(30 \frac{\text{kN}}{\text{m}^2} \right) (22\text{m}) (1.57) = \mathbf{1036.2 \text{ kN}}$$

➤ **Combined Frictional Resistance:**

$$Q_s = 323.1 + 1036.2 \approx \mathbf{1359.3 \text{ kN}}$$

3. Therefore, the maximum allowable load on any single pile, with a diameter of 0.5 m is:

$$Q_u = 172.788 \text{ kN} + 1359.3 \text{ kN} = 1532.1 \text{ kN}$$

$$Q_{all} = \frac{1532.1 \text{ kN}}{4} \approx \mathbf{383 \text{ kN}}$$

5.5 Shallow Foundation; Isolated Footings

Isolated footings are the most common type of foundations used for buildings (the term “footing” and “foundation” are often interchangeable when referring to this type of element). They are also the most economical solution when wishing to install a shallow foundation.

A major drawback of an isolated footing used in combination with piles, is that they no longer function as a shallow foundation. Instead, they merely “cap” the piles and do little more than connect the deep foundation to the structure it supports.

Furthermore, they are designed to only bear the load of a single column. Thus, they often prove insufficient to bear the loads of commercial or industrial structures. However, being the most economical and simplest shallow foundation solution, we are compelled to subject our design to a process of elimination and test to utilization of isolated footings for our application.

In this section, we will be ascertaining if the usage of isolated footings, supported by the piles discussed and calculated in the previous section, can withstand the load of our structure.

5.5.1 Isolated Footing Design Method

For the design of the isolated footings, we will be using the Terzaghi method to calculate the dimensions of a square foundation. This can be achieved by using the obtained critical stress on a column:

Equations Used:

$$q_u = 1.3 c N_c + q N_q + 0.4 B \gamma N_\gamma$$

Equation 22: Terzaghi's Equation for an Isolated, Square Foundation

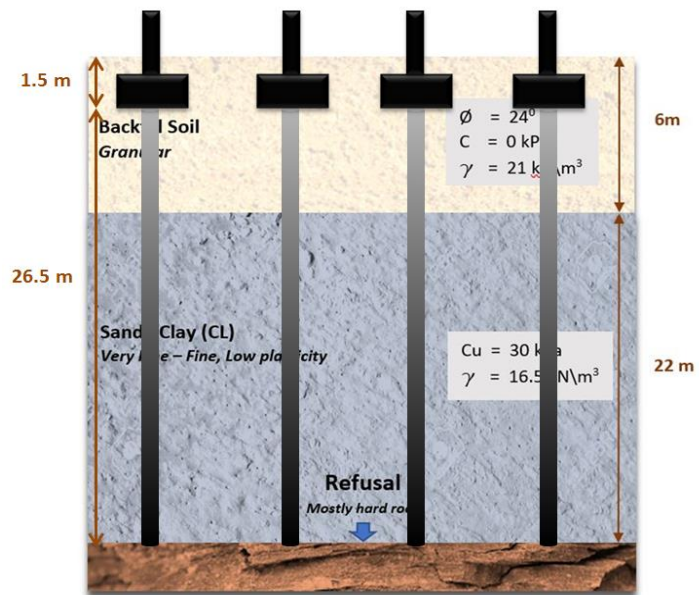


Figure 36: Representation of Isolated Footings on Piles

Where:

- q_u the ultimate bearing capacity of the soil ($\frac{kN}{m^2}$).
- C is the cohesion of the soil ($\frac{kN}{m^2}$).
- γ is the unit weight of the soil ($\frac{kN}{m^3}$).
- q soil stress acting on footing on ($\frac{kN}{m^2}$).
- ➔ $q = \gamma D_f$ where D_f is the embedment depth (m).
- B is the base of the foundation (m).
- $N_c, N_q, \text{ and } N_\gamma$ Terzaghi's bearing capacity factors (unitless).

Terzaghi's Bearing Capacity Factors can be determined by locating the values that correlate to the soil's characteristics, as found Table 11:

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)

Table 11: Terzaghi's Bearing Capacity Factors

5.5.2 Isolated Footing Design Example; Main Structure

In order to ascertain the dimensions (B) of the isolated, square footing we wish to design, we need the following information:

- The load acting on the footing: as attained from the analysis of our model, the load acting on a critical column of the main structure is **2869 kN**⁴⁷.
- The Characteristics of the base soil: →
- The embedment depth of the foundation: assumed here to be $D_f = 1.5m$.
- Terzaghi's Bearing Capacity Factors:
 $N_c = 172$, $N_q = 173.28$, and $N_\gamma = 325.35$

Sand Characteristics

- $\Delta L (\text{sand}) = 6 m$
- $\phi = 45^\circ$
- $\gamma_{\text{sand}} = 21 \frac{kN}{m^3}$
- $c = 0$

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)

Figure 37: Method of determining Meyerhof's Bearing Capacity Factors

⁴⁷ Refer to **Appendix C** for full details of loads found from analysis.

I. Solving for the footing's width (B):

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

$$q_u = 1.3 (0)(172) + \left(31.5 \frac{kN}{m^2}\right) (173.28) + 0.4 B \left(21 \frac{kN}{m^3}\right) (235.35)$$

$$\rightarrow q_u = 5458.32 + 2732.94 B$$

Considering $q_{all} = \frac{q_u}{FS} = \frac{q_u}{3}$: FS for a square footing = 3

$$\rightarrow q_{all} = \frac{5458.32 + 2732.94 B}{3} = 1819.44 + 910.98 B$$

$$\rightarrow \text{Also, from the critical column load; } q_u = \frac{Q_{all}}{A} = \frac{2869 kN}{B^2}$$

$$\rightarrow \frac{2869 kN}{B^2} = 1819.44 + 910.98 B$$

$$B = 1.02 m$$

Considering the calculated width is impractical for physical applications

$$\therefore B \approx 1.5 m$$

II. Determining the usability of an isolated footing as a shallow foundation system:

Since an isolated footing resting on piles will simply act as a capping for the deep foundation system, the actual bearing capacity or ultimate load that can be carried by the footing is equal to that of each individual pile beneath it. The minimum allowable spacing between single piles in order to classify them as such, is **3 x the diameter of the pile**. Otherwise, piles that do not satisfy this rule have been observed to act as a unit (referred to as group piles). If we were to utilize the footing's dimensions to its fullest extent, placing **4 piles** underneath (one on each corner), the spacing between each pile would be less than the above rule, resulting in a decreased bearing capacity for the group ⁴⁸.

⁴⁸ The stress transmitted to the soil by a Group of piles is always assumed to overlap, resulting in a decreased bearing capacity when working as a unit [20].

Each individual pile can carry a maximum allowable load of **383 kN**. Thus, a summation of the individual capacities of each pile (disregarding the decrease in efficiency related with to group effect) would result in a combined bearing capacity of **1532 kN** per footing. Compared with the value of a critical column's load (**2869 kN**), it is apparent that the footing designed above will not sufficiently bear the load of the structure.

Thus, we shall address this by designing a Raft foundation system in the following section.

5.6 Shallow Foundation; Raft (Mat)

Raft foundations (sometimes referred to as raft footings or mat foundations) are formed by reinforced concrete slabs of uniform thickness that cover a wide area, often the entire footprint of a building. They spread the load imposed by a number of columns or walls over the area of foundation, and can be considered to “float” on the ground as a raft floats on water [25]. Our system is categorized as a “Piled Raft” foundation, as we have implemented piles in our design, making use of the incompressible (rock) strata beneath the clay layer to support the structure. Thus, effectively negating any settlement that may occur from the installation of a raft foundation alone. Settlement issues aside, we can now solely focus on designing the raft's slab, with the necessary dimensions and attributes for our application.

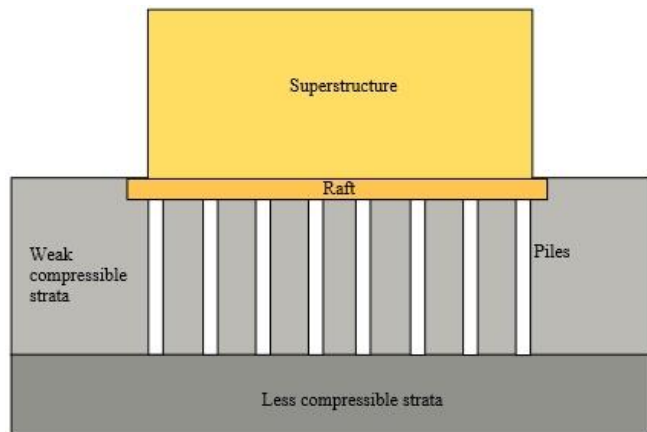


Figure 38: Representation of a Piled Raft Foundation System [25]

5.6.1 Raft Foundation Design Method

The design of a raft foundation usually adheres to the following steps:

1. Calculating the **depth** of the raft foundation; considering the width of the raft is identical to the footprint of the building, the depth of the raft is the only dimension required to solve for.
2. Calculating the **soil pressure** underneath the critical columns; this step is integral in assessing the soil's bearing capacity afterwards.

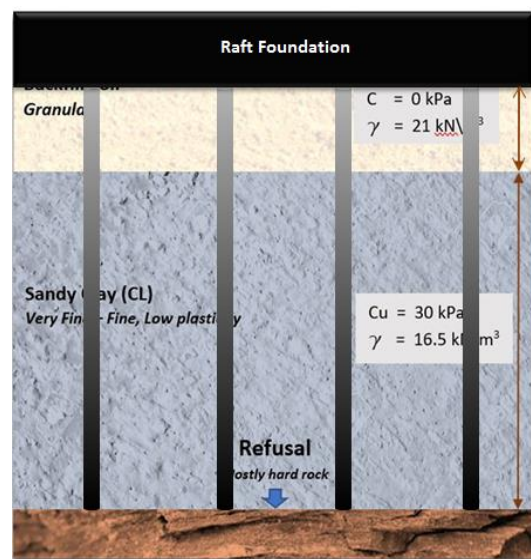


Figure 39: Representation of Our Project's Piled Raft Foundation System

Equations Used:

$$U = b_o d \left[\phi (0.34) \sqrt{f'_c} \right]$$

Equation 23: Critical Column Load (Used for Raft Depth Calculation) [20].

Where:

- U critical column load (kN)
- ϕ reduction factor (unitless, taken as 0.85)
- f'_c compressive strength of concrete at 28 days ($\frac{MN}{m^2}$)
- d thickness of the mat foundation (m)
- b_o modified width of a square column (m)

b_o depends on the column's location, as is clarified in **Figure 48**.

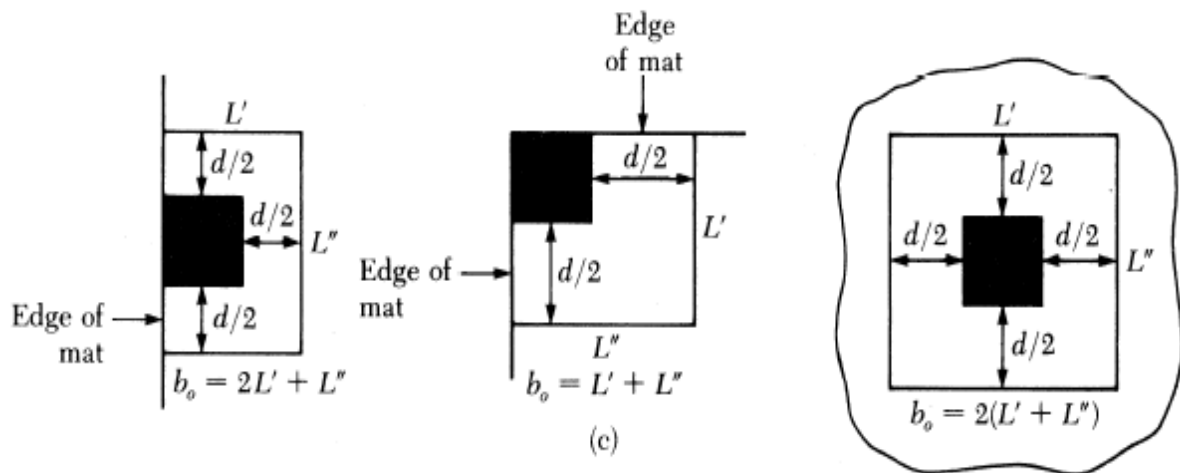


Figure 40: Location-Dependent Column Width Modifications b_o [20]

$$q_{(x,y)} = \frac{Q_{total}}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

Equation 24: Soil Point-Pressure Under a Specific Column

Where:

- $q_{(x,y)}$ soil point-pressure, under chosen column ($\frac{kN}{m^2}$)
- Q_{total} total combined load of all structural columns (**kN**)
- A surface area of the structure (mat/raft) (m^2)
- M_y moment of force about y-axis ; $M_y = Q_{total} (e_x) = (kN.m)$
- M_x moment of force about x-axis ; $M_x = Q_{total} (e_y) = (kN.m)$
- I_y moment of inertia about y-axis (m^4)
- I_x moment of inertia about x-axis (m^4)

$$e_x = x' - \frac{b}{2}$$

&

$$e_y = y' - \frac{h}{2}$$

Equation 25: Eccentricity Distances on the X and Y Axes.

Where:

- e_x, e_y eccentricity on each axis (**m**)
- b base of Raft from an overhead perspective (**m**)
- h height of Raft from an overhead perspective (**m**)

$$x' = \frac{\sum_{i=0}^{n-1} Q_i(x'_i)}{\sum Q}$$

&

$$y' = \frac{\sum_{i=0}^{n-1} Q_i(y'_i)}{\sum Q}$$

Equation 26: Equations for Calculating Distances from a Coordinate System

Where:

- x', y' distance from the coordinate system (**m**)
- Q column point load (**kN**)

5.6.2 Raft Foundation Design Example; Main Structure

1. Determining the Depth of the Raft (Thickness):

- b_o calculation; Using the dimensions of the critical column section (**W14 x 283**), we attain a width of **14 inches (0.4 m)**. Further, considering the critical column load will be acting in the middle of mat, the method for calculating the modified column width will be:

Given Data

- $U \approx 2.9 \text{ MN}$
- $\phi = 0.85$
- $f'_c = 25 \frac{\text{MN}}{\text{m}^2}$

$$b_o = 2 (0.4 + 0.4) = 1.6 \text{ m}$$

- Afterwards, we can re-write **Equation 23** to solve for the desired depth (**d**), as follows:

$$d = \frac{U}{b_o \left[\phi (0.34) \sqrt{f'_c} \right]} \quad \longrightarrow \quad d = \frac{2.9 \text{ MN}}{(1.6 \text{ m}) \left[(0.85) (0.34) \sqrt{25 \frac{\text{MN}}{\text{m}^2}} \right]}$$

$$\therefore \text{The Thickness of the Raft (d)} = 1.25 \text{ m} \approx 1.5 \text{ m}$$

2. Finding the soil pressure under a critical column:

- Since the shape of our mat is circular; **Diameter = B = L = 94 m**.
Hence;

$$A = \frac{\pi}{4} (94 \text{ m})^2 = 6989.78 \text{ m}^2$$

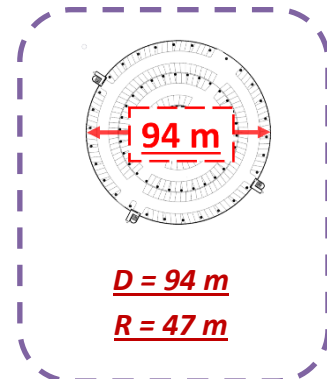
$$I_x = \frac{\pi (47 \text{ m})^4}{4} = 3,832,492.495 \text{ m}^4$$

$$I_y = \frac{\pi (47 \text{ m})^4}{4} = 3,832,492.495 \text{ m}^4$$

$$x' = 47.767 \text{ m} \text{ (Refer to App. D)}$$

$$y' = 47.505 \text{ m} \text{ (Refer to App. D)}$$

$$Q_{total} = 156,758 \text{ kN} \text{ (Refer to App. D)}$$



$$e_x = 47.767 \text{ m} - \frac{94}{2} = 0.767 \text{ m}$$

$$e_y = 47.505 \text{ m} - \frac{94 \text{ m}}{2} = 0.505 \text{ m}$$

$$M_y = 156,758 \text{ KN}(0.767 \text{ m}) = 120,233.386 \text{ kN.m}$$

$$M_x = 156,758 \text{ KN}(0.505 \text{ m}) = 79,162.790 \text{ kN.m}$$

- Therefore, our equation for the soil point-pressure under a critical column will be;

$$q_{(x,y)} = \frac{156,758 \text{ kN}}{6989.78 \text{ m}^2} \pm \frac{120,233.386 \text{ x}}{3,832,492.495 \text{ m}^4} \pm \frac{79,162.790 \text{ y}}{3,832,492.495 \text{ m}^4}$$

$$q_{(x,y)} = 22.34 + 0.0313 \text{ x} + 0.0207 \text{ y}$$

- Finally, by entering the critical column's coordinates, we can find the maximum point-pressure acting on the soil at that location;

$$X = 28.35 \text{ m} \quad ; \quad Y = 6.03 \text{ m}$$

$$q_{(28.35,6.03)} = 22.34 + 0.0313 (28.35) + 0.0207 (6.03) = 23.602 \frac{\text{kN}}{\text{m}^2}$$

5.7 Site Soil Bearing Capacity

In order to finalize the design of our foundation systems, we must compare the value of the point-pressure with the maximum stress the soil can bear.

We shall use Meyerhof's Bearing Capacity Theorem, as it is the most wide-spread and accurate and encompassing way of finding the ultimate bearing capacity of a soil.

5.7.1 Bearing Capacity Calculation

Equations used:

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

Equation 27: Meyerhof's Formula for Soil Bearing Capacity

Where:

- q_u bearing capacity of the soil ($\frac{kN}{m^2}$)
- γ unit weight of soil ($\frac{kN}{m^3}$)
- c cohesion of soil ($\frac{kN}{m^2}$)
- q effective stress at the bottom of foundation = $\gamma D_f = (\frac{kN}{m^2})$
- B width or diameter of the foundation (m)
- $F_{cs}, F_{qs},$ and $F_{\gamma s}$ shape factors (**unitless**)
- $F_{cd}, F_{qd},$ and $F_{\gamma d}$ depth factors (**unitless**)
- $F_{ci}, F_{qi},$ and $F_{\gamma i}$ inclination factors (**unitless**)
- $N_c, N_q,$ and N_γ Meyerhof Bearing Capacity Factors (**unitless, see Table 12**)

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

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \tan \phi$$

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

$$F_{ci} = \left(1 - \frac{\beta}{90}\right)^2$$

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

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

Equation 28: Shape Factor Formulas

Equation 29: Inclination Factor Formulas

Where:

- β : inclination of load acting on the foundation with respect to the vertical (**degrees**)
- ϕ : angle of shearing resistance (**degrees**)

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

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

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

Equation 30: Depth Factor Formulas

Table 3.3 Bearing Capacity Factors

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

Table 12: Meyerhof's Bearing Capacity Factors

Calculating Soil Bearing Capacity:

- Considering the structure is circular; $B = L = D = 94 \text{ m}$. Moreover, a slight eccentricity exists in the effective point of pressure underneath the column ⁴⁹, resulting in a need to modify B ;

$$B' = B - 2e_x = 94 - 2(0.767 \text{ m}) = \mathbf{92.466 \text{ m}}$$

L' will remain the same as **94 m**

Soil and Foundation Data

- $\gamma = 21 \frac{\text{kN}}{\text{m}^3}$
- $\phi = 45^\circ$
- $C = 0$
- $D_f = 1.5 \text{ m}$
- $N_q = 134.87$
- $N_\gamma = 271.75$

- Formulating Meyerhof's Equation for Bearing Capacity:

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

$$F_{qs} = 1 + \left(\frac{92.466 \text{ m}}{94 \text{ m}} \right) \tan 45 = 1.98$$

$$F_{qd} = 1 + 2 \tan 45 (1 - \sin 45)^2 \frac{1.5 \text{ m}}{92.466 \text{ m}} = 1$$

$$F_{qi} = \left(1 - \frac{\beta}{90} \right)^2 = 1 \text{ (since there is no inclination } \beta = 0^\circ \text{)}$$

$$F_{\gamma s} = 1 - 0.4 \left(\frac{92.466 \text{ m}}{94 \text{ m}} \right) = 0.61$$

$$F_{\gamma d} = 1, \quad F_{\gamma i} = \left(1 - \frac{\beta}{\phi} \right)^2 = 1 \text{ (since there is no inclination } \beta = 0^\circ \text{)}$$

$$q_u = 0 + 31.5 \frac{\text{kN}}{\text{m}^2} (134.87)(1.98) + \frac{1}{2} (92.466 \text{ m}) \left(21 \frac{\text{kN}}{\text{m}^3} \right) (271.75)(0.61)$$

$$q_u = 169354.37 \frac{\text{kN}}{\text{m}^2} \longrightarrow q_{all} = \frac{q_u}{FS} = \frac{169354.37 \frac{\text{kN}}{\text{m}^2}}{3} = \mathbf{56,451.45 \frac{\text{kN}}{\text{m}^2}}$$

\therefore Since the soil bearing capacity is larger than the soil point-pressure under each column, our chosen depth of 1.5 m for the raft foundation is considered safe and suitable for its intended use.

