

**Design of 8-storey hotel “Asyl Nomad” in Houston,
Texas, USA
Final Report**

**Bachelor of Engineering
(Civil and Environmental Engineering)**



Group 6

Aisha Nurymgali

Alikhan Kurmangaliyev

Arsen Bekzhan

Assylbek Bakzhan

Azamat Yerassyluly

Sanzhar Urmanov

Yeladil Yelaman

Declaration

We hereby declare that this report, entitled “Design of the multi-story hotel 'Asyl Nomad' in Humble, Houston, Texas” is the result of our own project work except for quotations and citations which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University

Aisha Nurymgali



Alikhan Kurmangaliyev



Arsen Bekzhan



Assylbek Bakzhan



Azamat Yerassyluly



Sanzhar Urmanov



Yeladil Yelaman



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Abstract

Humble, Texas, continues to grow as part of the expanding Greater Houston metropolitan area, attracting new businesses and investors. To meet the increasing demand for modern hospitality and commercial space, it is proposed to construct an eight-story hotel with one basement level at 6819 Manor Way, Humble, Texas. To implement the project, the existing vacant lot will be cleared and prepared for development. The building will include contemporary guest rooms, a restaurant and café on the ground floor, and a spacious lobby for the comfort of visitors. The surrounding area will be landscaped using sustainable and green design strategies to ensure the project remains environmentally responsible. This report presents the design project, covering the structural, architectural, geotechnical, environmental, and construction management aspects of the proposed development.

1. Introduction

The capstone project aims to design an 8-story hotel located at 6819 Manor Way, Humble, Houston, Texas. Along with structural design, it consists of geotechnical design, environmental engineering, and construction management. The group members and the job distribution are presented below in Table 1.1

Table 1.1. Distribution of responsibilities

Section	Weight, %	Members
Architectural and Structural	35	Azamat Yerassyluly Sanzhar Urmanov Yeladil Yelaman
Geotechnical	35	Assylbek Bakzhan Alikhan Kurmangaliyev
Environmental	15	Aisha Nurymgali
Construction Management	15	Arsen Bekzhan

To address all requirements for the structural design part of the project, various design codes were used, including the International Building Code, ASCE-7-22, the Texas Industrialized Housing and Buildings (IHB) Code, and ACI 318-19 - Building Code Requirements for Structural Concrete. Additionally, the LEED grade met the minimum requirements for sustainable architectural design. In the geotechnical section, multiple calculations were performed to determine the appropriate foundation type and to design it. The construction manager evaluated the hotel construction's profitability, conducting feasibility studies and scheduling, while the environmental team conducted multiple literature reviews on the quantity and composition of waste for occupancy type R1.

The project aims to design a safe, modern, and functional hotel for all visitors to Houston, Texas. The objectives are

1. To assess the geotechnical report provided and determine an appropriate foundation design.

2. To conduct an in-depth feasibility study for the chosen site and type of construction.
3. To guarantee a sustainable design in alignment with LEED standards.
4. To outline a thorough planning and scheduling process for the construction project.
5. To create the structural and architectural design for both the exterior and interior of the facility in compliance with relevant codes and regulations.

Problem Statement

The strategic planning of the hotel at 6819 Manor Way in Humble is a direct response to the obvious gap in the market in one of the most important parts of Houston. The particular place is highly effective and well-located, as it is near George Bush Intercontinental Airport. This special location will guarantee a large number of customers, which will be beneficial for both business and leisure travellers in search of convenient accommodation.

Houston is a city worth hosting with its highly developed, diversified economy, which is an international energy, aerospace, and healthcare centre. This financial strength brings about a strong and all-year-round business travelling demand. Moreover, the stable increase in the number of tourists in Houston supports the necessity of temporary accommodation. The project, by creating a limited service hotel on this location, will be able to tap into a clearly defined opportunity that will meet an essential need, utilizing a viable and strategically located hospitality solution.

2. Architectural Design

2.1 Overview of the building

The intended project is a 1-basement 8-story hotel on 6819 Manor Way, Humble (Houston), Floors 2-8 (3 m floor each): guest rooms under three categories, i.e., Standard, Standard +, and Luxury suites. The hotel has a total of 78 rooms. The architectural design emphasizes functionality, efficient space distribution, and a balance of hospitality, retail, and recreational services. The structure is a combination of a podium where people will conduct their functions and a tower where guests will be accommodated. The hotel consists of a one-story basement, three floors with dimensions of 60×40 m resulting in a 2400 square meters area each, and 5 floors with a floor area of 1500 square meters each (37.5×40 m). Vertical circulation includes at least two staircases and elevator cores in compliance with IBC requirements for fire safety and egress. The basement retaining walls are designed for lateral earth pressure and hydrostatic loads. The roof level may accommodate green roofing, solar panels, or service areas, supporting both sustainability goals and building services.

Basement and first floor

The first floor consists of the hall, serving as a gathering place for the visitors and reception, for the registration of visitors. Billiards and the supermarket are located on the west and east sides of the building, respectively. The stairs and elevators are located behind the billiards and supermarket rooms, in front of the restaurant. This floor has a height of 5 meters, which is the tallest in the entire building. The area of the restaurant "Assyl As" is 460 square meters, while the billiards and supermarket have the same area of 166 square meters.

Second floor

The second floor is built for commercial purposes, because no malls or clothing stores are located in this area, and it is not convenient for residents of this area to go far away for shopping. In total, the floor contains six large commercial spaces with a total area of 231 square meters, as well as two smaller premises with an area of 115.5 square meters. These spaces will be rented out.

Third floor

Gym, SPA salon, playground area for children, and conference rooms can be seen from the plan of the third floor. The visitors can conduct conferences, do sports, and have SPA procedures without going out of the hotel. The conference halls have a total area of 600 square meters. These rooms can accommodate various meetings and company presentations. The gym is 337.5 square meters, while the lounge area and spa are each 225 square meters.

4-8 floors

Floors from the 4th to the 8th are identical, and each floor has 16 rooms: 4 standard, 8 standard +, and 4 luxury rooms. However, the 4th floor is 2 standard rooms short because there is an entrance to the terrace. The total number for the standard rooms is 18, for standard it is 40 and 20 for Lux rooms. The room size depends on its type. For example, a standard room is the smallest, equalling 37.5 square meters, followed by a standard+ at 56.25 square meters, and the largest is a Lux at 67.5 square meters. All rooms have their own toilet and showers.

2.2 Site Selection and Design Conditions

2.2.1. Site Selection

Humble, Texas, is part of the Houston-area urban region and had an estimated population of about 16,489 (Census Reporter, n.d.). The broader five-mile radius around the site supports over 150,000 people, indicating a substantial surrounding catchment. Harris County as a whole has a median household income of approximately \$83,980 and is one of the major economic engines in Texas (*Humble, TX Radius Demographics | Current Census Data*, n.d.). The site is situated just off Highway 59 and Beltway 8, minutes from Bush Intercontinental Airport (IAH) and the Generation Park business hub, giving it strong regional connectivity and visibility.

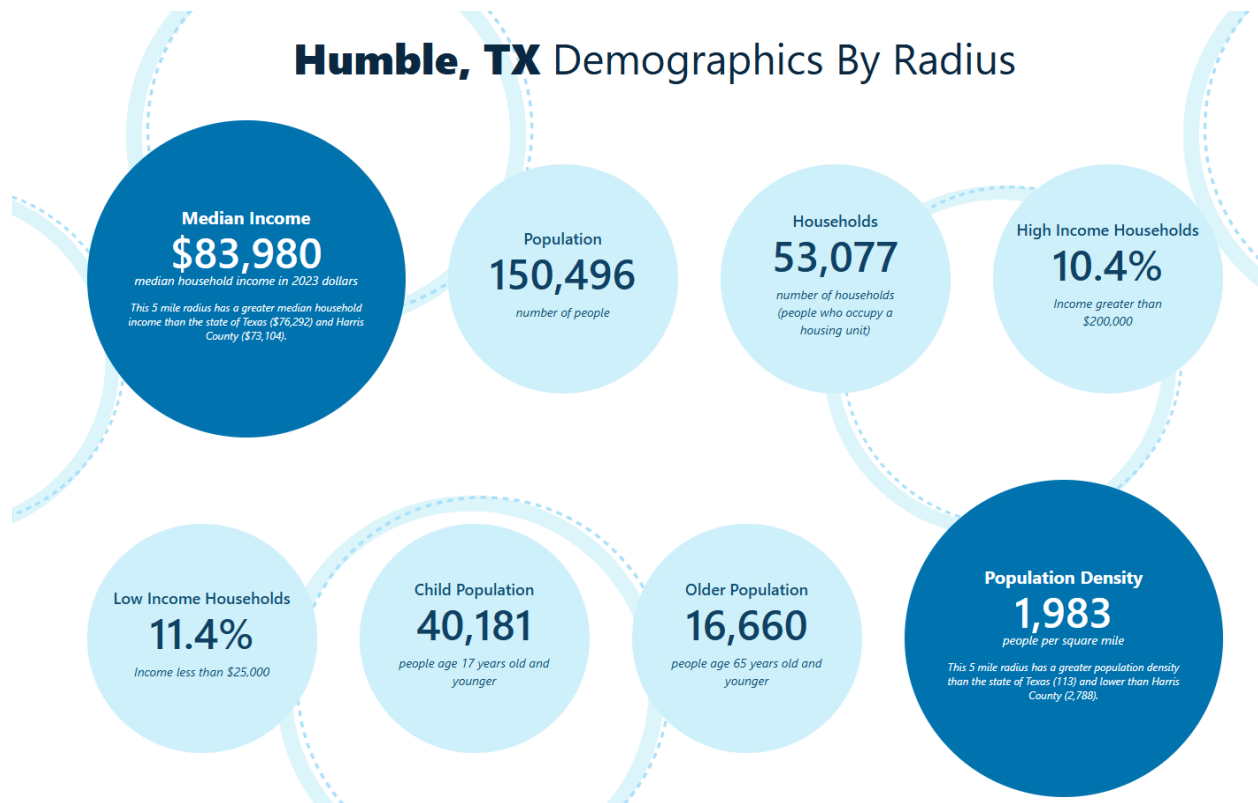


Figure 2.2.1.1. Humble Demographics from demographicsbyradius.com

Since Humble lies within the orbit of Houston, the region benefits from large labor pools, strong business growth, and scalable infrastructure. The location near major highways and transit corridors ensures good connectivity for guests and employees. The site is currently vacant and minimally developed, so the new hotel will not supplant a significant existing structure or heavily used community asset. The property is zoned to permit commercial, residential, industrial, or flex uses, providing flexibility for mixed-use development options.

Google Maps shows that the immediate area around 6819 Manor Way is a mix of entertainment, food service, industrial, and service-oriented businesses. On Manor Way itself, there is 1886 Humble Backyard, a local bar/restaurant described in reviews as prioritising customer service, specialising in mixing adult beverages, and featuring a patio for guests to unwind. Nearby fast-food outlets include chain restaurants like Jack in the Box and Church's Texas Chicken, while other informal food options, such as food trucks ("Tacos Y Mas Los Camaradas"), operate along the frontage roads.

Within a short drive of the site are industrial and service employers like American Crane School, Martin Marietta's Bender Yard (construction materials), and vehicle sales/service yards, which draw a steady flow of workers to the area. Southeast of the property, across Beltway 8, lies a cluster of residential neighborhoods with local churches and schools, while to the north, along the Eastex Freeway, are retail centers, hotels, and logistics businesses serving airport traffic. The combination of restaurants, bars, industrial employers, and residential subdivisions creates a diverse micro-market that can supply both guests and staff to a hotel.

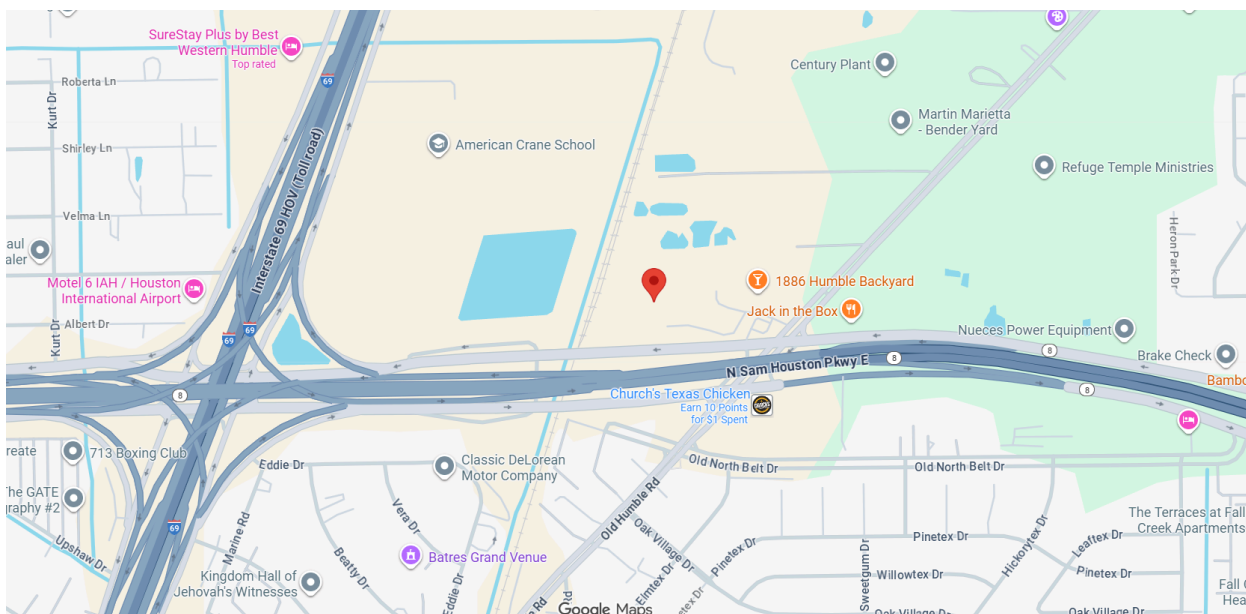


Figure 2.2.1.2 Snapshot of the surrounding facilities (Google Maps)

In summary, 6819 Manor Way offers an attractive mix of factors for a hotel development. Its prime location on two major highways provides high visibility and easy access. The property's proximity to Bush Intercontinental Airport (about 10 km) ensures a flow of visitors, while the growing population within a five-mile radius (≈ 150 km) with above-average household incomes supports local demand. The surrounding restaurants, bars, light industrial, and service businesses create a lively environment with opportunities for partnerships and cross-promotion. Together, these attributes collectively make 6819 Manor Way a strategic site for a hospitality venture aimed at high occupancy and long-term value.

2.2.2 Design Conditions

The project requirements dictate that the building should encounter certain environmental conditions, of which 2 of them will be addressed: a strong wind zone and a corrosion environment.

Strong Wind Zone

In this case, the wind design is guided by the ASCE 7-10, which states a 3-second gust wind speed at its basic. The Hazard Tool of ASCE (ASCE 7-10, Risk Category II, Exposure B) indicates a basic wind velocity of 59m/s at the project location in the Houston area. This was the value that was applied to all wind-load calculations. Despite the other wind speeds to various mean-recurrence periods (e.g., 10-year and 25-year and 50-year) that are also presented in the hazard tool, these are not applied to building design. ASCE 7-10 only requires the fundamental wind speed of 59 m/s. Because the site is in the Houston metropolitan area (~29.94° N, -95.28° W) and the basic wind speed exceeds 51.4, it lies in a “hurricane-prone region” (NCSEA, 2024).

Corrosion Environment

The IBC requires that building components exposed to the weather be protected from corrosion. Section 2304.10.6 of the 2015/2018 IBC and Section 2304.10.6 of the 2021 IBC stipulate that fasteners used with preservative-treated or fire-retardant-treated wood must be “hot-dip galvanized, stainless steel, silicon bronze, or copper” when manufacturer recommendations are absent (*Corrosion Information for All Simpson Strong-Tie Product Lines*, n.d.). Although Humble is slightly farther inland (~50 km), research by corrosion and galvanizing organizations shows that salt-laden air can affect metals 30–50 miles (48–80 km) inland and is amplified in industrial cities. Therefore, the hotel site likely experiences ISO C3 to C4 conditions, which corresponds to medium to high corrosivity (Poma, 2019).

2.3 Site Analysis

2.3.1. Climate

The range of temperature in the selected area in Houston is from 91°F - 94°F (33 °C and 34 °C) in the summer months and 62°F - 65°F in the winter (NCEI.Monitoring.Info@noaa.gov,

n.d.). The Houston area is considered to be humid since the average precipitation is 49 inches for the year (NCEI.Monitoring.Info@noaa.gov, n.d.). Figure X shows the temperature and precipitation values graphically.

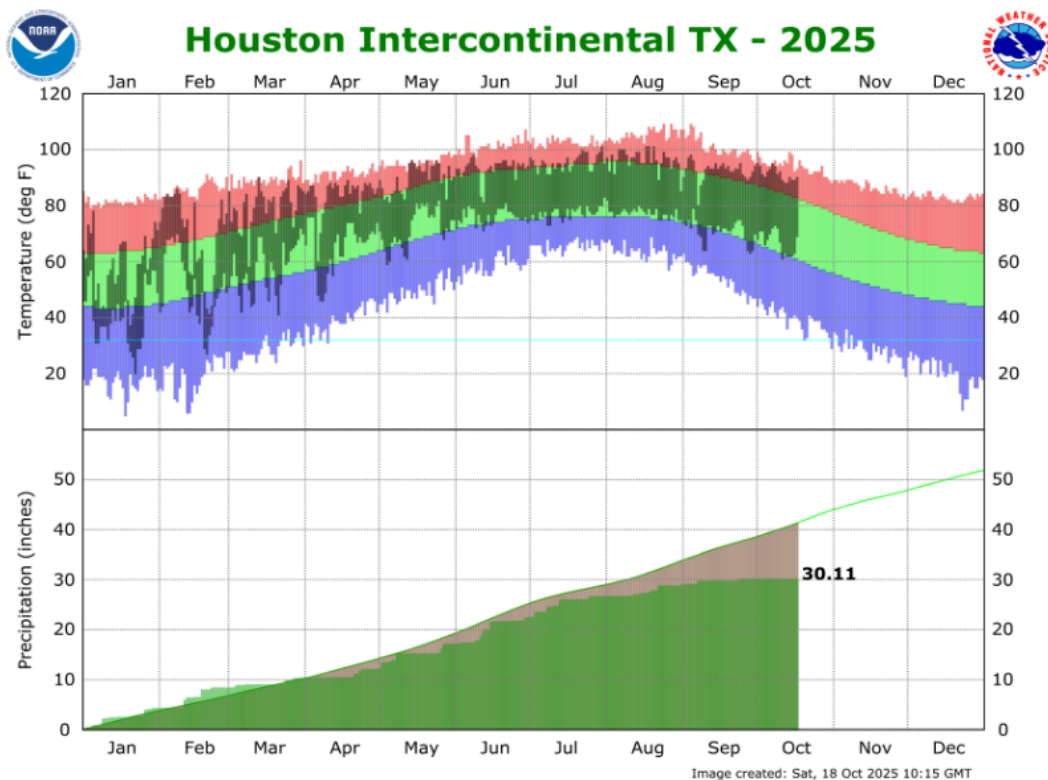


Figure 2.3.1.1. The temperature and precipitation throughout the year in Houston, Texas

2.3.2. Historical Background

The area surrounding the 6819 Manor Way in Humble, Texas, has evolved alongside the broader development of the Greater Houston region. Originally a semi-rural zone supporting agriculture and small industry in the late 19th and early 20th centuries, Humble gained oil in 1904, which transformed this area into Texas's earliest energy hubs. Over the past few decades, the city has transitioned from an industrial suburb to a growing mixed-use corridor, combining residential neighbourhoods, logistic facilities for the nearby airport and highways, and light commercial zones.

The chosen site is not a historically designated or protected property and is also not listed in any state or county cultural resource inventories. The quick review of Harris County GIS and Humble city planning data tells us that the land is unoccupied and previously undeveloped, with no registered community facilities or commercial assets. Consequently, the proposed project does not involve the removal of any valued private or public property. Instead, it represents a strategic reuse of an underdeveloped lot in a fast-growing commercial corridor, which will greatly contribute to the city's urban and economic development.

2.3.3. Orientation

The orientation of our building in our location is shown in the figure below. The main entrance is shown with a red arrow. This orientation allows us to effectively use the sunlight and to efficiently use the space for parking lots and outside decoration. The parking lot will be located at the front side of the hotel and the park will be located the south side of the hotel.

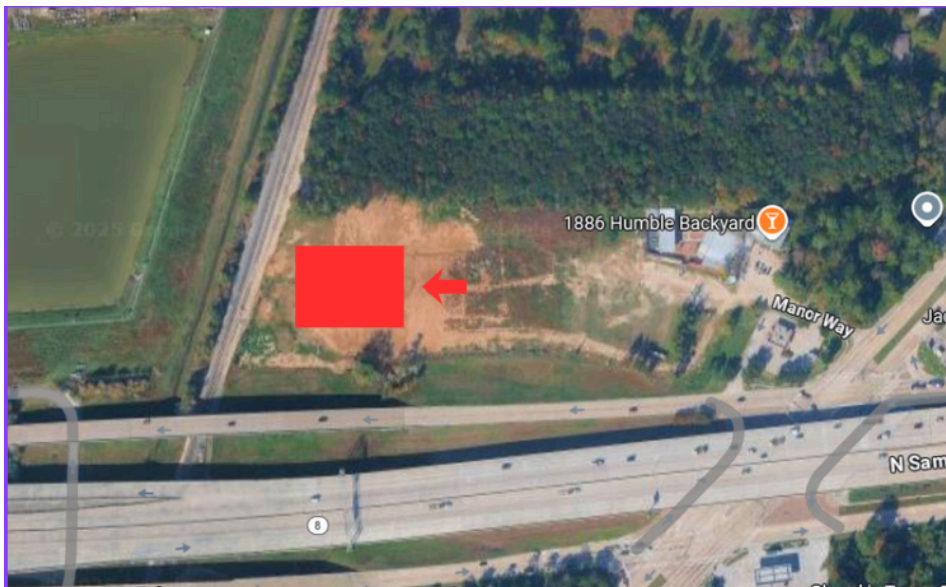


Figure 2.3.3.1. The orientation of the hotel with regard to the location.

2.3.4. Solar access

According to the figure below, the sun direction is from east to west along with the Sam Houston Pkwy W. The total duration of the sun is 11 hours and 20 minutes. The sunrise for October 19 is at 7 a.m and the dawn is at 7 p.m and it reaches its peak at 1 pm. The shadow length reaches the value of 0.8 meters. (*SunCalc Sun Position- Und Sun Phases Calculator*, n.d.)

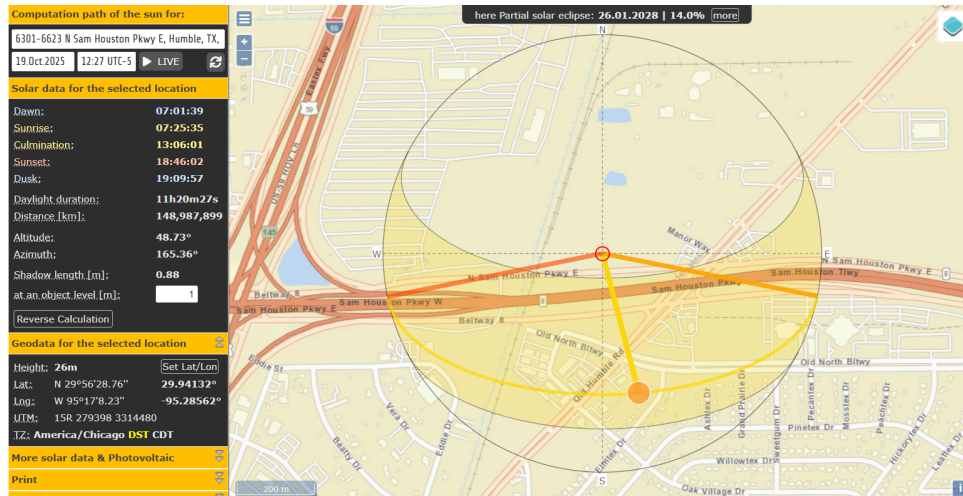


Figure 2.3.4.1. The sun access illustration and data.

2.3.5. Views

The north side of the hotel opens the view to the forest, while the south view shows the city side and the highway. The north view is shown in Figure 2.3.5.1.



Figure 2.3.5.1. The north view of the hotel.

2.3.6. Site survey

The location survey of 6819 Manor Way, Humble, Houston, shows some major features that are important in undertaking construction or solar installation. In case of overshadowing, the influence of landforms is not very big as the selected location is flat. However, overshadowing by tall trees is a major concern because of the thickening of the trees, especially in the backyard.

Trees will cast large shadows, which would seriously affect the level of production of sun energy. The amount of shadow created by buildings is minimal.

The selected location is mostly flat with a very small slope estimated to be 1-2, which is appropriate to the Houston region. The site is also stable, which is conducive to foundation stability because topographical differences can reduce the chances of settlement in the site. The most important geotechnical issue in this case is not the slope, but the control of water drainage in order to control the behavior of the underlying soil.