⁴⁹ From prior calculations and observations of effective point-pressure locations. Refer to **Appendix D: 12.4.2** for details.

5.8 Conclusion and Recommendations

As mentioned in the body of this chapter, the soil of the site we wish to construct our multi-story parking on is currently unsuitable for such an application.

A complete exchange of the topsoil with a granular fill of higher bearing capacity is our first recommendation. Followed by the installation of a deep foundation system in the form of piles, which will help stabilize the structure against the expected consolidation settlement that will result from the deep clay layer. We also recommend that the distribution of the piles match the locations of the main structure's columns.

For the shallow foundation system, the possibility of using isolated footings under each column has been investigated, producing unsatisfactory results and failing to bear the loads that would be imposed. Alternatively, a structure-wide, raft (mat) foundation, resting on the aforementioned piles has been found to sufficiently bear the loads and offer the structural rigidity required.

Thus, a **Piled-Raft** system is our recommended solution for this project's foundations.

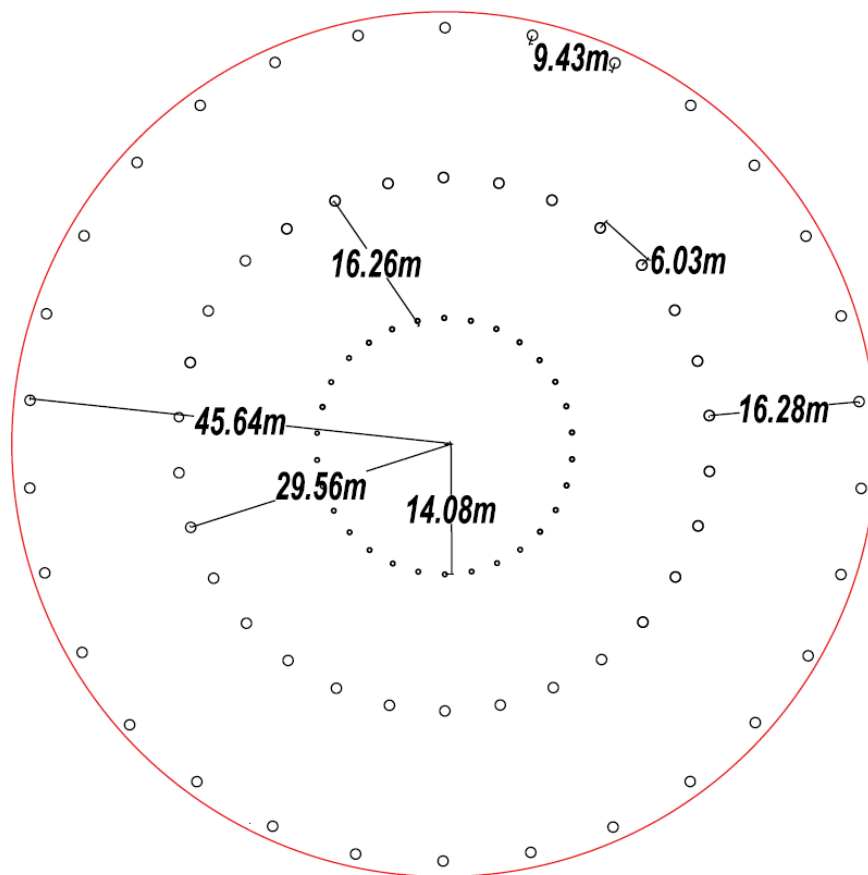


Figure 41: Recommended Locations of Piles Under Main Structure

Chapter 6

Reinforcement

6 Reinforcement

6.1 Introduction

Since it was first introduced in the late 1880s, Reinforced Cement Concrete (RC or RCC) has proved to be one of the most influential, effective and versatile construction materials known to humankind.

Ordinary cement concrete on its own, can only withstand in tension a fraction of the forces it could when in compression. Thus, the addition of steel reinforcement to concrete, is mainly to counteract tensile stresses and offer ductility to the structural element. Although other materials have been tried and tested as reinforcement for concrete, none have proven as reliable or predictable as steel reinforcement bars (rebars).

The type and designation of steel reinforcement bars (rebars) to be utilized in our design is:

Table 13: Type and Designation of Steel Rebars Used for this Project's Reinforcement

<i>ASTM Type</i> ⁵⁰	A615/A615M
<i>Grade</i>	$f_y = 60 \text{ ksi} / 420 \text{ MPa}$

The type and grade of rebar will stay nominal throughout the project. However, the diameter and number of bars used will differ from one element's reinforcement to another.

For the purpose of designing the concrete elements of our structure, we will be calculating the reinforcement required for each of the following;

- The Slabs
- The Shear Walls
- The Raft (Mat) Foundation

The method of calculating the reinforcement for the slabs and shear walls are very similar to each other and follow the same procedure. However, the Raft foundation's calculation slightly differs, especially in terms of calculating the maximum moment acting on the Raft.

⁵⁰ American Society for Testing and Materials; Designations of Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement [26].

6.2 Slab and Shear Wall Reinforcement

The project comprises of multiple types of slabs and shear walls, as can be found in the figure below (**Figure 50**):

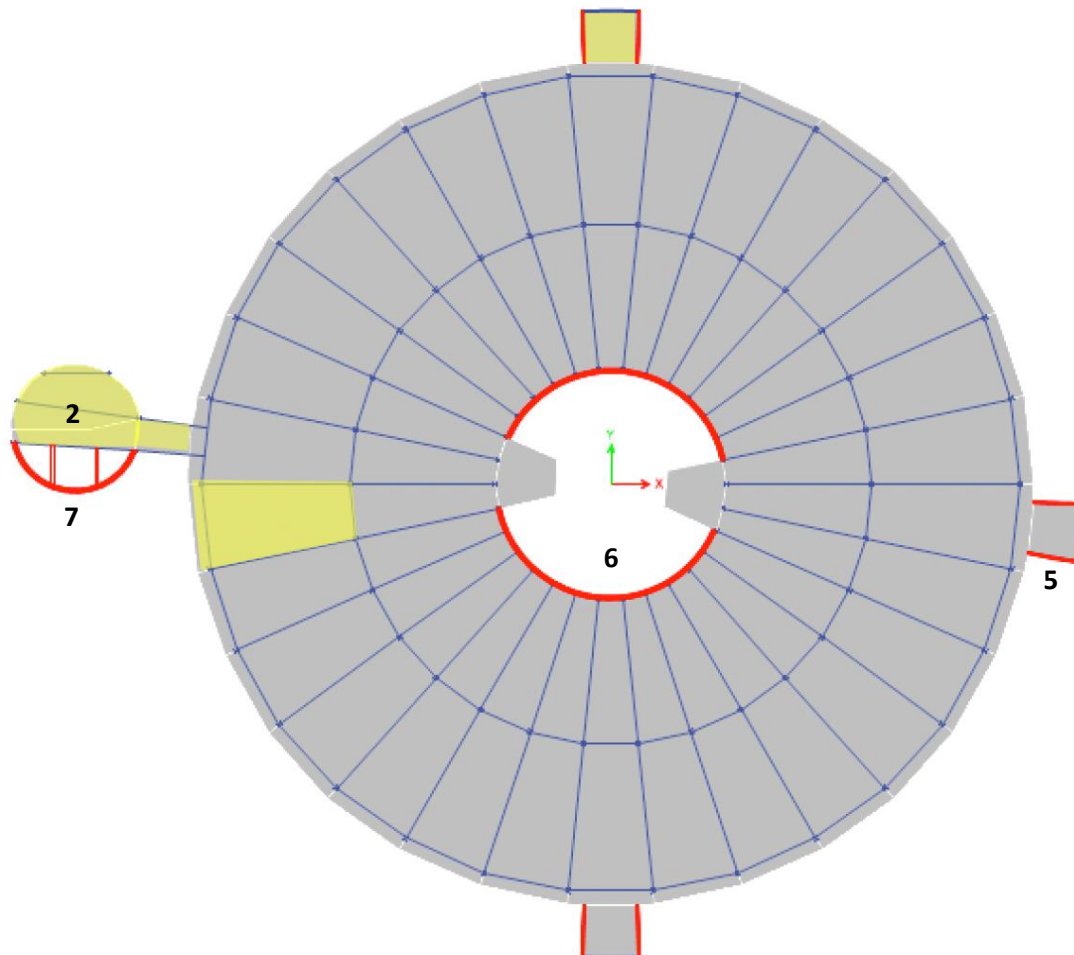


Figure 42: Locations of Element Considered for Reinforcement Design

As can be found in the figure above, there are 3 types of slab segments, and 3 types of shear walls:

- 1) Main Structure Slab
- 2) Elevator Structure Slab
- 3) Pedestrian Bridge Slab
- 4) Stair Structure Slab
- 5) Stair Structure Shear Wall
- 6) Ramp Shear Wall
- 7) Elevator Structure Shear Wall

6.2.1 Slab and Shear Wall Reinforcement Design Method

All the reinforcement will be designed on the basis of (1 m x 1 m) segments. In order to calculate the reinforcement for each element, we shall use the maximum moment values retrieved from the ETABS analysis of our model (indicated in Red), acting on each element:

Table 14: Load Cases Acting on Structural Elements and Moment Analysis from Modelling (kN.m)

Load Case / Location	DL	Combo	DL + W _x	DL + W _y	DL + E _x	DL + E _y
Main Structure	49.344	50.811	18.856	35.078	42.332	40.672
Stair Structure	8.577	7.619	11.34	6.956		
Pedestrian Bridge	12.304	21.371	11.69	11.79		
Elevator Slab	35.281	44.357	21.446	25.749		

Where:

- **DL** Self-weight only
- **Combo** Self-weight + Live Load
- **W_x, W_y** Wind acting on X – axis , Y – axis
- **E_x, E_y** Earthquake acting on X – axis , Y – axis

Equations Used ⁵¹:

$$\bar{k} = \frac{M_{max}}{\phi b d^2}$$

Equation 31: Factored Reinforcement Stress

⁵¹ Based on interpretations and provided equations from [19].

Where:

- \bar{k} Factored Reinforcement Stress ($\frac{kN}{m^2}$)
- M_{max} Maximum Moment acting on chosen point (kN.m)
- ϕ Safety Factor (**unitless, taken here as 0.9**)
- b Width of the element (m)
- d Distance from top of the element to centroid of steel (cross-section) (m)

$$\rho = \frac{0.85 f_c'}{f_y} \left[1 - \sqrt{1 - \frac{2 \bar{k}}{0.85 f_c'}} \right]$$

Equation 32: Required Ratio of Steel Reinforcement to Concrete

$$\rho_{min} = \frac{200}{f_y}$$

Equation 33: Minimum Ratio of Steel Reinforcement to Concrete

$$\rho_{max} = 0.75 \left[\frac{(0.85)(0.65)f_c'}{f_y} \right] \left[\frac{600}{600 + f_y} \right]$$

Equation 34: Maximum Ratio of Steel Reinforcement to Concrete

Where:

- ρ Steel reinforcement ratio (**unitless**)
- f_c' Compressive strength of concrete at 28 days ($\frac{MN}{m^2}$)
- f_y Steel yield strength (MPa)

$$A_s = \rho b d$$

Equation 35: Area of Steel Reinforcement ⁵²

Where:

- A_s Area of steel reinforcement (m^2 or mm^2)

$$S_{max} = A_{one\ bar} \frac{b}{A_s\ (required)}$$

Equation 36: Maximum Clear Spacing Between Individual Rebars

Where:

- S_{max} Maximum clear spacing between individual rebars (mm)

Note: Maximum clear spacing is defined in the SBC ⁵³.

6.2.2 Slab and Shear Wall Reinforcement Design Examples

Main Structure Slab Segment:

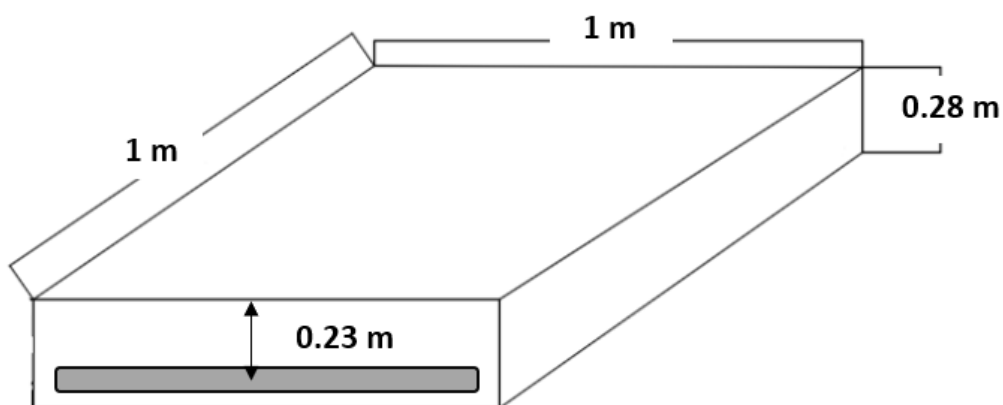


Figure 43: Dimensions of Main Structure Slab Segment

⁵² After finding the required area of steel reinforcement for the element, refer to **Appendix E** to choose a diameter and number of bars.

⁵³ SBC: 304 [4] states the maximum clear spacing is **2 x thickness of the slab**. Also stated, the minimum cover from rebars to surface of element is to no less than **50 mm**.

$$\bar{k} = \frac{50.8 \text{ kN.m}}{(0.9)(1000 \text{ mm})(230 \text{ mm})^2} \times \frac{1000 \text{ N/kN}}{1000 \text{ mm/m}}$$

$$\rightarrow \bar{k} = 1.067 \times 10^{-4} \left(\frac{\text{N}}{\text{mm}^2} \right) = 1.067 \left(\frac{\text{MN}}{\text{m}^2} \right)$$

Obtained / Given Data

- $M_{max} \approx 50.8 \text{ kN.m}$
- $f'_c = 28 \text{ MPa}$
- $f_y = 420 \text{ MPa}$
- $b = 1 \text{ m}$
- $d = 0.23 \text{ m}$

$$\rho = \frac{0.85(28)}{420} \left[1 - \sqrt{1 - \frac{2(1.067)}{0.85(28)}} \right] = 0.0026$$

$$\rho_{min} = \frac{3\sqrt{28,000}}{420,000} = 0.000476$$

$$\rho_{max} = 0.75 \left[\frac{(0.85)(0.65)28}{420} \right] \left[\frac{600}{600+420} \right] = 0.0163$$

$$A_s = (0.0026)(1000)(0.23) = 598 \text{ mm}^2$$

$$A_{s \text{ min}} = (0.000476)(1000)(0.23) = 109.52 \text{ mm}^2$$

$$A_{s \text{ max}} = (0.0163)(1000)(0.23) = 3737.5 \text{ mm}^2$$

From the obtained values of the required, minimum and maximum areas of steel reinforcement, we can know compare the findings with the available number of bars and area configurations, in **Table B5**⁵⁴:

From the available configurations: 10 #10 bars will be sufficient to reinforce the main structure's slab.

$$S_{max} = \pi \left(\frac{10 \text{ mm}}{2} \right)^2 \frac{1000 \text{ mm}}{598 \text{ mm}^2} = 131.302 \text{ mm}$$

$$S_{min} = 25 \text{ mm} ; \text{ Minimum spacing obtained from }^{55}$$

Based on the calculated values above and considering we need a cover of at least 50 mm from the rebars to the surface of the concrete element; we recommend $S = 50 \text{ mm}$

⁵⁴ Refer to **Appendix E**.

⁵⁵ **SBC; 304: 7.6.1 [4]**

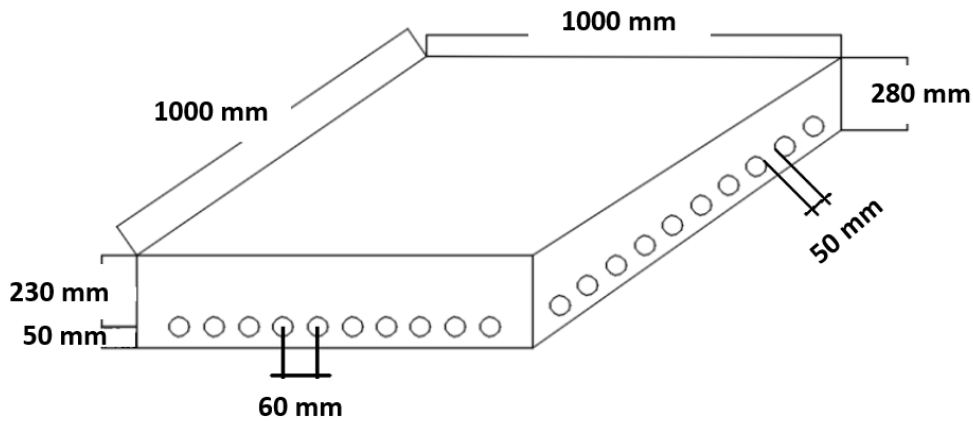


Figure 44: Representation of Slab Design Results

Ramp Structure Shear Wall Segment:

Similar to the calculation of reinforcement required for a slab, the calculation of a shear wall's reinforcement differs in the need to design a two meshes of rebars, top and bottom of the wall. Using the data obtained from the ETABS model:

Table 15: ETABS Generated Analyses of Shear Wall Reinforcement Requirements

Structure	Location	A_s (mm ² /m) required	No. Bar	Diameter (mm)	A_s (mm ²) provided	Area of one bar (mm ²)
Ramp Structure	Top	13950	10	43	14742	1452.201
	Bottom	13950	10	43	14742	1452.201

$$S_{max} = 1452.201 \text{ mm}^2 \frac{1000 \text{ mm}}{13950 \text{ mm}^2} = 104 \text{ mm}$$

Therefore, we recommend a clear spacing of $S = 50 \text{ mm}$

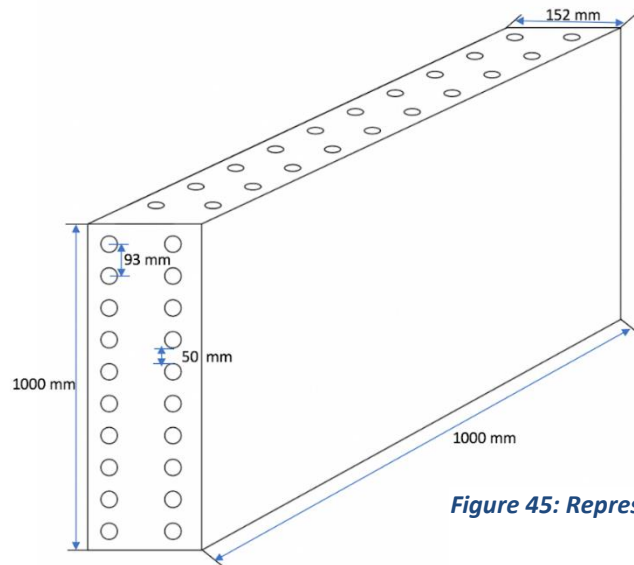


Figure 45: Representation of Ramp Shear Wall With Reinforcement

6.3 Raft Foundation (Mat) Reinforcement Design Method

In order to find the required amount of reinforcement for a raft's slab, we will use the conventional "Rigid Slab" technique ⁵⁶:

- A. Divide the Raft into "strips", spanning the length and width of the slab, from the edge of the first column on each direction (X and Y), to the last.
- B. Calculate or find the maximum positive and negative moments acting on each strip, to determine the critical ones on the X and Y axes.
- C. Treat the critical strips found as simply supported beams, with respect to all the acting forces on them (pile reactions, soil pressure and structural loads) and solve for the area of steel reinforcement (A_s) accordingly.