The land lies in the area of soft clays, which is a very plastic and expansive soil. The shrinking and swelling of this clay in regard to its moisture content has enormous pressure on the foundations, which may cause movement, cracking, and structural damage

2.3.7 Access and Transport

The highway N Sam Houston Pkwy E is located south of the hotel, and the entrance to the hotel territory is on the left side of the Old Humble Rd, turning to Manor Way (Figure 2.3.7.1).

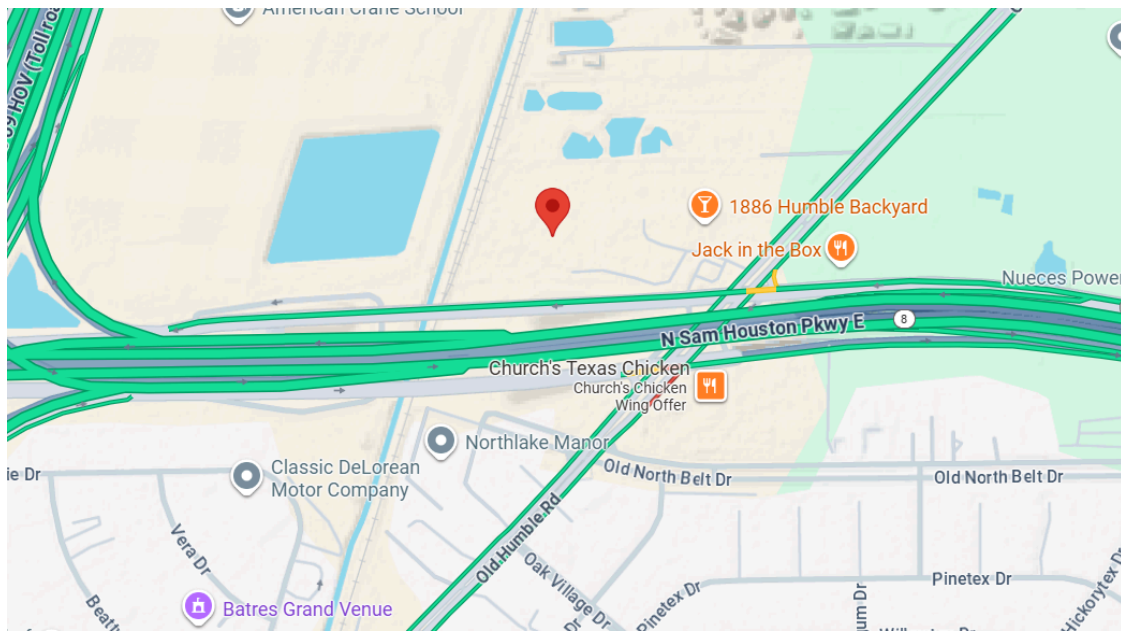


Figure 2.3.7.1. Traffic Map

2.3.8 Bushfire Risks

The risk of wildfire in the property on the 6819 Manor Way in the Harris County area around Humble Texas is relatively low -to moderate as compared to the drier areas. Although the

general area has been characterized by management of wildfire, the site issues must be handled especially in a 3-acre land that has the possibility of having extensive natural vegetation or uncontrolled growth. The significant factors in this case include the proximity of any vegetation that is thick, flammable (pine trees, thick brush) to the intended structures and its accessibility to fire suppression equipment. The proposed development will address safety standards of the Harris County Fire Marshal Office (HCFMO) by ensuring that the area surrounding the building has a 30-foot defensible space buffer and the proposed building materials are fire resistant. The fire map is shown in the figure below.



Figure 2.3.8.1. Fire map of the site

2.3.9 Stormwater Drainage

The Houston stormwater drainage system revolves around the Bayou Network which is an essential natural and artificially constructed system of waterways that gathers runoff in the incredibly flat 1,777 square miles of Harris County. These bayous, as Buffalo, Brays, and White Oak, are the principal specialized vessels, and transport the water slowly into the Houston Ship Channel and Galveston Bay (The Harris County Flood Control District (HCFCD)).

The collection system is part of the underground storm sewers within the dense urban areas, and large roadside ditches within the suburban and unincorporated areas. An essential part of flood control is the regulation where new developments are required to build on site detention basins. These basins store temporary overflow of impervious surfaces (such as pavement and roofs) water in the bayou network, which guarantees that the water is introduced into the bayou system at a slower controlled release rate, rather than causing overcapacity to the downstream areas and reduce the chances of structural flooding.

2.3.10 Services

Houston works in the Texas electricity market, which has developed a system of dual service delivery. The regulated utility company, CenterPoint Energy, maintains the physical infrastructure, such as the power lines, poles, and the transmission grid and this company acts as the Transmission and Distribution Utility (TDU). Their sole duty is to make the grid reliable and solve the outages. Natural gas service is monopolized by CenterPoint Energy, it provides the integrated service of supplying both the pipeline system and the gas to the consumers.

In Houston, the water and sanitary sewer services are operated by the Public Works department, however some sides of the Houston such as Manor Way 6819 can be operated by Municipal Utility Districts.

Telecommunication is operated by several mobile companies such as T-mobile, Verizon etc.

2.4 Design concept

At the beginning of the project, our team envisioned a pyramid-style hotel design with a gradual decrease in area on each floor. However, this design was neither architecturally nor aesthetically pleasing. The hotel would have stood out from the other buildings' designs. Therefore, the decision was made to maintain the same floor area for the first three floors (60 x 40) and the subsequent five floors (37.5 x 40). In the picture below, you can see that we also plan to have a terrace on the 3rd floor of the building, which will serve as an additional source of relaxation for our guests.

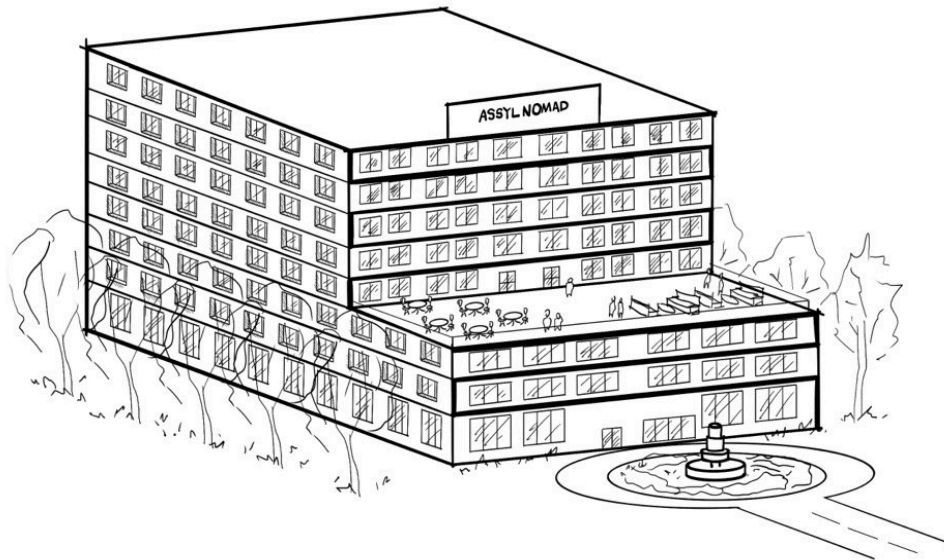


Figure 2.4.1. First sketch of the “Assyl Nomad” hotel

Houston is located in a specific wind zone in the United States. According to the ASCE Hazard Tool, the wind speed is 60 m/s, with the 50-year mean recurrence interval (MRI) wind speed being 46 m/s. Besides, according to **Table 3-2** of the Houston Region borehole data in Appendix C: Engineering Report, a number of metals that are native in the groundwater can be used to accelerate the corrosion of construction material. The building layout will, therefore,

incorporate the provision of precautionary measures to avoid corrosion of the materials.

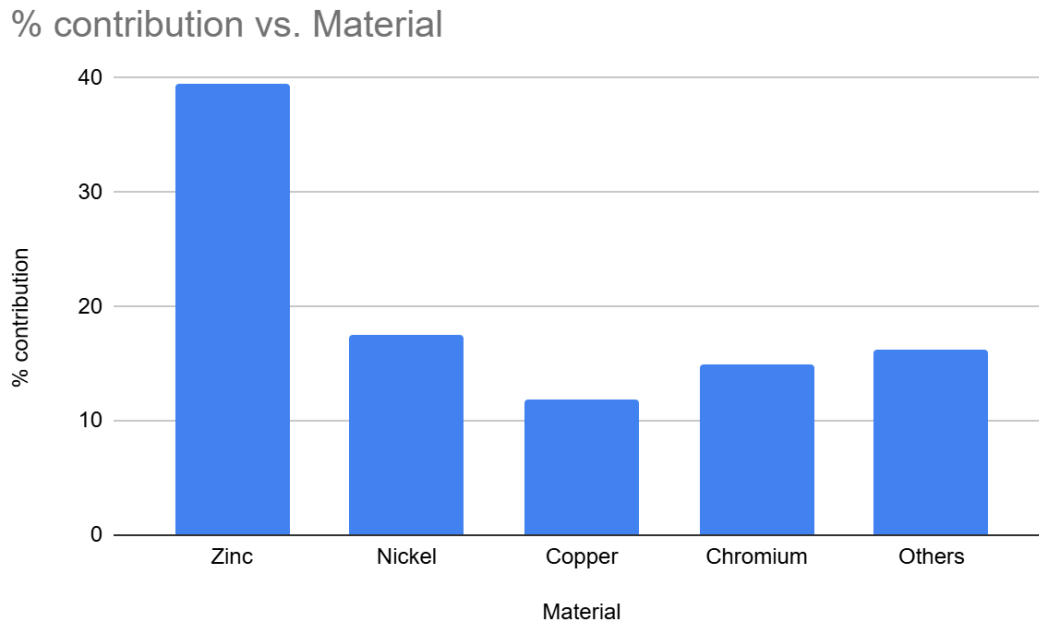


Figure 2.4.2. Percentage of different metals in the corrosion of building materials

2.5. Geometry Design and Site Layout

2.5.1 Design Codes

ASCE 7-10 - Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7-22 standard gives out all the load criteria and load combinations to be applied in the structural design. It gives specifications of dead loads, live loads, roof loads, wind loads, rain loads, seismic effects, and serviceability specifications, including deflection and drift. In the case of Houston, the controlling lateral load is wind load, while seismic effects are insignificant (Seismic Design Category A or B). The category of risk in the hotels as stipulated by the standard, is Risk II and the following importance factors. Strength and serviceability load combinations, pressure coefficient, and enclosure categories are directly based on ASCE 7-10. These guidelines inform the development of the reinforced concrete structural system, which includes the podium and tower.

ACI 318-19 - Building Code Requirements for Structural Concrete. This code has been used to regulate the analysis, design, and detailing of reinforced concrete elements, which

are used in the structure of the hotel. It specifies strength design methodology, load factors (ϕ – *factors*), and minimum reinforcement ratios of beams, slabs, columns, and walls. Other rules in this code include: shear, bond, anchorage, and also durability, which has to be provided according to the conditions of exposure. The design of the retaining walls in the basement of the hotel follows ACI 318-19, which will resist lateral earth pressure and hydrostatic loads. The upper stories will have flat slabs and columns, which will be checked on punching shear and deflection limits as a measure to secure long-term serviceability and safety of the building. It will be designed according to the exposure class and requirements in the Houston region.

IBC - International Building Code (IBC, 2021/2024). This code (IBC) defines the basic rules of building design, construction, and the classification of the occupancy. It gives the lowest requirements in the structures of safety, means of escape, accessibility, and building services. In this project, “Assyl Nomad”, which is a one-basement and eight-story hotel located on 6819 Manor Way, Humble (Houston area), the building can be classified as Group R-1 (transient lodging), with accessory occupancies of A-2 (dining areas), B (administrative offices), and S-2 (parking or storage space as needed). The IBC regulated the total fire-resistance rating, egress width, stair and elevator core design, and retaining wall requirements in the basement. It also requires the market to adhere to the standards that have been referenced, like ASCE 7-22 and ACI 318-19 on structural design. The IBC is locally embraced via the Texas Industrialized Housing and Buildings Code system.

Occupant Load = Area/Occupant load factor. Below in the table, there are calculated values for the occupant load for each premises. It was made according to the IBC.

Table 2.5.1. Occupant load of each premises

Premises	Area, m ²	Occupant load factor, m ²	Occupant load, persons
Restaurant	460	1.39	330.9
Conference rooms	600	1.39	431.7
Billiards	166	1.39	119.4

Supermarket	166	5.57	29.8
Commercial	346.5	5.57	62.2
GYM	337.5	4.65	72.6
SPA Salon	225	13.9	16.2
Lounge	225	1.39	161.9
Standard	37.5	18.6	2.02
Standard+	56.25	18.6	3.02
Lux	67.5	18.6	3.63

2.5.2 Architectural design

The project conditions required were to design a large span building having a minimum of 20 meters between supports. The design was to be made in a rectangular shape, but due to some thoughtful ideas, the area of the 4th to 8th floor was to be downsized by 37.5 meters to 60 meters. This is because of the narrow width of 60 meters, giving only 4 hotel rooms per floor, but the cost of construction is higher than the projected income. The podium facilitates the large-scale activities of the people, whereas the tower provides privacy and silence to the hotel customer. The decrease in the floorplate size of the upper levels enhances stability under lateral loads and adds to the formation of the visual difference in the architecture. The selection of materials and facade solutions is made to withstand humid and corrosive environments, which is also a problem in the climate of Houston. The figure above shows that the x-axis is a 40-meter-long large span with columns spaced 20 meters. The y-axis demonstrates a 7.5-meter span, resulting in nine columns. Furthermore, in accordance with architectural standards, we added additional columns to ensure the staircase remains stable. The number of columns along the x-axis on floors 4-8 was changed, as maintaining a large span was no longer necessary. Therefore, seven columns were used, with the first 3 and last 3 columns spaced 7.5 meters apart, and a 5-meter gap in the middle between the 3 columns.

According to the IBC code for the architectural designs, this hotel is included in the R1 occupancy group. The hotel is equipped with the automated sprinklers and the longest distance between the room and emergency exit is 42 meters (below 200 feet or 61 meters is acceptable (IBC)). The number of stairs is 2 and located between the grids C and D by the y-axis and 1-2 and 6-7 grids by the x-axis. The number of parking spaces in the site is 124 which should include spaces for guests and personnel. In order to get access to the sunlight in the rooms the minimum requirement for the living room's window area is taken into consideration (min 8% of the rooms area). The roof of the hotel was designed with 1% slope in order to meet the requirements for drainage of the water.

The figures below are the plans and 3d models of the hotel which are made in REVIT software.

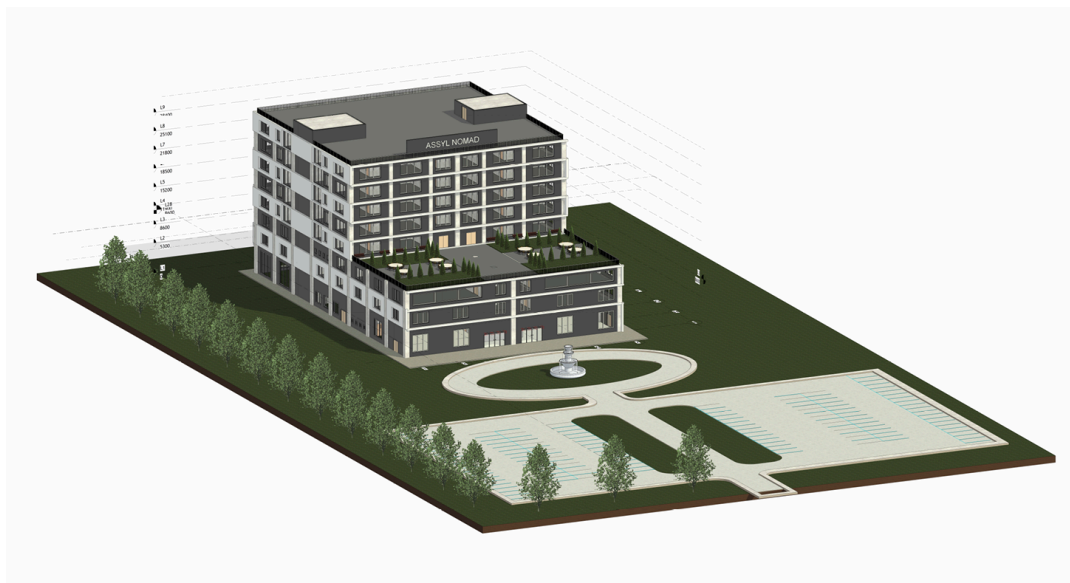


Figure 2.5.2.1. 3D model of the residential hotel “Assyl Nomad”.

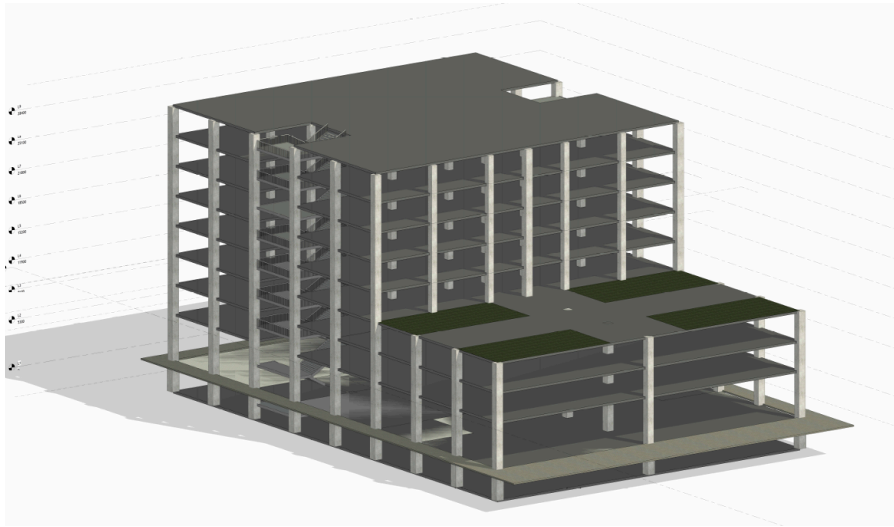


Figure 2.5.2.2. Structural elements of the hotel

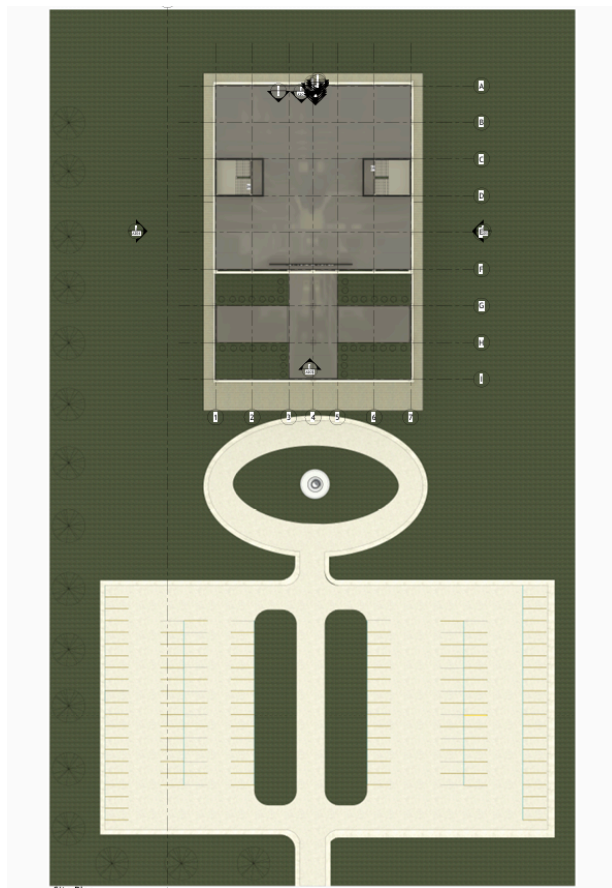


Figure 2.5.2.3. Site layout

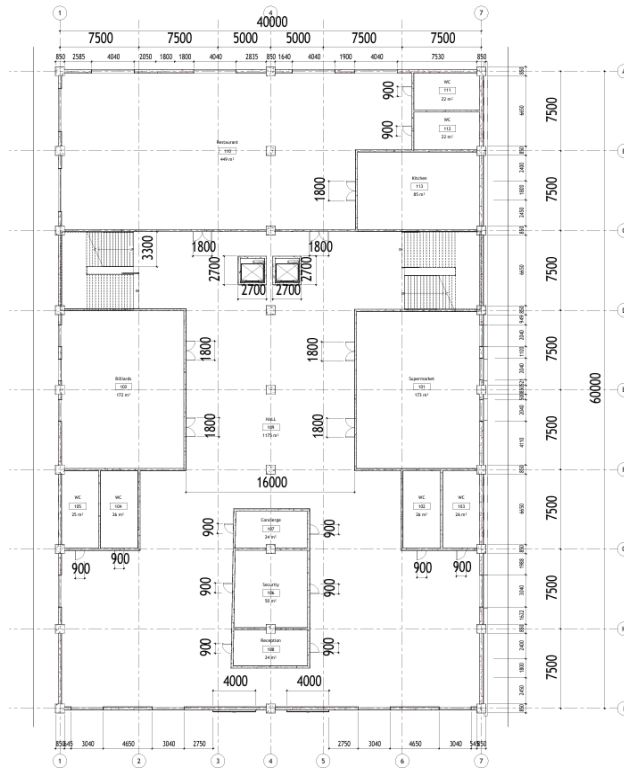


Figure 2.5.2.4. 1st floor plan of the hotel

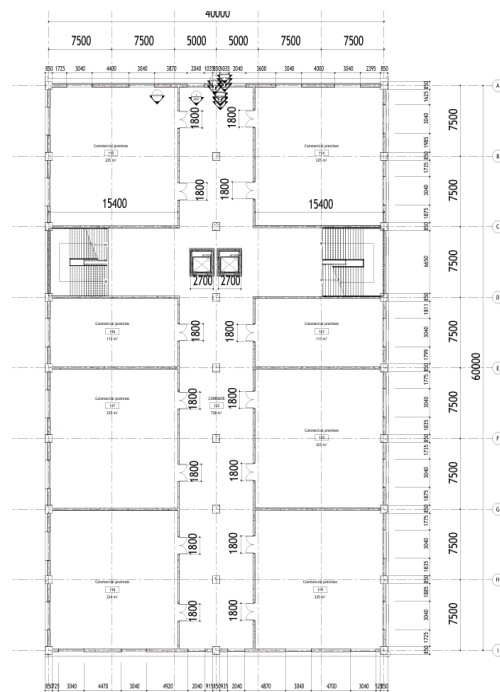


Figure 2.5.2.5. 2nd floor plan of the hotel

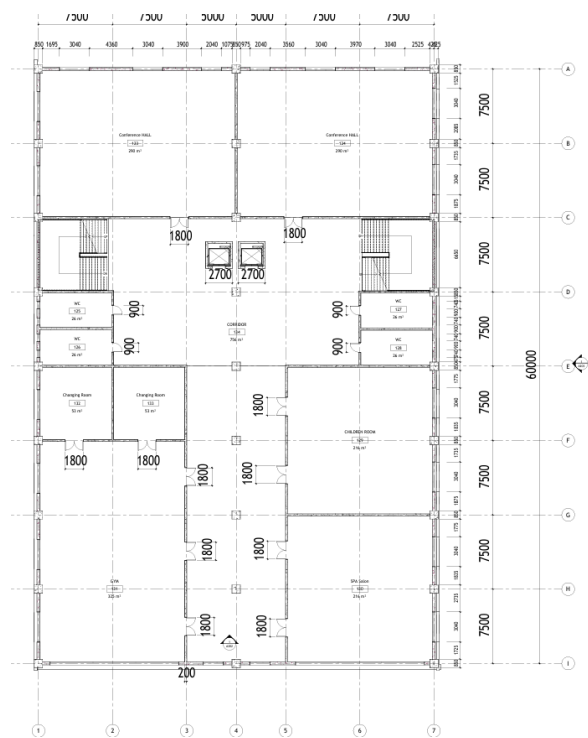


Figure 2.5.2.6. 3rd floor plan of the hotel

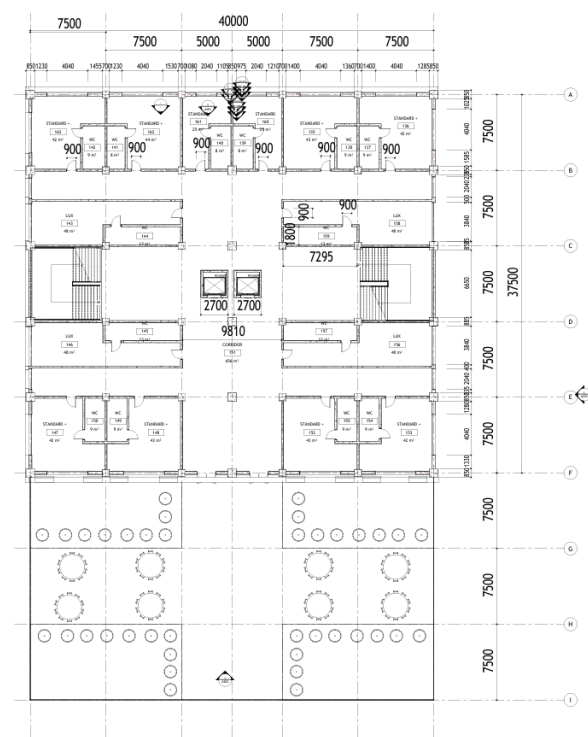


Figure 2.5.2.7. 4th floor plan of the hotel

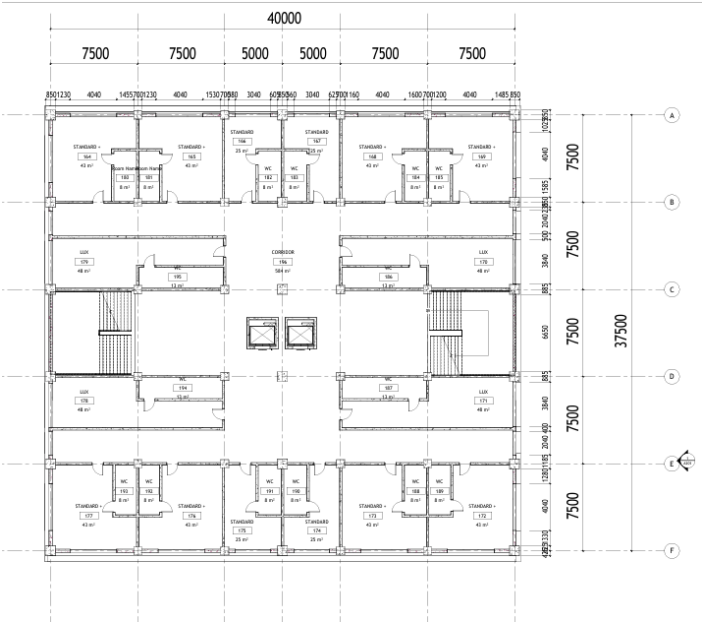
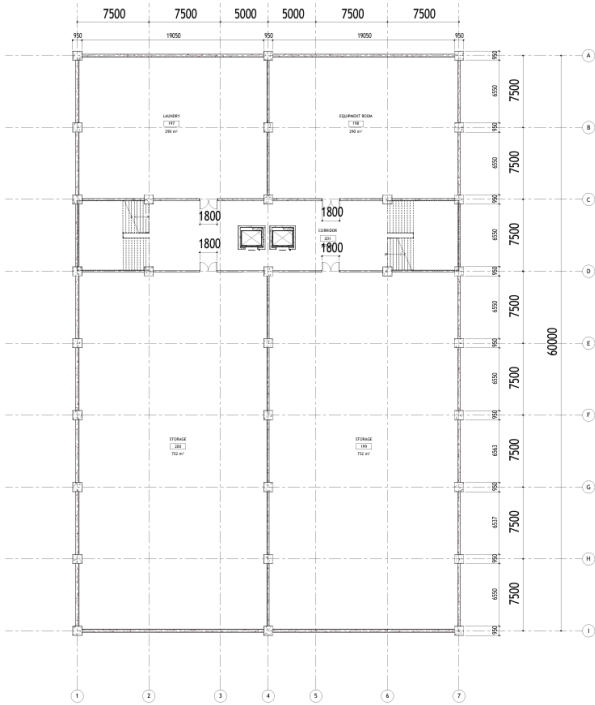