Equations Used:

$$q_{av} = \frac{q_{first} + q_{last}}{2}$$

Equation 37: Average Soil pressure Across a Raft Slab Strip

Where:

- q_{av} average soil point pressure across strip $\left(\frac{kN}{m^2}\right)$
- q_{first} point pressure at the first (top) edge of the strip $\left(\frac{kN}{m^2}\right)$
- q_{last} point pressure at the last (bottom) edge of the strip $\left(\frac{kN}{m^2}\right)$

⁵⁶ As interpreted from [20].

$$\mathbf{Soil\ Reaction = } q_{av} L B = kN$$

Equation 38: Soil Reaction Under a Raft Slab Strip

Where:

- q_{av} average soil point pressure $\left(\frac{kN}{m^2}\right)$
- L length of the strips (m)
- B base of the strips (m)

$$\mathbf{Load}_{avg} = \frac{\mathbf{soil\ reaction} + \Sigma Q}{2}$$

Equation 39: Average Load Acting on a Raft Slab Strip

Where:

- Q summation of loads acting on a strip (kN)

$$\mathbf{q}_{av\ (modi)} = \frac{\mathbf{Load}_{avg}}{L B}$$

Equation 40: Modified Average Soil Pressure Under A Raft Slab Strip

Where:

- $q_{av\ (modi)}$ modified average soil pressure $\left(\frac{kN}{m^2}\right)$
- L length of the strips (m)
- B base of the strips (m)

$$F = \frac{\sum Q}{\text{Load}_{avg}}$$

Equation 41: Modifying Load Factor

Where:

- F load factor (**unitless**)

$$q' = q_{av(modi)} B$$

Equation 42: Modified Soil Reaction per Unit Length for a Raft Slab Strip

Where:

- q' soil reaction per unit length ($\frac{kN}{m}$)

$$M' = \frac{\text{Moment max}}{B}$$

Equation 43: Moment per Unit Length Acting on a Strip

Where:

- M' moment per unit length ($\frac{kN.m}{m}$)
- **Moment max** maximum positive or negative moment acting on the strip (**kN.m**)
- B width of the strip (**m**)

$$M_u = M'(F)$$

Figure 46: Factored Moment Acting on a Raft Slab Strip

Where:

- M_u factored moment (**kN.m**)
- F Load factor (**unitless, taken here as 1.7**)

$$M_u = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

Equation 44: Alternative Equation for Factored Moment Calculation (to find A_s)

Where:

- A_s area of steel per unit width (mm^2)
- f_y yield stress of steel reinforcement (MPa)
- ϕ reduction factor (unitless, taken here as 0.9)

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

Equation 45: Ratio of Steel to Concrete

6.3.1 Raft Reinforcement Design Example

When first attempting preliminary analysis of the raft's slab, it was found that the critical strip lies on the X-axis (as shown in **Figure 49**). Seeing as how our raft slab is circular in shape, the obtained critical strip can be applied nominally in every direction. Thus, is representative of the entire raft's reinforcement design.

Table 16: Critical Strip Information

Critical point loads (kN)				
No.	Inner	Middle	Outer	Direction
9	358	2897	2150	X - axis
23	335	2868	2309	
Soil point pressures kN/m^2				
9	23.03	23.6	24.1	X - axis
23	22.18	21.67	21.16	

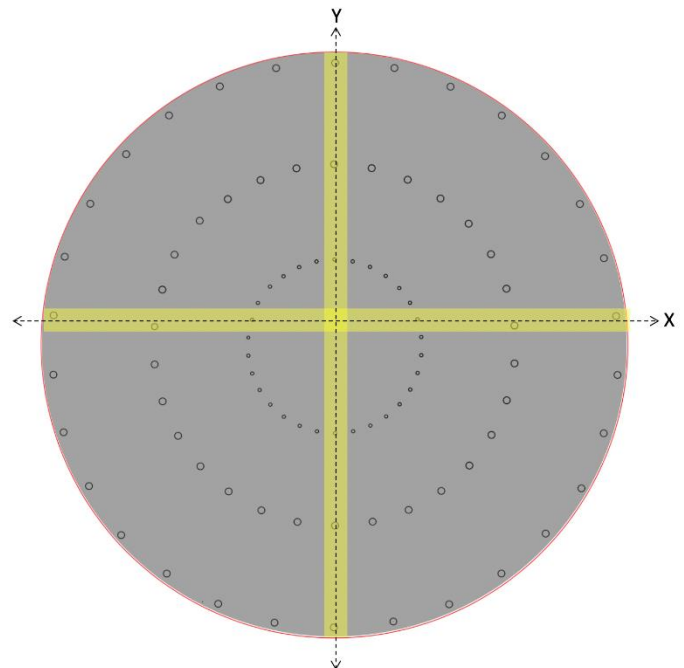


Figure 47: Location of Critical Strips on X and Y axes.

$$q_{av} = \frac{24.1 \frac{kN}{m^2} + 21.16 \frac{kN}{m^2}}{2} = 22.63 \frac{kN}{m^2}$$

soil reaction = $22.63 \frac{kN}{m^2} (94 m)(9.6m)$
 soil reaction = 20,421.312 kN

- Obtained / Given Data**
- $\sum Q = 10,917 kN$
 - $f'_c = 28 MPa$
 - $f_y = 420 MPa$
 - $B = 9.6 m$
 - $L = 94 m$

$$Load_{avg} = \frac{20,421.312 kN + 10,917 kN}{2} = 15,669.156 kN$$

$$q_{av(modi)} = \frac{15,669.156 kN}{94 m (9.6m)} = 17.346 \frac{kN}{m^2}$$

$$q' = 17.346 \frac{kN}{m^2} (9.6 m) = 166.639 \frac{kN}{m^2}$$

Afterwards, we calculate the load factor to modify the actual load acting on the raft;

$$F = \frac{10,917 kN}{15,669.156 kN} = 1.435$$

Table 17: Modified Critical Strip Point Loads

<i>Modified Critical Strip Point Loads (kN)</i>				
<i>NO.</i>	<i>Inner</i>	<i>Middle</i>	<i>Outer</i>	<i>direction</i>
9	513.387	4158.060	3085.892	<i>X direction</i>
23	480.825	4116.437	3314.105	

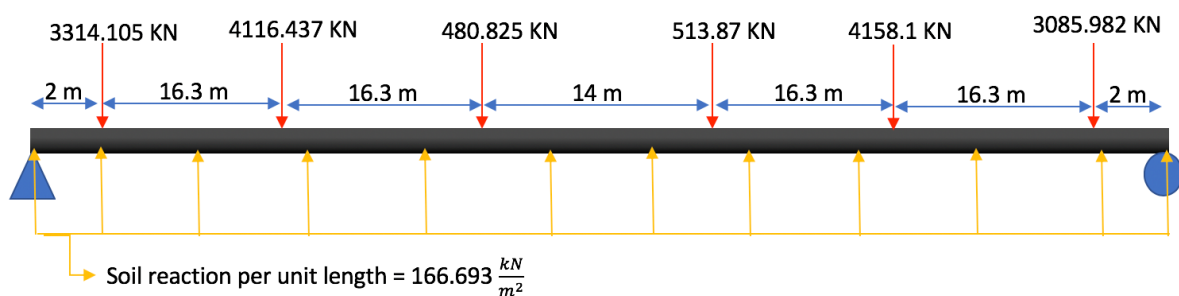


Figure 48: Representation of Strip as a Simply Supported Beam

Positive moment calculation:

Assuming that a **25 mm** bar diameter will be used with **175 mm** center to center spacing.
And considering **1.25 m** was the calculated theoretical depth of the mat foundation:

$$D_{\text{actual}} = (\text{Theoretical Depth}) + (\text{Diameter}) + (c/c \text{ spacing})$$

$$D_{\text{actual}} = 1.25 + 0.025 + 0.175 = 1.45 \approx \mathbf{1.5 \text{ m}}$$

$$\text{Maximum positive moment} = \mathbf{7171.353 \text{ kN.m}} \quad (\text{From ETABS Analysis})$$

$$M' = \frac{7171.353 \text{ kN.m}}{9.6 \text{ m}} = 747.016 \frac{\text{kN.m}}{\text{m}}$$

$$M_u = 747.016 \frac{\text{kN.m}}{\text{m}} (1.7) = 1269.927 \frac{\text{kN.m}}{\text{m}}$$

Also;

$$M_u = 0.9 A_s 420 \frac{\text{MN}}{\text{m}^2} \left(1.5 - \frac{a}{2} \right)$$

$$1269.927 \frac{\text{kN.m}}{\text{m}} = 0.9 A_s 420 \frac{\text{MN}}{\text{m}^2} \left(1.5 - \frac{a}{2} \right)$$

And;

$$a = \frac{A_s 420 \frac{\text{MN}}{\text{m}^2}}{0.85 \left(28 \frac{\text{kN}}{\text{m}^2} \right) (1 \text{ m})}$$

Solving for A_s

$$A_s = 2223 \frac{\text{mm}^2}{\text{m}}$$

Checking our assumption for rebars with 25 mm diameter and 175 mm c/c spacing

$$A_s (\text{provided}) = \frac{\pi(25\text{mm})^2}{4} \div 0.175 \text{ m} = \mathbf{2805.7143 \frac{\text{mm}^2}{\text{m}}}$$

Therefore, 25 mm rebars with 175 mm c/c spacing, will be a sufficient configuration for the reinforcement of the Raft Slab

Chapter 7
ETABS Structural Modelling

7. Structural Modelling

7.1 Software used for structural modelling

CSI® ETABS® [14]: Extended Three-Dimensional Analysis of Building Systems: A finite element analysis program, predominantly used for the structural design and analysis of this project.

7.2 Gridlines

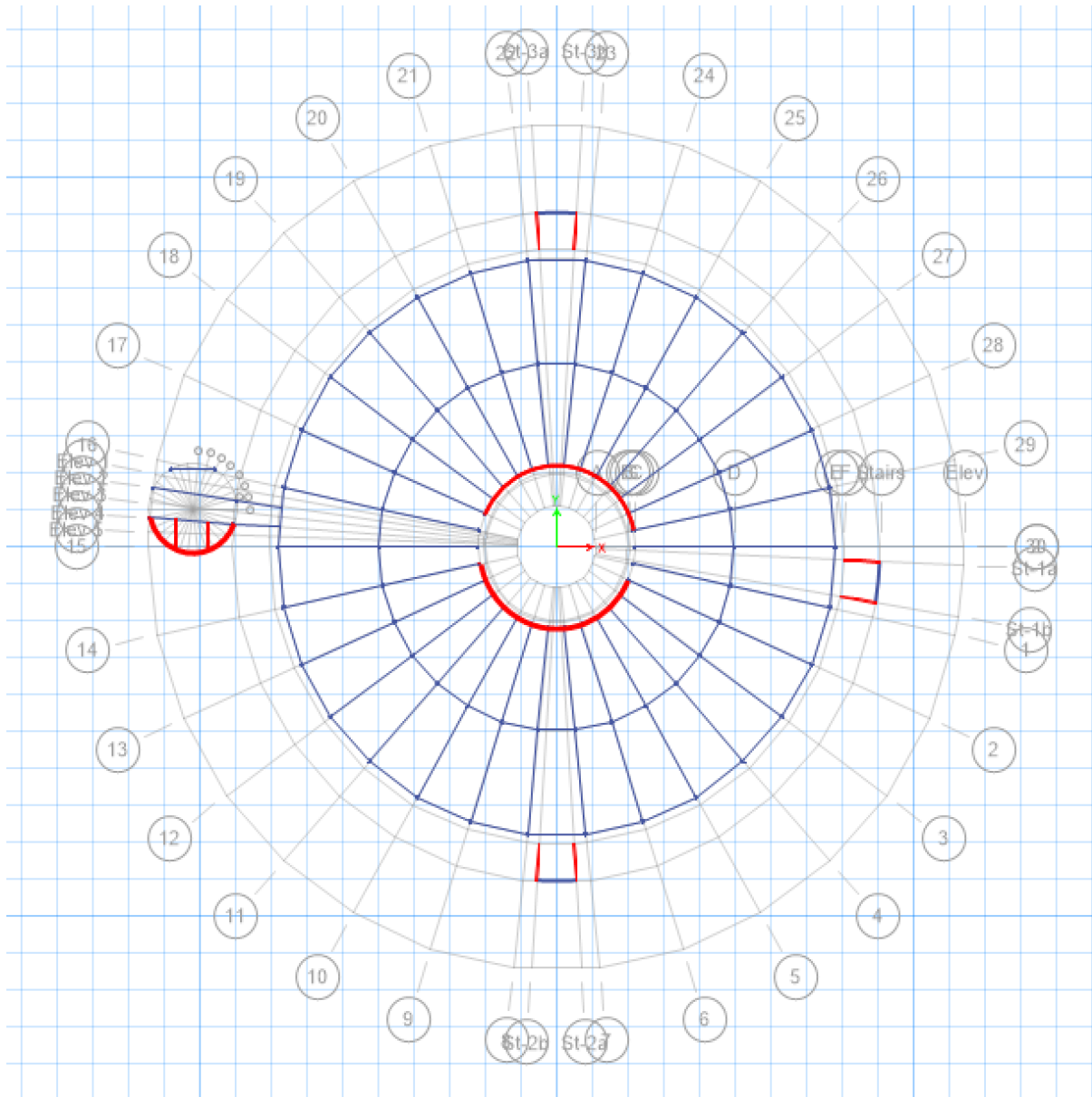


Figure 49: Gridlines used to model the structure in ETABS

7.3 3D overview of model

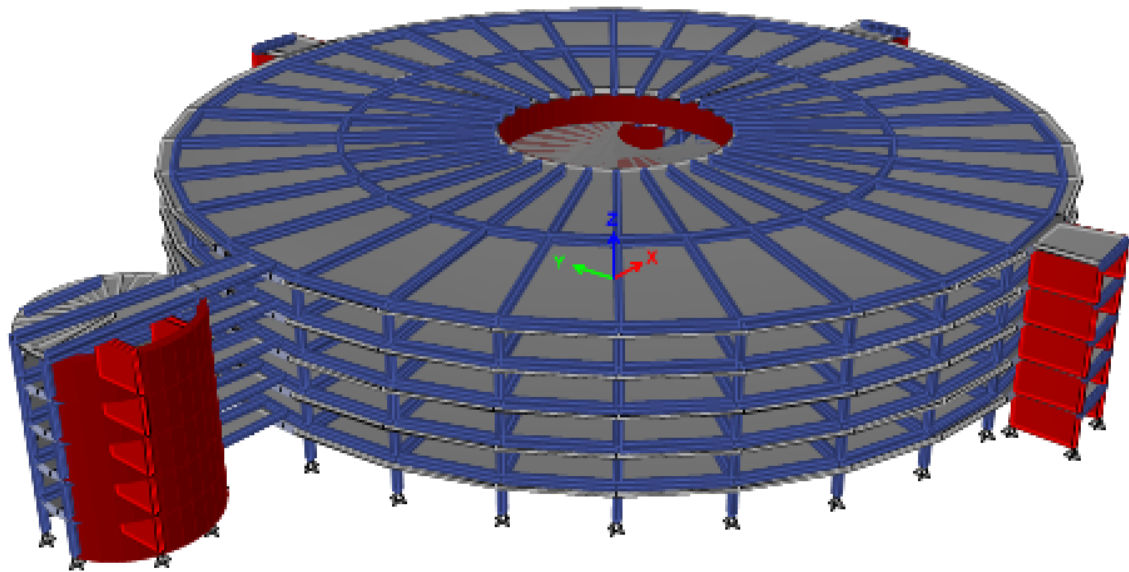


Figure 50: 3D view of structure model; 1

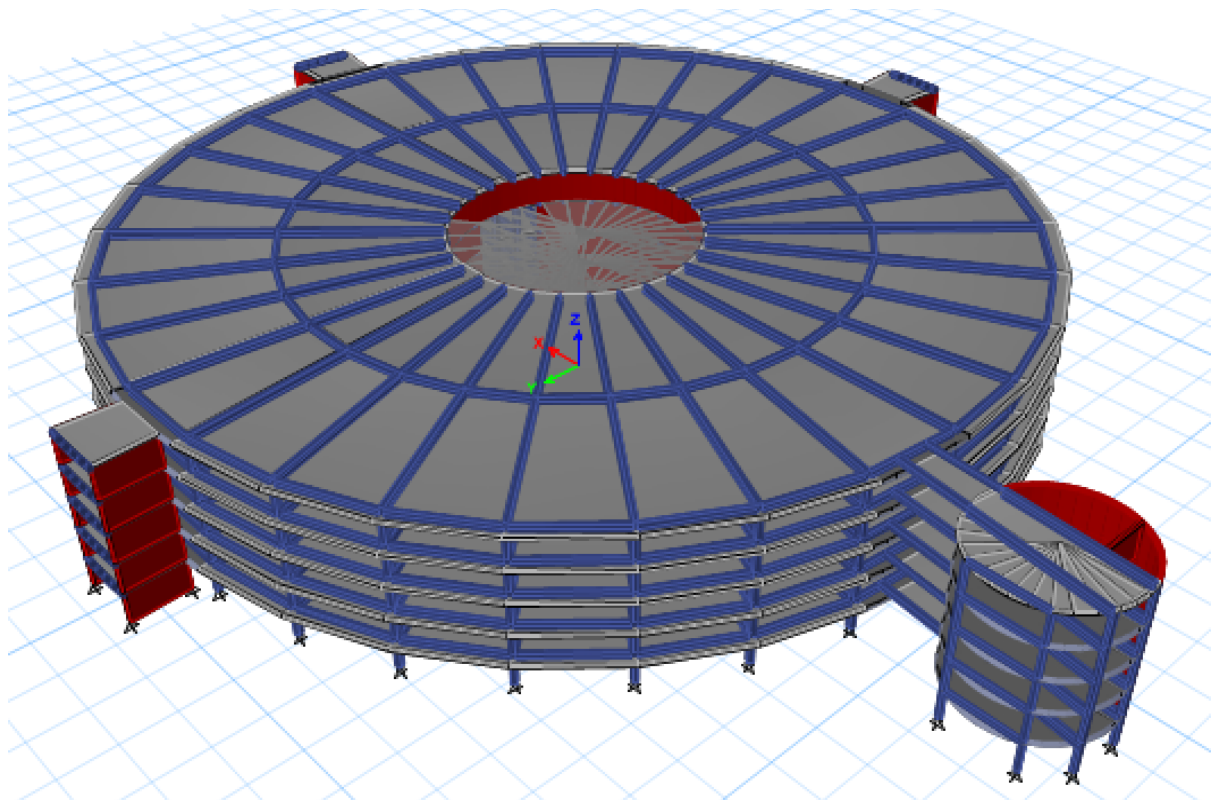


Figure 51: 3D view of structure model; 2

7.4 Elevator Structure Shear Wall

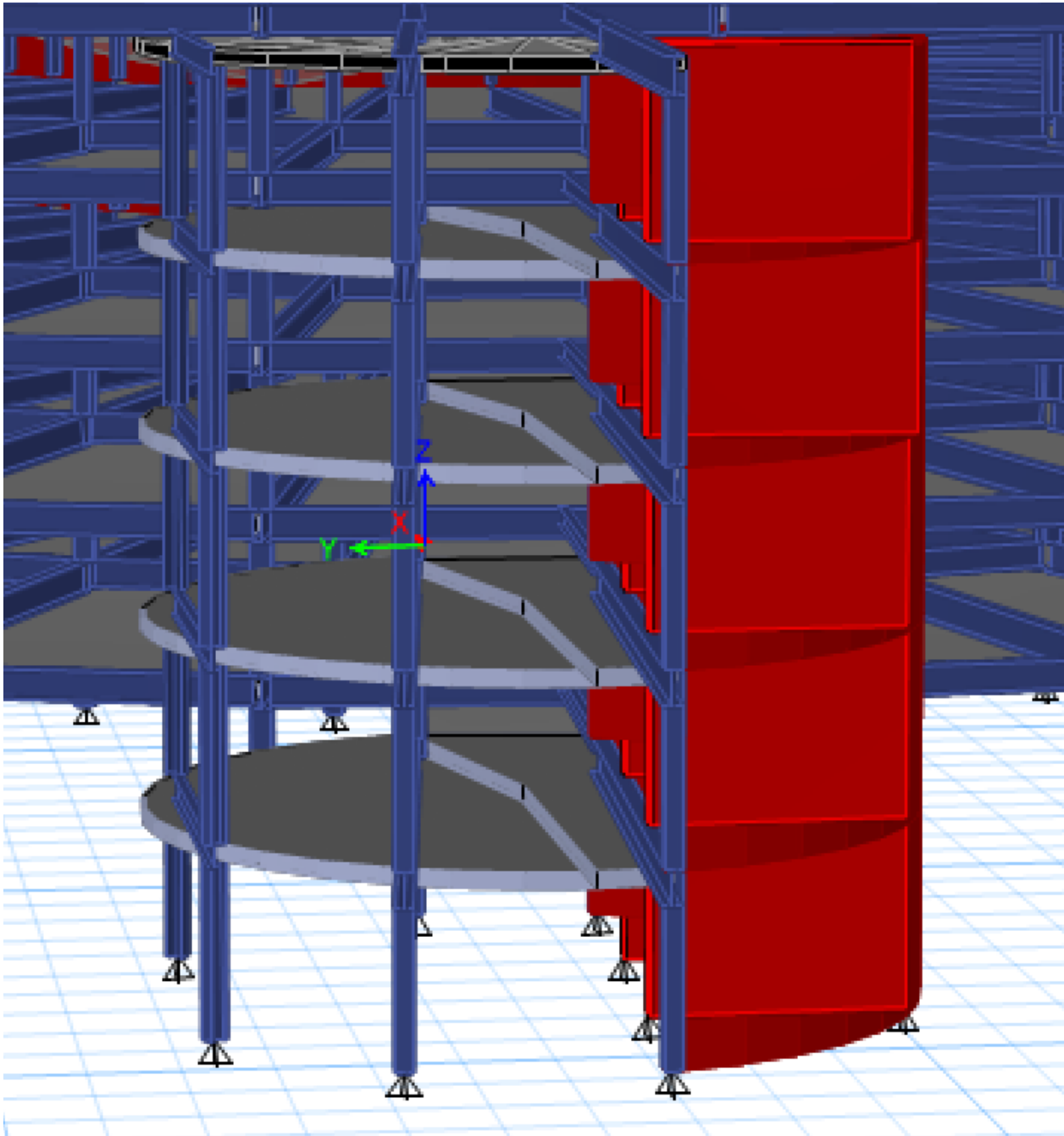


Figure 52: A segmental view of the elevator building with the shear walls

7.5 Analysis Results Representation

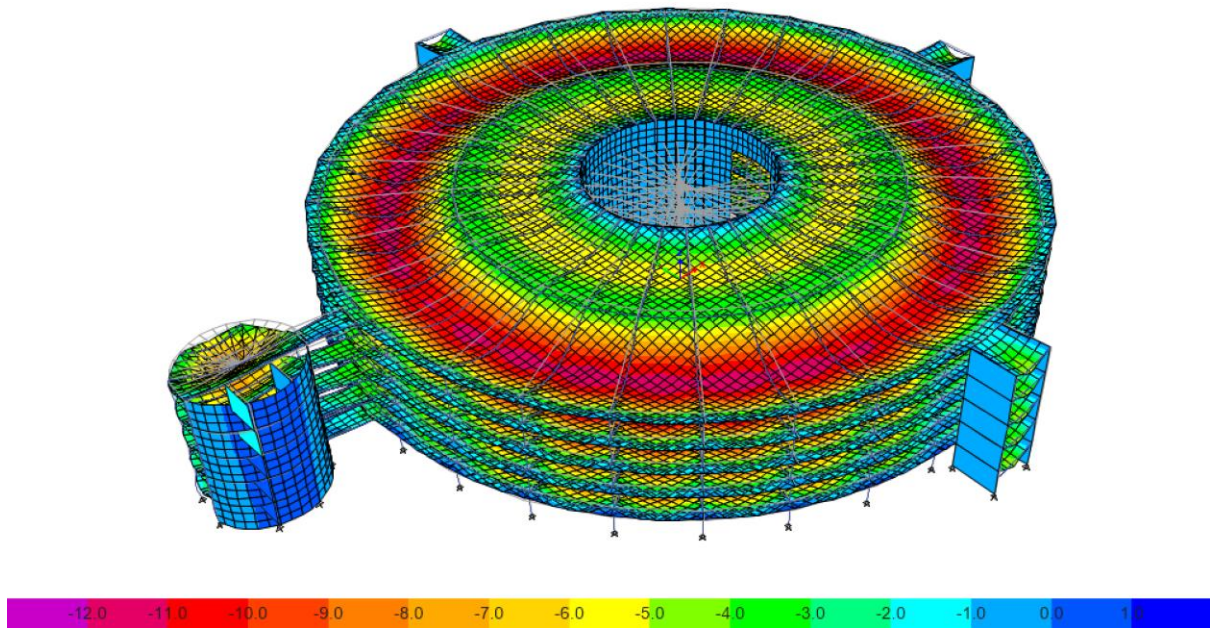


Figure 53: 3D view of the structure's deflection, with legend

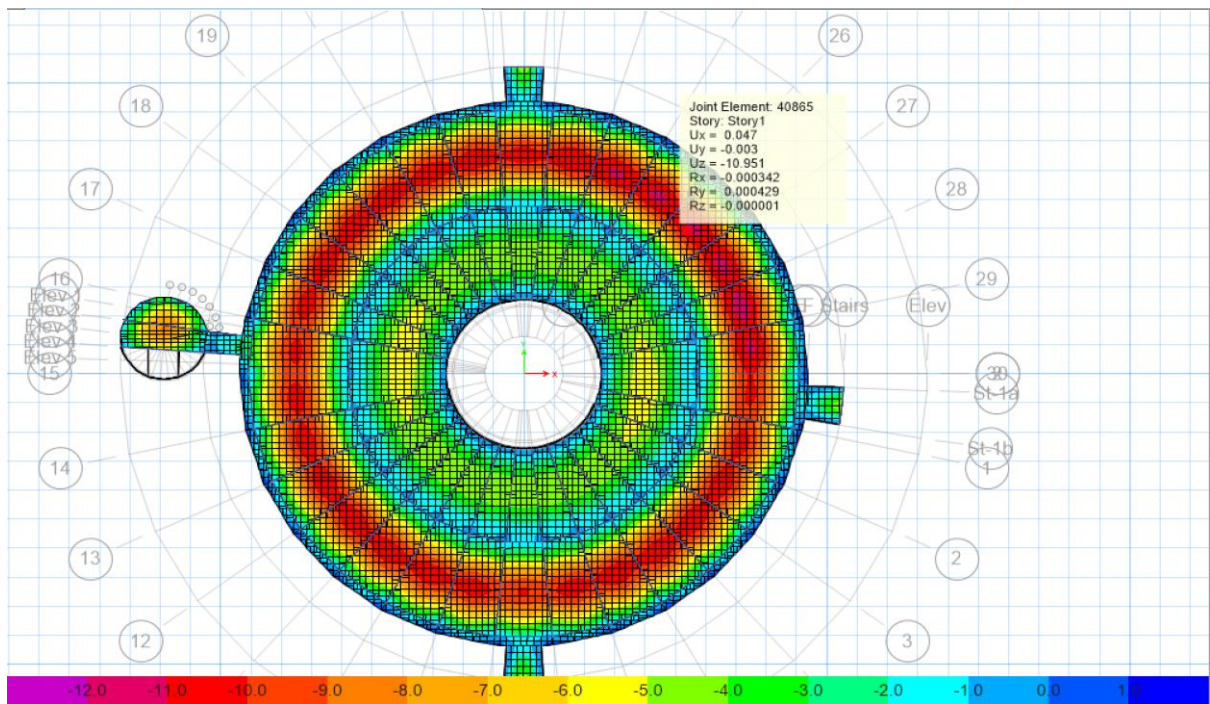


Figure 54: Top view of deflected shape, with legend

7.6 Deflection due to a moving, vehicular load

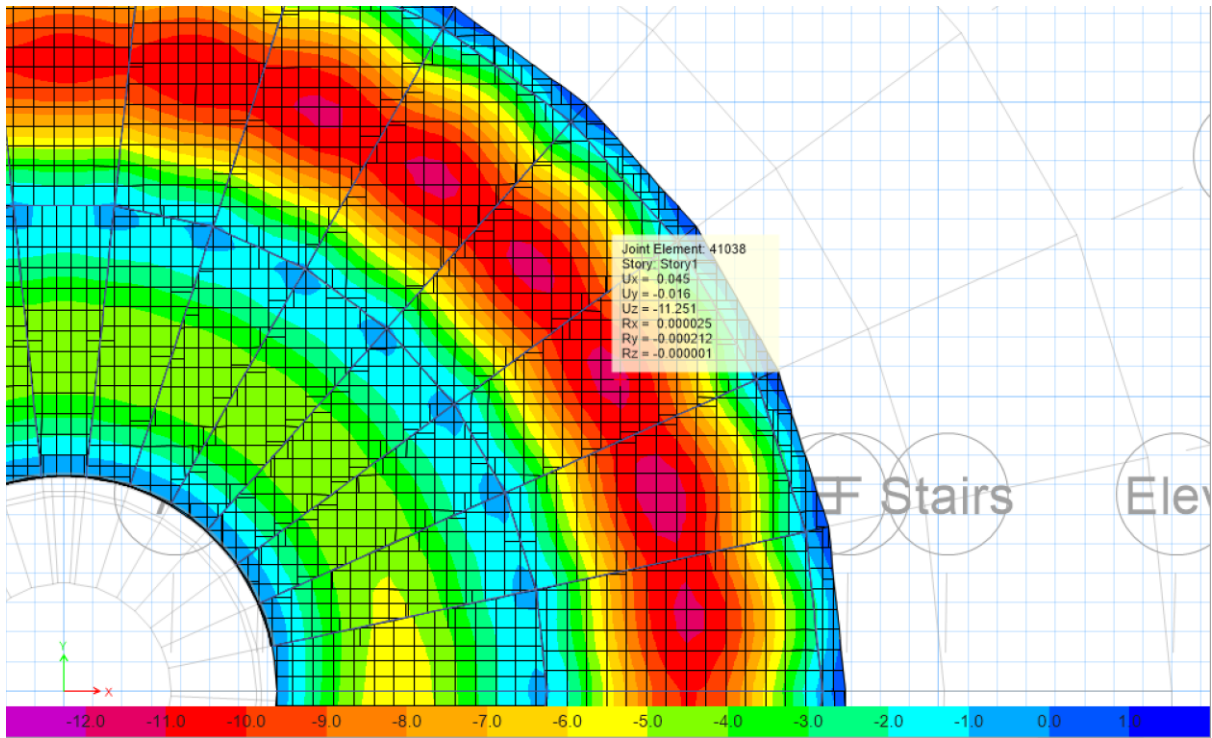


Figure 55: Deflection due to a moving vehicle; position 1

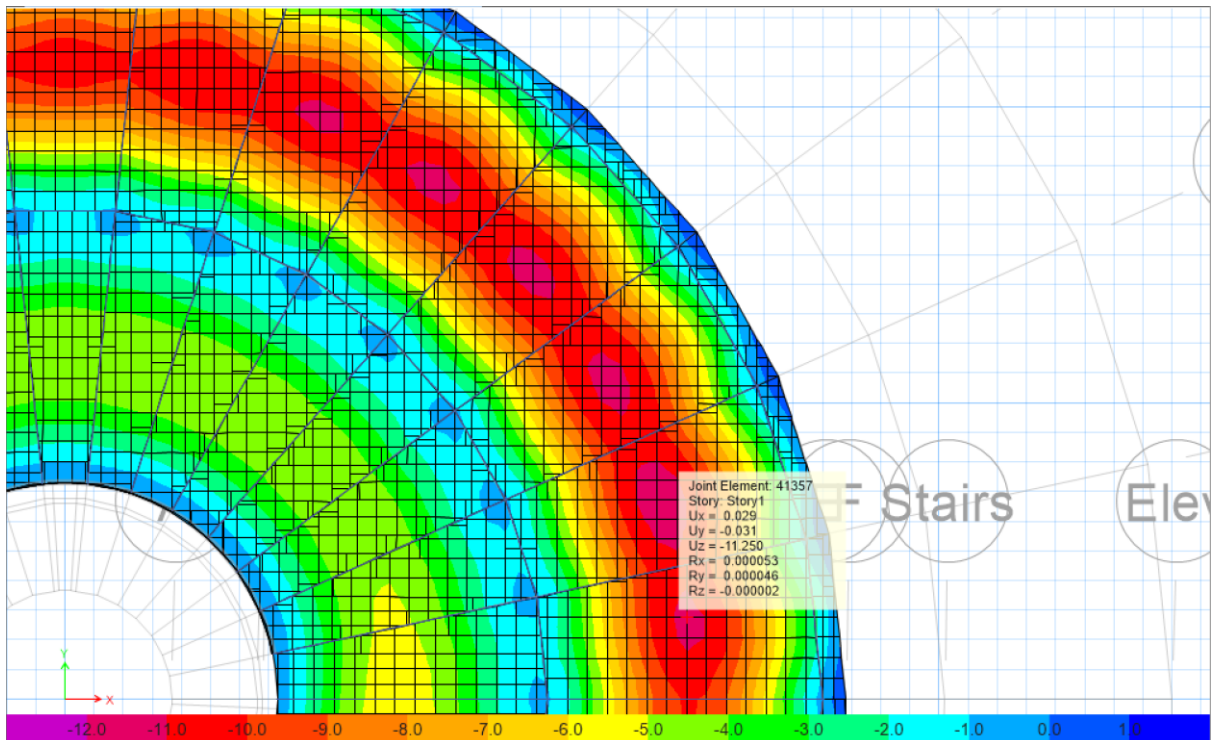


Figure 56: Deflection resulting from a moving vehicle; position 2

7.7 Steel Connection Design

Connections are structural elements used to join members of a steel frame. Adding rigidity and lending increased strength to the structure. In terms of connections used for our project's structure, there were three types utilized, modelled and analyzed ⁵⁷:

- 1- Beam-beam connections
- 2- Beam-column connections
- 3- Column-footing connections

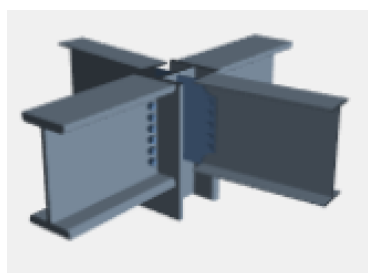


Figure 59: Beam - Beam Connection

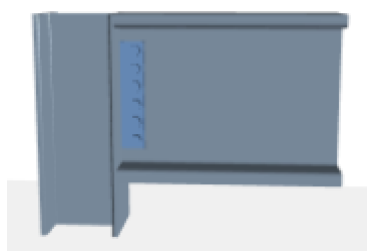


Figure 58: Beam - Column Connection

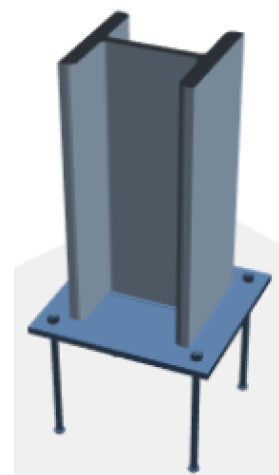


Figure 57: Column - Footing Connection

Table 18: Steel Connection Characteristics

Item	Beam-Beam	Beam-Column	Column-Footing
Bolt types	A325-N	A325-N	A325-N
Bolt size	M32	M32	M32
Hole type	STD	STD	STD
Plate Material	A992 f_y 50	A992 f_y 50	A992 f_y 50
Plate thickness (mm)	9.4	12.5	25
Weld Material	E70XX	E70XX	E70XX
Weld thickness (mm)	100	100	150

As shown in the figures the connections were designed with shear for reactions only. This is due to the presence of shear walls in the core and on the edges of structure. This adds rigidity, increases the building's resistance to the lateral forces imposed, and minimizes the creation of moments at the supports and their connections. Moreover, considering the building is relatively short in height, less moment connections would be needed. Thus, the structure can be considered of adequate rigidity and structural integrity, necessitating the installation of shear-resistant connections only.

⁵⁷ For further details and summary on connections, refer to **Appendix C**.

7.8 Steel Connections Passing Check

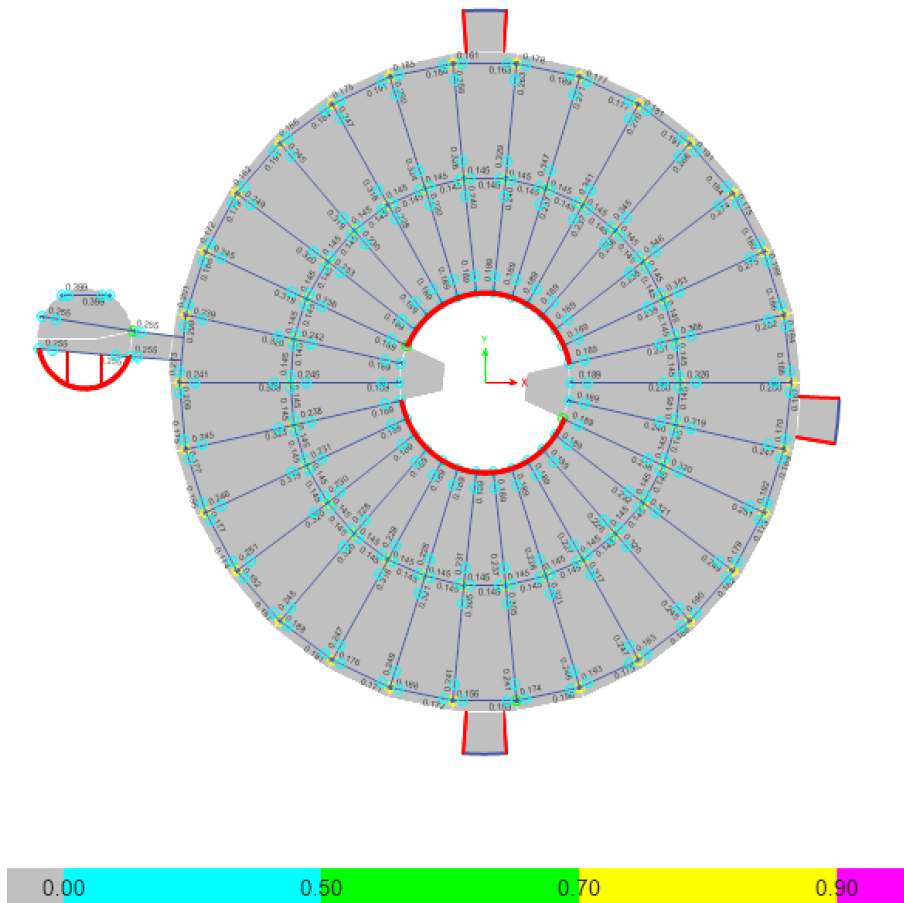


Figure 61: Plan View of Structure with Steel Connections Passing Testing

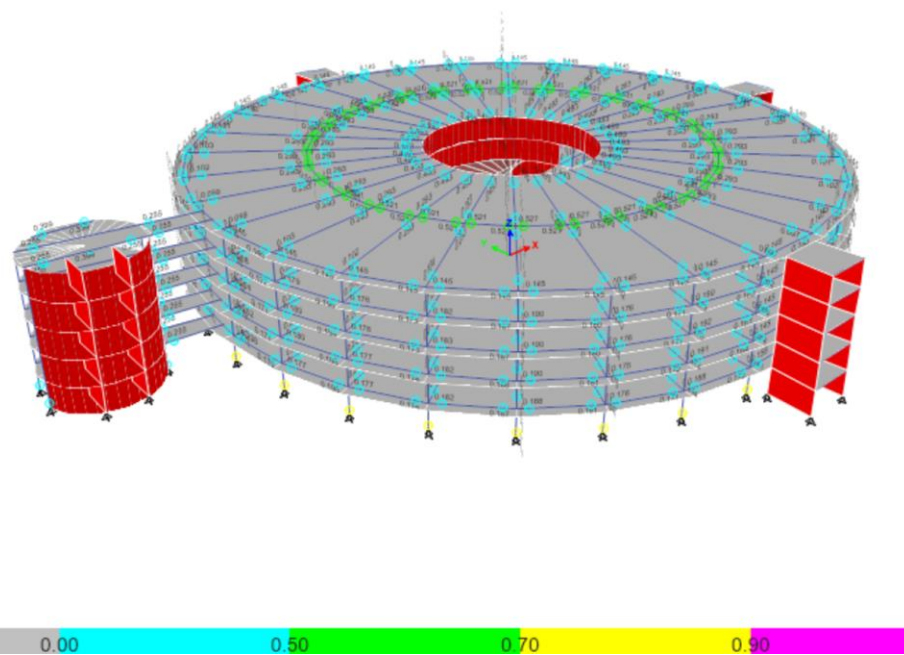


Figure 60: 3D View of Structure with Steel Connections Passing Testing

Chapter 8

Cost Estimation

8 Cost Estimation

In this section, we shall describe a general breakdown of the estimated cost of this project. The prices provided herein have been obtained from local material suppliers, vendors, contractors and manufacturers. The below estimates are indicative of current, local market prices, as of the writing of this report.