Figure 2.5.2.8. 5-8 floor plan of the hotel



[illegible]

Figure 2.5.2.11. Section cut by y-axis

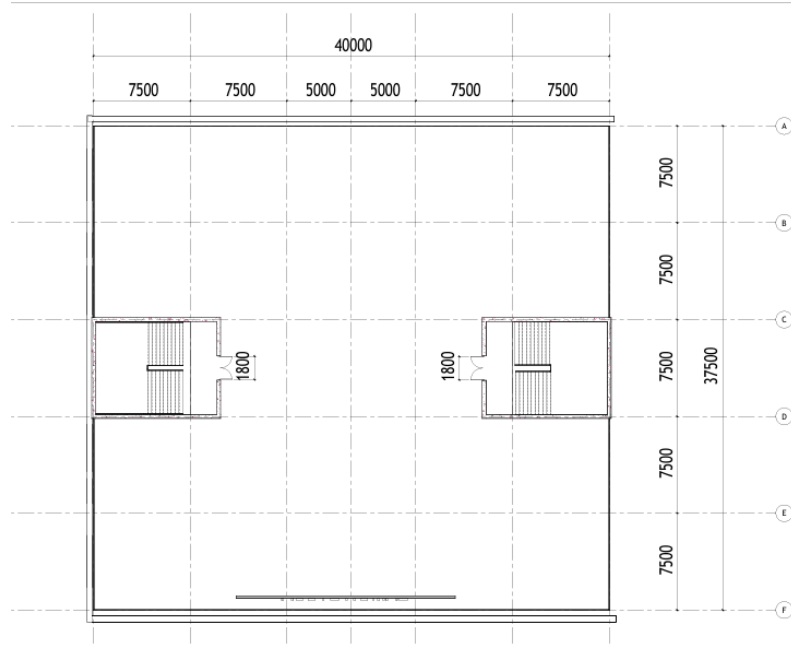


Figure 2.5.2.12. Roof plan



Figure 2.5.2.13. North elevation



Figure 2.5.2.14. South elevation



Figure 2.5.2.15. East elevation



Figure 2.5.2.16. West elevation

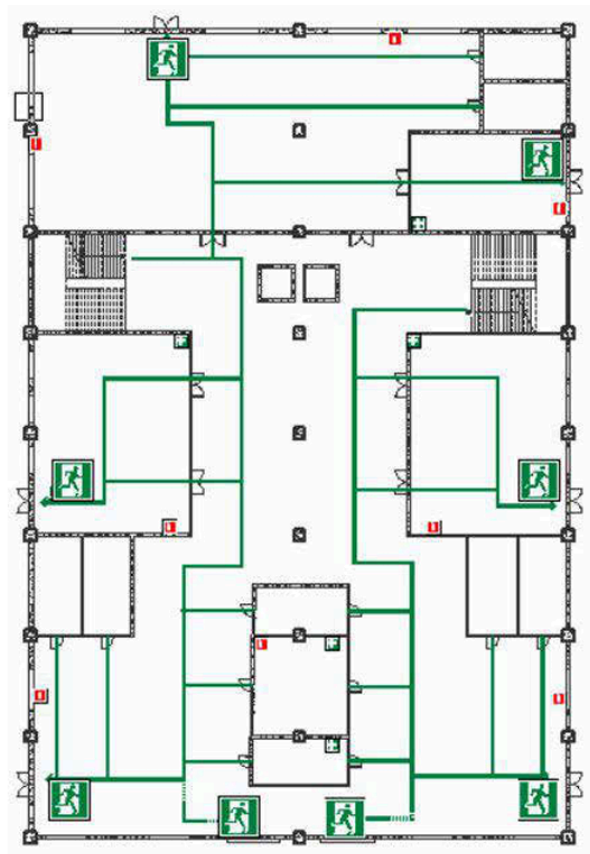


Figure 2.5.2.17. Evacuation and emergency exits (1st floor)

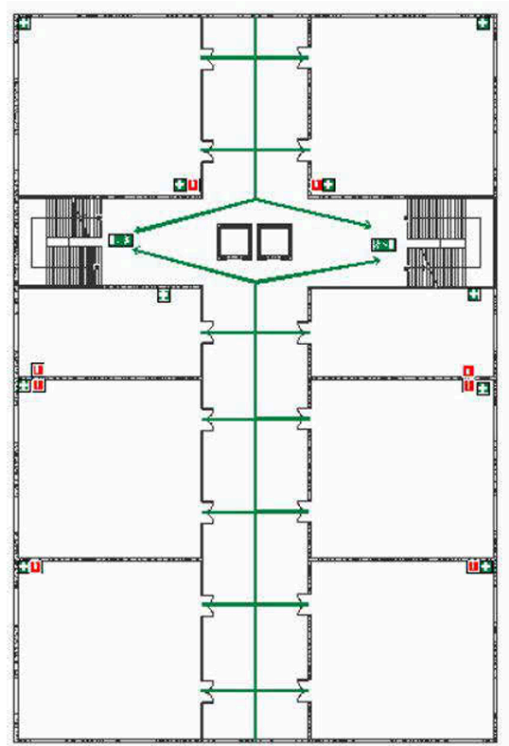


Figure 2.5.2.18. Evacuation and emergency exits (2nd floor)

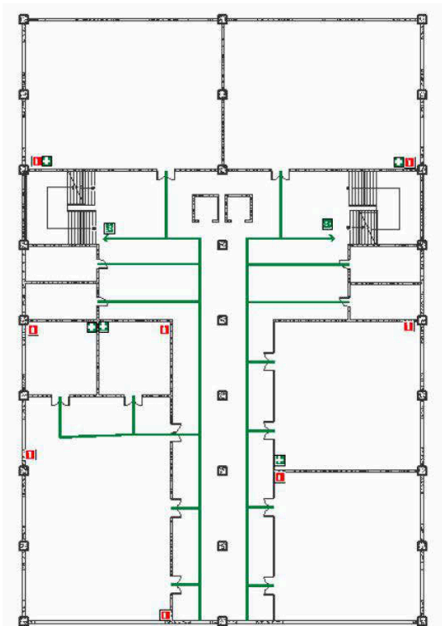


Figure 2.5.2.19. Evacuation and emergency exits (3rd floor)

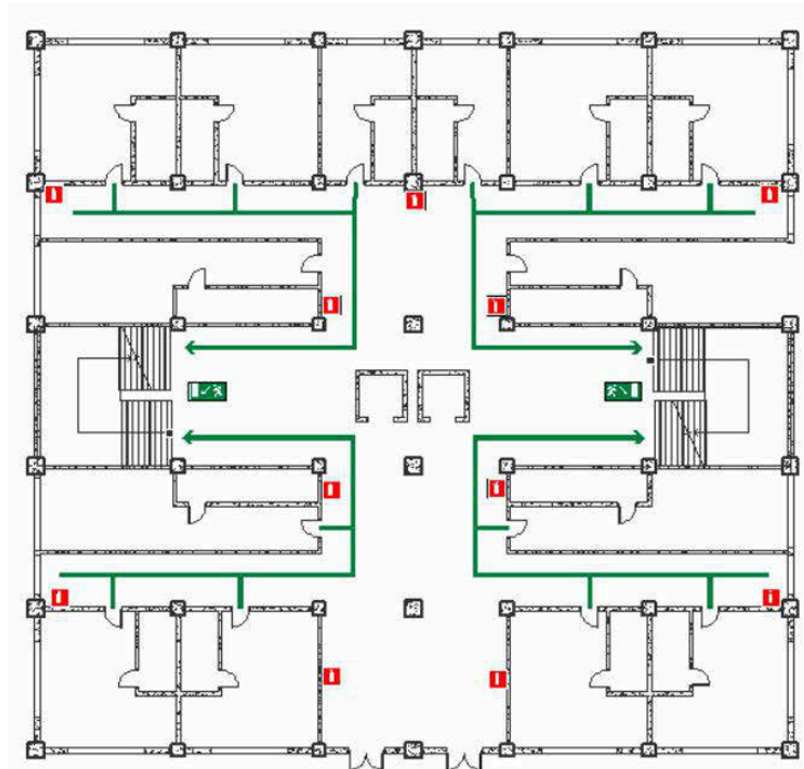


Figure 2.5.2.20. Evacuation and emergency exits (4 - 8 floor)

2.6. Leadership in Energy and Environmental Design (LEED)

LEED is a green building rating system that aims to provide environmentally friendly, cost-effective, and healthy buildings. At least 40 credits should be earned to get a certification. Silver accreditation is awarded for 50-59 points, gold accreditation for 60-79 points, and above 80 is platinum certification. The hotel now achieves 87 points, earning Platinum certification (USGBC, n.d.).

Location and transportation category

The project received a score of 12 points in Location and Transportation, as detailed in Figure 2.6.1. This category scored for protecting sensitive land, as the building sits on a sensitive site. The hotel received five points for Surrounding Density and Diverse Uses, which promotes development in areas with existing infrastructure to reduce vehicle travel and encourage health through physical activity. Further points were awarded for Access to Quality Transit, gaining five

points for the hotel being well-served by public transport, helping to reduce environmental damage from private vehicle use. Bicycle Facilities received one point, and Reduced Parking Footprint received another point, promoting less car dependence.

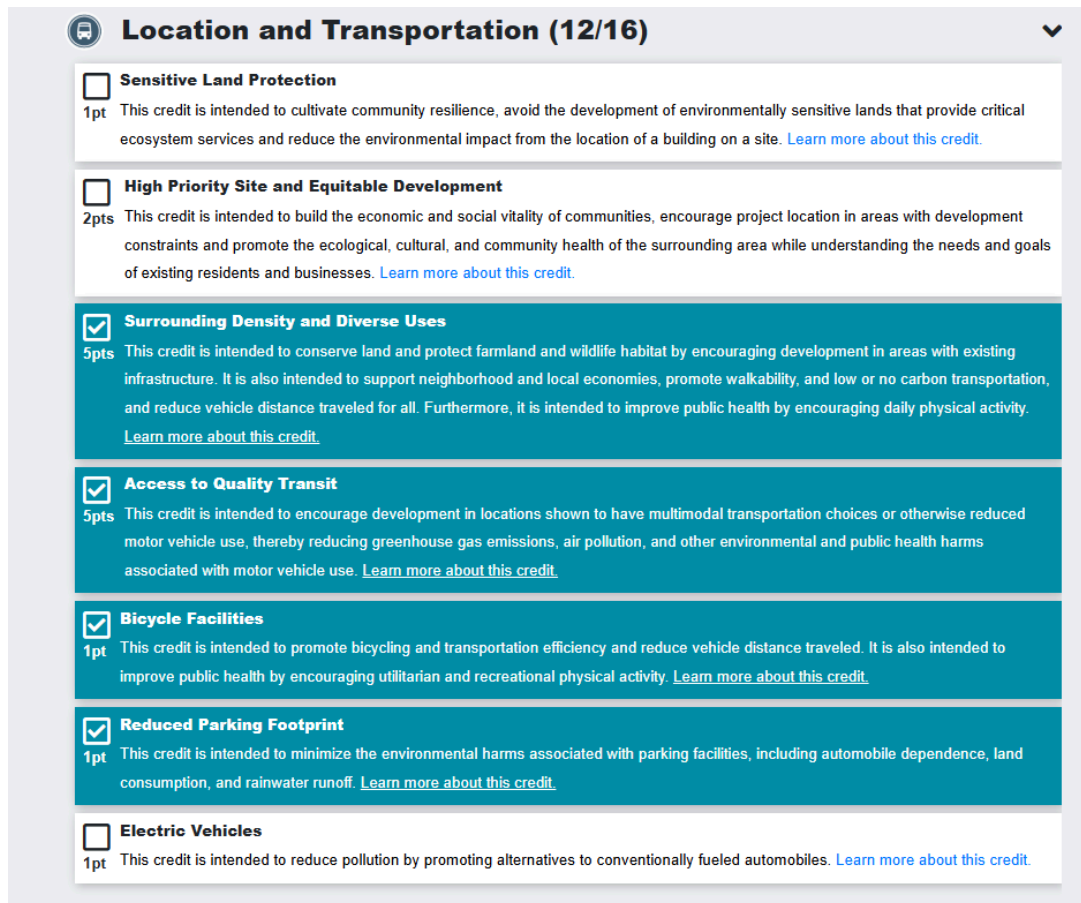


Figure 2.6.1. LEED scoreboard for Location and Transportation

Sustainable sites

The Sustainable Sites category was fully scored at 10 points out of 10 as shown in Figure 2.6.2. The hotel earned full points for the site assessment, preservation and restoration of habitats and development of open space for recreation. Other points were earned through strategies to manage rainwater and mitigate heat island effect, enhancing local environmental conditions and the project's overall sustainability.

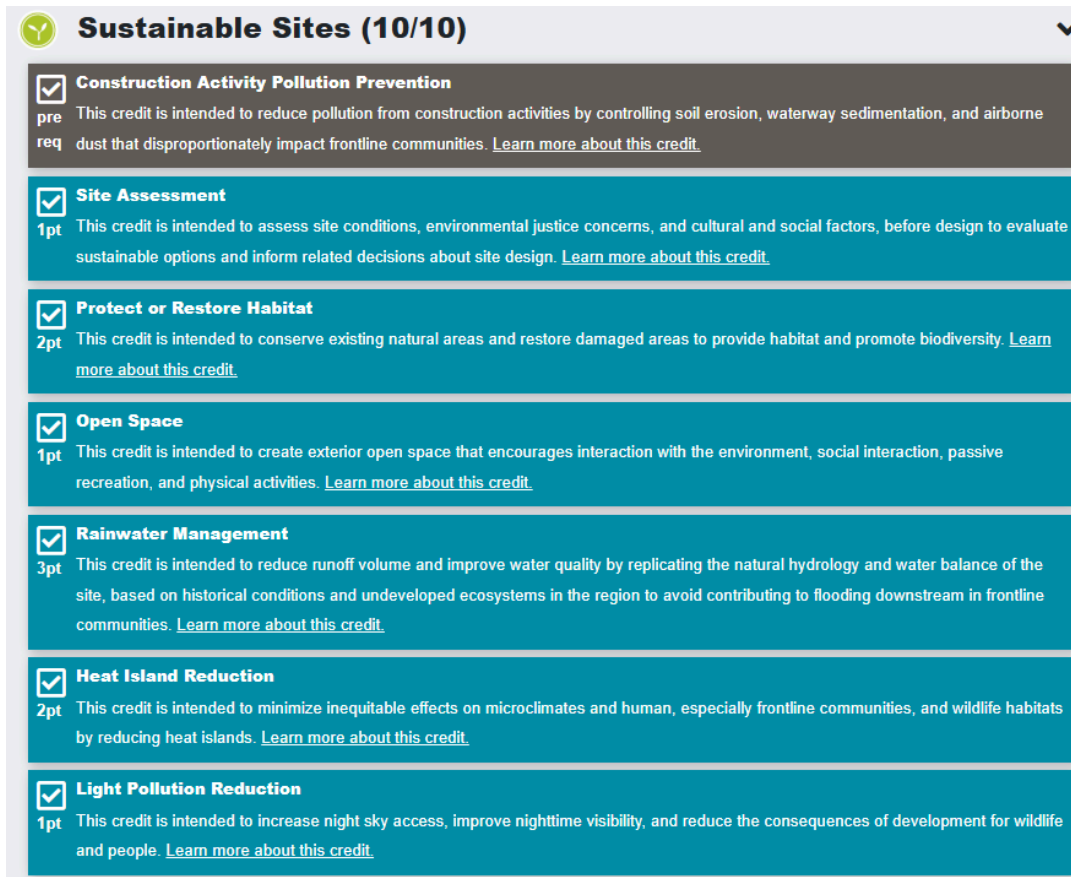


Figure 2.6.2. LEED scoreboard for Sustainable Sites

Water efficiency

Figure 2.6.3 shows that the hotel scored 9 out of 11 points in the Water Efficiency category. The project achieved all the water conservation prerequisites such as Outdoor Water Use Reduction, Indoor Water Use Reduction, and Building-Level Water Metering. An additional (2 points) was awarded for Outdoor Water Use Reduction where strategies were implemented to reduce outdoor water use. The project attained 6 points for Indoor Water Use Reduction with water efficient fixtures and technologies that reduce water consumption. Finally, 1 point was awarded for Water Metering to monitor and control water use.

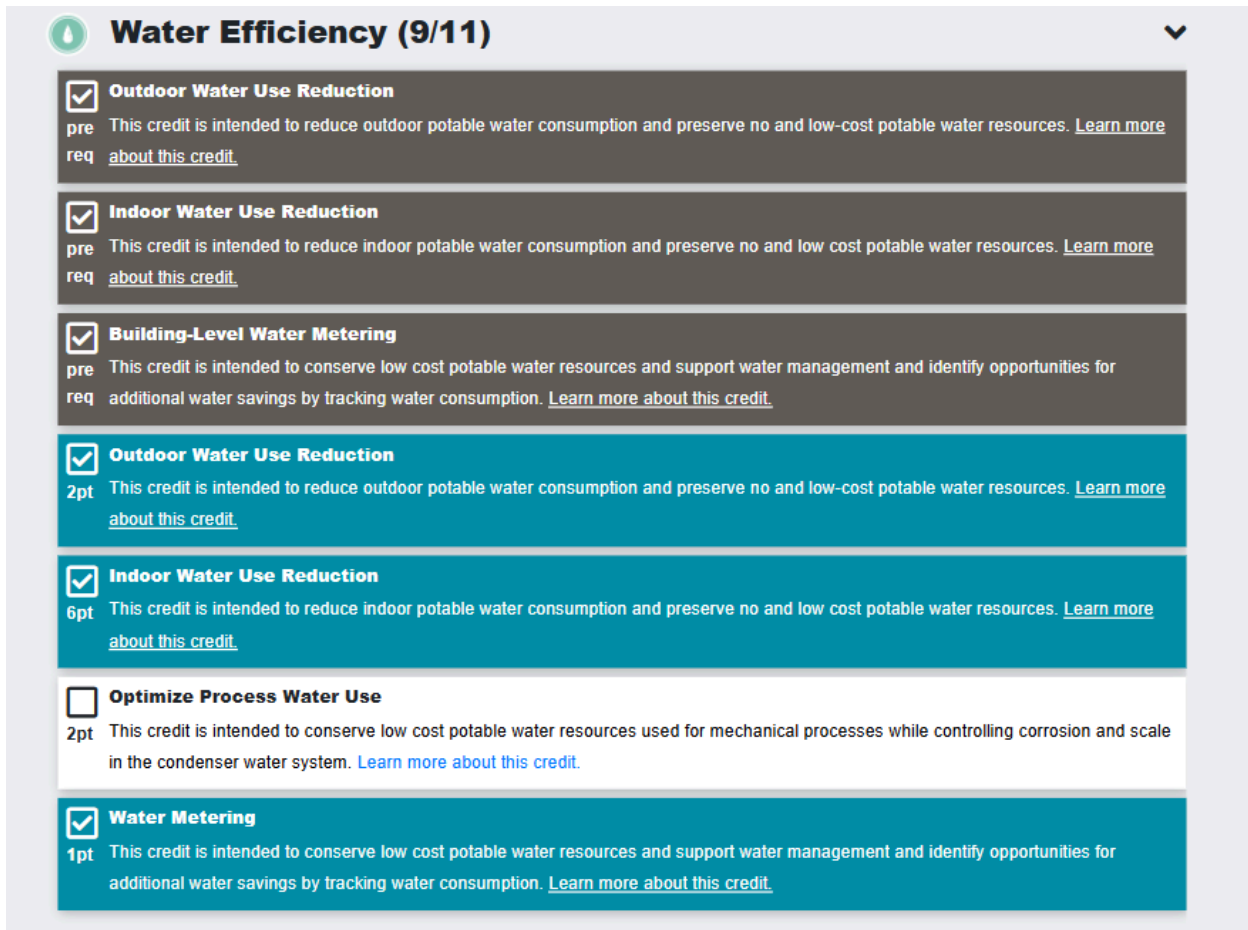


Figure 2.6.3. LEED scoreboard for Water Efficiency

Energy and Atmosphere

The hotel scored 24 points out of a total of 33 in the Energy and Atmosphere category (see Figure 2.6.4). The prerequisites met included Fundamental Commissioning and Verification, Minimum Energy Performance, Building-Level Energy Metering, and Fundamental Refrigerant Management. The project earned 6 points for Enhanced Commissioning, which helps ensure that building systems operate efficiently and meet sustainability goals. Optimize Energy Performance 18 points The project exceeded the energy performance standards and therefore achieved a high number of points.