Prices shown are in Saudi Arabian Riyal SAR (1 SAR ≈ \$ 0.27 USD)⁵⁸

8.1 Structural Steel Cost

Based on an approximated metric tonnage of **2587**, the estimation includes; steel sections requested for quotation, suitable strength and thickness connection plates, bolts, anti-corrosive galvanization and transportation to project site.

Table 19: Cost Estimation of Structural Steel Sections and Periphery Items

Structural Steel				
No.	Total length (m)	Total length (MT)	Unit Price (SAR/MT)	Cost (SAR)
1	9220	2587	7650	19,790,550

Quotation provided by *Zamil Structural Steel Fabrication and Manufacturing*

8.2 Premixed Concrete Material Cost

Table 20: Cost Estimation of Premixed Concrete

4000 psi Premixed Concrete				
No.	Element	Volume (m ³)	Unit Price (SAR/m ³)	Cost (SAR)
1	Slabs	40312.40	189	7619043.634
2	Shear Walls	547.235	189	103427.415
3	Raft Foundation	10609.5	189	2005195.5
4	Concrete Piles	498.624	189	94239.936
Total Cost (SAR)				9,821,906.485

Prices provided by *Saudi Ready-Mix Concrete Company LTD.*

⁵⁸ As of 19/4/2019 exchange rate.

8.3 Steel Reinforcement (Rebars) Material Cost

Cost of required rebar tonnage was estimated using an industry defined ratio of rebar requirement by volume of concrete, which is 140 Kg of rebars per m³ of concrete⁵⁹

Table 21: Cost Estimation for Rebars

Cost Estimation of Reinforcements						
No.	Element	Volume (m ³)	Estimation Ratio (Kg/m ³)	Tonnage (MT)	Unit Price (SAR/MT Ton)	Cost (SAR)
1	Slab	40312.40	140	5643.736025	2880	16253959.8
2	Shear Wall	547.235		76.6129	2880	220645.152
3	Raft Foundation	10609.5		1485.33	2880	4277750.4
4	Concrete Piles	498.624		69.80736	2880	201045.197
Total Cost (SAR)						20,953,400.5

Prices provided by *Abdullah Ababtain General Contracting Company LTD.*

8.4 Total Project Cost

Total estimated cost; including only the aforementioned criteria:

≈ 50.566 Million SAR

⁵⁹ As per *Al-Mulhim et al. 2018 [17]*.

Chapter 9

Additional Elements

9 Additional

9.1 Solar roof fixture

Environmental considerations are increasingly becoming a focal point of national interest across the globe. Rising atmospheric temperatures, coupled with a scarcity of resources are slowly pushing the planet's inhabitants to the brink of unsustainability. However, the current state is also pushing them to innovate far beyond the conventional and ushering forth an age where the impossible has seemingly stopped becoming so. Out of the many solutions that are being offered to minimize our carbon footprint and sustainably power our cities, is the promise of Solar.

The surface area above the fifth floor, that comprises the roof of our facility's main structure offers an expanse of space that has not been designed to be utilized in any manner. We propose the installation of a ring-shaped steel frame, spanning the diameter of the building that can house a permanent solar panel fixture. This installation could provide the structure and PMU with a surplus of power, that can either be used to offset the costs of the project or power the campus sustainably.

The following is a quick breakdown of the possible benefits from such an installation on our facility's roof:

Using an average Monocrystalline solar panel (Figure on the right) with the following specifications:

Size (surface area) = 1.96 m²/panel

Watts Produced per Panel = 370 W

Usable space on main structure roof = 3500 m²

*∴ Watts of power generated by solar installation
(3500 / 1.96) x 370 = 660.714 kW*

Average Annual Hours of Sunlight on the City of Dammam, KSA ⁶⁰ = 3350 hrs./annum.

*∴ Theoretical Amount of Power Generated by Installation/Annum.
= 3350 hrs./Annum. x 660.714 kW = 2,213,392 kWh / Annum.*

For large commercial purchases, solar panels are usually quoted in terms of Watts. This makes it easier to for the purchaser to set up the supporting services and estimate the return on investment that could be made from generating power through their panels.

Average prices for these types of panels, in this output rating = \$ 0.48/ W

Assuming steel framing, inverters and wiring = 15% of total panel cost

*Total price of solar installation = (\$0.48 x 3.75 SAR/\$) x 660,714 W x 1.15 = **1,367,677 SAR***



MONOCRYSTALLINE 72 CELLS

Figure 62: Monocrystalline Solar Panel. Image Courtesy of Alibaba.com

⁶⁰ Value obtained from [27].

The electricity tariff for an educational institution in Saudi Arabia = 0.18 SAR/kWh ⁶¹

If the power generated by the solar installation was to be resupplied to the grid and sold to the SEC, Annual cost recovery would be

*= 0.18 SAR/ kWh x 2,200,000 kWh = **396,000 SAR/Annum.***

∴ Time to full Recovery of Costs Incurred by Solar Installation

*= (1,367,677 SAR ÷ 396,000 SAR/Annum.) = **3.45 Years to Full Cost Recovery***

Even when the simple calculation above does not take into consideration inflation rates, the rising demand on electricity or planned governmental raising of electricity tariffs, it can be easily understood that such an installation may prove to be very effective in the very near future.

The falling prices of solar panels to meet the growing global demand, coupled with the sunny geographical location of Saudi Arabia offers a compelling argument for the future of solar energy production in our region and the world over.

9.2 Pedestrian Crossing

We propose a pedestrian crossing to be constructed, linking the parking structure to the male campus gate on the opposite side of the road.



Figure 63: Proposed Pedestrian Crossing Location

⁶¹ As per tariffs declared by the Saudi Electricity Company [28].

Chapter 10
Project Summary

10 Project Summary

In the design of our structure, our attempts in addressing the pre-existing soil issues have been met with great success. After the exchange of the topsoil layer with a granular backfill, the soil has shown an improvement in bearing capacity and the piled raft foundation is has proven rigid enough to carry the structure without allowing any consolidation settlement.

The structural steel members have been found to sufficiently bear the weight of the structure, whilst offering greater rigidity and less weight than comparable building constructed solely of reinforced concrete. The construction of shear walls in key locations around the building has also proved effective in resisting lateral loading and offering increased structural integrity.

The ETABS model has also proved the safety and sufficiency of our structural frame design, while also exhibiting minimal deformation throughout the structure when subjected to different load combinations and scenarios. The modelling of the connections has also attained sufficient results, indicating an exclusive requirement for shear-resistant connections between steel elements, only.

However, the cost of the structure is quite high and may be considered disadvantageous when opting to utilize this particular design.

Chapter 11

References

11 References

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Chapter 12
Appendices

12 Appendices

12.1 Appendix A: Structural loading details

12.1.1 Load combination breakdown

The minimum uniformly distributed dead loads (UDL LLs) utilized in the design are as follows:

No.	Load Unit	Dead	
		$\frac{kN}{m^2}$	$\frac{kips}{ft^2}$
1	Tiles	0.75	0.01566
2	mortar	0.51	0.01065
3	Plaster	0.25	0.005221
4	profile metallic deck	0.77475	0.01618
5	E and M	0.5	0.10442
6	Ceiling	1.8	0.03759
7	Windows, glass, frame.	0.4	0.00835
8	Concrete filling	0.023	0.00048
9	Solar panel support structure	0.1	0.00208
10	Solar panel	0.2	0.00418

- 1- Main structure typical floor slab uses **1,2,3,4,5, and 6.**
- 2- Main structure roof floor slab uses **1,2,3,4,5,6,9, and 10.**
- 3- Elevator structure slab uses **1,2,3,4,5,6,7, and 8.**
- 4- Stairs structure slab uses **1,2,3,4,5,6,7, and 8.**

12.2 Appendix B: Structural design

12.2.1 Horizontal Elements

Main Structure Horizontal Elements				
No.	Roof		Typical floor	
	Girder	Beam	Girder	Beam
1	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
2	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
3	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
4	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
5	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
6	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
7	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
8	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
9	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
10	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
11	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
12	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
13	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
14	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
15	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
16	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
17	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
18	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68

19	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
20	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
21	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
22	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
23	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
24	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
25	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68

Main Structure Horizontal Elements				
No.	Roof		Typical floor	
	Girder	Beam	Girder	Beam
26	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
27	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
28	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
29	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
30	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
31	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
32	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
33	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
34	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
35	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
36	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
37	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68

38	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
39	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
40	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
41	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
42	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
43	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
44	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
45	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
46	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
47	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
48	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
49	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
50	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68
51	W 24 x 250	W 24 x 94	W 24 x 279	W 24 x 68

Properties

Section	Depth (in)	$I_x(\text{in}^4)$	$\phi_n V_x (\text{Kip})$	$\phi_n M_x (\text{Kip} \cdot \text{ft})$	weight $\frac{\text{Kip}}{\text{ft}}$
W 24 x 279	24	13400	1280	4240	0.279
W 24 x 250	24	9600	929	3130	0.25
W 24 x 94	24	2700	375	949	0.094
W 24 x 68	24	1830	295	664	0.068
W 14 x 38	14	385	131	231	0.038

12.2.2 Vertical Elements

Designed Section

Story No.	Inner columns	Middle columns	Outer columns
5	W12x26	W12x58	W12x45
4	W12x65	W14x176	W12x120
3	W12x65	W14x176	W12x120
2	W12x106	W14x283	W12x190
1	W12x106	W14x283	W12x190

Member sections

Main girder	W24x306
-------------	---------

Roof girder	W24x279
Main beam	W24x68
Roof beam	W24x94

UDL (kip/ft)			
W24x306	W24x279	W24x68	W24x94
4.491	3.943	3.192	2.81

Loads acting on the columns (kips)			
Story No.	Inner columns	Middle columns	Outer columns
5	106.148	301.521	195.373
4	227.047	644.677	417.629
3	347.947	987.832	639.885
2	468.847	1330.988	862.140
1	589.747	1674.143	1084.396

Area required for the section			
Story No.	Inner columns	Middle columns	Outer columns
5	2.831	8.041	5.210

4	6.055	17.191	11.137
3	9.279	26.342	17.064
2	12.503	35.493	22.990
1	15.727	44.644	28.917

Checking the selected section									
Story	Location	Section	A(in ²)	r _x (in)	r _y (in)	KL/r	F _{cr}	P _n	Check
5	Inner	W12x26	7.65	5.17	1.51	91.25	24.6	169.371	Yes
	Middle	W12x58	17	5.28	2.51	54.90	36.1	552.330	Yes
	outer	W12x45	13.1	5.15	1.95	70.66	31.1	366.669	Yes
4	Inner	W12x65	19.1	5.28	3.02	45.63	38.5	661.815	Yes
	Middle	W14x176	51.8	6.43	4.02	34.28	41.4	1930.068	Yes
	outer	W12x120	35.2	5.51	3.13	44.02	39.1	1238.688	Yes
3	Inner	W12x65	19.1	5.28	3.02	45.63	38.5	661.815	Yes
	Middle	W14x176	51.8	6.43	4.02	34.28	41.4	1930.068	Yes
	outer	W12x120	35.2	5.51	3.13	44.02	39.1	1238.688	Yes
2	Inner	W12x106	31.2	5.47	3.11	44.31	39.1	1097.928	Yes
	Middle	W14x283	83.3	6.79	4.17	33.04	41.6	3118.752	Yes
	outer	W12x190	56	5.82	3.25	42.40	39.5	1990.800	Yes
1	Inner	W12x106	31.2	5.47	3.11	44.31	39.1	1097.928	Yes

	Middle	W14x283	83.3	6.79	4.17	33.04	41.6	3118.752	Yes
	outer	W12x190	56	5.82	3.25	42.40	39.5	1990.800	Yes

12.2.3 Remaining Shear Wall Dimensions

Element	Thickness (mm)	Thickness (in)
Stairs	100	4
Elevator	203	8
Ramp	152	6

Structure	Slab thickness (mm)	Cover (mm)	d (mm)	b (mm)	M max (KN.m)
Main Structure	280	50	230	1000	50.811
Stairs	153	50	103	1000	11.34
Elevator	229	50	179	1000	21.371
Pedestrian	229	50	179	1000	44.357
Ramp	500	50	450	1000	121
Structure	M max (N.mm)	κ (N/mm ²)	f'c (Mpa)	fy (Mpa)	ρ
Main Structure	50811000	1.067233774	28	420	0.00260
Stairs	11340000	1.187670846	28	420	0.00290
Elevator	21371000	0.741099078	28	420	0.00179
Pedestrian	44357000	1.538202789	28	420	0.00379
Ramp	121000000	0.663923182	28	420	0.00160
Structure	ρ_{min}	ρ_{max}	As (mm ² /m)	As min (mm ² /m)	As max (mm ² /m)
Main Structure	0.000476	0.016250	598.1638872	109.5238095	3737.5

Stairs	0.000476	0.016250	298.916421 5	49.0476190 5	1673.75
Elevator	0.000476	0.016250	320.926283 8	85.2380952 4	2908.75
Pedestrian	0.000476	0.016250	678.243042 4	85.2380952 4	2908.75
Ramp	0.000476	0.016250	721.554923 9	214.285714 3	7312.5

12.3 Appendix C: Modelling Analysis Results

12.3.1.1 Modelling Summary

1 Structure Data

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

1.1 Story Data

Table 1.1 - Story Data

Name	Height mm	Elevation mm	Master Story	Similar To	Splice Story
Story5	3500	17500	No	None	No
Story4	3500	14000	No	Story1	No
Story3	3500	10500	No	Story1	No
Story2	3500	7000	No	Story1	No
Story1	3500	3500	Yes	None	No
Base	0	0	No	None	No

2 Loads

This chapter provides loading information as applied to the model.

2.1 Load Patterns

Table 2.1 - Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Other	0	
Live	Live	0	
windx	Wind	0	ASCE 7-10
windy	Wind	0	ASCE 7-10
earthquake x	Seismic (Drift)	0	ASCE 7-10
earthquake y	Seismic (Drift)	0	ASCE 7-10

2.2 Load Cases

Table 2.2 - Load Cases - Summary

Name	Type
Dead	Linear Static
Live	Linear Static
windx	Linear Static
windy	Linear Static
earthquake x	Linear Static
earthquake y	Linear Static

3 Analysis Results

This chapter provides analysis results.

3.1 Structure Results

Table 3.1 - Base Reactions

Load Case/Combo	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m	Y m	Z m
Dead	1.744E-05	0.0003	72926.1425	21856.2198	160710.4685	0.0016	0	0	0
Live	7.635E-06	0.0001	57607.639	11514.7409	56520.5772	0.0009	0	0	0
windx	-5988.1132	-0.0002	3.865E-05	0.0012	-64444.7497	491.2373	0	0	0
windy	0.0008	-6242.7777	3.272E-05	67289.1151	0.0122	15811.7258	0	0	0

Load Case/Combo	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m	Y m	Z m
earthquake x	-16544.6965	-0.0008	0.0001	0.0041	-210387.994	1544.28	0	0	0
earthquake y	0.0024	-16535.7848	0.0001	210467.946 9	0.0384	21040.608	0	0	0
Comb1	3.314E-05	0.0005	179683.593 5	44651.0491	283285.485 7	0.0034	0	0	0
SL	2.508E-05	0.0004	130533.781 6	33370.9606	217231.045 7	0.0025	0	0	0
DL+EQX	-16544.6965	-0.0006	72926.1427	21856.2238	-49677.5255	1544.2815	0	0	0
DL+EXY	0.0024	-16535.7846	72926.1426	232324.166 7	160710.506 9	21040.6096	0	0	0
DL+WINDX	-9580.9812	-0.0002	65633.5283	19670.5997	41527.8222	785.981	0	0	0
DL+WINDY	0.0012	-9988.4441	65633.5283	127333.181 9	144639.441 2	25298.7626	0	0	0
DStIS1	1.222E-05	0.0002	92172.2224	18423.5854	90432.9235	0.0015	0	0	0
DStIS2	-5988.1132	-0.0001	57607.6391	11514.7421	-7924.1725	491.2382	0	0	0
DWal1	1.222E-05	0.0002	92172.2224	18423.5854	90432.9235	0.0015	0	0	0
DWal2	-5988.1132	-0.0001	57607.6391	11514.7421	-7924.1725	491.2382	0	0	0
DWal3	5988.1133	0.0004	57607.639	11514.7396	120965.326 8	-491.2363	0	0	0
DWal4	0.0008	-6242.7776	57607.6391	78803.856	56520.5894	15811.7267	0	0	0
DWal5	-0.0008	6242.7779	57607.639	-55774.3743	56520.565	-15811.7248	0	0	0
DWal6	-5988.1132	-0.0002	3.865E-05	0.0012	-64444.7497	491.2373	0	0	0
DWal7	5988.1132	0.0002	-3.865E-05	-0.0012	64444.7497	-491.2373	0	0	0
DWal8	0.0008	-6242.7777	3.272E-05	67289.1151	0.0122	15811.7258	0	0	0
DWal9	-0.0008	6242.7777	-3.272E-05	-67289.1151	-0.0122	-15811.7258	0	0	0
DStID1	7.635E-06	0.0001	57607.639	11514.7409	56520.5772	0.0009	0	0	0

3.2 Story Results

Table 3.2 - Story Drifts

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story5	Dead	X	6.3E-05	165	-62.945	12.2614	17.5
Story5	Dead	Y	0.000135	170	-66.56	6	17.5
Story5	Live	Y	5.4E-05	170	-66.56	6	17.5
Story5	windx	X	4E-05	126	52.0956	-8.7178	17.5
Story5	windx	Y	1.6E-05	170	-66.56	6	17.5
Story5	windy	Y	8.2E-05	161	-57	-0.575	17.5
Story5	earthquake x	X	0.000152	161	-57	-0.575	17.5
Story5	earthquake x	Y	7.3E-05	170	-66.56	6	17.5
Story5	earthquake y	Y	0.000278	131	-3.2246	-52.7215	17.5
Story5	Comb1	X	0.00011	165	-62.945	12.2614	17.5
Story5	Comb1	Y	0.000247	170	-66.56	6	17.5
Story5	SL	X	8.5E-05	165	-62.945	12.2614	17.5
Story5	SL	Y	0.000188	170	-66.56	6	17.5
Story5	DL+EQX	X	0.000204	165	-62.945	12.2614	17.5
Story5	DL+EQX	Y	0.000208	170	-66.56	6	17.5
Story5	DL+EXY	Y	0.00034	161	-57	-0.575	17.5
Story5	DL+WINDX	X	0.000105	165	-62.945	12.2614	17.5
Story5	DL+WINDX	Y	0.000147	170	-66.56	6	17.5
Story5	DL+WINDY	X	6.3E-05	158	-55.715	12.2614	17.5
Story5	DL+WINDY	Y	0.000229	145	-65.8726	9.2578	17.5
Story5	DStIS1	Y	8.6E-05	170	-66.56	6	17.5
Story5	DStIS2	X	5.2E-05	165	-62.945	12.2614	17.5
Story5	DStIS2	Y	6.9E-05	170	-66.56	6	17.5
Story5	DWal1	Y	8.6E-05	170	-66.56	6	17.5
Story5	DWal2	X	5.2E-05	165	-62.945	12.2614	17.5

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story5	DWal2	Y	6.9E-05	170	-66.56	6	17.5
Story5	DWal3	X	5E-05	126	52.0956	-8.7178	17.5
Story5	DWal3	Y	3.8E-05	145	-65.8726	9.2578	17.5
Story5	DWal4	Y	0.000126	161	-57	-0.575	17.5
Story5	DWal5	X	3.6E-05	161	-57	-0.575	17.5
Story5	DWal5	Y	9E-05	127	-3.2246	52.7215	17.5
Story5	DWal6	X	4E-05	126	52.0956	-8.7178	17.5
Story5	DWal6	Y	1.6E-05	170	-66.56	6	17.5
Story5	DWal7	X	4E-05	126	52.0956	-8.7178	17.5
Story5	DWal7	Y	1.6E-05	170	-66.56	6	17.5
Story5	DWal8	Y	8.2E-05	161	-57	-0.575	17.5
Story5	DWal9	Y	8.2E-05	161	-57	-0.575	17.5
Story5	DStID1	Y	5.4E-05	170	-66.56	6	17.5
Story4	Dead	Y	0.000106	145	-65.8726	9.2578	14
Story4	Live	X	1.5E-05	158	-55.715	12.2614	14
Story4	Live	Y	4.4E-05	145	-65.8726	9.2578	14
Story4	windx	X	4.6E-05	126	52.0956	-8.7178	14
Story4	windx	Y	2E-05	199	-66.358	4.6402	14
Story4	windy	Y	0.000113	79	12.5301	-2.6633	14
Story4	earthquake x	X	0.00019	154	-57	-0.5	14
Story4	earthquake x	Y	8.2E-05	199	-66.358	4.6402	14
Story4	earthquake y	Y	0.000402	79	12.5301	-2.6633	14
Story4	Comb1	Y	0.000197	145	-65.8726	9.2578	14
Story4	SL	Y	0.00015	145	-65.8726	9.2578	14
Story4	DL+EQX	X	0.000198	158	-55.715	12.2614	14
Story4	DL+EQX	Y	0.000186	138	-66.5779	6	14
Story4	DL+EXY	Y	0.000401	3	12.81	0	14
Story4	DL+WINDX	X	9.3E-05	158	-55.715	12.2614	14
Story4	DL+WINDX	Y	0.000125	145	-65.8726	9.2578	14
Story4	DL+WINDY	Y	0.000207	178	-61.975	-0.575	14
Story4	DStIS1	X	2.4E-05	158	-55.715	12.2614	14
Story4	DStIS1	Y	7E-05	145	-65.8726	9.2578	14
Story4	DStIS2	X	5.7E-05	158	-55.715	12.2614	14
Story4	DStIS2	Y	6.3E-05	138	-66.5779	6	14
Story4	DWal1	X	2.4E-05	158	-55.715	12.2614	14
Story4	DWal1	Y	7E-05	145	-65.8726	9.2578	14
Story4	DWal2	X	5.7E-05	158	-55.715	12.2614	14
Story4	DWal2	Y	6.3E-05	138	-66.5779	6	14
Story4	DWal3	X	5.3E-05	131	-3.2246	-52.7215	14
Story4	DWal3	Y	2.8E-05	178	-61.975	-0.575	14
Story4	DWal4	Y	0.000116	154	-57	-0.5	14
Story4	DWal5	Y	0.000114	79	12.5301	-2.6633	14
Story4	DWal6	X	4.6E-05	126	52.0956	-8.7178	14
Story4	DWal6	Y	2E-05	199	-66.358	4.6402	14
Story4	DWal7	X	4.6E-05	126	52.0956	-8.7178	14
Story4	DWal7	Y	2E-05	199	-66.358	4.6402	14
Story4	DWal8	Y	0.000113	79	12.5301	-2.6633	14
Story4	DWal9	Y	0.000113	79	12.5301	-2.6633	14
Story4	DStID1	X	1.5E-05	158	-55.715	12.2614	14
Story4	DStID1	Y	4.4E-05	145	-65.8726	9.2578	14
Story3	Dead	X	8.7E-05	205	-57	3.9	10.5
Story3	Dead	Y	6.1E-05	199	-66.358	4.6402	10.5
Story3	Live	X	3.5E-05	205	-57	3.9	10.5
Story3	Live	Y	2.5E-05	199	-66.358	4.6402	10.5
Story3	windx	X	5.6E-05	126	52.0956	-8.7178	10.5
Story3	windx	Y	2.4E-05	199	-66.358	4.6402	10.5

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story3	windy	Y	0.000112	103	45.8653	9.749	10.5
Story3	earthquake x	X	1549067566	155	-62	4.3	10.5
Story3	earthquake y	Y	0.000398	95	52.7697	-2.304	10.5
Story3	Comb1	X	0.00016	205	-57	3.9	10.5
Story3	Comb1	Y	0.000113	199	-66.358	4.6402	10.5
Story3	SL	X	0.000121	205	-57	3.9	10.5
Story3	SL	Y	8.6E-05	199	-66.358	4.6402	10.5
Story3	DL+EQX	X	1549067566	155	-62	4.3	10.5
Story3	DL+EXY	Y	0.000389	95	52.7697	-2.304	10.5
Story3	DL+WINDX	X	0.000105	165	-62.945	12.2614	10.5
Story3	DL+WINDX	Y	9.3E-05	199	-66.358	4.6402	10.5
Story3	DL+WINDY	X	6.4E-05	205	-57	3.9	10.5
Story3	DL+WINDY	Y	0.000177	205	-57	3.9	10.5
Story3	DStIS1	X	5.6E-05	205	-57	3.9	10.5
Story3	DStIS1	Y	4E-05	199	-66.358	4.6402	10.5
Story3	DStIS2	X	6.3E-05	165	-62.945	12.2614	10.5
Story3	DStIS2	Y	4.9E-05	199	-66.358	4.6402	10.5
Story3	DWal1	X	5.6E-05	205	-57	3.9	10.5
Story3	DWal1	Y	4E-05	199	-66.358	4.6402	10.5
Story3	DWal2	X	6.3E-05	165	-62.945	12.2614	10.5
Story3	DWal2	Y	4.9E-05	199	-66.358	4.6402	10.5
Story3	DWal3	X	6.6E-05	205	-57	3.9	10.5
Story3	DWal4	X	2.6E-05	205	-57	3.9	10.5
Story3	DWal4	Y	0.000109	64	19.3914	-21.5363	10.5
Story3	DWal5	X	4.3E-05	205	-57	3.9	10.5
Story3	DWal5	Y	0.000116	103	45.8653	9.749	10.5
Story3	DWal6	X	5.6E-05	126	52.0956	-8.7178	10.5
Story3	DWal6	Y	2.4E-05	199	-66.358	4.6402	10.5
Story3	DWal7	X	5.6E-05	126	52.0956	-8.7178	10.5
Story3	DWal7	Y	2.4E-05	199	-66.358	4.6402	10.5
Story3	DWal8	Y	0.000112	103	45.8653	9.749	10.5
Story3	DWal9	Y	0.000112	103	45.8653	9.749	10.5
Story3	DStID1	X	3.5E-05	205	-57	3.9	10.5
Story3	DStID1	Y	2.5E-05	199	-66.358	4.6402	10.5
Story2	Dead	X	2.2E-05	165	-62.945	12.2614	7
Story2	Dead	Y	4.6E-05	199	-66.358	4.6402	7
Story2	Live	X	1.1E-05	165	-62.945	12.2614	7
Story2	Live	Y	1.9E-05	199	-66.358	4.6402	7
Story2	windx	X	6.2E-05	165	-62.945	12.2614	7
Story2	windx	Y	2.7E-05	199	-66.358	4.6402	7
Story2	windy	Y	0.000105	137	3.2246	-52.7215	7
Story2	earthquake x	X	1549067566	155	-62	4.3	7
Story2	earthquake y	Y	0.000358	95	52.7697	-2.304	7
Story2	Comb1	X	4.4E-05	165	-62.945	12.2614	7
Story2	Comb1	Y	8.5E-05	199	-66.358	4.6402	7
Story2	SL	X	3.3E-05	165	-62.945	12.2614	7
Story2	SL	Y	6.5E-05	199	-66.358	4.6402	7
Story2	DL+EQX	X	1549067566	155	-62	4.3	7
Story2	DL+EXY	Y	0.000351	126	52.0956	-8.7178	7
Story2	DL+WINDX	X	0.00012	165	-62.945	12.2614	7
Story2	DL+WINDX	Y	8.4E-05	199	-66.358	4.6402	7
Story2	DL+WINDY	X	2.6E-05	205	-57	3.9	7
Story2	DL+WINDY	Y	0.00017	130	3.2246	52.7215	7
Story2	DStIS1	X	1.8E-05	165	-62.945	12.2614	7
Story2	DStIS1	Y	3E-05	199	-66.358	4.6402	7

Story	Load Case/Combo	Direction	Drift	Label	X m	Y m	Z m
Story2	DStIS2	X	7.3E-05	165	-62.945	12.2614	7
Story2	DStIS2	Y	4.6E-05	199	-66.358	4.6402	7
Story2	DWal1	X	1.8E-05	165	-62.945	12.2614	7
Story2	DWal1	Y	3E-05	199	-66.358	4.6402	7
Story2	DWal2	X	7.3E-05	165	-62.945	12.2614	7
Story2	DWal2	Y	4.6E-05	199	-66.358	4.6402	7
Story2	DWal3	X	6.3E-05	137	3.2246	-52.7215	7
Story2	DWal3	Y	1.6E-05	7	41.5663	18.5065	7
Story2	DWal4	Y	0.000106	130	3.2246	52.7215	7
Story2	DWal5	Y	0.00011	10	36.8103	26.7442	7
Story2	DWal6	X	6.2E-05	165	-62.945	12.2614	7
Story2	DWal6	Y	2.7E-05	199	-66.358	4.6402	7
Story2	DWal7	X	6.2E-05	165	-62.945	12.2614	7
Story2	DWal7	Y	2.7E-05	199	-66.358	4.6402	7
Story2	DWal8	Y	0.000105	137	3.2246	-52.7215	7
Story2	DWal9	Y	0.000105	137	3.2246	-52.7215	7
Story2	DStID1	X	1.1E-05	165	-62.945	12.2614	7
Story2	DStID1	Y	1.9E-05	199	-66.358	4.6402	7
Story1	Dead	X	6.2E-05	205	-57	3.9	3.5
Story1	Live	X	2.4E-05	205	-57	3.9	3.5
Story1	windx	X	8.8E-05	131	-3.2246	-52.7215	3.5
Story1	windx	Y	3E-05	199	-66.358	4.6402	3.5
Story1	windy	Y	0.000125	95	52.7697	-2.304	3.5
Story1	earthquake x	X	1549067566	155	-62	4.3	3.5
Story1	earthquake y	Y	0.00036	95	52.7697	-2.304	3.5
Story1	Comb1	X	0.000114	205	-57	3.9	3.5
Story1	SL	X	8.7E-05	205	-57	3.9	3.5
Story1	DL+EQX	X	1549067566	155	-62	4.3	3.5
Story1	DL+EXY	Y	0.000349	95	52.7697	-2.304	3.5
Story1	DL+WINDX	X	0.000161	205	-57	3.9	3.5
Story1	DL+WINDX	Y	5.4E-05	199	-66.358	4.6402	3.5
Story1	DL+WINDY	Y	0.000191	95	52.7697	-2.304	3.5
Story1	DStIS1	X	3.9E-05	205	-57	3.9	3.5
Story1	DStIS2	X	9E-05	205	-57	3.9	3.5
Story1	DStIS2	Y	3.1E-05	199	-66.358	4.6402	3.5
Story1	DWal1	X	3.9E-05	205	-57	3.9	3.5
Story1	DWal2	X	9E-05	205	-57	3.9	3.5
Story1	DWal2	Y	3.1E-05	199	-66.358	4.6402	3.5
Story1	DWal3	X	9.5E-05	137	3.2246	-52.7215	3.5
Story1	DWal4	Y	0.00012	126	52.0956	-8.7178	3.5
Story1	DWal5	Y	0.000131	95	52.7697	-2.304	3.5
Story1	DWal6	X	8.8E-05	131	-3.2246	-52.7215	3.5
Story1	DWal6	Y	3E-05	199	-66.358	4.6402	3.5
Story1	DWal7	X	8.8E-05	131	-3.2246	-52.7215	3.5
Story1	DWal7	Y	3E-05	199	-66.358	4.6402	3.5
Story1	DWal8	Y	0.000125	95	52.7697	-2.304	3.5
Story1	DWal9	Y	0.000125	95	52.7697	-2.304	3.5
Story1	DStID1	X	2.4E-05	205	-57	3.9	3.5

Table 3.3 - Story Forces

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story5	Dead	Top	13943.4686	-52.0548	95.4285	-3243.5989	3809.8618	22789.1355
Story5	Dead	Bottom	13859.0984	-51.6768	129.1792	-6034.5302	3430.6329	21836.9394
Story5	Live	Top	4270.3598	-59.7998	49.8675	-1426.6426	1201.3704	9089.6758

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story5	Live	Bottom	4212.3778	-54.0411	57.9723	-2456.1087	1083.4462	8660.4492
Story5	windx	Top	1.1831	92.2201	-5.3761	-71.8632	4.2603	-246.7099
Story5	windx	Bottom	21.3587	-123.9362	9.4129	-256.4228	-82.2095	-595.7226
Story5	windy	Top	38.9809	-37.1153	101.4752	3348.1954	352.4357	566.4558
Story5	windy	Bottom	0.5632	4.0123	43.9359	-1261.0157	-224.0315	935.7311
Story5	earthquake x	Top	3.9904	-253.026	-35.963	953.2603	12.0014	-972.6475
Story5	earthquake x	Bottom	87.2461	-163.4462	34.7614	-980.1283	-266.9461	-2090.6174
Story5	earthquake y	Top	135.0553	-107.4483	-296.2186	9980.1911	1292.4882	1995.0665
Story5	earthquake y	Bottom	3.0043	18.8701	245.6808	-3402.9005	195.7445	3018.0688
Story5	Comb1	Top	23564.7379	-158.1454	194.3023	-6174.9468	6494.0268	41890.4439
Story5	Comb1	Bottom	23370.7225	-148.4779	247.7706	-11171.2102	5850.2735	40061.0461
Story5	SL	Top	18213.8284	-111.8546	145.296	-4670.2415	5011.2322	31878.8113
Story5	SL	Bottom	18071.4762	-105.7179	187.1514	-8490.6389	4514.0792	30497.3887
Story5	DL+EQX	Top	13947.459	-305.0808	59.4655	-2290.3386	3821.8632	21816.4881
Story5	DL+EQX	Bottom	13946.3446	-215.123	163.9406	-7014.6586	3163.6868	19746.322
Story5	DL+EXY	Top	14078.5239	-159.503	-200.7901	6736.5922	5102.35	24784.202
Story5	DL+EXY	Bottom	13862.1028	-32.8067	374.86	-9437.4307	3626.3774	24855.0082
Story5	DL+WINDX	Top	12551.0148	100.7029	77.284	-3034.2202	3435.6922	20115.4862
Story5	DL+WINDX	Bottom	12507.3625	-244.807	131.3219	-5841.3537	2956.0344	18700.0893
Story5	DL+WINDY	Top	12611.4911	-106.2337	248.246	2437.8737	3992.7728	21416.5513
Story5	DL+WINDY	Bottom	12474.0897	-40.0894	186.5587	-7448.7024	2729.1193	21150.4152
Story5	DStIS1	Top	6832.5756	-95.6797	79.7881	-2282.6281	1922.1926	14543.4812
Story5	DStIS1	Bottom	6739.8044	-86.4658	92.7556	-3929.7739	1733.5139	13856.7188
Story5	DStIS2	Top	4271.5429	32.4203	44.4915	-1498.5058	1205.6307	8842.9659
Story5	DStIS2	Bottom	4233.7365	-177.9773	67.3852	-2712.5315	1001.2367	8064.7266
Story5	DWal1	Top	6832.5756	-95.6797	79.7881	-2282.6281	1922.1926	14543.4812
Story5	DWal1	Bottom	6739.8044	-86.4658	92.7556	-3929.7739	1733.5139	13856.7188
Story5	DWal2	Top	4271.5429	32.4203	44.4915	-1498.5058	1205.6307	8842.9659
Story5	DWal2	Bottom	4233.7365	-177.9773	67.3852	-2712.5315	1001.2367	8064.7266
Story5	DWal3	Top	4269.1766	-152.0199	55.2436	-1354.7794	1197.1101	9336.3856
Story5	DWal3	Bottom	4191.019	69.8951	48.5593	-2199.6859	1165.6557	9256.1718
Story5	DWal4	Top	4309.3406	-96.9151	151.3428	1921.5529	1553.8061	9656.1316
Story5	DWal4	Bottom	4212.9409	-50.0288	101.9082	-3717.1244	859.4147	9596.1803
Story5	DWal5	Top	4231.3789	-22.6845	-51.6077	-4774.838	848.9347	8523.22
Story5	DWal5	Bottom	4211.8146	-58.0534	14.0363	-1195.0929	1307.4777	7724.7182
Story5	DWal6	Top	1.1831	92.2201	-5.3761	-71.8632	4.2603	-246.7099
Story5	DWal6	Bottom	21.3587	-123.9362	9.4129	-256.4228	-82.2095	-595.7226
Story5	DWal7	Top	-1.1831	-92.2201	5.3761	71.8632	-4.2603	246.7099
Story5	DWal7	Bottom	-21.3587	123.9362	-9.4129	256.4228	82.2095	595.7226
Story5	DWal8	Top	38.9809	-37.1153	101.4752	3348.1954	352.4357	566.4558
Story5	DWal8	Bottom	0.5632	4.0123	43.9359	-1261.0157	-224.0315	935.7311
Story5	DWal9	Top	-38.9809	37.1153	-101.4752	-3348.1954	-352.4357	-566.4558
Story5	DWal9	Bottom	-0.5632	-4.0123	-43.9359	1261.0157	224.0315	-935.7311
Story5	DStID1	Top	4270.3598	-59.7998	49.8675	-1426.6426	1201.3704	9089.6758
Story5	DStID1	Bottom	4212.3778	-54.0411	57.9723	-2456.1087	1083.4462	8660.4492
Story4	Dead	Top	27267.614	-117.208	131.2165	-6621.3605	8185.6437	58295.015
Story4	Dead	Bottom	26989.8191	-127.9528	107.6774	-4951.9612	7399.8478	54959.1835
Story4	Live	Top	16257.5254	-76.4755	59.9352	-2622.1074	3492.9591	24094.7512
Story4	Live	Bottom	16104.0665	-79.5189	49.918	-1974.5981	2782.9604	20315.7902
Story4	windx	Top	3.9066	-76.386	-87.1703	383.7683	185.6477	-1918.0504
Story4	windx	Bottom	16.5362	-277.382	-33.5989	-469.2521	-76.1949	-3240.8214
Story4	windy	Top	7.3926	-94.4952	-851.5418	2270.4615	3805.5929	1037.6176
Story4	windy	Bottom	63.2081	-19.191	-696.6117	166.6244	3270.927	4345.0391
Story4	earthquake x	Top	21.3357	-874.4739	-310.6789	1465.6963	612.5225	-7540.8005