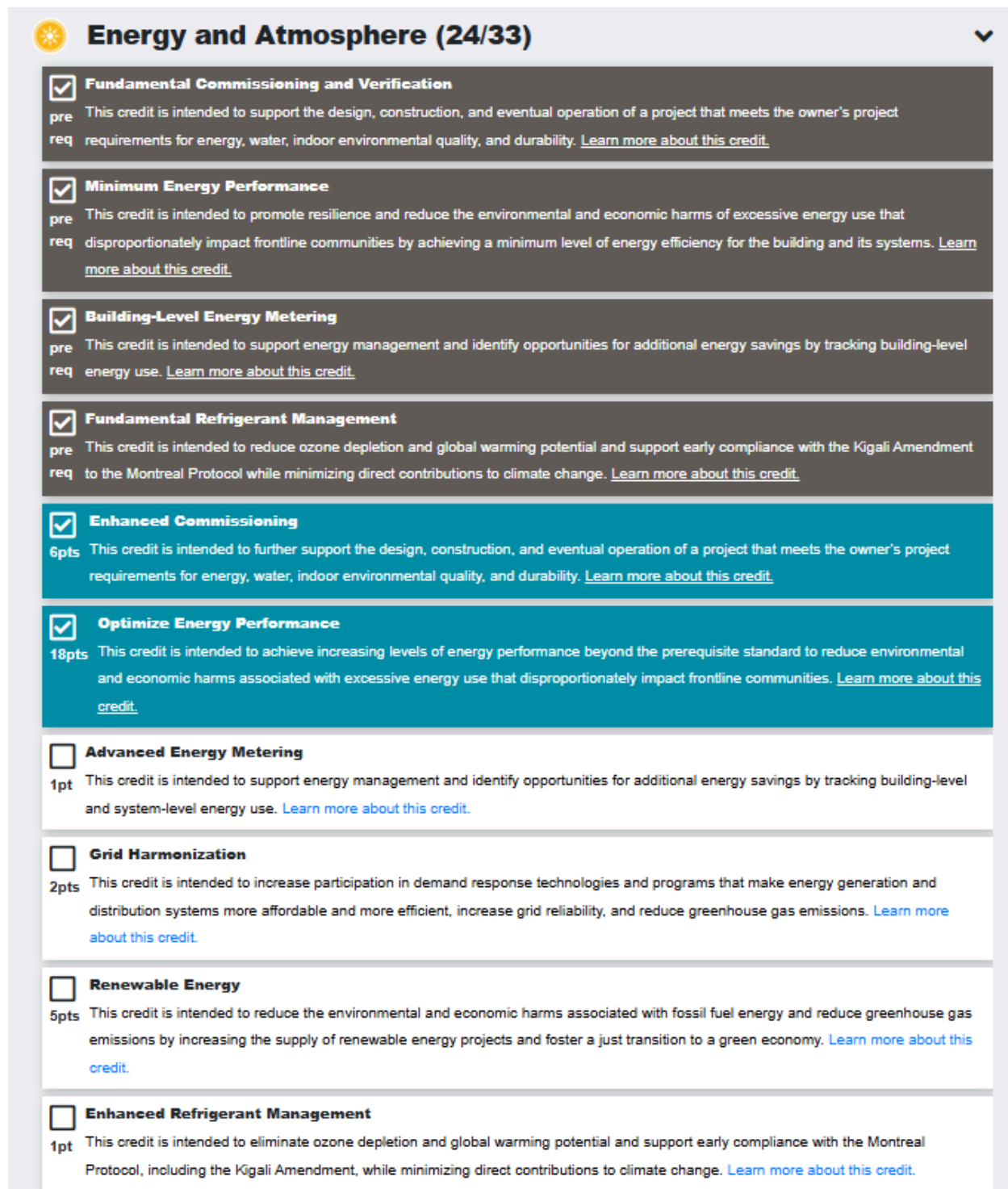


Figure 2.6.4. LEED scoreboard for Energy and Atmosphere

Materials and Resources

As shown in Figure 2.6.5, the hotel scored 8 out of 13 points in the Materials and Resources category. Points were awarded for Storage and Collection of Recyclables (prerequisite), to ensure that the hotel has a proper recycling system. 2 points for Environmental Product Declarations which means the environmental impact of the materials used in the building has been assessed. The hotel gained a further 2 points for Sourcing of Raw Materials ensuring materials used were sourced responsibly.

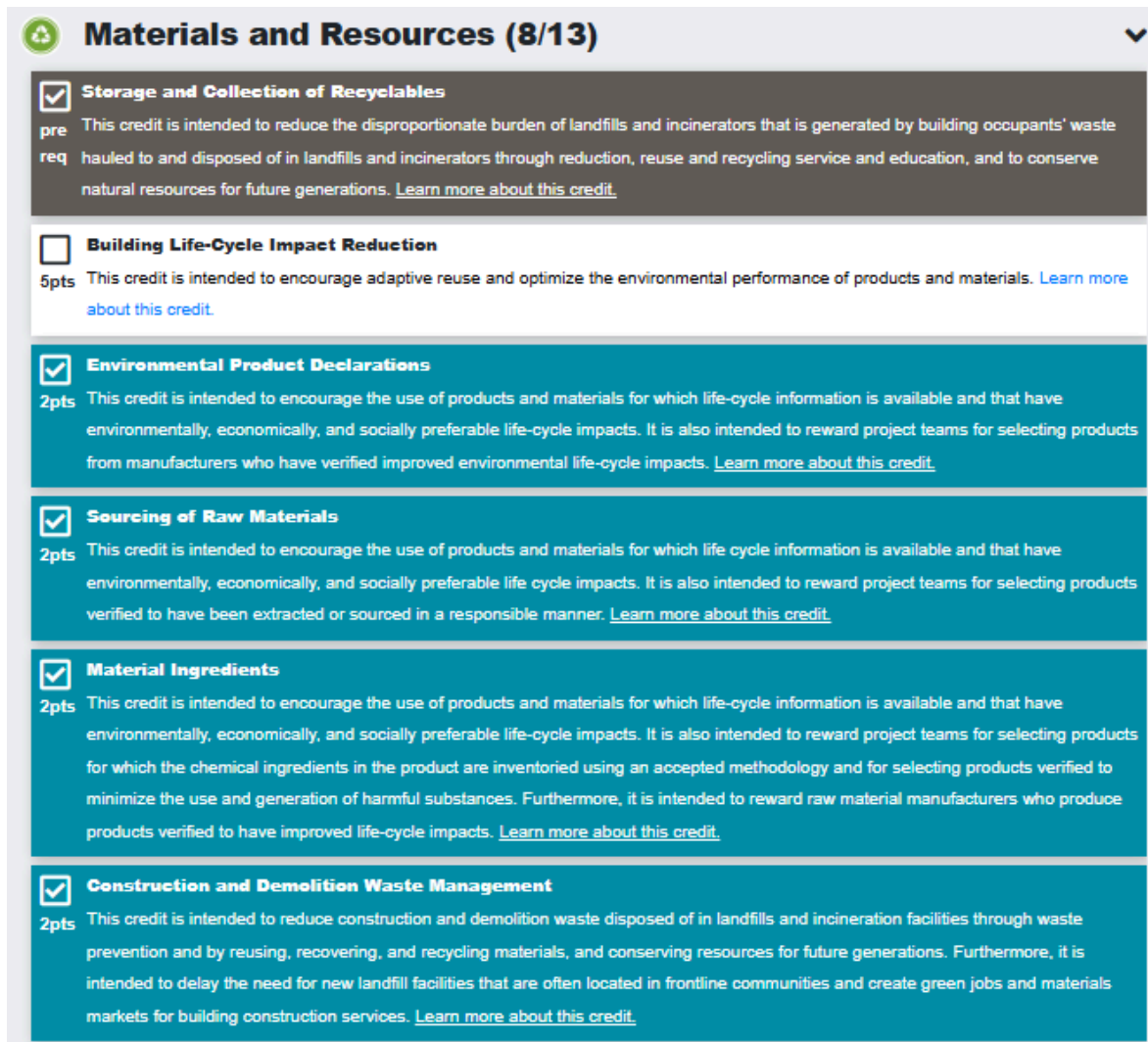


Figure 2.6.5. LEED scoreboard for Material and Resources

Indoor Environmental Quality

Figure 2.6.6 shows that the hotel scored 14 out of 16 points for Indoor Environmental Quality. The hotel inside is smoke-free to ensure a healthy environment for guests, also air

purifiers and air quality control systems will be installed in corridors. Rooms and halls will have thermostats and lighting so people feel comfortable. Big windows in each room provide lots of daylight and views outside. Masonry walls have an excellent acoustic performance; therefore, guests will not hear each other. The Indoor Air Quality Assessment point is not achieved as we did not implement a formal process to assess and ensure high indoor air quality during both the post-construction phase and occupancy, which is necessary to earn this credit.


Indoor Environmental Quality (14/16)


☒ **Minimum Indoor Air Quality Performance**
pre This credit is intended to contribute to the comfort and well-being of all building occupants by establishing minimum standards for indoor
req air quality (IAQ). [Learn more about this credit.](#)

☒ **Environmental Tobacco Smoke Control**
pre This credit is intended to prevent or minimize exposure of building occupants, indoor surfaces, and ventilation air distribution systems to
req environmental tobacco smoke. [Learn more about this credit.](#)

☒ **Enhanced Indoor Air Quality Strategies**
2pts This credit is intended to promote occupants' comfort, well-being, and productivity by improving indoor air quality. [Learn more about this credit.](#)

☒ **Low-Emitting Materials**
3pts This credit is intended to reduce concentrations of chemical contaminants that can damage air quality and the environment, and to protect the health, productivity, and comfort of installers and building occupants. [Learn more about this credit.](#)

☒ **Construction Indoor Air Quality Management Plan**
1pt This credit is intended to promote the well-being of construction workers and building occupants by minimizing indoor air quality problems associated with construction and renovation. [Learn more about this credit.](#)

☐ **Indoor Air Quality Assessment**
2pts This credit is intended to establish better quality indoor air in the building after construction and during occupancy to protect human health, productivity, and wellbeing. [Learn more about this credit.](#)

☒ **Thermal Comfort**
1pt This credit is intended to promote occupants' productivity, comfort, and well-being by providing quality thermal comfort. [Learn more about this credit.](#)

☒ **Interior Lighting**
2pts This credit is intended to promote occupants' productivity, comfort, and well-being by providing high-quality lighting. [Learn more about this credit.](#)

☒ **Daylight**
3pts This credit is intended to connect building occupants with the outdoors, reinforce circadian rhythms, and reduce the use of electrical lighting by introducing daylight into the space. [Learn more about this credit.](#)

☒ **Quality Views**
1pt This credit is intended to give building occupants a connection to the natural outdoor environment by providing quality views. [Learn more about this credit.](#)

☒ **Acoustic Performance**
1pt This credit is intended to provide workspaces and classrooms that promote occupants' well-being, productivity, and communications through effective acoustic design. [Learn more about this credit.](#)

Figure 2.6.6. LEED scoreboard for Indoor Environmental Quality

2.7 Life Cycle Cost

The concrete mix strength in the structural system design for the hotel is designed to be 6000 psi (~42 MPa). Therefore, to accurately simulate this in the software (Life-365), the water/cementitious material (w/cm) ratio is set to 0.40 compared to the 0.45 w/cm ratio used in the standard case (base case) for the conventional concrete mix design of 4000 psi. To comply with the sustainability criteria stipulated by the LEED grade, and improve chloride resistance for increased durability, the project alternative considers a 20% Class F Fly Ash content.

<u>Mixture</u>	<u>Rebar</u>	<u>Barriers</u>
w/cm *	Rebar Steel Type *	Select Barrier *
0,4	Black Steel	<none>
Class F fly ash (%) *	Rebar % vol. concrete. *	
20	1,2	
Slag (%) *	<u>Inhibitor</u>	
0	Select Inhibitor *	
Silica Fume (%) *	<none>	
0		

Figure 2.7.1. Concrete mix ratio for the adjusted case

A site-specific analysis was performed for Houston, Texas, replacing default values to meet the specific environmental conditions of the area. As depicted in Figure 2.7.2, the heat in Houston is much greater than that in cooler climates, with average monthly temperatures varying between a minimum of 10.2°C in January to a maximum of 28.1°C in July. The rate of diffusion of chloride ions is temperature-dependent, and the higher temperatures increase the rate of penetration. To counteract this effect, and in compliance with ACI 318-19, basement retaining walls were covered by 3 inches (75 mm) of concrete cover for a better physical barrier.

Temperature Cycle (°C) (Automatically set)

Month	Temperature (°C)
January	10.2
February	12.2
March	15.9
April	20.2
May	23.6
June	26.9
July	28.1
August	27.9
September	25.7
October	20.9
November	16.1
December	11.9

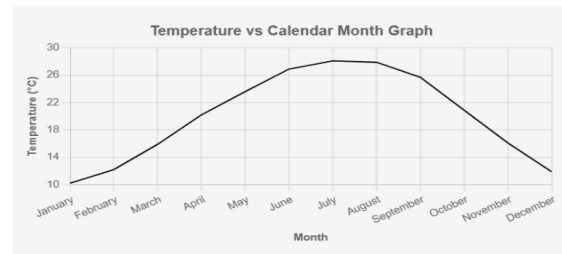


Figure 2.7.2. Temperature vs. Calendar Month graph

The life cycle costs were considered for a 75-year period. The simulations results show the economic benefits of the high-performance design:

Base Case: Pays a huge Repair Cost of \$3,237,688 due to earlier onset of corrosion and shorter time for maintenance.

Alternative 1 (High Performance Design): The use of the 6000 psi mixture containing Fly Ash brings the future repair cost to \$1,244,803: more than 60% lower.

Total Savings: The Life-Cycle Cost for the Proposed Design is \$2,624,583 compared to \$4,617,468 for the original design.

As illustrated above, the alternative design eliminates the "cost steps" that result from major renovations. This shows that the 6000 psi mix is not only a structural requirement but also the most economic decision to ensure profitability for the hotel.

Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Base case	\$1,379,780	\$0	\$3,237,688	\$4,617,468
Alternative 1	\$1,379,780	\$0	\$1,244,803	\$2,624,583
alternative 2	\$1,379,780	\$152,000	\$0	\$1,531,780
alternative 3	\$1,288,124	\$304,000	\$0	\$1,592,124

Figure 2.7.3. Total life cycle costs

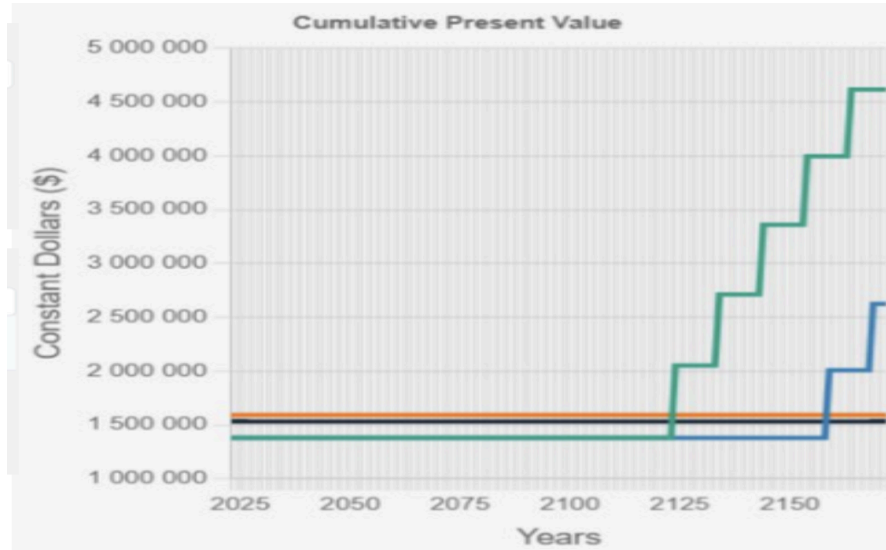


Figure 2.7.4. Cumulative present values for base and alternative cases

Corrosion Prevention and Durability

For the Houston hotel project with a 75-year design life, a two-pronged approach was applied to the durability design, involving chemical refinement of the concrete matrix and physical barriers.

Chemical Refinement and SCMs

In the project, 20% Class F Fly Ash is used to densify the concrete matrix. Fly ash, via pozzolanic reactions, enhances durability by decreasing permeability and limiting chloride diffusion (Thomas, 2007). In severe environments, silica fume (5-8%) is added to further refine the cementitious matrix and enhance electrical resistivity to corrosive currents (Holland, 2005). Moreover, slag cement greater than 40% can help prevent sulfate attack and alkali-silica reactions (ACI Committee 233, 2017).

Physical Barriers and Environmental Factors

Houston's hot climate means faster diffusion of the chloride ions. To offset this, and as per ACI 318-19, structural elements such as basement walls are covered by a 3-inch (75 mm) layer of concrete. This offers vital protection against oxidation of the reinforcement (ACI Committee 318, 2019). Additional measures, such as the use of corrosion inhibitors and silane seals, can also provide a chemical barrier film on the steel reinforcing bar, increasing the threshold of the chloride ions and the onset of corrosion (ACI Committee 222, 2019).

Corrosion Repair methods

The simplest method of dealing with reinforcement corrosion in concrete structures such as columns and beams is the Manual Patch Repair. This approach is the most common repair method for isolated defects when the concrete has "spalled" (cracked). This involves drilling a hole in the concrete to expose the corroded steel, including the area behind the reinforcing bars to facilitate bonding. After cleaning the rusted steel and applying a primer, the steel is patched with high-strength mortar to return the structure to its original form. This approach is very successful for minor damage, as long as the cause of the corrosion is also resolved.

In more corrosive environments, such as coastal regions, we can use Electrochemical Methods for long-term protection. For example, Sacrificial Anodes can be used, small pieces of zinc that are attached to the steel reinforcement. Zinc is more prone to corroding than steel, so it "sacrifices" itself to protect the steel from rusting. In critical structures or large areas, Cathodic Protection may be applied, where a small electrical current is applied to the steel. This prevents the chemical reaction of rusting, even in highly salt-laden concrete.

3. Structural Design

3.1. Material Selection

Reinforced concrete is used for columns, beams, and slabs because the compressive strength of concrete is high. Steel rebars have high tensile strength, which is necessary to resist lateral forces from strong winds. Reinforced concrete has a high durability in the humid and corrosive environment of Houston. According to Table 3.1.1, RC is corrosion-resistant, fire-resistant, has high thermal mass, and has excellent acoustic performance. Additionally, it has a low material cost and is quicker to build.

Masonry is used for exterior and interior walls due to fire resistance, high thermal mass, good acoustics, and easy to build. The cladding panels are applied to a metal frame to the front, which isolates the main edifice against corrosion, minimizes thermal bridges, enhances acoustic aspects, and makes things easier to maintain.

Table 3.1.1. Comparison between RC and Steel Frames

Parameter	Reinforced concrete	Steel
-----------	---------------------	-------

Strength	Higher compressive strength	Higher tensile strength
Ductility	Moderate	High
Weight	Heavy	Light
Stiffness	High	Moderate
Corrosion resistance	A coating is needed	Vulnerable
Fire resistance	High	Low
Speed of construction	Low	High
Cost	Low material cost	High material cost
Sound isolation	Good	Low
Thermal mass	High	Low
Maintenance	Hard to repair	Easy to repair

3.2. Analysis and design of Gravity Load Resisting System

3.2.1. Calculation of Live Load

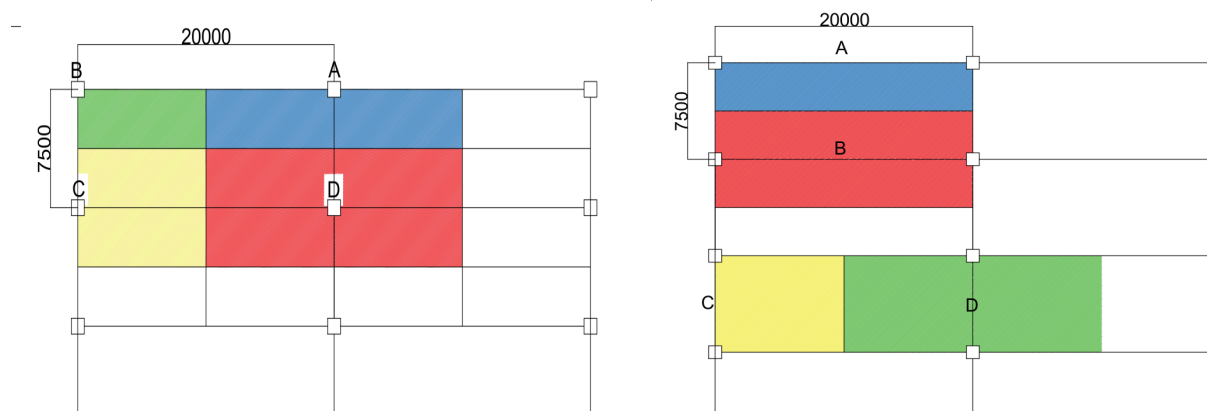


Figure 3.2.1.1. Tributary area for columns and beams for basement and floor 1-3

$$L = L_0 \left(0.25 + \frac{4.57}{\sqrt{K_{LL} * A_t}} \right)$$

$L \geq 0.4L_0$ for members supporting two or more floors

$L \geq 0.5L_0$ for members supporting one floor

$L_0=4.79 \text{ kN/m}^2$ for public rooms from ASCE 7-4 is used to calculate live load reduction for Basement.

Table 3.2.1.1. Live load reduction for columns of Floor 0 (Basement).

Structural element	Tributary area, m ²	K_{LL}	L, kN/m ²
Column A	75	4	2.46
Column B	37.5	1	4.77
Column C	75	4	2.46
Column D	150	4	2.09
Beam A	75	2	2.98
Beam B	150	2	2.46
Beam C	75	2	2.98
Beam D	150	2	2.46

$L_0=3.59 \text{ kN/m}^2$ for retail stores for upper floors in residential buildings from ASCE 7-4 is used to calculate live load reduction for Floor 1.

Table 3.2.1.2. Live load reduction for columns and beams of Floor 1.

Structural element	Tributary area, m ²	K_{LL}	L, kN/m ²
Column A	75	4	1.84
Column B	37.5	1	3.58

Column C	75	4	1.84
Column D	150	4	1.57
Beam A	75	2	2.24
Beam B	150	2	1.84
Beam C	75	2	2.24
Beam D	150	2	1.84

$L_0=4.79 \text{ kN/m}^2$ for gymnasium from ASCE 7-4 is used to calculate live load reduction for Floor 2.

Table 3.2.1.3. Live load reduction for columns of Floor 2.

Structural element	Tributary area, m²	K_{LL}	L, kN/m²
Column A	75	4	2.46
Column B	37.5	1	4.77
Column C	75	4	2.46
Column D	150	4	2.09
Beam A	75	2	2.98
Beam B	150	2	2.46
Beam C	75	2	2.98
Beam D	150	2	2.46

$L_0=1.92 \text{ kN/m}^2$ for public rooms in residential buildings from ASCE 7-4 is used to calculate live load reduction for Floor 3.

Table 3.2.1.4. Live load reduction for columns of Floor 3.

Structural element	Tributary area, m ²	K_{LL}	L, kN/m ²
Column A	75	4	0.99
Column B	37.5	1	1.91
Column C	75	4	0.99
Column D	150	4	0.84
Beam A	75	2	1.2
Beam B	150	2	0.99
Beam C	75	2	1.2
Beam D	150	2	0.99

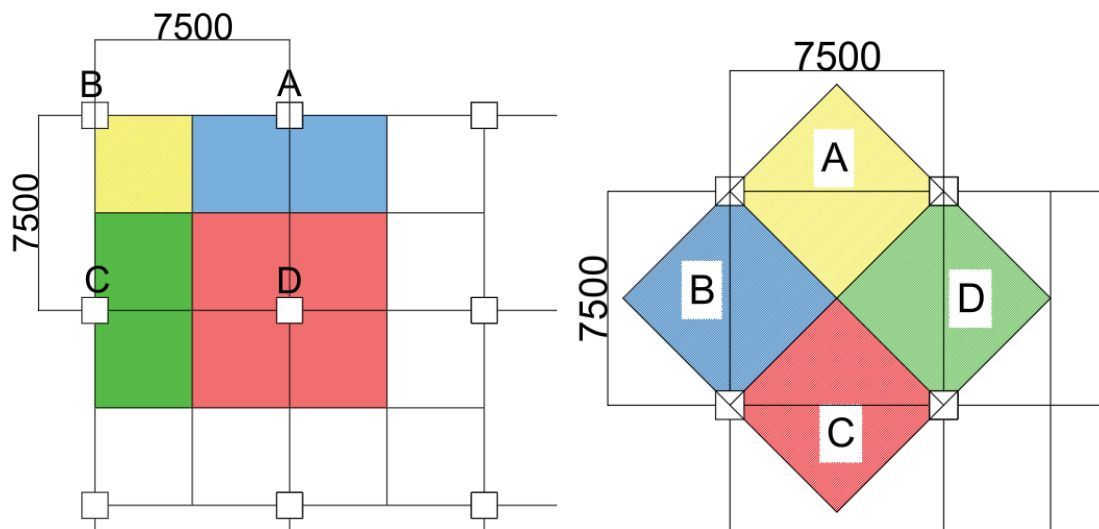


Figure 3.2.1.2. Column placement for Floor 4-8

$L_0=1.92 \text{ kN/m}^2$ for public rooms in residential buildings from ASCE 7-4 is used to calculate live load reduction for Floor 4-7.

Table 3.2.1.5. Live load reduction for columns and beams of Floors 4-7.

Structural element	Tributary area, m^2	K_{LL}	L , kN/m^2
Column A	28.1	4	1.31
Column B	14.1	1	Not applicable
Column C	28.1	4	1.31
Column D	56.2	4	1.07
Beam A, B, C, D	56.3	2	1.31

3.2.1.1 Roof live load reduction

To find the roof live load reduction, the following formula is used:

$$L_r = L_0 * R_1 * R_2, \text{ where}$$

L_r - roof reduction live load, must be in the range of 0.58 to 0.96 kPa.

L_0 - minimum distributed load according to ASCE 7-10.

R_1 - reduction factor based on tributary area:

R_2 - reduction factor based on roof slope

$L_0=0.96 \text{ kN/m}^2$ for public rooms in residential buildings

Table 3.2.1.6. Roof live load reduction.