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story4	earthquake x	Bottom	72.6604	-687.5072	-127.2612	-1428.3448	-398.2892	-12519.4417
Story4	earthquake y	Top	24.1066	-315.3992	-3582.176	4061.0421	14175.6995	3402.0211
Story4	earthquake y	Bottom	192.6684	-75.2602	-2299.3104	-995.0668	12563.1871	13126.3211
Story4	Comb1	Top	58733.1776	-263.0104	253.3561	-12141.0044	15411.507	108505.62
Story4	Comb1	Bottom	58154.2894	-280.7736	209.0816	-9101.7103	13332.554	98456.2845
Story4	SL	Top	43525.1395	-193.6835	191.1517	-9243.4679	11678.6028	82389.7662
Story4	SL	Bottom	43093.8857	-207.4717	157.5954	-6926.5592	10182.8082	75274.9737
Story4	DL+EQX	Top	27288.9498	-991.6818	-179.4624	-5155.6642	8798.1662	50754.2145
Story4	DL+EQX	Bottom	27062.4795	-815.46	-19.5838	-6380.306	7001.5586	42439.7418
Story4	DL+EXY	Top	27291.7206	-432.6072	-3450.9595	-2560.3184	22361.3432	61697.0362
Story4	DL+EXY	Bottom	27182.4876	-203.2129	-2191.633	-5947.028	19963.0349	68085.5046
Story4	DL+WINDX	Top	24547.1032	-227.7047	-21.3777	-5345.1952	7664.1157	49396.633
Story4	DL+WINDX	Bottom	24317.2951	-558.9687	43.1514	-5207.5683	6537.9512	44277.9509
Story4	DL+WINDY	Top	24552.6809	-256.6795	-1244.372	-2326.4861	13456.028	54125.7018
Story4	DL+WINDY	Bottom	24391.9701	-145.8631	-1017.6691	-4190.166	11893.3463	56415.3277
Story4	DStIS1	Top	26012.0407	-122.3608	95.8963	-4195.3718	5588.7346	38551.6019
Story4	DStIS1	Bottom	25766.5064	-127.2303	79.8687	-3159.3569	4452.7366	32505.2643
Story4	DStIS2	Top	16261.432	-152.8615	-27.2352	-2238.3391	3678.6068	22176.7008
Story4	DStIS2	Bottom	16120.6027	-356.9009	16.319	-2443.8501	2706.7655	17074.9688
Story4	DWal1	Top	26012.0407	-122.3608	95.8963	-4195.3718	5588.7346	38551.6019
Story4	DWal1	Bottom	25766.5064	-127.2303	79.8687	-3159.3569	4452.7366	32505.2643
Story4	DWal2	Top	16261.432	-152.8615	-27.2352	-2238.3391	3678.6068	22176.7008
Story4	DWal2	Bottom	16120.6027	-356.9009	16.319	-2443.8501	2706.7655	17074.9688
Story4	DWal3	Top	16253.6189	-0.0896	147.1055	-3005.8757	3307.3114	26012.8016
Story4	DWal3	Bottom	16087.5304	197.8631	83.5169	-1505.346	2859.1553	23556.6116
Story4	DWal4	Top	16264.9181	-170.9707	-791.6066	-351.6459	7298.552	25132.3688
Story4	DWal4	Bottom	16167.2746	-98.7099	-646.6938	-1807.9737	6053.8874	24660.8293
Story4	DWal5	Top	16250.1328	18.0197	911.477	-4892.5688	-312.6338	23057.1336
Story4	DWal5	Bottom	16040.8585	-60.3279	746.5297	-2141.2225	-487.9666	15970.7511
Story4	DWal6	Top	3.9066	-76.386	-87.1703	383.7683	185.6477	-1918.0504
Story4	DWal6	Bottom	16.5362	-277.382	-33.5989	-469.2521	-76.1949	-3240.8214
Story4	DWal7	Top	-3.9066	76.386	87.1703	-383.7683	-185.6477	1918.0504
Story4	DWal7	Bottom	-16.5362	277.382	33.5989	469.2521	76.1949	3240.8214
Story4	DWal8	Top	7.3926	-94.4952	-851.5418	2270.4615	3805.5929	1037.6176
Story4	DWal8	Bottom	63.2081	-19.191	-696.6117	166.6244	3270.927	4345.0391
Story4	DWal9	Top	-7.3926	94.4952	851.5418	-2270.4615	-3805.5929	-1037.6176
Story4	DWal9	Bottom	-63.2081	19.191	696.6117	-166.6244	-3270.927	-4345.0391
Story4	DStID1	Top	16257.5254	-76.4755	59.9352	-2622.1074	3492.9591	24094.7512
Story4	DStID1	Bottom	16104.0665	-79.5189	49.918	-1974.5981	2782.9604	20315.7902
Story3	Dead	Top	40203.9004	-90.5394	105.2335	-5246.5527	12444.53	91625.9299
Story3	Dead	Bottom	39960.0288	-91.3685	54.2583	-2299.4162	11540.0392	81826.9812
Story3	Live	Top	28162.2408	-64.9395	50.583	-2115.8278	4952.4417	36028.9025
Story3	Live	Bottom	28063.5321	-65.863	29.8095	-944.9159	4204.5733	29729.091
Story3	windx	Top	15.2002	-245.3439	-76.4231	460.8988	2.5787	-4239.2894
Story3	windx	Bottom	42.4328	-441.0981	-47.7045	-532.6168	-208.1971	-6046.8128
Story3	windy	Top	65.7011	-52.3124	-1129.0359	4072.7539	5142.2479	4261.8299
Story3	windy	Bottom	126.5662	-12.9201	-1227.9292	1800.2331	7112.7035	7415.2236
Story3	earthquake x	Top	70.0787	-1284.1387	-252.6377	1544.3044	-118.9648	-16028.7786
Story3	earthquake x	Bottom	163.2631	-1191.382	-175.5611	-1397.9024	-870.325	-22162.8803
Story3	earthquake y	Top	201.73	-159.4493	-4056.98	7231.3298	18974.2606	12843.5917
Story3	earthquake y	Bottom	408.93	-64.2975	-3825.6368	2840.1077	25222.1389	23306.6583
Story3	Comb1	Top	93304.2657	-212.5505	207.213	-9681.1878	22857.3427	167597.3598
Story3	Comb1	Bottom	92853.6859	-215.023	112.8053	-4271.1649	20575.3644	145758.923

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story3	SL	Top	68366.1412	-155.479	155.8165	-7362.3805	17396.9717	127654.8324
Story3	SL	Bottom	68023.5609	-157.2315	84.0679	-3244.3321	15744.6126	111556.0722
Story3	DL+EQX	Top	40273.9791	-1374.6782	-147.4042	-3702.2483	12325.5652	75597.1513
Story3	DL+EQX	Bottom	40123.2919	-1282.7505	-121.3028	-3697.3186	10669.7142	59664.1009
Story3	DL+EXY	Top	40405.6304	-249.9887	-3951.7465	1984.777	31418.7907	104469.5216
Story3	DL+EXY	Bottom	40368.9588	-155.666	-3771.3785	540.6915	36762.1781	105133.6395
Story3	DL+WINDX	Top	36207.8307	-474.0357	-27.5668	-3984.4594	11204.203	75680.4738
Story3	DL+WINDX	Bottom	36031.9184	-787.9886	-27.4948	-2921.6614	10052.9199	63969.3826
Story3	DL+WINDY	Top	36288.6322	-165.1853	-1711.7473	1794.5088	19427.6737	89282.2647
Story3	DL+WINDY	Bottom	36166.5318	-102.9039	-1915.8543	810.8983	21766.3609	85508.6408
Story3	DStIS1	Top	45059.5852	-103.9032	80.9328	-3385.3245	7923.9067	57646.244
Story3	DStIS1	Bottom	44901.6514	-105.3808	47.6953	-1511.8654	6727.3173	47566.5456
Story3	DStIS2	Top	28177.441	-310.2834	-25.8402	-1654.9291	4955.0204	31789.613
Story3	DStIS2	Bottom	28105.9649	-506.9611	-17.895	-1477.5327	3996.3762	23682.2782
Story3	DWal1	Top	45059.5852	-103.9032	80.9328	-3385.3245	7923.9067	57646.244
Story3	DWal1	Bottom	44901.6514	-105.3808	47.6953	-1511.8654	6727.3173	47566.5456
Story3	DWal2	Top	28177.441	-310.2834	-25.8402	-1654.9291	4955.0204	31789.613
Story3	DWal2	Bottom	28105.9649	-506.9611	-17.895	-1477.5327	3996.3762	23682.2782
Story3	DWal3	Top	28147.0406	180.4044	127.0061	-2576.7266	4949.863	40268.1919
Story3	DWal3	Bottom	28021.0993	375.2351	77.5141	-412.2991	4412.7705	35775.9038
Story3	DWal4	Top	28227.9419	-117.2519	-1078.4529	1956.9261	10094.6896	40290.7324
Story3	DWal4	Bottom	28190.0983	-78.7831	-1198.1197	855.3172	11317.2768	37144.3145
Story3	DWal5	Top	28096.5396	-12.6271	1179.6189	-6188.5817	-189.8062	31767.0726
Story3	DWal5	Bottom	27936.9659	-52.9429	1257.7388	-2745.149	-2908.1302	22313.8674
Story3	DWal6	Top	15.2002	-245.3439	-76.4231	460.8988	2.5787	-4239.2894
Story3	DWal6	Bottom	42.4328	-441.0981	-47.7045	-532.6168	-208.1971	-6046.8128
Story3	DWal7	Top	-15.2002	245.3439	76.4231	-460.8988	-2.5787	4239.2894
Story3	DWal7	Bottom	-42.4328	441.0981	47.7045	532.6168	208.1971	6046.8128
Story3	DWal8	Top	65.7011	-52.3124	-1129.0359	4072.7539	5142.2479	4261.8299
Story3	DWal8	Bottom	126.5662	-12.9201	-1227.9292	1800.2331	7112.7035	7415.2236
Story3	DWal9	Top	-65.7011	52.3124	1129.0359	-4072.7539	-5142.2479	-4261.8299
Story3	DWal9	Bottom	-126.5662	12.9201	1227.9292	-1800.2331	-7112.7035	-7415.2236
Story3	DStID1	Top	28162.2408	-64.9395	50.583	-2115.8278	4952.4417	36028.9025
Story3	DStID1	Bottom	28063.5321	-65.863	29.8095	-944.9159	4204.5733	29729.091
Story2	Dead	Top	53452.4991	-67.8249	47.4753	-2294.201	16493.076	118520.2187
Story2	Dead	Bottom	53261.1074	-64.397	42.5209	-2105.3437	15736.5919	110359.7157
Story2	Live	Top	40339.0572	-54.4632	25.3906	-932.8656	6305.7307	45472.8049
Story2	Live	Bottom	40270.5655	-54.4642	23.2326	-805.4096	5631.8245	39839.3845
Story2	windx	Top	41.6194	-513.9529	-75.8999	419.743	-87.3102	-8504.9387
Story2	windx	Bottom	81.7057	-699.4252	-52.9215	-528.8476	-427.145	-11246.2592
Story2	windy	Top	129.4696	-35.9714	-1724.2675	5202.3751	10908.3319	7294.8215
Story2	windy	Bottom	249.1187	-6.8983	-1835.6546	2382.9003	14623.6388	13872.2706
Story2	earthquake x	Top	163.32	-1933.5908	-233.726	1203.6108	-477.7998	-30432.8523
Story2	earthquake x	Bottom	289.9276	-1900.1115	-190.1694	-1166.8648	-1504.7326	-38851.3024
Story2	earthquake y	Top	417.4425	-97.5437	-5372.9019	8880.4005	37643.6324	22919.7385
Story2	earthquake y	Bottom	721.6015	-52.4266	-5338.3051	5032.4285	48262.6807	39209.9673
Story2	Comb1	Top	128685.4905	-168.5309	97.5952	-4245.6262	29880.8604	214980.7502
Story2	Comb1	Bottom	128346.2336	-164.4192	88.1971	-3815.0677	27894.8294	196174.6741
Story2	SL	Top	93791.5564	-122.288	72.8658	-3227.0666	22798.8067	163993.0236

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story2	SL	Bottom	93531.6729	-118.8612	65.7534	-2910.7532	21368.4163	150199.1002
Story2	DL+EQX	Top	53615.8191	-2001.4156	-186.2508	-1090.5902	16015.2762	88087.3663
Story2	DL+EQX	Bottom	53551.0349	-1964.5085	-147.6486	-3272.2085	14231.8592	71508.4133
Story2	DL+EXY	Top	53869.9416	-165.3686	-5325.4267	6586.1995	54136.7084	141439.9572
Story2	DL+EXY	Bottom	53982.7088	-116.8236	-5295.7843	2927.0848	63999.2725	149569.683
Story2	DL+WINDX	Top	48173.8402	-883.367	-78.712	-1393.1922	14704.0721	93060.2949
Story2	DL+WINDX	Bottom	48065.7257	-1177.0377	-46.4057	-2740.9655	13479.5007	81329.7294
Story2	DL+WINDY	Top	48314.4006	-118.5967	-2716.1003	6259.0192	32297.0995	118339.9113
Story2	DL+WINDY	Bottom	48333.5865	-68.9946	-2898.7786	1917.8312	37560.7548	121519.377
Story2	DStIS1	Top	64542.4916	-87.1411	40.6249	-1492.5849	10089.1691	72756.4878
Story2	DStIS1	Bottom	64432.9048	-87.1427	37.1721	-1288.6553	9010.9192	63743.0153
Story2	DStIS2	Top	40380.6766	-568.4161	-50.5093	-513.1226	6218.4205	36967.8662
Story2	DStIS2	Bottom	40352.2712	-753.8894	-29.689	-1334.2572	5204.6795	28593.1253
Story2	DWal1	Top	64542.4916	-87.1411	40.6249	-1492.5849	10089.1691	72756.4878
Story2	DWal1	Bottom	64432.9048	-87.1427	37.1721	-1288.6553	9010.9192	63743.0153
Story2	DWal2	Top	40380.6766	-568.4161	-50.5093	-513.1226	6218.4205	36967.8662
Story2	DWal2	Bottom	40352.2712	-753.8894	-29.689	-1334.2572	5204.6795	28593.1253
Story2	DWal3	Top	40297.4378	459.4897	101.2904	-1352.6086	6393.0409	53977.7436
Story2	DWal3	Bottom	40188.8598	644.961	76.1541	-276.562	6058.9695	51085.6437
Story2	DWal4	Top	40468.5269	-90.4346	-1698.877	4269.5095	17214.0626	52767.6264
Story2	DWal4	Bottom	40519.6842	-61.3625	-1812.4221	1577.4907	20255.4633	53711.6551
Story2	DWal5	Top	40209.5876	-18.4917	1749.6581	-6135.2407	-4602.6012	38177.9834
Story2	DWal5	Bottom	40021.4468	-47.5659	1858.8872	-3188.3099	-8991.8143	25967.114
Story2	DWal6	Top	41.6194	-513.9529	-75.8999	419.743	-87.3102	-8504.9387
Story2	DWal6	Bottom	81.7057	-699.4252	-52.9215	-528.8476	-427.145	-11246.2592
Story2	DWal7	Top	-41.6194	513.9529	75.8999	-419.743	87.3102	8504.9387
Story2	DWal7	Bottom	-81.7057	699.4252	52.9215	528.8476	427.145	11246.2592
Story2	DWal8	Top	129.4696	-35.9714	-1724.2675	5202.3751	10908.3319	7294.8215
Story2	DWal8	Bottom	249.1187	-6.8983	-1835.6546	2382.9003	14623.6388	13872.2706
Story2	DWal9	Top	-129.4696	35.9714	1724.2675	-5202.3751	-10908.3319	-7294.8215
Story2	DWal9	Bottom	-249.1187	6.8983	1835.6546	-2382.9003	-14623.6388	-13872.2706
Story2	DStID1	Top	40339.0572	-54.4632	25.3906	-932.8656	6305.7307	45472.8049
Story2	DStID1	Bottom	40270.5655	-54.4642	23.2326	-805.4096	5631.8245	39839.3845
Story1	Dead	Top	66635.9192	-95.6213	39.7791	-2514.9383	21170.4163	137965.8408
Story1	Dead	Bottom	66545.2964	-95.8595	47.0771	-3502.2131	20455.8641	130710.7715
Story1	Live	Top	52546.7427	-82.1671	21.9416	-1031.8455	12470.86	48071.09
Story1	Live	Bottom	52560.6394	-84.0481	20.4939	-1417.4874	11816.9782	42703.965
Story1	windx	Top	86.8796	-612.0799	-60.6219	734.0803	-274.3299	-13076.9494
Story1	windx	Bottom	144.5694	-834.6079	-82.0338	1044.3843	-1114.8563	-15855.5618
Story1	windy	Top	255.5145	-5.0864	-2069.3883	6030.2332	18001.2142	13712.9722
Story1	windy	Bottom	504.5623	-22.6976	-2470.3095	7436.7015	20456.5693	27794.0561
Story1	earthquake x	Top	303.0067	-1901.8476	-171.0931	2016.0506	-1068.0352	-44310.9859
Story1	earthquake x	Bottom	474.6859	-1997.8178	-252.8113	3224.1438	-3301.8289	-51993.4595
Story1	earthquake y	Top	739.8938	4.3079	-5651.7993	11917.9067	58081.7255	38764.9068
Story1	earthquake y	Bottom	1239.4981	-68.0804	-6350.1466	15915.8416	64193.3616	66515.8753
Story1	Comb1	Top	164037.8914	-246.213	82.8415	-4668.8788	45357.8755	242472.7529
Story1	Comb1	Bottom	163951.3786	-249.5084	89.2827	-6470.6355	43454.202	225179.2698
Story1	SL	Top	119182.6619	-177.7884	61.7207	-3546.7838	33641.2763	186036.9307
Story1	SL	Bottom	119105.9357	-179.9077	67.571	-4919.7004	32272.8423	173414.7365
Story1	DL+EQX	Top	66938.9259	-1997.469	-131.314	-498.8877	20102.3811	93654.8549

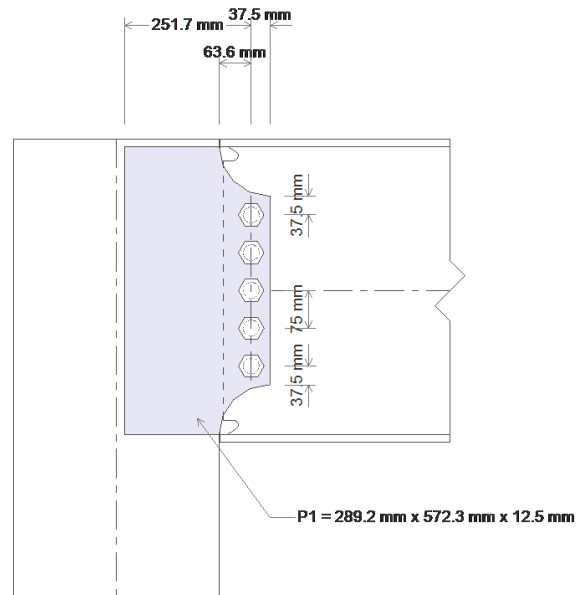
Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story1	DL+EQX	Bottom	67019.9822	-2093.6773	-205.7342	-278.0692	17154.0352	78717.312
Story1	DL+EXY	Top	67375.813	-91.3134	-5612.0202	9402.9684	79252.1418	176730.7476
Story1	DL+EXY	Bottom	67784.7945	-163.9399	-6303.0695	12413.6286	84649.2257	197226.6468
Story1	DL+WINDX	Top	60111.3346	-1065.3871	-61.1939	-1088.916	18614.4468	103246.1376
Story1	DL+WINDX	Bottom	60122.0778	-1421.6462	-88.8847	-1480.9768	16626.5077	92270.7955
Story1	DL+WINDY	Top	60381.1504	-94.1974	-3275.2201	7384.9287	47855.3173	146110.0122
Story1	DL+WINDY	Bottom	60698.0664	-122.5897	-3910.1259	8746.7306	51140.7886	162110.1841
Story1	DStIS1	Top	84074.7884	-131.4674	35.1066	-1650.9529	19953.376	76913.744
Story1	DStIS1	Bottom	84097.023	-134.477	32.7902	-2267.9798	18907.1651	68326.344
Story1	DStIS2	Top	52633.6224	-694.2471	-38.6803	-297.7653	12196.5301	34994.1405
Story1	DStIS2	Bottom	52705.2088	-918.6561	-61.54	-373.103	10702.1219	26848.4032
Story1	DWal1	Top	84074.7884	-131.4674	35.1066	-1650.9529	19953.376	76913.744
Story1	DWal1	Bottom	84097.023	-134.477	32.7902	-2267.9798	18907.1651	68326.344
Story1	DWal2	Top	52633.6224	-694.2471	-38.6803	-297.7653	12196.5301	34994.1405
Story1	DWal2	Bottom	52705.2088	-918.6561	-61.54	-373.103	10702.1219	26848.4032
Story1	DWal3	Top	52459.8631	529.9128	82.5635	-1765.9258	12745.1899	61148.0394
Story1	DWal3	Bottom	52416.0699	750.5598	102.5277	-2461.8717	12931.8345	58559.5268
Story1	DWal4	Top	52802.2572	-87.2535	-2047.4467	4998.3877	30472.0741	61784.0622
Story1	DWal4	Bottom	53065.2016	-106.7458	-2449.8157	6019.2141	32273.5475	70498.0211
Story1	DWal5	Top	52291.2283	-77.0807	2091.3299	-7062.0788	-5530.3542	34358.1178
Story1	DWal5	Bottom	52056.0771	-61.3505	2490.8034	-8854.1888	-8639.5911	14909.9089
Story1	DWal6	Top	86.8796	-612.0799	-60.6219	734.0803	-274.3299	-13076.9494
Story1	DWal6	Bottom	144.5694	-834.6079	-82.0338	1044.3843	-1114.8563	-15855.5618
Story1	DWal7	Top	-86.8796	612.0799	60.6219	-734.0803	274.3299	13076.9494
Story1	DWal7	Bottom	-144.5694	834.6079	82.0338	-1044.3843	1114.8563	15855.5618
Story1	DWal8	Top	255.5145	-5.0864	-2069.3883	6030.2332	18001.2142	13712.9722
Story1	DWal8	Bottom	504.5623	-22.6976	-2470.3095	7436.7015	20456.5693	27794.0561
Story1	DWal9	Top	-255.5145	5.0864	2069.3883	-6030.2332	-18001.2142	-13712.9722
Story1	DWal9	Bottom	-504.5623	22.6976	2470.3095	-7436.7015	-20456.5693	-27794.0561
Story1	DStID1	Top	52546.7427	-82.1671	21.9416	-1031.8455	12470.86	48071.09