Structural element	Tributary area, m^2	R1	R2	L , kN/m^2
Column A	28.1	0.89	1	0.85

Column B	14.1	1	1	0.96
Column C	28.1	0.89	1	0.85
Column D	56.2	0.6	1	0.58
Beam A, B, C, D	56.3	0.6	1	0.58

3.2.2. Calculation of Dead Load

Table 3.2.2.1. Floor dead load for basement

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed	50	1.15
Industrial epoxy finish	2-3	0.12
Bituminous Smooth Surface Membrane	-	0.03
Total		10.675

First Floor

Table 3.2.2.2. Floor dead load for Lobby/Hall

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed	50	1.15
Ceramic Tile (19 mm) on Mortar Bed (13 mm)	32	0.77
Bituminous Smooth Surface	-	0.03

Membrane		
Total		11.325

Table 3.2.2.3. Floor dead load for Restaurant

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed	60	1.38
Ceramic / Quarry Tile (19 mm) on 25 mm Mortar	44	1.10
Bituminous Smooth Surface Membrane	-	0.03
Total		11.885

Table 3.2.2.4. Floor dead load for billiards

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed	40	0.92
Wood or Vinyl Floor Finish on 15 mm plywood	20	0.40
Bituminous Smooth Surface Membrane	-	0.03
Total		10.725

Table 3.2.2.5. Floor dead load for supermarket

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed	50	1.15
Ceramic Tile (19 mm) on 20 mm Mortar Bed	39	0.9
Bituminous Smooth Surface Membrane	-	0.03
Total		11.455

Table 3.2.2.6. Floor dead load for toilets

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed	60	1.38
Bituminous Gravel-Covered Waterproof Membrane	-	0.26
Ceramic Tile (19 mm) on 25 mm Mortar Bed	44	1.10
Total		12.115

Table 3.2.2.7. Floor dead load for kitchen

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Slab	375	9.375
Cement Screed – Stone Concrete	70	1.61
Bituminous Gravel-Covered Waterproof Membrane	-	0.26
Ceramic Tile (19 mm) on 25 mm Mortar Bed	44	1.10
Total		12.345

Table 3.2.2.8. Floor dead load for staircase

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete Stair Slab / Flight (Normal Weight)	375	9.375
Cement Finish (25 mm) on Stone-Concrete Fill	25	1.53
Ceramic or Granite Tile (19 mm) on 25 mm Mortar Bed	44	1.10
Bituminous Smooth Surface Membrane	-	0.03
Total		12.035

Second floor

Table 3.2.2.9. Floor dead load for retail units

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete – Normal Weight	375	9.375
Cement screed (stone concrete)	50	1.15
Ceramic / Quarry tile (19 mm) on 25 mm mortar bed	44	1.10
Bituminous Smooth Surface Membrane	-	0.03
Total		10.555

Table 3.2.2.10. Floor dead load for ceiling

Material	Thickness (mm)	Load (KPa)
Gypsum board	13 - 16	0.131
Suspended steel grid	100 - 150	0.096
Mechanical ducts + allowance	250 - 300	0.192
Total		0.42

Third floor

Table 3.2.2.11. Floor dead load for conference room

Material	Thickness (mm)	Load (KPa)
Carpet with padding	10	0.15
Lightweight concrete fill	50	0.75
Reinforced concrete slab	375	9.375
Suspended gypsum board + ducts	350	0.42
Steel studs + gypsum on both sides	100	0.38
Electrical and HVAC fixtures	-	0.10
Total		11.175

Table 3.2.2.12. Floor dead load for gym

Material	Thickness (mm)	Load (KPa)
Rubber flooring on a concrete base	25	0.50
Lightweight concrete fill	50	0.75
Reinforced concrete slab	375	9.375
Suspended gypsum board + ducts	350	0.42

Steel studs + gypsum on both sides	100	0.38
Ducts, fans, conduits	-	0.10
Total		11.525

Table 3.2.2.13. Floor dead load for SPA salon

Material	Thickness (mm)	Load (KPa)
Ceramic tile (19 mm) on 13 mm mortar	32	0.77
Lightweight concrete fill	50	0.75
Reinforced concrete slab	375	9.375
Suspended gypsum board + ducts	350	0.42
Moisture-resistant gypsum board	100	0.40
Plumbing, HVAC	-	0.15
Total		11.865

Table 3.2.2.14. Floor dead load for children's playroom

Material	Thickness (mm)	Load (KPa)
Vinyl/linoleum flooring	6	0.05
Lightweight concrete fill	50	0.75
Reinforced concrete slab	375	9.375

Suspended gypsum board + ducts	350	0.42
Steel studs + gypsum both sides	100	0.38
Lights, ducts, wiring	-	0.10
Total		11.075

4-8 floors

Table 3.2.2.15. Floor dead load for 4-8

Material	Thickness (mm)	Load (KPa)
Reinforced Concrete – Normal Weight	200	5.0
Lightweight Concrete Leveling Fill	40	0.60
Vinyl or Linoleum Floor Covering	6	0.05
Ceramic Tile (19 mm) on 13 mm Mortar Bed	32	0.77
Suspended Gypsum Board + Steel Grid + Duct Allowance	350	0.42
Light Steel Studs with Gypsum Both Sides	100	0.38
Plumbing, Wiring, Fixtures	-	0.10
Total		7.32

Table 3.2.2.16. Floor dead load for interior walls

Material	Thickness (mm)	Load (KPa)
Gypsum board (each side)	12.5 × 2	0.20
Fibrous glass insulation (between studs)	50	0.105
Studs/tracks/fasteners (allowance)	-	0.07
Interior paint	-	0.06
CMU wall	100	1.05
Plumbing, Wiring, Fixtures	-	0.10
Total		1.54

Table 3.2.2.17. Floor dead load for exterior walls

Material	Thickness (mm)	Load (KPa)
Hollow/solid concrete masonry (CMU)	254	4.06
Exterior cement plaster	20–30	0.24
Interior plaster	20	0.24
Fibrous glass insulation (continuous)	40	0.084
Vapor/wind barrier (single-ply sheet)	-	0.03
Interior gypsum board (room	13	0.104

side)		
Total		4.758

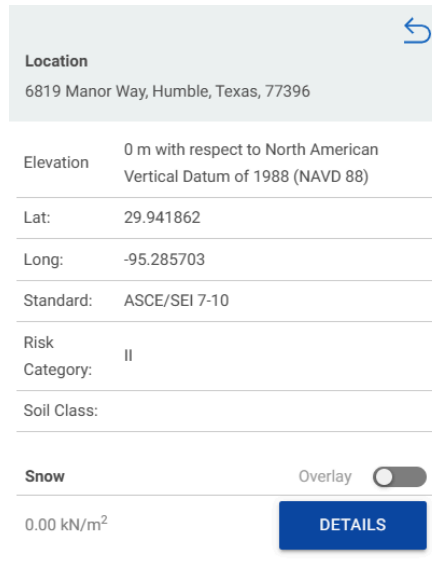
Table 3.2.2.18. Dead load from interior walls to slab

Location	Height, m	Length, m	Load, KN	Total area, m ²	Floor load, KPa
Basement	3.3	129.9	660.152	2400	0.275
Level 1	5.3	250.95	2048.254		0.853
Level 2	5.3	230.2	1878.892		0.783
Level 3	5.3	227.9	1860.120		0.775
Level 4	3.3	379.1	1926.586	1500	1.284
Level 5-8	3.3	409.8	2082.604		1.388
Level 9	2.5	66	254.1	135	1.882

Table 3.2.2.19. Total dead load values

Location	Pressure Load, KPa	Exterior wall load, kN/m
Basement Slab	11.370	15.7014
Slab 1	13.618	25.2174
Slab 2	11.758	25.2174
Slab 3	13.060	25.2174
Slab 4	9.024	15.7014
Slab 5-8	9.128	15.7014
Slab 9	9.622	0
Slab 10	7.74	0

3.2.3. Calculation of Snow Load



The screenshot shows a web-based tool for calculating snow load. It includes a location input field with the address "6819 Manor Way, Humble, Texas, 77396". Below this, there are fields for Elevation (0 m with respect to North American Vertical Datum of 1988 (NAVD 88)), Latitude (29.941862), Longitude (-95.285703), Standard (ASCE/SEI 7-10), Risk Category (II), and Soil Class. A "Snow" section at the bottom shows a value of 0.00 kN/m² and an "Overlay" toggle switch. A "DETAILS" button is located at the bottom right of the form.

Figure 3.2.3.1 Snow load values from the Hazard tool

According to the ASCE 7 – 10 code, the $p_g = 0 \text{ kN/m}^2$

3.2.4. Structural member sizing

3.2.4.1. Beam size estimation

$$M_u = \frac{\omega l^2}{8} = \frac{12.2456 * 3.75 * 7.5^2}{8} = 322.88 \text{ kN} * \text{m}$$

$$\frac{M_u}{\phi b d^2} = \rho f_y \left(1 - \frac{1}{1.7} \frac{\rho f_y}{f_c}\right)$$

$$\frac{322.88 * 2}{0.9 b d^2} = 0.012 * 420000 \left(1 - \frac{1}{1.7} \frac{0.012 * 420000}{34470}\right) \Rightarrow d = 0.54 \text{ m}$$

Depth of the beam for upper floors will be taken as 0.6 m, and width as 0.3m.

3.2.4.2. Column size estimation

In order to estimate the column size, the axial load on each column must be calculated.

The load combination for the axial load is the following:

$$U = 1.2DL + 1.6LL + 0.5L_r$$

$$R = \text{Factored load} * \text{Area.}$$

Table 3.2.4.1. Calculation of axial loads on columns of each floor

	Live load, kN/m ²	Dead load, kN/m ²	Roof live load, kN/m ²	Area, m ²	Factored load, kN/m ²	R, kN
Slab 10	0	7.74	0.58	56.25	9.59	701
Slab 9	0	9.622	0.58	56.25	10.8236	771
Slab 5-8	1.07	9.128	0	56.25	12.2456	1283
Slab 4	1.07	9.024	0	150	12.1796	2682
Slab 3	0.84	13.060	0	150	15.9456	3121
Slab 2	2.09	11.758	0	150	18.7064	3535
Slab 1	1.57	13.618	0	150	18.6736	3530
Basement	2.09	11.370	0	150	16.154	3152

The lengths of the walls and beams applied on the column are calculated using the following formula:

$$L_{bw} = (L_b + L_b + L_c + L_c)/2 = (20 + 20 + 7.5 + 7.5)/2 = 27.5m$$

$$SW_{b/w} = 24kN/m^3 * L_{bw} * A$$

Table 3.2.4.2. Calculation of loads for each column at different floors

Loads	Load on slabs, kN	Load from beams, kN	Load from walls, kN	Total load, kN
Slab 10	539	161.7	0	
Slab 9	609	161.7	0	1785
Slab 5-8	689	161.7	432	2430
Slab 4	1827	161.7	693	2682
Slab 3	2392	297	432	3121
Slab 2	2806	297	432	3535

Slab 1	2801	297	432	3530
Basement	2423.1	297	432	3152

Table 3.2.4.3. Calculation of combined axial loads on columns

Column	Axial load, kN
Floor 9	771
Floor 8	2053
Floor 7	3336
Floor 6	4618
Floor 5	5901
Floor 4	8582
Floor 3	11703
Floor 2	15238
Floor 1	18767
Basement	21920

The building will have a high-strength concrete of 41.37 MPa for the basement and first 3 floors and 35MPa for the rest floors, for the rebar reinforcement steel of 520MPa. According to ACI 318, the value ϕ for the tied columns is 0.65. The minimum reinforcement ratio for the column is 1%. However, for the future purposes and avoiding recalculations 1% is taken as an assumption for the safety reasons, since our building is in a strong wind zone, which requires high tensile strength of the column.

Table 3.2.4.4. Column size estimation

Column parameters	B, mm	Reinforcement ratio	f'_c , kPa	f_y , MPa	ϕ
Basement	1050	1%	41370	520	0.65
Floor 1-3	950	1%	41370	520	0.65
Floor 4-8	800	1%	34470	420	0.65

The strength of columns must be greater than the applied loads. Therefore, the gross area of columns is found by equalizing $\phi P_n \geq P_u$.

Column size estimation for the basement:

$$\phi P_n = 0.8\phi(0.85f'_c(A_g - A_s) + f_y A_s)$$

$$\phi P_n = 0.8 * 0.65(0.85 * 41370 * 0.99A_g + 520000 * 0.01A_g) = 21920$$

$$b=1.026\text{m}$$

1050 mm will be taken as the column dimensions for the basement

Column size estimation for 1-3 floors:

$$\phi P_n = 0.8\phi(0.85f'_c(A_g - A_s) + f_y A_s)$$

$$\phi P_n = 0.8 * 0.65(0.85 * 41370 * 0.99 + 520000 * 0.01A_g) = 18767$$

$$b=0.95\text{m}$$

950 mm will be taken as column dimensions for 1-3 floors

Column size estimation for floors 4-8:

$$\phi P_n = 0.8\phi(0.85f'_c(A_g - A_s) + f_y A_s)$$

$$\phi P_n = 0.8 * 0.65(0.85 * 34470 * 0.99 + 420000 * 0.01) = 8582$$

$$b=0.705$$

800 mm will be taken as the column dimension for 4-8 floors.

Maximum axial strength is calculated according to ACI 318, R 22.4.2.1:

$$\phi P_n = 0.8\phi(0.85f_c(A_g - A_s) + f_y A_s)$$

Reinforcement of concrete will be taken as 1% for the first 3 floors, since the building has long spans. For 4-8 floors 1% of reinforcement can be considered as appropriate.

3.2.4.3. Assigning structural properties in SAP2000

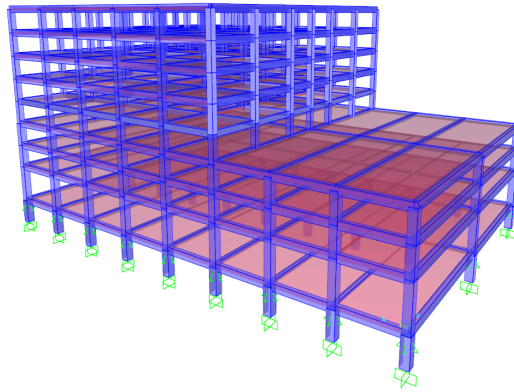


Figure 3.2.4.1. Model geometry in SAP2000

Material Property Data	
General Data	
Material Name and Display Color	A615Gr75
Material Type	Rebar
Material Notes	Modify/Show Notes...
Weight and Mass	
Weight per Unit Volume	76.9729
Mass per Unit Volume	7.849
Uniaxial Property Data	
Modulus of Elasticity, E	1.999E+08
Poisson	0.3
Coefficient of Thermal Expansion, A	1.170E-05
Shear Modulus, G	76903069
Other Properties for Rebar Materials	
Minimum Yield Stress, Fy	517106.8
Minimum Tensile Stress, Fu	689475.8
Expected Yield Stress, Fye	566817.5
Expected Tensile Stress, Fue	758423.4
☐ Switch To Advanced Property Display	
OK Cancel	

Figure 3.2.4.2. Properties of rebar

Material Property Data

General Data

Material Name and Display Color: 6000Psi [Color: Blue]

Material Type: Concrete

Material Notes: [Modify/Show Notes...]

Weight and Mass

Weight per Unit Volume: 24

Mass per Unit Volume: 2.4028

Units

Units: KN, m, C

Isotropic Property Data

Modulus of Elasticity, E: 30441742

Poisson: 0.2

Coefficient of Thermal Expansion, A: 9.900E-06

Shear Modulus, G: 12684059

Other Properties for Concrete Materials

Specified Concrete Compressive Strength, f_c: 41370

☐ Lightweight Concrete

Shear Strength Reduction Factor: []

☐ Switch To Advanced Property Display

[OK] [Cancel]

Figure 3.2.4.3. Properties for concrete

Rectangular Section

Section Name: beam 0-3

Display Color: [Color: Yellow]

Section Notes: [Modify/Show Notes...]

Dimensions

Depth (t₃): 0.9

Width (t₂): 0.45

Section

[Diagram showing a rectangular section on a grid with dimensions 0.9 and 0.45]

Material

+ 6000Psi

Property Modifiers

[Set Modifiers...]

Properties

[Section Properties...]

[Time Dependent Properties...]

[Concrete Reinforcement...]

[OK] [Cancel]

Figure 3.2.4.4. Beam dimensions for basement and 1-3 floors

Rectangular Section

Section Name: Display Color: ■

Section Notes:

Dimensions

Depth (t3):

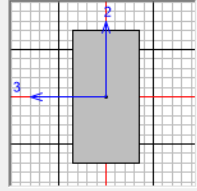
Width (t2):

Material

+ ▾

Property Modifiers

Section



Properties

Figure 3.2.4.5. Beam dimensions for 4-8 floors

Rectangular Section

Section Name: Display Color: ■

Section Notes:

Dimensions

Depth (t3):

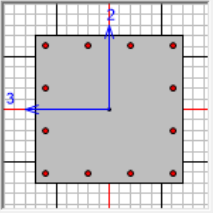
Width (t2):

Material

+ ▾

Property Modifiers

Section



Properties

Figure 3.2.4.6. Column dimensions for basement

Rectangular Section

Section Name: column floor 1-3 Display Color: ■

Section Notes: [Modify/Show Notes...](#)

Dimensions

Depth (t3): 0.95

Width (t2): 0.95

Material

+ 6000Psi

Property Modifiers

[Set Modifiers...](#)

[Concrete Reinforcement...](#)

[Section Properties...](#)

[Time Dependent Properties...](#)

[OK](#) [Cancel](#)

Section

Figure 3.2.4.7. Column dimensions for 1-3 floors

Rectangular Section

Section Name: column 4-8 floors Display Color: ■

Section Notes: [Modify/Show Notes...](#)

Dimensions

Depth (t3): 0.8

Width (t2): 0.8

Material

+ 5000Psi

Property Modifiers

[Set Modifiers...](#)

[Concrete Reinforcement...](#)

[Section Properties...](#)

[Time Dependent Properties...](#)

[OK](#) [Cancel](#)

Section

Figure 3.2.4.8. Column dimensions for 4-8 floors

3.2.4.4. Assigning gravity loads in SAP2000

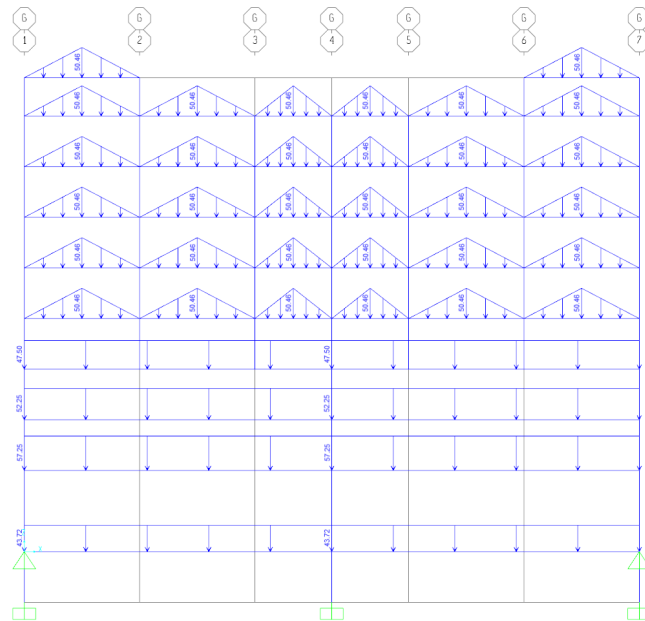


Figure 3.2.4.9. Dead Load Assignment for 2D Frame

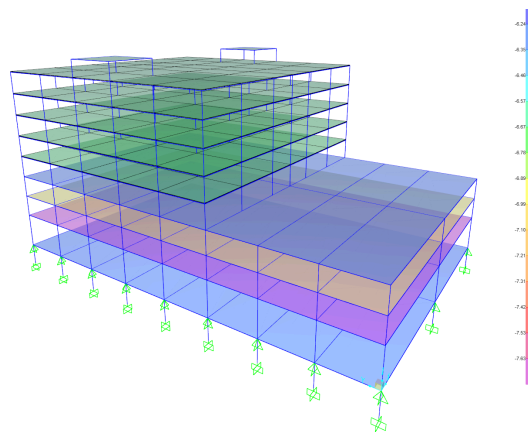


Figure 3.2.4.10. Dead Load Assignment for 3D Frame

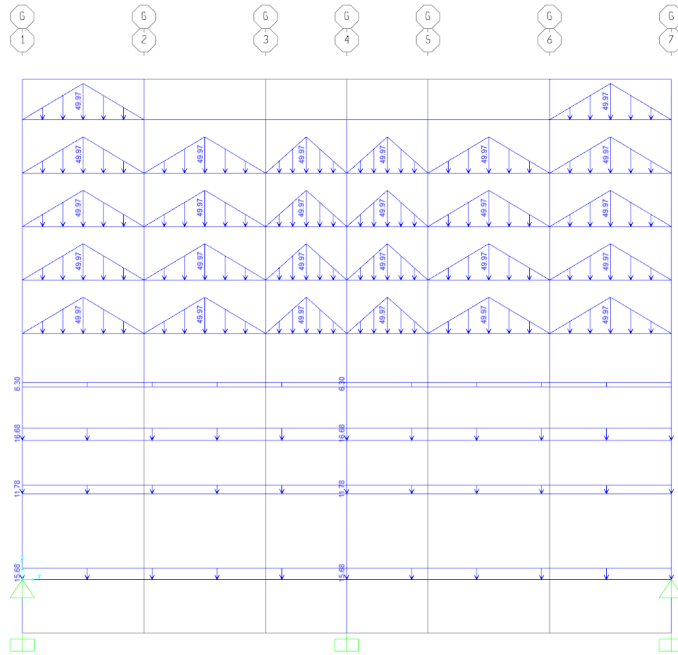


Figure 3.2.4.11. Live Load Assignment for 2D Frame

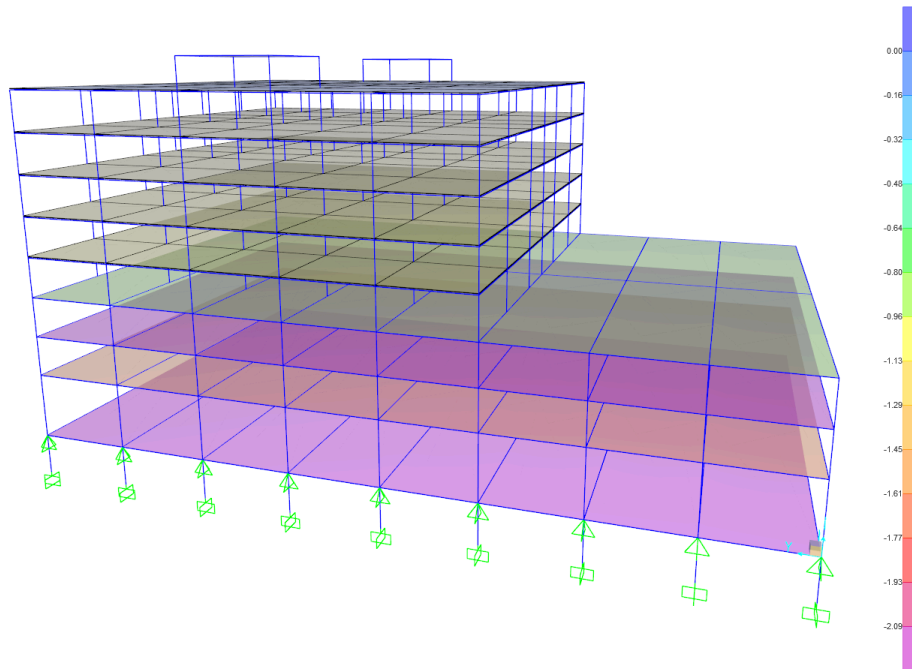


Figure 3.2.4.12. Live load assignment for 3D Frame

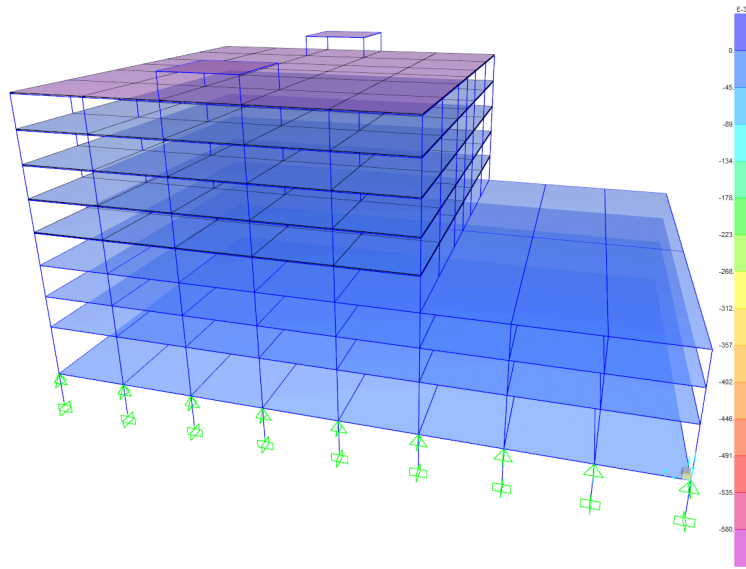


Figure 3.2.4.13. Live roof assignment 3D Frame

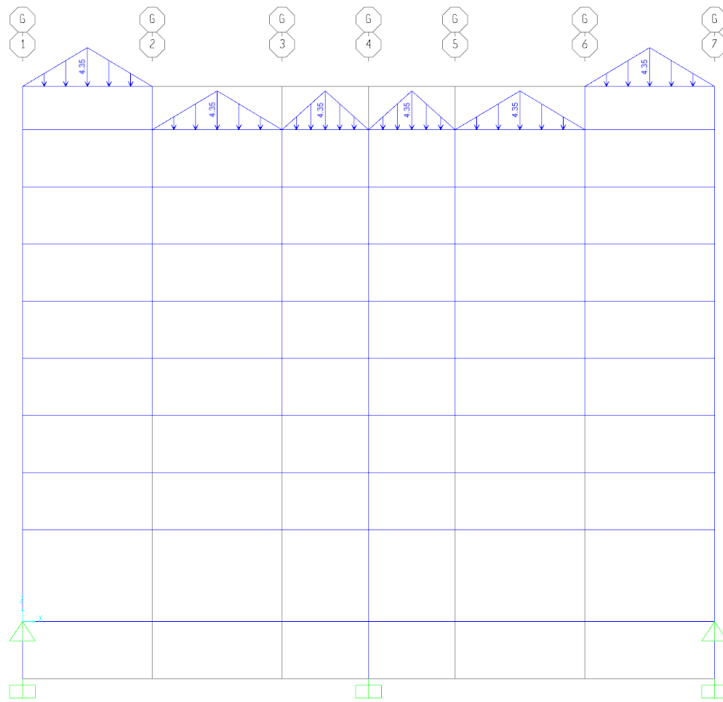


Figure 3.2.4.14. Live Roof load Assignment for 2D Frame

3.3. Analysis and design of Lateral Load Resisting System

3.3.1. Wind force

According to ASCE 7-10, the 50-year MRI wind velocity is 46 m/s

Wind Details	
Wind Speed	59 m/s
10-year MRI	34 m/s
25-year MRI	41 m/s
50-year MRI	46 m/s
100-year MRI	50 m/s

Figure 3.3.1. Wind Speed according to the Hazard tool.