12.3.1.2 Connection Design Summary

Connection Design: B39-CJ

Story: Story1

Design Code: AISC 360-10



Beam-Column Moment Minor Axis Connection

Summary of results

	Design Check Type	D/C Ratio	Result	Reference
1	Beam design flexural strength	0.037	Passed	Spec. Eq F13-1
2	Strength of bolt group	0.032	Passed	Pg 7-18 AISC manual
3	Shear yielding of web plate	0.011	Passed	J4-3
4	Shear rupture of web plate	0.017	Passed	J4-4
5	Block shear rupture strength of web plate	0.031	Passed	J4-5
6	Design strength of weld	0.015	Passed	J2-3
7	Weld strength at tension flange	0.189	Passed	J2-3
8	Shear rupture of column web	0.145	Passed	Manual Eq 9-2

Material Properties

Beam W24X68	A992Fy50	$F_y = 344.74$ MPa	$F_u = 448.16$ MPa
Column W14X283	A992Fy50	$F_y = 344.74$ MPa	$F_u = 448.16$ MPa
Web Plate	A992Fy50	$F_y = 344.74$ MPa	$F_u = 448.16$ MPa

Geometric Properties

Beam W24X68	$t_w = 10.5$ mm	$d = 602$ mm	$t_f = 377.4$ mm	$b_f = 227.8$ mm
Column W14X283	$t_w = 32.8$ mm	$d = 424.2$ mm	$t_f = 52.6$ mm	$b_f = 408.9$ mm
Preferences	$s = 75$ mm	$L_{ev} = 37.5$ mm	$L_{eh} = 37.5$ mm	

Bolts, Plate & Weld

Weld	Size, $D(1/16) = 100$ mm	
Web Plate	Thickness, $t = 12.5$ mm	
Bolt	Type = A325-N	diameter, $d_b = 32$ mm
Hole	Type = STD	diameter, $d_h = 33.3$ mm

12.4 Appendix D; Geotechnical design

12.4.1 Deep foundations; Other pile diameter calculations

Pile No.	D (m)	Ap (m ²)	Rc (KN/m ²)	qall (rock) Kpa	Qp (KN)
1	0.3	0.0707	11000	880	62.204
2	0.4	0.1257	11000	880	110.584
3	0.5	0.1963	11000	880	172.788
4	0.6	0.2827	11000	880	248.814
Pile No.	P (m)	κ	ϕ	θ	L'
1	0.94	0.3	38	45	4.5
2	1.26	0.3	38	45	6
3	1.57	0.3	38	45	7.5
4	1.88	0.3	38	45	9
Pile No.	σ	Tan(Φ)	α	f(Kpa)	Cu
1	94.5	0.781285627	1	21.62474326	30
2	126	0.781285627	1	28.83299101	30
3	157.5	0.781285627	1	36.04123876	30
4	189	0.781285627	1	43.24948651	30
Pile No.	Qs sand (KN)	Qs Clay (KN)	Qs total (KN)	Qu (KN)	Qall (KN)
1	122.285	622.035	744.320	806.524	201.631
2	217.396	829.380	1046.776	1157.360	289.340
3	339.681	1036.726	1376.406	1549.194	387.298
4	489.140	1244.071	1733.211	1982.025	495.506

Main Structure Forces (KN)			
No.	Inner	Middle	Outer
1	75	2821	1760
2	87	2846	2124
3	92	2846	2117
4	107	2846	2114
5	125	2857	2117
6	356	2867	2123
7	355	2859	1760
8	358	2862	1756
9	349	2897	2150
10	358	2878	2132
11	349	2867	2131
12	149	2862	2127
13	136	2869	2130
14	131	2847	2135
15	128	2842	1775
16	131	2849	1766
17	126	2849	2127
18	130	2845	2117
19	128	2859	2115
20	130	2867	2119
21	132	2867	295
22	139	2856	2336
23	335	2868	2309

24	361	1854	2101
25	375	2850	2116
26	397	2845	2117
27	143	2844	2114
28	116	2849	2118
29	101	2844	2123
30	86	2823	1768

12.4.2 Soil Pressure Data

Stairs Structure			
No.	Top	Bottom	Right
1	553	635	514
2	660	577	389
3	662	574	334
4	55	633	460

Elevator	
No.	
1	204
2	1096
3	723
4	757
5	265
6	374
8	400

Structure Calculation		
D (m)	R (m)	Area (m ²)
94	47	6939.77817
Q (KN)	I _x (m ⁴)	I _y (m ⁴)
156758	3832492.495	3832492.495
M _y (KN.m)	M _x (KN.m)	Soil qall(Kpa)
120206.45	79151.73	56451.5
e _x (m)	e _y (m)	
0.767	0.505	
X' (m)	Y' (m)	
47.767	47.505	

Main Structure Forces (KN) , Coordinate (m), and Soil Point Pressure (Kpa)

No.	Inner						
	Forces	X'	Y'	X	Y	q(x,y) (Kpa)	Check
1	75	47	28.92	1.33	-12.9	22.364	SAFE
2	87	49.34	31.58	3.96	-12.18	22.461	SAFE
3	92	51.95	34.42	6.41	-11.09	22.560	SAFE
4	107	54.61	36.65	8.57	-9.52	22.661	SAFE
5	125	57.35	39.39	10.36	-7.53	22.758	SAFE
6	356	59.76	42.05	11.7	-5.21	22.848	SAFE
7	355	62.42	44.66	12.53	-2.66	22.926	SAFE
8	358	65.08	47	12.81	0	22.990	SAFE
9	349	65.08	47	12.53	2.66	23.036	SAFE
10	358	62.42	49.34	11.7	5.21	23.063	SAFE
11	349	59.58	51.95	10.36	7.53	23.069	SAFE
12	149	57.35	54.61	8.4	9.5	23.048	SAFE
13	136	54.61	57.61	6.4	11.09	23.018	SAFE
14	131	51.95	59.42	3.96	12.18	22.964	SAFE
15	128	49.34	65.08	1.33	12.73	22.893	SAFE
16	131	47	65.08	-1.33	12.74	22.810	SAFE
17	126	44.66	62.42	-3.96	12.18	22.716	SAFE
18	130	42.05	59.58	-6.41	11.09	22.616	SAFE
19	128	39.39	57.35	-8.57	9.52	22.516	SAFE
20	130	36.65	54.61	-10.36	7.53	22.419	SAFE
21	132	34.24	51.95	-11.7	5.21	22.329	SAFE
22	139	31.58	49.34	-12.53	2.66	22.250	SAFE
23	335	28.92	47	-12.81	0	22.187	SAFE

24	361	28.92	44.66	-12.53	-2.66	22.140	SAFE
25	375	31.58	42.05	-11.7	-5.21	22.114	SAFE
26	397	34.42	39.39	-10.36	-7.53	22.108	SAFE
27	143	36.65	36.65	-8.4	-9.5	22.129	SAFE
28	116	39.39	34.24	-6.4	-11.09	22.159	SAFE
29	101	42.05	31.52	-3.96	-12.18	22.213	SAFE
30	86	44.66	32.62	-1.33	-12.9	22.280	SAFE

Main Structure Forces (KN) , Coordinate (m), and Soil Point Pressure (Kpa)							
Middle							
Forces	X'	Y'	X	Y	q(x,y) (Kpa)	Check	
2821	47	17.93	3.03	-28.82	22.088	SAFE	
2846	53.22	19.31	8.96	-27.56	22.300	SAFE	
2846	59.05	21.68	14.49	-25.09	22.525	SAFE	
2846	64.12	25.23	19.39	-21.54	22.752	SAFE	
2857	68.77	29.88	23.44	-17.03	22.972	SAFE	
2867	72.32	34.95	26.47	-11.79	23.175	SAFE	
2859	74.69	40.78	28.346	-6.03	23.353	SAFE	
2862	76.07	47	28.98	0	23.497	SAFE	
2897	76.07	47	28.35	6.03	23.602	SAFE	
2878	74.69	53.22	26.48	11.79	23.662	SAFE	
2867	72.32	59.05	23.45	17.03	23.676	SAFE	

2862	68.77	64.12	19.39	21.54	23.641	SAFE
2869	64.12	68.77	14.49	25.09	23.561	SAFE
2847	59.05	72.32	8.96	27.56	23.439	SAFE
2842	53.22	74.69	3.03	28.82	23.279	SAFE
2849	47	76.07	-3.03	28.82	23.089	SAFE
2849	40.78	76.07	-8.96	27.56	22.876	SAFE
2845	34.95	74.69	-14.49	25.09	22.652	SAFE
2859	29.88	72.32	-19.39	21.54	22.425	SAFE
2867	25.23	68.77	-23.44	17.03	22.205	SAFE
2867	21.68	64.12	-26.47	11.79	22.002	SAFE
2856	19.31	59.05	-28.346	6.03	21.824	SAFE
2868	17.93	53.22	-28.98	0	21.679	SAFE
1854	17.93	47	-28.35	-6.03	21.575	SAFE
2850	19.31	40.78	-26.48	-11.79	21.514	SAFE
2845	21.68	34.95	-23.45	-17.03	21.501	SAFE
2844	25.23	29.88	-19.39	-21.54	21.535	SAFE
2849	29.88	25.23	-14.49	-25.09	21.616	SAFE
2844	34.95	21.68	-8.96	-27.56	21.738	SAFE
2823	40.78	19.31	-3.03	-28.82	21.898	SAFE

Main Structure Forces (KN) , Coordinatate (m), and Soil Point Prussure (Kpa)

Outer						
Forces	X'	Y'	X	Y	q(x,y) (Kpa)	Check
1760	47	1.51	4.75	-45.25	21.803	SAFE
2124	56.46	3.81	14.06	-43.27	22.136	SAFE
2117	65.48	7.63	22.75	-39.4	22.488	SAFE
2114	74.69	13.1	30.45	-33.81	22.845	SAFE
2117	80.9	19.31	36.81	-26.74	23.191	SAFE
2123	86.37	28.52	41.57	-18.51	23.510	SAFE
1760	90.19	37.54	44.51	-9.46	23.789	SAFE
1756	92.49	47	45.5	0	24.015	SAFE
2150	92.5	47	44.51	9.46	24.180	SAFE
2132	90.19	56.46	41.57	18.51	24.274	SAFE
2131	86.37	65.48	36.81	26.74	24.295	SAFE
2127	80.9	74.69	30.45	33.81	24.242	SAFE
2130	74.69	80.9	22.75	39.4	24.116	SAFE
2135	65.48	86.37	14.06	43.27	23.923	SAFE
1775	56.46	90.19	4.75	45.25	23.672	SAFE
1766	47	92.49	-4.75	45.25	23.374	SAFE
2127	37.54	92.5	-14.06	43.27	23.041	SAFE
2117	28.52	90.19	-22.75	39.4	22.688	SAFE
2115	19.31	86.37	-30.45	33.81	22.332	SAFE

2119	13.1	80.9	-36.81	26.74	21.986	SAFE
295	7.63	74.69	-41.57	18.51	21.667	SAFE
2336	3.81	65.48	-44.51	9.46	21.388	SAFE
2309	1.51	56.46	-45.5	0	21.161	SAFE
2101	1.51	47	-44.51	-9.46	20.997	SAFE
2116	3.81	37.54	-41.57	-18.51	20.902	SAFE
2117	7.63	28.52	-36.81	-26.74	20.882	SAFE
2114	13.1	19.31	-30.45	-33.81	20.935	SAFE
2118	19.31	13.1	-22.75	-39.4	21.061	SAFE
2123	28.52	7.63	-14.06	-43.27	21.254	SAFE
1768	37.54	3.81	-4.75	-45.25	21.505	SAFE

Stairs Structure Forces (KN), Cooridenate (m), and Soil Point Prussure (Kpa)

No.	Top						
	Forces	X'	Y'	X	Y	q(x,y) (Kpa)	Check
1	553	71.95	85.4	2.86	46.81	23.645	SAFE
2	660	71.95	87.53	3.22	52.72	23.778	SAFE
3	662	67.49	87.53	-3.22	52.72	23.576	SAFE
4	55	67.49	85.4	-2.86	46.81	23.465	SAFE

Stairs Structure Forces (KN), Coordinate (m), and Soil Point Pressure (Kpa)

Bottom

Forces	X'	Y'	X	Y	q(x,y) (Kpa)	Check
635	25.54	6.02	3.22	-52.72	21.601	SAFE
577	25.54	8.61	2.86	-46.81	21.711	SAFE
574	21.56	8.61	-2.86	-46.81	21.532	SAFE
633	21.56	6.02	-3.22	-52.72	21.399	SAFE

Stairs Structure Forces (KN), Coordinate (m), and Soil Point Pressure (Kpa)

Right

Forces	X'	Y'	X	Y	q(x,y) (Kpa)	Check
514	87.26	5.85	52.09	-8.72	24.042	SAFE
389	87.26	8.65	52.76	-2.3	24.196	SAFE
334	86.99	8.65	46.8	-2.3	24.009	SAFE
460	86.99	58.25	46.24	-7.74	23.879	SAFE

Structure	Slab thickness (mm)	Cover (mm)	d (mm)	b (mm)	M max (KN.m)
Main Structure	280	50	230	1000	50.811
Stairs	153	50	103	1000	11.34
Elevator	229	50	179	1000	21.371
Pedestrian	229	50	179	1000	44.357
Ramp	500	50	450	1000	121
Structure	M max (N.mm)	κ (N/mm ²)	f'c (Mpa)	fy (Mpa)	ρ
Main Structure	50811000	1.067233774	28	420	0.00260
Stairs	11340000	1.187670846	28	420	0.00290
Elevator	21371000	0.741099078	28	420	0.00179
Pedestrian	44357000	1.538202789	28	420	0.00379
Ramp	121000000	0.663923182	28	420	0.00160
Structure	ρ_{min}	ρ_{max}	As (mm ² /m)	As min (mm ² /m)	As max (mm ² /m)
Main Structure	0.000476	0.016250	598.1638872	109.5238095	3737.5
Stairs	0.000476	0.016250	298.9164215	49.04761905	1673.75
Elevator	0.000476	0.016250	320.9262838	85.23809524	2908.75
Pedestrian	0.000476	0.016250	678.2430424	85.23809524	2908.75
Ramp	0.000476	0.016250	721.5549239	214.2857143	7312.5

12.5 Appendix E; Reinforcement

12.5.1 Areas of Steel Reinforcement with corresponding Rebar Number and Diameters:

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

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

TABLE B.4 Areas of Groups of Standard Metric Bars (mm²)

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

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

12.5.2 Slab Reinforcement

Slab Reinforcement (1mx1m)					
Structure	As (mm ² /m) required	No. Bar	Diameter (mm)	As (mm ²) provided	As (mm ² /m) provided
Main Structure	598.1639	10	10	710	710
Stairs	298.9164	5	10	355	355
Elevator	320.9263	5	10	355	355
Pedestrian	678.2430	10	10	710	710
Ramp	721.5549	6	13	774	774
Structure	Area of one bar (mm ²)	Clear Spacing Max (mm)	Clear Spacing Min (mm)	Clear Spacing used (mm)	Spacing c/c (mm)
Main Structure	78.540	131.302	25	95	100
Stairs	78.540	262.748	25	180	185
Elevator	78.540	244.729	25	180	185
Pedestrian	78.540	115.799	25	95	100
Ramp	132.732	183.953	25	100	106.5

12.5.3 Shear Wall Reinforcement

Shear Wall Reinforcement (1mx1m)				
Structure	Location	As (mm ² /m) required	No. Bar	Diameter (mm)
Main Structure	Top	13950	10	43
	Bottom	13950	10	43
Stairs	Top	3009	20	16
	Bottom	3009	20	16
Elevator	Top	9693	19	25
	Bottom	9693	19	25
Structure	As (mm ²) provided	As (mm ² /m) provided	Area of one bar (mm ²)	Clear Spacing Max (mm)
Main Structure	14742	14520	1452.201	104.100
	14742	14520	1452.201	104.100
Stairs	3980	3980	201.062	66.820
	3980	3980	201.062	66.820
Elevator	9690	9690	490.874	50.642
	9690	9690	490.874	50.642
Structure	Clear Spacing Min (mm)	Clear Spacing used (mm)	Spacing c/c (mm)	
Main Structure	25	55	76.5	
	25	55	76.5	
Stairs	25	30	38	
	25	30	38	
Elevator	25	26	38.5	
	25	26	38.5	

12.5.4 Raft Slab Reinforcement

Mat Reinforcement			
Forces of the Critical Strips (KN)			
No.	Inner	Middle	Outer
1	75	2821	1760
16	131	2849	1766
9	358	2897	2150
23	335	2868	2309

Soil Point Pressure at the critical Strips (KPa)				
No.	Inner	Middle	Outer	direction
1	22.36	22.08	21.8	Y-direction
16	22.8	23.08	23.37	
9	23.03	23.6	24.1	X-direction
23	2.18	21.67	21.16	

q av (Kpa)	soil reaction (KN)	Total column load (KN)	Average load (KN)	qav (modi) (KPa)	Load factor; F	Soil reaction per unit length (KN/m)	Direction
22.585	20380.704	9402	14891.352	16.502	1.584	158.419	Y-direction
22.63	20421.312	10917	15669.156	17.364	1.435	166.693	X-direction

Mat Reinforcement

Forces of the Critical Strips (KN)							
No.	Inner	Middle	Outer	Inner load (modi)	Middle load (modi)	Outer load (modi)	direction
1	75	2821	1760	118.789	4468.039	2787.575	Y-direction
16	131	2849	1766	207.484	4512.387	2797.078	
9	358	2897	2150	513.837	4158.060	3085.892	X-direction
23	335	2868	2309	480.825	4116.437	3314.105	

Direction	Positive Moment (KN.m)	Negative Moment (KN.m)	Cover min (mm)	Bar Diameter (mm)	D (theory) (mm)	D (cal) (mm)	D (actual) (mm)
Y-direction	6815.394	5177.5563	76	25	1300	1401	1500
X-direction	7171.353	8279.7445	76	25	1300	1401	1500

Positive moment mat Reinforcement per Unit Length							
M' (KN.m/m)	Load Factor	Mu (KN.m/m)	ϕ	f'c (Kpa)	fy (Kpa)	a	As (mm ² /m)
747.016	1.7	1269.927	0.9	28000	420000	0.0039	2223
Negative moment mat Reinforcement per Unit Length							
M' (KN.m/m)	Load Factor	Mu (KN.m/m)	ϕ	f'c (Kpa)	fy (Kpa)	a	As (mm ² /m)
862.473	1.7	1466.205	0.9	28000	420000	0.0454	25878

Spacing	Diameter (mm)	Spacing (mm) c/c	As provided (mm ² /m)
Top	25	175	2805.7143
Bottom	25	175	2805.7143

Purposefully Left Blank