Exposure	α	z_g (m)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (m)	$\bar{\epsilon}$	z_{min} (m)*
B	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
C	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
D	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

Figure 3.3.2. Exposure categories.

The location is in the suburbs, which means there are many surrounding buildings, trees, and other fixed structures that minimize the exposure to wind. Thus, it may be concluded that on the basis of ASCE 7-10 definitions, the terrain falls under the Exposure Category B (urban and suburban area with closely spaced obstructions).

Wind Directionality Factor (k_d): 0.85

Topographic Factor (k_{zt}): 1.0

Constants for Exposure Category B:

Gradient height (z_g): 365.76 m

Power law exponent (α): 7.0

For a height $z \geq 4.57$ m (15 ft):

$$k_z = 2.01 \left(\frac{z}{z_g} \right)^{2/\alpha}$$

For a height $z < 4.57$ m (15 ft):

$$k_z = 2.01 \left(\frac{4.57}{z_g} \right)^{2/\alpha}$$

Example Calculation for Floor 1

Height of Floor 1, $z_1=5.3$ m

Since $5.3 \text{ m} > 4.57 \text{ m}$, the first formula (for heights higher 15 ft) is used.

$$k_z = 2.01 \left(\frac{5.3}{365.76} \right)^{2/7} = 0.599$$

Table 3.3.1. k_z values.

Floor #	Height	k_z
8	32.4	1.0056
7	29.1	0.9752
6	25.8	0.9423
5	22.5	0.9061
4	19.2	0.8660
3	15.9	0.8206
2	10.6	0.7308
1	5.3	0.5995

$$h = 32.4 \text{ m} \approx 106.299 \text{ ft}$$

$$n_a = \frac{43.5}{h^{0.9}} = \frac{43.5}{106.299^{0.9}} = 0.652 \text{ Hz}; n_a < 1 \Rightarrow \text{the building is flexible}$$

$$\bar{z} = \max(0.6h, z_{min}) = \max(19.44, 9.14) = 19.44 \text{ m}$$

$$g_Q = g_v = 3.4$$

$$\beta = 0.02 (2\%)$$

Case I : Wind parallel to Y with L (1-3) = 60 m, L (4-8) = 37.5 m, and B = 40m

$$I_z = c * \left(\frac{10}{\bar{z}}\right)^{1/6} = 0.3 * \left(\frac{10}{19.44}\right)^{1/6} = 0.2685$$

$$L_z = l * \left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} = 97.54 * \left(\frac{19.44}{10}\right)^{1/3} = 121.734$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{40+32.4}{121.734}\right)^{0.63}}} = 0.829$$

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} = \sqrt{2\ln(3600 * 0.652)} + \frac{0.577}{\sqrt{2\ln(3600*0.652)}} = 4.086$$

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{a}} * V = 0.45\left(\frac{19.44}{10}\right)^{0.25} * 46 = 24.442 \text{ m/s}$$

$$N_1 = \frac{n_1 * L_z}{\bar{V}_z} = \frac{0.652 * 121.734}{24.442} = 3.247$$

$$R_n = \frac{N_1 * 7.47}{(1+10.3N_1)^{5/3}} = \frac{3.247 * 7.47}{(1+10.3 * 3.247)^{5/3}} = 0.0665$$

$$\eta_B = \frac{4.6 * n_1 * B}{\bar{V}_z} = \frac{4.6 * 0.652 * 40}{24.442} = 4.908 \Rightarrow R_B = \frac{1}{4.908} - \frac{1}{2 * 4.908^2} (1 - e^{-2 * 4.908}) = 0.183$$

$$\eta_h = \frac{4.6 * n_1 * h}{\bar{V}_z} = \frac{4.6 * 0.652 * 32.4}{24.442} = 3.975 \Rightarrow R_h = \frac{1}{3.975} - \frac{1}{2 * 3.975^2} (1 - e^{-2 * 3.975}) = 0.219$$

$$\eta_{L(1-3)} = \frac{15.4 * n_1^L}{\bar{V}_z} = \frac{15.4 * 0.652 * 60}{24.442} = 24.65 \Rightarrow R_{L(1-3)}$$

$$\eta_{L(1-3)} = \frac{1}{24.65} - \frac{1}{2 * 24.65^2} (1 - e^{-2 * 24.65}) = 0.0397$$

$$\eta_{L(4-8)} = \frac{15.4 * n_1^L}{\bar{V}_z} = \frac{15.4 * 0.652 * 37.5}{24.442} = 15.405 \Rightarrow R_{L(4-8)} = \frac{1}{15.405} - \frac{1}{2 * 15.405^2} (1 - e^{-2 * 15.405}) = 0.0628$$

$$R_{(1-3)} = \sqrt{\frac{1}{\beta} R_h R_B R_N (0.53 + 0.47 R_L)}$$

$$R_{(1-3)} = \sqrt{\frac{1}{0.02} 0.219 * 0.183 * 0.0665 (0.53 + 0.47 * 0.0397)} = 0.270$$

$$R_{(4-8)} = \sqrt{\frac{1}{\beta} R_h R_B R_N (0.53 + 0.47 R_L)}$$

$$R_{(4-8)} = \sqrt{\frac{1}{0.02} 0.219 * 0.183 * 0.0665 (0.53 + 0.47 * 0.0628)} = 0.273$$

$$G_{f(1-3)} = 0.925 \left(\frac{1 + 1.7 * I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 * g_v * I_z} \right) = 0.925 \left(\frac{1 + 1.7 * 0.2685 \sqrt{3.4^2 * 0.829^2 + 4.086 * 0.27^2}}{1 + 1.7 * 3.4 * 0.2685} \right) = 0.837$$

$$G_{f(4-8)} = 0.925 \left(\frac{1 + 1.7 * I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 * g_v * I_z} \right) = 0.925 \left(\frac{1 + 1.7 * 0.2685 \sqrt{3.4^2 * 0.829^2 + 4.086 * 0.273^2}}{1 + 1.7 * 3.4 * 0.2685} \right) = 0.8376$$

Case II: Wind perpendicular to the long axis with L = 40 m, and B (1-3)=60 m, B (4-8) = 37.5 m

$$I_z = c * \left(\frac{10}{z} \right)^{1/6} = 0.3 * \left(\frac{10}{19.44} \right)^{1/6} = 0.2685$$

$$L_z = l * \left(\frac{z}{10} \right)^{\bar{\epsilon}} = 97.54 * \left(\frac{19.44}{10} \right)^{1/3} = 121.734$$

$$Q_{1-3} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B+h}{L_z} \right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{60+32.4}{121.734} \right)^{0.63}}} = 0.8085$$

$$Q_{4-8} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B+h}{L_z} \right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{37.5+32.4}{121.734} \right)^{0.63}}} = 0.832$$

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} = \sqrt{2\ln(3600 * 0.652)} + \frac{0.577}{\sqrt{2\ln(3600*0.652)}} = 4.086$$

$$\bar{V}_z = \bar{b}(\frac{\bar{z}}{10})^{\bar{a}} * V = 0.45(\frac{19.44}{10})^{0.25} * 46 = 24.44 \text{ m/s}$$

$$N_1 = \frac{n_1 * L_z}{\bar{V}_z} = \frac{0.652 * 121.734}{24.44} = 3.247$$

$$R_n = \frac{N_1^{*7.47}}{(1+10.3N_1)^{5/3}} = \frac{3.247^{*7.47}}{(1+10.3*3.247)^{5/3}} = 0.0665$$

$$\eta_{B(1-3)} = \frac{4.6 * n_1^{*B}}{\bar{V}_z} = \frac{4.6 * 0.652 * 60}{24.44} = 7.363 \Rightarrow R_{B(1-3)} = \frac{1}{7.363} - \frac{1}{2 * 7.363^2} (1 - e^{-2 * 7.363}) = 0.127$$

$$\eta_{B(4-8)} = \frac{4.6 * n_1^{*B}}{\bar{V}_z} = \frac{4.6 * 0.652 * 37.5}{24.44} = 4.6 \Rightarrow R_{B(4-8)} = \frac{1}{4.6} - \frac{1}{2 * 4.6^2} (1 - e^{-2 * 4.6}) = 0.194$$

$$\eta_h = \frac{4.6 * n_1^{*h}}{\bar{V}_z} = \frac{4.6 * 0.652 * 32.4}{24.44} = 3.976 \Rightarrow R_h = \frac{1}{3.976} - \frac{1}{2 * 3.976^2} (1 - e^{-2 * 3.976}) = 0.219$$

$$\eta_L = \frac{15.4 * n_1^{*L}}{\bar{V}_z} = \frac{15.4 * 0.652 * 40}{24.44} = 16.43 \Rightarrow R_L = \frac{1}{16.43} - \frac{1}{2 * 16.43^2} (1 - e^{-2 * 16.43}) = 0.059$$

$$R_{(1-3)} = \sqrt{\frac{1}{\beta} R_h R_B R_N (0.53 + 0.47 R_L)}$$

$$R_{(1-3)} = \sqrt{\frac{1}{0.02} 0.219 * 0.127 * 0.0665 (0.53 + 0.47 * 0.059)} = 0.227$$

$$R_{(4-8)} = \sqrt{\frac{1}{\beta} R_h R_B R_N (0.53 + 0.47 R_L)}$$

$$R_{(4-8)} = \sqrt{\frac{1}{0.02} 0.219 * 0.194 * 0.0665 (0.53 + 0.47 * 0.059)} = 0.2807$$

$$G_{f(1-3)} = 0.925 \left(\frac{1+1.7 * I_z \sqrt{g_Q^2 * Q^2 + g_R^2 * R^2}}{1+1.7 * g_v * I_z} \right) = 0.925 \left(\frac{1+1.7 * 0.2685 \sqrt{3.4^2 * 0.8085^2 + 4.086^2 * 0.227^2}}{1+1.7 * 3.4 * 0.2685} \right) = 0.842$$

$$G_{f(4-8)} = 0.925 \left(\frac{1+1.7 * I_z \sqrt{g_Q^2 * Q^2 + g_R^2 * R^2}}{1+1.7 * g_v * I_z} \right) = 0.925 \left(\frac{1+1.7 * 0.2685 \sqrt{3.4^2 * 0.832^2 + 4.086^2 * 0.2807^2}}{1+1.7 * 3.4 * 0.274} \right) = 0.857$$

Design Velocity pressure:

$$q_z = 0.613 * k_z * k_{zt} * k_d * V^2$$

Table 3.3.2. q_z values.

Floor #	k_{zt}	k_d	k_z	q_z (KPa)
8	1	0.85	1.0056	1.109
7			0.9752	1.075
6			0.9423	1.039
5			0.9061	0.999
4			0.8660	0.955
3			0.8206	0.905
2			0.7308	0.806
1			0.5995	0.661

According to the table, $q_{h=8} = 1.109 \text{ KPa}$

Design net pressure:

$$p^w = q_z * G_f * C_p^w$$

$$p^l = q_z * G_f * C_p^l$$

$$P = p^w - p^l$$

Table 3.3.3. C_p^w and C_p^l values regarding wind direction parallel to the longitude.

Floor #	L	B	L/B	C_p^w	C_p^l
1-3	60	40	1.5	0.8	-0.4
4-8	37.5	40	0.9375	0.8	-0.5

Table 3.3.4 C_p^w and C_p^l values regarding wind direction perpendicular to the longitude.

Floor #	L	B	L/B	C_p^w	C_p^l
1-3	40	60	0.67	0.8	-0.5
4-8	40	37.5	1.067	0.8	-0.49

Table 3.3.5. Net pressure for wind direction parallel to the long one.

Floor #	Cwp	Clp	Gf	qh	qz	Pw	Pl	Pnet
8	0.8	-0.5	0.8376	1.109	1.109	0.743	-0.464	1.207
7	0.8	-0.5	0.8376	1.109	1.075	0.720	-0.450	1.171
6	0.8	-0.5	0.8376	1.109	1.039	0.696	-0.435	1.131
5	0.8	-0.5	0.8376	1.109	0.999	0.669	-0.418	1.088
4	0.8	-0.5	0.8376	1.109	0.955	0.640	-0.400	1.040
3	0.8	-0.4	0.837	1.109	0.905	0.606	-0.303	0.909
2	0.8	-0.4	0.837	1.109	0.806	0.540	-0.270	0.809
1	0.8	-0.4	0.837	1.109	0.661	0.443	-0.221	0.664

Table 3.3.6. Net pressure for wind direction perpendicular to the long one.

Floor #	Cwp	Clp	Gf	qh	qz	Pw	Pl	Pnet
8	0.8	-0.49	0.857	1.109	1.109	0.760	-0.466	1.226
7	0.8	-0.49	0.857	1.109	1.075	0.737	-0.452	1.189
6	0.8	-0.49	0.857	1.109	1.039	0.712	-0.436	1.149
5	0.8	-0.49	0.857	1.109	0.999	0.685	-0.420	1.104
4	0.8	-0.49	0.857	1.109	0.955	0.655	-0.401	1.056
3	0.8	-0.5	0.842	1.109	0.905	0.609	-0.381	0.990
2	0.8	-0.5	0.842	1.109	0.806	0.543	-0.339	0.882
1	0.8	-0.5	0.842	1.109	0.661	0.445	-0.278	0.724

Wind Load in each direction was calculated using the following 2 formulas:

$$F_i = (p_{i+1} * \frac{h_{i+1}}{2} + p_i * \frac{h_i}{2}) * B$$

$$F_i = \frac{Force\ Y * C_f}{\Sigma C_f}$$

Table 3.3.7 Wind Forces in Y and X directions.

Floor	Height, m	Force on Y	Force on X
Roof	2.50	11.318	13.789
8.00	3.30	79.679	75.841
7.00	3.30	156.949	149.390
6.00	3.30	151.934	144.616
5.00	3.30	146.459	139.404

4.00	3.30	140.413	133.650
3.00	3.30	164.943	261.964
2.00	3.30	182.109	297.695
1.00	5.30	156.155	255.268

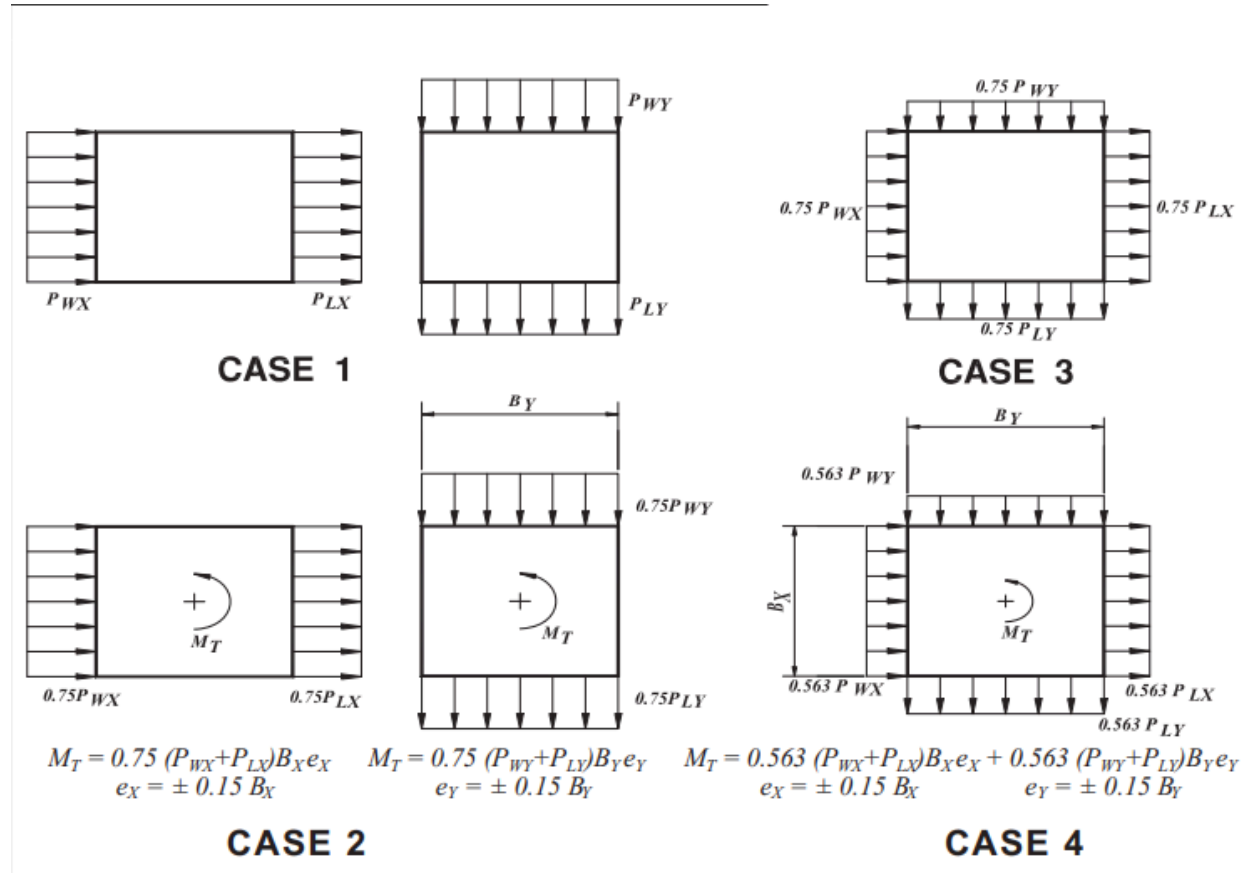


Figure 3.3.3. Wind load cases.

Table 3.3.8. Parameters of columns and values of D

Floor	Height, m	Ic, m ⁴	Ib, m ⁴	E, MPa	D, kN/m
Roof	2.50	0.0104128645	0.0046142578	27805.570	31693.699
8.00	3.30	0.0104128645	0.0046142578	27805.570	18189.680

7.00	3.30	0.0104128645	0.0046142578	27805.570	18189.680
6.00	3.30	0.0104128645	0.0046142578	27805.570	18189.680
5.00	3.30	0.0104128645	0.0046142578	27805.570	18189.680
4.00	3.30	0.0104128645	0.0046142578	27805.570	18189.680
3.00	5.30	0.0475128645	0.01063125	27805.570	18315.132
2.00	5.30	0.0475128645	0.01063125	27805.570	18315.132
1.00	5.30	0.0475128645	0.01063125	27805.570	18315.132
Basement	3.30	0.0709045312	0.01063125	27805.570	49356.233

$$I_{c,cr} = 0.7 * \frac{1}{12} * \left(\frac{h}{1000}\right)^4$$

$$I_{b,cr} = 0.35 * \frac{1}{12} * \left(\frac{h}{1000}\right)^3 * \left(\frac{b}{1000}\right)$$

$$D = 12 * E * 10^6 * \left(\frac{I_{b,cr} * I_{c,cr}}{5 * I_{c,cr} + I_{b,cr} * h}\right) * \frac{1}{h^2}$$

Table 3.3.9. Cf values for Y direction

Frame	1	2	3	4	5	6	7	8	Roof
1	776561.613	776561.613	776561.613	300129.725	300129.725	300129.725	300129.725	300129.725	0
2	0.000	0.000	0.000	300129.725	300129.725	300129.725	300129.725	300129.725	79234.24747
3	0.000	0.000	0.000	300129.725	300129.725	300129.725	300129.725	300129.725	79234.24747
4	776561.613	776561.613	2092704.290	300129.725	300129.725	300129.725	300129.725	300129.725	0
5	0.000	0.000	0.000	300129.725	300129.725	300129.725	300129.725	300129.725	79234.24747

6	0.000	0.000	0.000	300129. 725	300129. 725	300129. 725	300129. 725	300129. 725	79234.2 4747
7	776561. 613	776561. 613	209270 4.290	300129. 725	300129. 725	300129. 725	300129. 725	300129. 725	0
Sum	232968 4.840	232968 4.840	496197 0.193	210090 8.077	210090 8.077	210090 8.077	210090 8.077	210090 8.077	316936. 990

$$C_{fy} = D * h * \# \text{ of frames in this direction}$$

Table 3.3.10. Cf values for X direction

Frame	1	2	3	4	5	6	7	8	Roof
A	194140. 403	194140. 403	194140. 403	360155. 670	360155. 670	360155. 670	360155. 670	360155. 670	0.000
B	194140. 403	194140. 403	194140. 403	360155. 670	360155. 670	360155. 670	360155. 670	360155. 670	0.000
C	194140. 403	194140. 403	194140. 403	360155. 670	360155. 670	360155. 670	360155. 670	360155. 670	158468. 495
D	194140. 403	194140. 403	194140. 403	360155. 670	360155. 670	360155. 670	360155. 670	360155. 670	158468. 495
E	194140. 403	194140. 403	194140. 403	360155. 670	360155. 670	360155. 670	360155. 670	360155. 670	0.000
F	194140. 403	194140. 403	194140. 403	360155. 670	360155. 670	360155. 670	360155. 670	360155. 670	0.000
G	194140. 403	194140. 403	194140. 403	0.000	0.000	0.000	0.000	0.000	0.000
H	194140. 403	194140. 403	194140. 403	0.000	0.000	0.000	0.000	0.000	0.000
I	194140. 403	194140. 403	194140. 403	0.000	0.000	0.000	0.000	0.000	0.000
sum	174726 3.630	174726 3.630	174726 3.630	216093 4.022	216093 4.022	216093 4.022	216093 4.022	216093 4.022	316936. 990

$$C_{fx} = D * h * \# \text{ of frames in this direction}$$

1. Mass center without elevator

$$\bar{y}(1 - 3) = \frac{60}{2} = 30 \text{ m}$$

$$\bar{y}(4 - 8) = 60 - (37.5/2) = 41.25 \text{ m}$$

$$\bar{x}(1 - 8) = 40/2 = 20 \text{ m}$$

2. Stiffness center

$$\bar{y}(1 - 3) = \frac{\sum C_{fy} * y(SC)}{\text{Sum of } C_{fy}} = 30 \text{ m}$$

$$\bar{y}(4 - 8) = \frac{\sum C_{fy} * y(SC)}{\text{Sum of } C_{fy}} = 30 \text{ m}$$

$$\bar{x}(1 - 8) = \frac{\sum C_{fy} * x(SC)}{\text{Sum of } C_{fy}} = 20 \text{ m}$$

3. Eccentricity

$$e_{y(1-3)} = \bar{y}(MC) - \bar{y}(SC) = 30 - 30 = 0$$

$$e_{y(4-8)} = \bar{y}(MC) - \bar{y}(SC) = 41.25 - 30 = 11.25$$

$$e_x = \bar{x}(MC) - \bar{x}(SC) = 20 - 20 = 0$$

$$F_{\text{per frame}} = \frac{\text{Force on floor}}{\text{Total \# of spans in direction}} * \# \text{ of frame's span}$$

3.3.1.1. Case 1

Table 3.3.11. Design Wind Load Case 1 for Y direction

Floor	1	2	3	4	5	6	7	Total
Roof	2.830	2.830	0.000	0.000	0.000	2.830	2.830	11.318
8.00	11.383	11.383	11.383	11.383	11.383	11.383	11.383	79.679
7.00	22.421	22.421	22.421	22.421	22.421	22.421	22.421	156.949
6.00	21.705	21.705	21.705	21.705	21.705	21.705	21.705	151.934
5.00	20.923	20.923	20.923	20.923	20.923	20.923	20.923	146.459
4.00	20.059	20.059	20.059	20.059	20.059	20.059	20.059	140.413
3.00	25.814	0.000	0.000	25.814	0.000	0.000	25.814	77.442
2.00	60.703	0.000	0.000	60.703	0.000	0.000	60.703	182.109
1.00	52.052	0.000	0.000	52.052	0.000	0.000	52.052	156.155

Table 3.3.12. Design Wind Load Case 1 for X direction

Floor	A	B	C	D	E	F	G	H	I	Total
Roof	0.000	0.000	6.895	6.895	0.000	0.000	0.000	0.000	0.000	13.789
8.00	12.640	12.640	12.640	12.640	12.640	12.640	0.000	0.000	0.000	75.841
7.00	24.898	24.898	24.898	24.898	24.898	24.898	0.000	0.000	0.000	149.390
6.00	24.103	24.103	24.103	24.103	24.103	24.103	0.000	0.000	0.000	144.616
5.00	23.234	23.234	23.234	23.234	23.234	23.234	0.000	0.000	0.000	139.404
4.00	22.275	22.275	22.275	22.275	22.275	22.275	0.000	0.000	0.000	133.650
3.00	29.107	29.107	29.107	29.107	29.107	29.107	29.107	29.107	29.107	261.96

										4
2.00	33.077	33.077	33.077	33.077	33.077	33.077	33.077	33.077	33.077	297.69 5
1.00	28.363	28.363	28.363	28.363	28.363	28.363	28.363	28.363	28.363	255.26 8

$$M_T = 0.75(p^w + p^l) * B * e$$

$$e = 0.15 * B$$

$$F_{Torsion} = \frac{M_T * y^i * C_f}{\sum (y_i^2 * C_f + x_i^2 * C_f)}$$

$$F_{Total} = F_{direct} + F_{Torsion}$$

$$e_y = 0.15 * 40 = 6$$

$$e_{x(1-3)} = 0.15 * 60 = 9$$

$$e_{x(4-8)} = 0.15 * 37.5 = 5.25$$

3.3.1.2. Case 2

Table 3.3.13. Mt values for Y direction case 2

Floor	Force	0.75*force	Mt
Roof	11.318	8.489	146.427
8.00	79.679	59.759	1030.847
7.00	156.949	117.712	2030.528
6.00	151.934	113.951	1965.646
5.00	146.459	109.844	1894.813
4.00	140.413	105.310	659.066

3.00	164.943	123.707	631.859
2.00	182.109	136.582	742.244
1.00	156.155	117.116	819.491

Table 3.3.14. Mt values for X direction case 2

Floor	Force	0.75*Force	Mt
Roof	13.789	10.342	54.294
8.00	75.841	56.881	298.624
7.00	149.390	112.043	588.223
6.00	144.616	108.462	569.426
5.00	139.404	104.553	548.903
4.00	133.650	100.238	526.247
3.00	261.964	196.473	1768.257
2.00	297.695	223.271	2009.441
1.00	255.268	191.451	1723.059

Table 3.3.15. Design Wind Load Case 2 for Y direction

Floor	1	2	3	4	5	6	7	Total
Roof	1.247	1.575	1.903	2.122	2.341	2.669	2.997	8.489
8.00	2.375	4.686	6.996	8.537	10.078	12.388	14.699	59.759
7.00	4.678	9.229	13.781	16.816	19.851	24.402	28.954	117.712
6.00	4.528	8.935	13.341	16.279	19.216	23.623	28.029	113.951
5.00	4.365	8.613	12.860	15.692	18.524	22.771	27.019	109.844

4.00	11.104	12.582	14.059	15.044	16.029	17.507	18.984	105.310
3.00	11.673	0.000	0.000	0.000	19.361	0.000	27.049	58.082
2.00	35.556	0.000	0.000	45.527	0.000	0.000	55.498	136.582
1.00	29.068	0.000	0.000	39.039	0.000	0.000	49.010	117.116

Table 3.3.16. Design Wind Load Case 2 for X direction

Floor	A	B	C	D	E	F	G	H	I	Total
Roof	-0.315	-0.236	5.013	5.092	0.000	0.079	0.157	0.236	0.315	10.342
8.00	7.748	8.181	8.614	9.047	9.480	9.913	0.866	1.299	1.732	56.881
7.00	15.262	16.115	16.968	17.821	18.674	19.527	1.706	2.559	3.412	112.043
6.00	14.774	15.600	16.426	17.251	18.077	18.903	1.651	2.477	3.303	108.462
5.00	14.242	15.038	15.834	16.630	17.426	18.221	1.592	2.388	3.184	104.553
4.00	13.654	14.417	15.180	15.943	16.706	17.469	1.526	2.289	3.052	100.238
3.00	13.762	15.779	17.796	19.813	21.830	23.847	25.864	27.881	29.898	196.473
2.00	15.639	17.932	20.224	22.516	24.808	27.100	29.392	31.684	33.976	223.271
1.00	13.410	15.376	17.341	19.307	21.272	23.238	25.203	27.169	29.134	191.451

3.3.1.3. Case 3

During that case 75% of wind load Case 1 were used. The calculated result is below:

Table 3.3.17. Design Wind Load Case 3 for Y direction

Floor	1	2	3	4	5	6	7	Total
Roof	2.122	2.122	0.000	0.000	0.000	2.122	2.122	8.489
8.00	8.537	8.537	8.537	8.537	8.537	8.537	8.537	59.759
7.00	16.816	16.816	16.816	16.816	16.816	16.816	16.816	117.712
6.00	16.279	16.279	16.279	16.279	16.279	16.279	16.279	113.951
5.00	15.692	15.692	15.692	15.692	15.692	15.692	15.692	109.844
4.00	15.044	15.044	15.044	15.044	15.044	15.044	15.044	105.310
3.00	19.361	0.000	0.000	19.361	0.000	0.000	19.361	58.082
2.00	45.527	0.000	0.000	45.527	0.000	0.000	45.527	136.582
1.00	39.039	0.000	0.000	39.039	0.000	0.000	39.039	117.116

Table 3.3.18. Design Wind Load Case 3 for X direction

Floor	A	B	C	D	E	F	G	H	I	Total
Roof	0.000	0.000	5.171	5.171	0.000	0.000	0.000	0.000	0.000	10.342
8.00	9.480	9.480	9.480	9.480	9.480	9.480	0.000	0.000	0.000	56.881
7.00	18.674	18.674	18.674	18.674	18.674	18.674	0.000	0.000	0.000	112.043
6.00	18.077	18.077	18.077	18.077	18.077	18.077	0.000	0.000	0.000	108.462
5.00	17.426	17.426	17.426	17.426	17.426	17.426	0.000	0.000	0.000	104.553
4.00	16.706	16.706	16.706	16.706	16.706	16.706	0.000	0.000	0.000	100.238

3.00	21.830	21.830	21.830	21.830	21.830	21.830	21.830	21.830	21.830	196.47 3
2.00	24.808	24.808	24.808	24.808	24.808	24.808	24.808	24.808	24.808	223.27 1
1.00	21.272	21.272	21.272	21.272	21.272	21.272	21.272	21.272	21.272	191.45 1

3.3.1.3. Case 4

For that case, 56.3% of wind load Case 2 was taken. The torsion was considered. The formula for torsional moment and results of the load are below:

$$M_T = 0.563 * (p_x^w + p_x^l) * B_x * e_x + 0.563 * (p_y^w + p_y^l) * B_y * e_y$$

Table 3.3.19. Mt values for Y and X direction case 4

Floor	0.563*Force Y	0.563*Force X	Mt
Roof	6.372	7.763	150.674
8.00	44.859	42.698	997.990
7.00	88.362	84.107	1965.809
6.00	85.539	81.419	1902.994
5.00	82.456	78.484	1834.417
4.00	79.053	75.245	1758.692
3.00	92.863	147.486	1331.478
2.00	102.527	167.602	1495.076
1.00	87.915	143.716	1282.000

Table 3.3.20. Design Wind Load Case 4 for Y direction

Floor	1	2	3	4	5	6	7	Total
Roof	0.692	1.030	-0.225	0.000	0.225	2.156	2.494	6.372
8.00	0.443	2.680	4.917	6.408	7.900	10.137	12.374	44.859
7.00	0.872	5.279	9.685	12.623	15.561	19.968	24.375	88.362
6.00	0.844	5.110	9.376	12.220	15.064	19.330	23.596	85.539
5.00	0.813	4.926	9.038	11.779	14.521	18.633	22.746	82.456
4.00	0.780	4.722	8.665	11.293	13.922	17.864	21.807	79.053
3.00	-1.667	0.000	0.000	14.533	0.000	0.000	30.734	43.600
2.00	15.985	0.000	0.000	34.176	0.000	0.000	52.367	102.527
1.00	13.707	0.000	0.000	29.305	0.000	0.000	44.904	87.915

Table 3.3.21. Design Wind Load Case 4 for X direction

Floor	A	B	C	D	E	F	G	H	I	Total
Roof	-0.687	-0.516	3.538	3.710	0.000	0.172	0.344	0.516	0.687	7.763
8.00	2.563	3.701	4.840	5.978	7.116	8.255	2.277	3.415	4.554	42.698
7.00	5.048	7.291	9.533	11.775	14.018	16.260	4.485	6.727	8.969	84.107
6.00	4.887	7.058	9.228	11.399	13.570	15.741	4.341	6.512	8.683	81.419
5.00	4.711	6.803	8.896	10.988	13.081	15.173	4.185	6.277	8.370	78.484
4.00	4.516	6.522	8.529	10.535	12.541	14.547	4.012	6.018	8.024	75.245
3.00	10.312	11.831	13.350	14.869	16.387	17.906	19.425	20.944	22.462	147.486
2.00	11.801	13.506	15.212	16.917	18.622	20.328	22.033	23.739	25.444	167.602
1.00	10.119	11.581	13.044	14.506	15.968	17.431	18.893	20.356	21.818	143.716

3.3.2. Seismic Force

According to ASCE 7-10, the following values were identified:

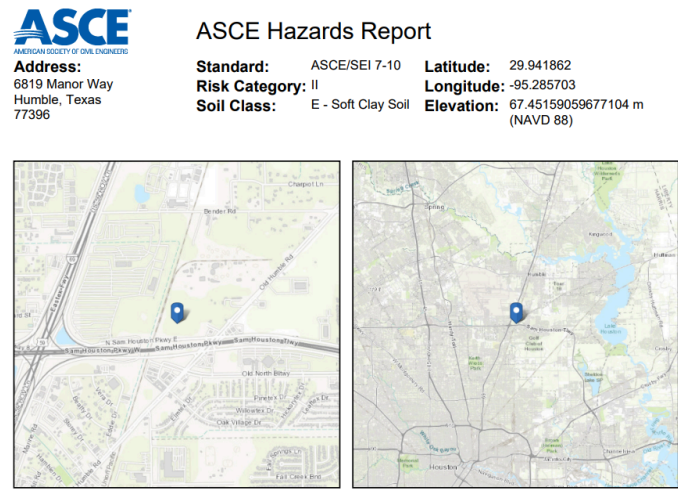


Figure 3.3.4. The location of the building in ASCE Hazards Report

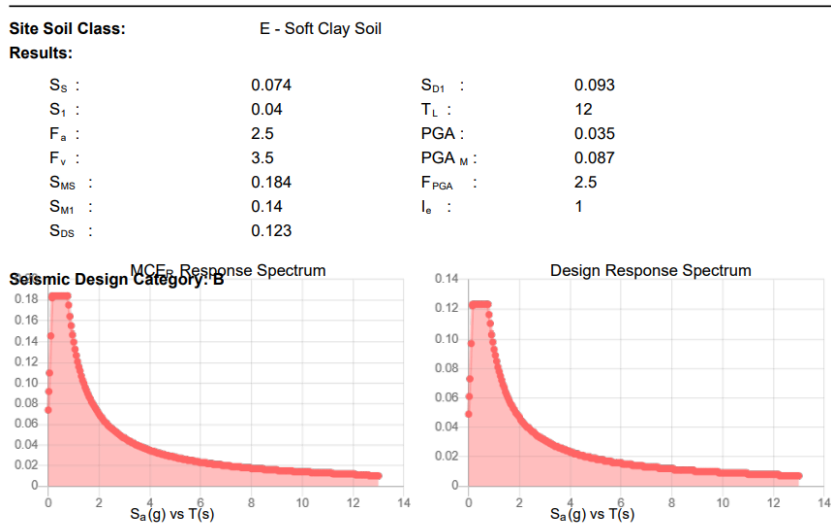


Figure 3.3.5. The parameters for seismic load values from hazard tools.

Table 3.3.22. Seismic values

S_s	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}	T_L
0.074	0.04	2.5	3.5	0.185	0.14	0.123	0.093	12

$$T_0 = 0.2S_{D1}/S_{DS} = 0.15$$

$$T_s = S_{D1}/S_{DS} = 0.75$$

$$T_a = C_t h_n^x = 0.0466 * 28.4^{0.9} = 0.947s$$

$$T = T_a C_u = 1.61$$

T* from Modal Analysis is 3.21.

Since T less than T*, T_a will be used for further calculations

$$S_a = \frac{S_{D1}}{T} = 0.098$$

According to ASCE 7-10 Table 11.6-2 the Risk Category is B.

$$C_s = S_{DS}/(R/I_e) = 0.123/(3/1) = 0.041$$

$$C_{s,max} = S_{D1}/T(R/I_e) = 0.093/0.947(3/1) = 0.0327$$

$$C_{s,min} = MAX[0.044S_{DS}I, 0.01] = 0.01$$

$$C_s = 0.0327$$

Since the building is located in very low earthquake risk zone, the ordinary concrete frame is used with $R_a = 3$, $\Omega = 3$, $C_d = 2.5$.

Table 3.3.23. Values for W in kN

Floor	Load	Area	W, kN
1	7.633	2400	18319
2	6.967	2400	16721

3	6.333	2400	15199
4	6.663	1500	9995
5-8	6.728	1500	10092
Roof	6.728	112.5	756.9

From linear interpolation the values of an exponent related to the structure is $k=1.2.2$

Table 3.3.24. Summary of the final forces and seismic load on each floor

Floors	hi (m)	Wx (kN)	Wx h _x k	C _v x	F _x (kN)
Roof	34.9	925	70503	14.000	67
8	32.4	15151	1055145	205.000	1002
7	29.1	14410	880277	171.000	836
6	25.8	14410	760057	147.000	722
5	2.5	14410	643180	125.000	611
4	19.2	14410	530026	103.000	503
3	15.9	14254	416531	81.000	396
2	10.6	32493	578981	112.000	550
1	5.3	29368	224644	44.000	213

3.3.2.1. Case 1

$$F_i = \frac{\text{Force } Y * C_f}{\Sigma C_f}$$

Table 3.3.25. Seismic load on each frame in Y direction

Floors	1	2	3	4	5	6	7	Total
Roof	0.0000	16.7500	16.7500	0.0000	16.7500	16.7500	0.0000	67.0000
8	143.1429	143.1429	143.1429	143.1429	143.1429	143.1429	143.1429	1002.0000
7	119.4286	119.4286	119.4286	119.4286	119.4286	119.4286	119.4286	836.0000
6	103.1429	103.1429	103.1429	103.1429	103.1429	103.1429	103.1429	722.0000
5	87.2857	87.2857	87.2857	87.2857	87.2857	87.2857	87.2857	611.0000
4	71.8571	71.8571	71.8571	71.8571	71.8571	71.8571	71.8571	503.0000
3	61.9751	0.0000	0.0000	61.9751	0.0000	0.0000	61.9751	185.9252
2	183.3333	0.0000	0.0000	183.3333	0.0000	0.0000	183.3333	550.0000
1	71.0000	0.0000	0.0000	71.0000	0.0000	0.0000	71.0000	213.0000

$$F_i = \frac{\text{Force } X * C_f}{\Sigma C_f}$$

Table 3.3.26. Seismic load on each frame in X direction

Floor	A	B	C	D	E	F	G	H	I	Total
Roof	0.000	0.000	33.500	33.500	0.000	0.000	0.000	0.000	0.000	67.000
8	167.000	167.000	167.000	167.000	167.000	167.000	0.000	0.000	0.000	1002.000
7	139.333	139.333	139.333	139.333	139.333	139.333	0.000	0.000	0.000	836.000

6	120.33 3	120.33 3	120.33 3	120.33 3	120.33 3	120.33 3	0.000	0.000	0.000	722.00 0
5	101.83 3	101.83 3	101.83 3	101.83 3	101.83 3	101.83 3	0.000	0.000	0.000	611.00 0
4	83.833	83.833	83.833	83.833	83.833	83.833	0.000	0.000	0.000	503.00 0
3	44.000	44.000	44.000	44.000	44.000	44.000	44.000	44.000	44.000	396.00 0
2	61.111	61.111	61.111	61.111	61.111	61.111	61.111	61.111	61.111	550.00 0
1	23.667	23.667	23.667	23.667	23.667	23.667	23.667	23.667	23.667	213.00 0

3.3.2.2. Case 2

In this case the total load was calculated with consideration of torsion. The following expressions were used to identify the torsional moment and eccentricity:

$$M_T = 0.75(p^w + p^l) * B * e$$

$$e = 0.15 * B$$

$$F_{Torsion} = \frac{M_T * y_i^l * C_f}{\sum (y_i^2 * C_f + x_i^2 * C_f)}$$

$$F_{Total} = F_{direct} + F_{Torsion}$$

Table 3.3.27. Mt values for Y direction

Floors	Fx (kN)	0.75*F	Mt
Roof	67	50.25	866.8125
8	1002	751.5	12963.375
7	836	627	10815.75

6	722	541.5	9340.875
5	611	458.25	7904.8125
4	503	377.25	6507.5625
3	396	297	1782
2	550	412.5	2475
1	213	159.75	958.5

Table 3.3.28. Mt values for X direction

Floors	F _x (kN)	0.75*F	Mt
Roof	67	50.25	452.25
8	1002	751.5	6763.5
7	836	627	5643
6	722	541.5	4873.5
5	611	458.25	4124.25
4	503	377.25	3395.25
3	396	297	2673
2	550	412.5	3712.5
1	213	159.75	1437.75

Table 3.3.29. Seismic load with torsion on each frame in Y direction

Floors	1	2	3	4	5	6	7	Total
Roof	-4.644	9.660	11.401	0.000	13.724	15.465	4.644	50.250
8	37.900	63.946	89.993	107.357	124.722	150.768	176.815	751.500
7	31.621	53.352	75.084	89.571	104.059	125.791	147.522	627.000

6	27.309	46.077	64.845	77.357	89.869	108.637	127.405	541.500
5	23.110	38.993	54.876	65.464	76.053	91.935	107.818	458.250
4	19.025	32.101	45.176	53.893	62.610	75.685	88.760	377.250
3	24.799	0.000	0.000	46.481	0.000	0.000	68.163	139.444
2	107.386	0.000	0.000	137.500	0.000	0.000	167.614	412.500
1	41.588	0.000	0.000	53.250	0.000	0.000	64.912	159.750

Table 3.3.30. Seismic load with torsion on each frame in X direction

Floors	A	B	C	D	E	F	G	H	I	Total
Roof	0.000	0.000	25.125	25.125	0.000	0.000	0.000	0.000	0.000	50.250
8	125.250	125.250	125.250	125.250	125.250	125.250	0.000	0.000	0.000	751.500
7	104.500	104.500	104.500	104.500	104.500	104.500	0.000	0.000	0.000	627.000
6	68.013	73.573	79.132	84.691	90.250	95.809	11.118	16.677	22.237	541.500
5	57.557	62.262	66.966	71.671	76.375	81.079	9.409	14.113	18.818	458.250
4	47.383	51.256	55.129	59.002	62.875	66.748	7.746	11.619	15.492	377.250
3	20.804	23.853	26.902	29.951	33.000	36.049	39.098	42.147	45.196	297.000
2	28.894	33.129	37.364	41.599	45.833	50.068	54.303	58.538	62.772	412.500
1	11.190	12.830	14.470	16.110	17.750	19.390	21.030	22.670	24.310	159.750

3.3.3. Assigning lateral loads in SAP2000

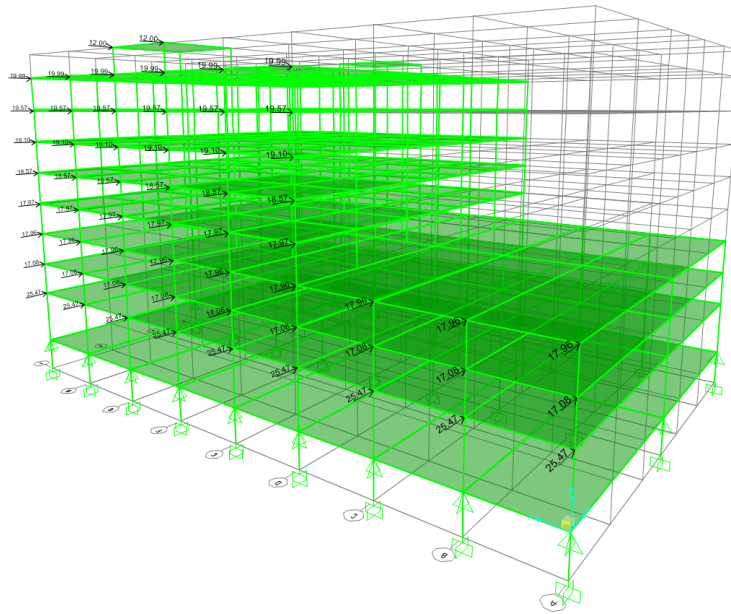


Figure 3.3.6. Wind Force in X direction for 3D Frame

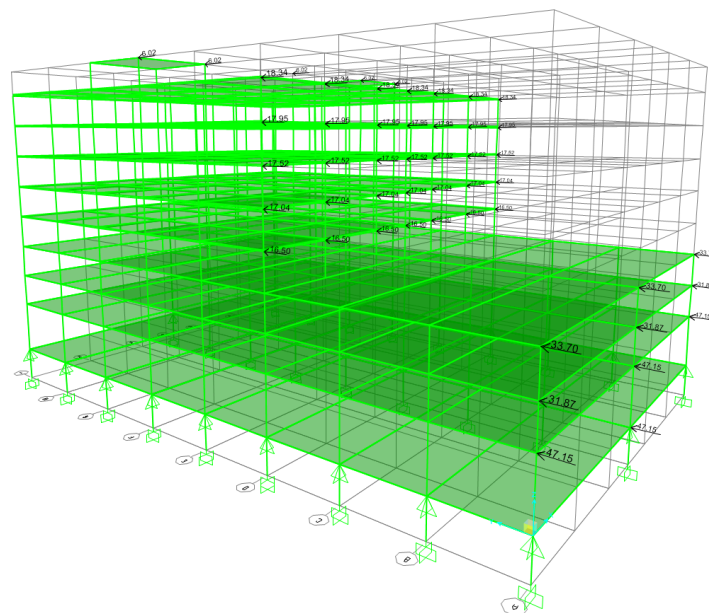


Figure 3.3.7. Wind Force in Y direction for 3D Frame

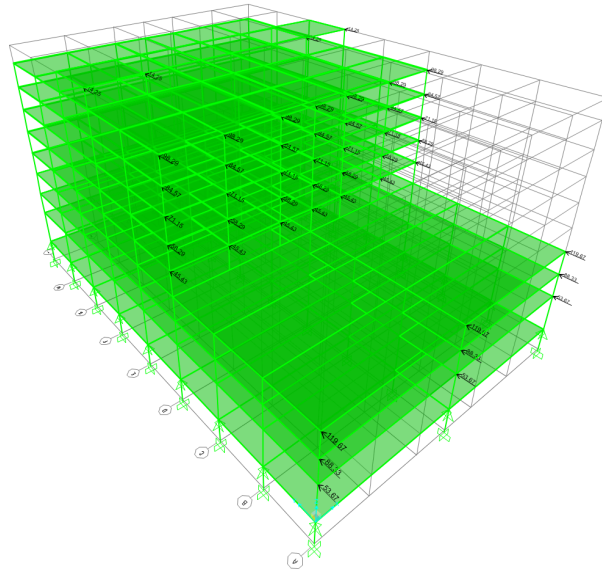


Figure 3.3.8. Seismic Force in Y direction for 3D Frame

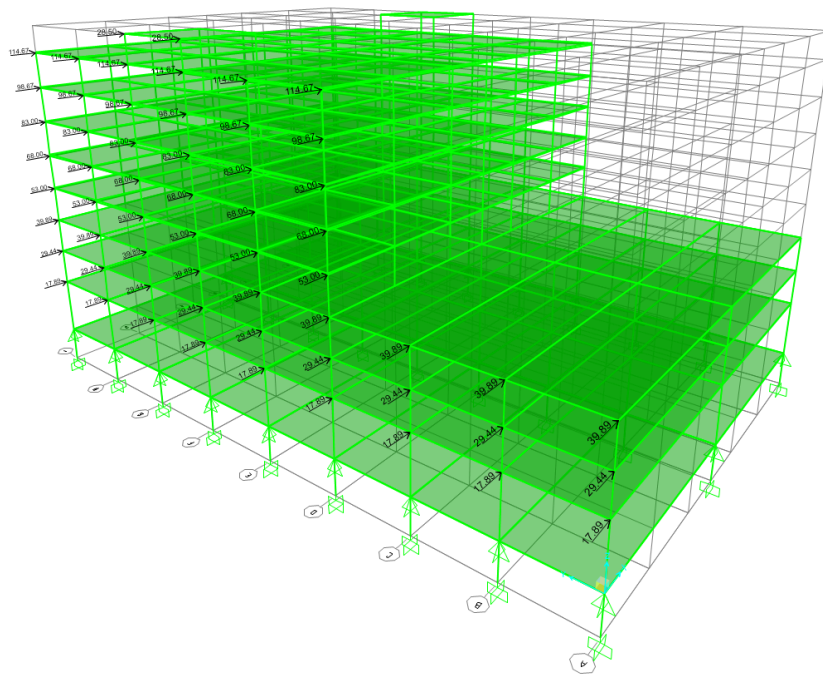


Figure 3.3.9. Seismic Force in X direction for 3D Frame

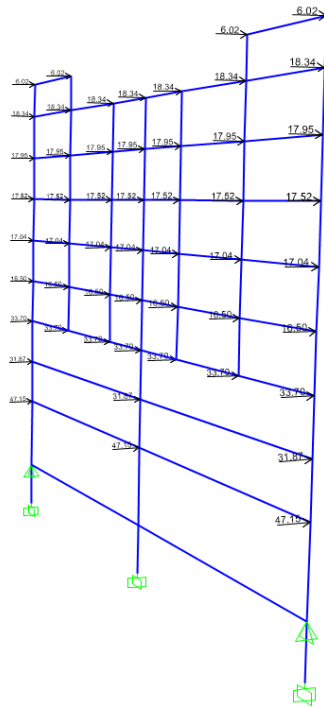


Figure 3.3.10. Wind Force in Y direction for 2D Frame

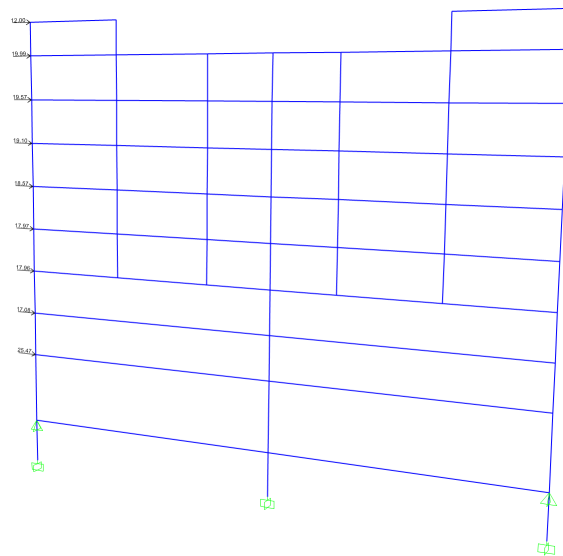


Figure 3.3.11. Wind Force in X direction for 2D Frame

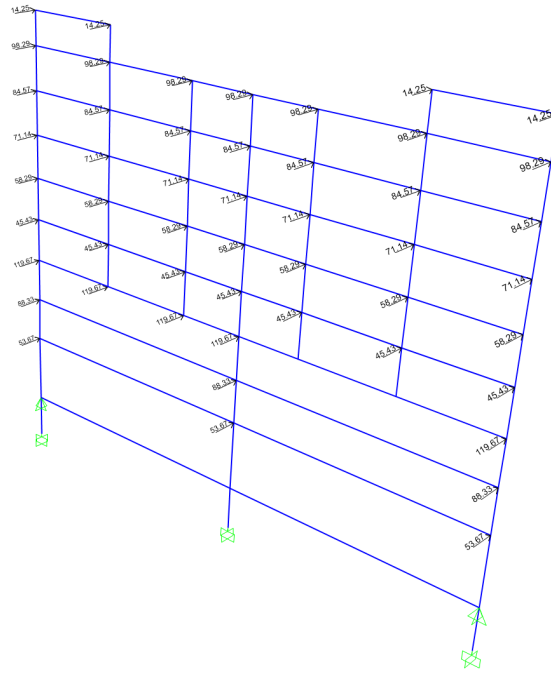


Figure 3.3.12. Seismic Force in Y direction for 2D Frame

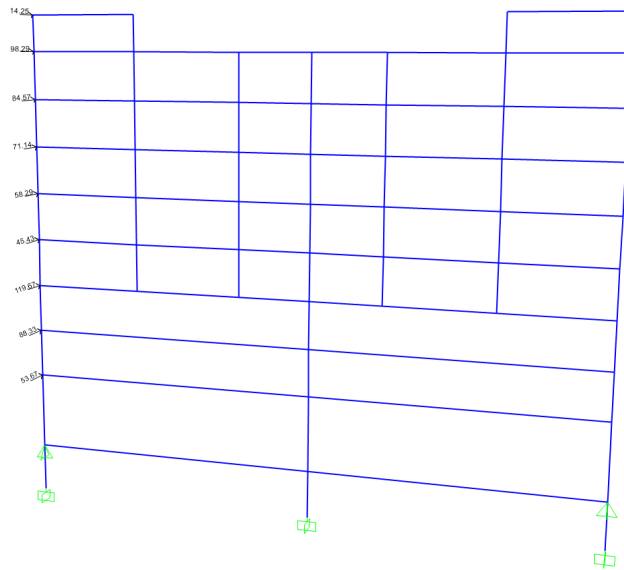


Figure 3.3.13. Seismic Force in X direction for 2D Frame

3.3.4. Load combinations in SAP2000

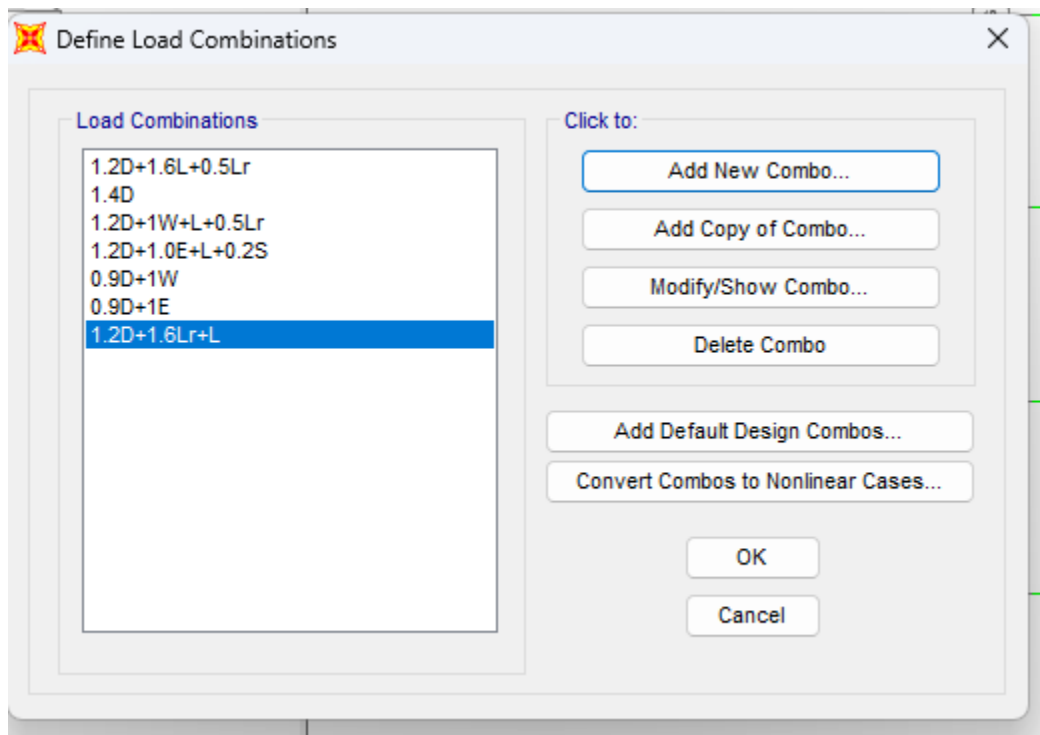


Figure 3.3.14. Possible Load Combinations

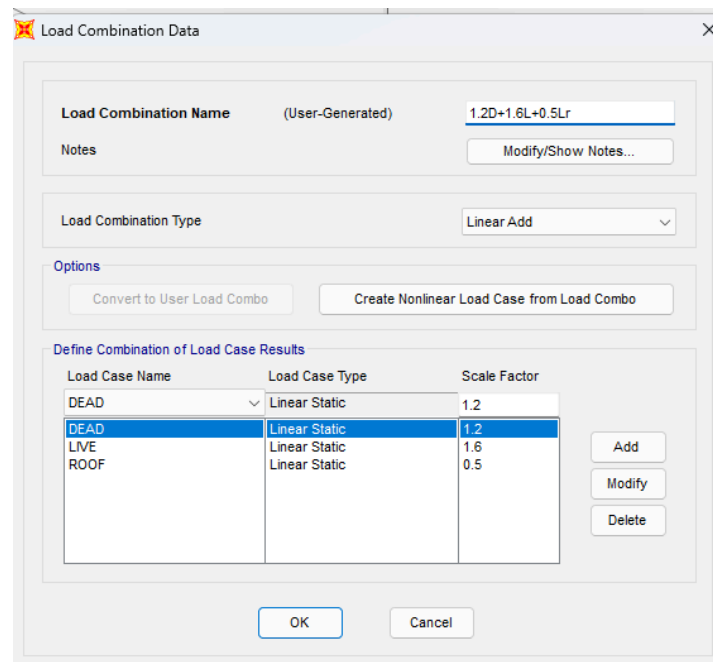


Figure 3.3.15. Load combination assignment

Element Forces - Frames					
File View Format-Filter-Sort Select Options					
Units: As Noted					
Filter:					
	Frame Text	Station m	OutputCase Text	CaseType Text	P KN
►	851	0	1.2D+1.6L+0.5Lr	Combination	-19725.563
	851	0	1.4D	Combination	-19893.301
	851	0	1.2D+1W+L+0.5Lr	Combination	-18741.306
	851	0	1.2D+1.0E+L+0.2S	Combination	-18691.828
	851	0	0.9D+1W	Combination	-12788.551
	851	0	0.9D+1E	Combination	-12788.551
	851	0	1.2D+1.6Lr+L	Combination	-18850.157

Figure 3.3.16. Reaction forces at the bottom under different load combinations

Table 3.3.31 Comparison of Axial Loads

Floor	Hand, kN	2D, kN	3D, kN
8	1416	1247.916	2208.715
7	3478	2842.297	4457.561
6	5540	4421.057	6767.004
5	7602	6014.712	9198.056
4	9913	7739.184	9929.716
3	11983	10139.43	13210.474
2	14467	13570.406	15550.209
1	16951	16967.415	17911.719
Basement	19305	20158.588	20149.121

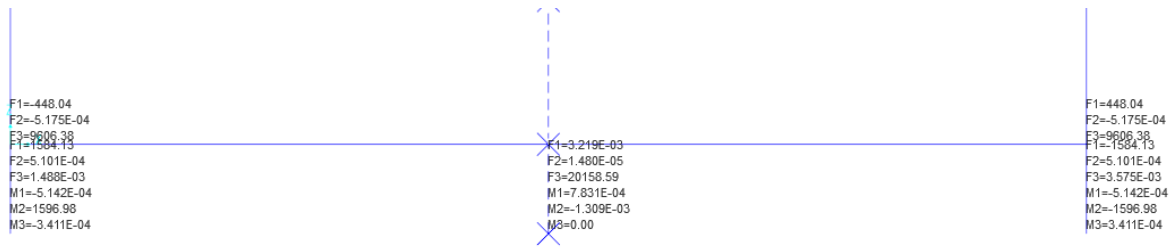


Figure 3.3.17. Basement reactions for 2D Frame

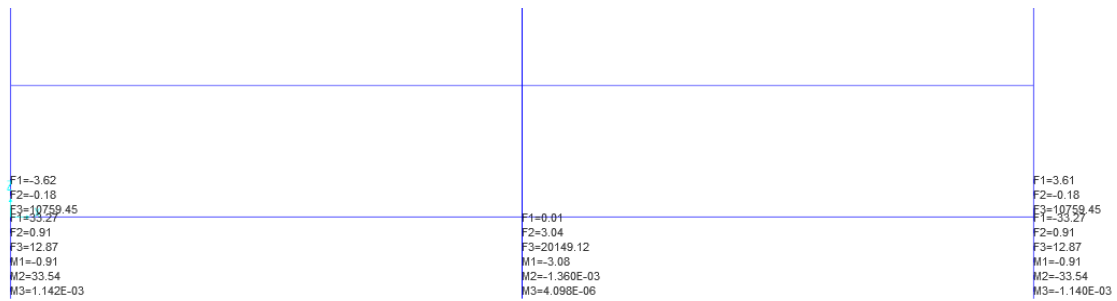


Figure 3.3.18. Basement reactions for 3D Frame

3.4. Lateral Drift Analysis and Check under Wind and Seismic Loads

3.4.1. Wind and Seismic Drift Calculations.

Lateral displacement analysis was conducted to determine the shear and flexural drift in both the Y- and X-directions for Frames 4 and C under the wind and seismic loads, respectively.

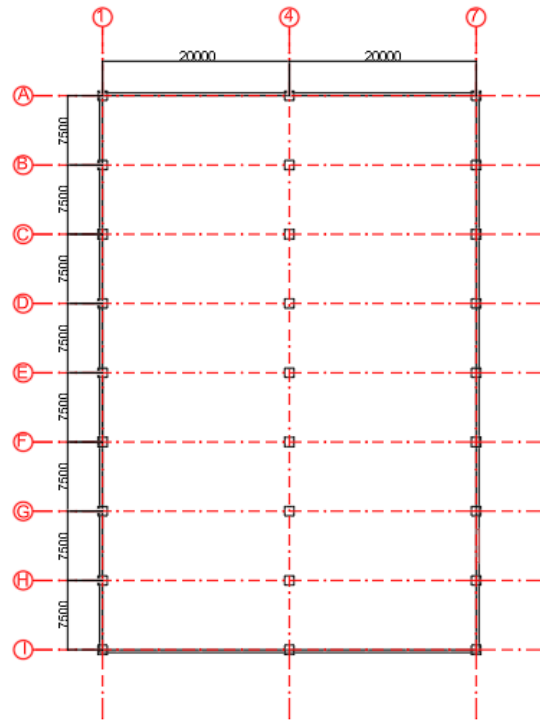


Figure 3.4.1. Layout of the 1-3 floors

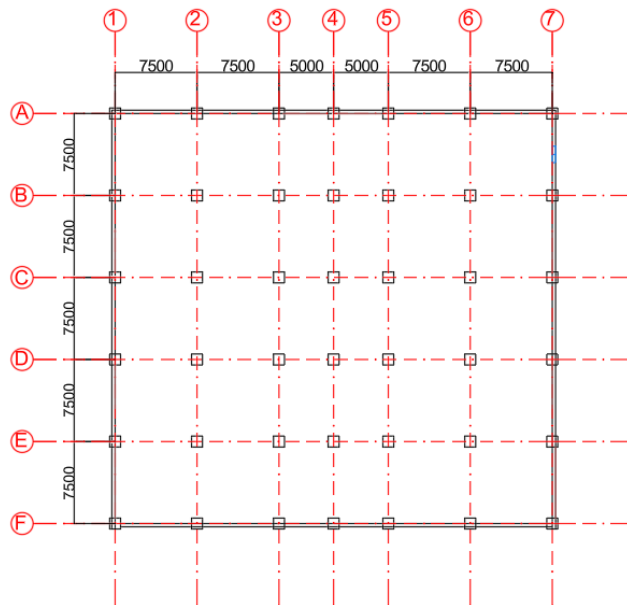


Figure 3.4.2. Layout of the 4-8 floors

The shear drift was computed using the previously calculated wind (Case I) and seismic loads to calculate the shear force and base shear that are determined by:

$$V_i = F_i + V_{i+1}$$

$$V_{i-col} = \frac{V_i}{n}$$

The deformation of the frame was found by using:

$$\delta_t^i = \delta_b + \delta_c$$

The deformation of the beams was calculated:

$$\delta_b^i = \frac{V_{i-avg} * h_i^2 * L}{12 * E * I_{b,cr}}$$

The deformation of the column was calculated:

$$\delta_c^i = \frac{V_{i-avg} * h_i^3}{12 * E * I_{c,cr}} \quad \text{for the floor 1-12}$$

The inter-story and absolute drifts were calculated using the following formulas:

$$\delta_i = \delta_t^{i-1} + \frac{\delta_t^i}{2}$$

$$\Delta_i = \delta_i + \Delta_{i-1}$$

$$drift\% = \frac{\delta_i}{H_{avg}} * 100$$

Table 3.4.1. Lateral Shear Drifts due to Wind Load for Frame 4

Floor	Column size	Beam size	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vi-avg (kN)
8	800x800	600x305	3300	3300	11.383	11.383	1.897	1.897
7	800x800	600x305	3300	3300	22.421	33.804	5.634	3.766
6	800x800	600x305	3300	3300	21.705	55.509	9.252	7.443
5	800x800	600x305	3300	3300	20.923	76.432	12.739	10.995
4	800x800	600x305	3300	3300	20.059	96.491	16.082	14.410
3	950x950	900x450	5300	4300	25.814	122.305	61.153	38.617
2	950x950	900x450	5300	4800	60.703	183.008	91.504	76.328
1	950x950	900x450	5300	5050	52.052	235.06	117.530	104.517

Table 3.4.2. Lateral Shear Drifts due to Wind Load for Frame 4 (Results)

Floor	Ic (mm ⁴)	Ib (mm ⁴)	Ic-avg (mm ⁴)	db (mm)	dc (mm)	dt (mm)	di (mm)	Di (mm)
8	1041286 4583	4614257 813	1041286 4583	0.109	0.0212	0.130	0.130	12.897
7	1041286 4583	4614257 813	1041286 4583	0.216	0.0421	0.258	0.384	12.767
6	1041286 4583	4614257 813	1041286 4583	0.426	0.0832	0.510	0.631	12.384
5	1041286 4583	4614257 813	1041286 4583	0.630	0.1228	0.753	0.870	11.752
4	1041286 4583	4614257 813	1041286 4583	0.826	0.1610	0.987	1.259	10.883
3	4751286 4583	1458333 3333	2896286 4583	1.189	0.3432	1.532	2.517	9.623
2	4751286 4583	1458333 3333	4751286 4583	2.928	0.5751	3.503	4.429	7.106
1	4751286 4583	1458333 3333	4751286 4583	4.437	0.9171	5.355	2.677	2.677

Table 3.4.3. Lateral Shear Drifts due to Wind Load for Frame C

Floor	Column size	Beam size	hi (mm)	hi-avg (mm)	Fi	Vi	Vi col	Vi avg
Roof	800x800	600x300	2500	2500	6.895	6.895	3.447	3.447
8	800x800	600x300	3300	2900	12.640	19.535	3.256	3.352
7	800x800	600x300	3300	3300	24.898	44.433	7.406	5.331
6	800x800	600x300	3300	3300	24.103	68.536	11.423	9.414
5	800x800	600x300	3300	3300	23.234	91.770	15.295	13.359
4	800x800	900x450	3300	3300	22.275	114.045	19.007	17.151
3	950x950	900x450	5300	4300	29.107	143.152	71.576	45.292
2	950x950	900x450	5300	5300	33.077	176.229	88.115	79.845
1	950x950	900x450	5300	5300	28.363	204.592	102.296	95.205

Table 3.4.4. Lateral Shear Drifts due to Wind Load for Frame C (Results)

Floor	Ic (mm ⁴)	Ib (mm ⁴)	Ic avg	db	dc	dt	di	Di
Roof	1.04E+10	4.61E+09	1.04E+10	0.093	0.016	0.109	0.127	12.557
8	1.04E+10	4.61E+09	1.04E+10	0.122	0.024	0.146	0.226	12.430
7	1.04E+10	4.61E+09	1.04E+10	0.251	0.055	0.306	0.424	12.204
6	1.04E+10	4.61E+09	1.04E+10	0.444	0.097	0.541	0.655	11.780
5	1.04E+10	4.61E+09	1.04E+10	0.630	0.138	0.768	0.877	11.125
4	1.04E+10	4.61E+09	1.04E+10	0.809	0.177	0.986	1.180	10.248
3	4.75E+10	1.46E+10	2.90E+10	1.147	0.227	1.374	2.599	9.068
2	4.75E+10	1.46E+10	4.75E+10	3.073	0.750	3.823	4.190	6.469
1	4.75E+10	1.46E+10	4.75E+10	3.664	0.894	4.558	2.279	2.279

Table 3.4.5. Lateral Shear Drifts due to Seismic Load for Frame 4

Floor	Column size	Beam size	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vi-avg (kN)
Roof	800x800	600x300	2500	2500	0.000	0.000	0	0
8	800x800	600x300	3300	2900	107.357	107.357	17.89285 714	8.946428 571
7	800x800	600x300	3300	3300	89.571	196.929	32.82142 857	25.35714 286
6	800x800	600x300	3300	3300	77.357	274.286	45.71428 571	39.26785 714
5	800x800	600x300	3300	3300	65.464	339.750	56.625	51.16964 286
4	800x800	900x450	3300	3300	53.893	393.643	65.60714 286	61.11607 143
3	950x950	900x450	5300	4300	46.481	440.124	220.0620 759	142.8346 094
2	950x950	900x450	5300	5300	137.500	577.624	288.8120 759	254.4370 759
1	950x950	900x450	5300	5300	53.250	630.874	315.4370 759	302.1245 759

Table 3.4.6. Lateral Shear Drifts due to Seismic Load for Frame 4 (Results)

Floor	Ic (mm ⁴)	Ib (mm ⁴)	Ic-avg (mm ⁴)	db (mm)	dc (mm)	dt (mm)	di (mm)	Di (mm)
Roof	1041286 4583	4614257 813	1041286 4583	0	0	0	0.000	0.000
8	1041286 4583	4614257 813	1041286 4583	0.395881 0369	0.067831 75744	0.463712 7943	0.232	41.319
7	1041286 4583	4614257 813	1041286 4583	1.452938 399	0.283290 1757	1.736228 574	1.100	41.087
6	1041286 4583	4614257 813	1041286 4583	2.250008 126	0.438700 7721	2.688708 898	2.212	39.987
5	1041286 4583	4614257 813	1041286 4583	2.931968 297	0.571667 6045	3.503635 901	3.096	37.775
4	1041286 4583	4614257 813	1041286 4583	3.501888 5	0.682789 1734	4.184677 673	3.844	34.679
3	4751286 4583	1458333 3333	2896286 4583	4.396776 812	1.269278 436	5.666055 248	4.925	30.834
2	4751286 4583	1458333 3333	4751286 4583	11.89860 899	2.580812 104	14.47942 11	10.073	25.909
1	4751286 4583	1458333 3333	4751286 4583	14.12868 853	3.064517 07	17.19320 56	15.836	15.836

Table 3.4.7. Lateral Shear Drifts due to Seismic Load for Frame C

Floor	Column size	Beam size	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vi-avg (kN)
Roof	800x800	600x300	2500	2500	33.500	33.500	16.75	16.75
8	800x800	600x300	3300	2900	167.000	200.500	33.41666 667	25.08333 333
7	800x800	600x300	3300	3300	139.333	339.833	56.63888 889	45.02777 778
6	800x800	600x300	3300	3300	120.333	460.167	76.69444	66.66666

							444	667
5	800x800	600x300	3300	3300	101.833	562.000	93.66666 667	85.18055 556
4	800x800	900x450	3300	3300	83.833	645.833	107.6388 889	100.6527 778
3	950x950	900x450	5300	4300	44.000	689.833	344.9166 667	226.2777 778
2	950x950	900x450	5300	5300	61.111	750.944	375.4722 222	360.1944 444
1	950x950	900x450	5300	5300	23.667	774.611	387.3055 556	381.3888 889

Table 3.4.8. Lateral Shear Drifts due to Seismic Load for Frame C (Results)

Floor	Ic (mm4)	Ib (mm4)	Ib avg (mm4)	Ic-avg (mm4)	db (mm)	dc (mm)	dt (mm)	di (mm)	Di (mm)
Roof	104128 64583	461425 7813	461425 7813	104128 64583	0.510	0.075	0.585	0.585	61.626
8	104128 64583	461425 7813	461425 7813	104128 64583	1.028	0.176	1.204	0.89448 78591	61.0405 3011
7	104128 64583	461425 7813	461425 7813	104128 64583	2.389	0.466	2.854	2.02904 0046	60.1460 4225
6	104128 64583	461425 7813	461425 7813	104128 64583	3.537	0.690	4.226	3.54026 2893	58.1170 0221
5	104128 64583	461425 7813	461425 7813	104128 64583	4.519	0.881	5.400	4.81294 3821	54.5767 3931
4	104128 64583	461425 7813	461425 7813	104128 64583	5.339	1.041	6.381	5.89016 6316	49.7637 9549
3	475128 64583	145833 33333	959879 5573	289628 64583	9.797	1.862	11.659	9.01979 4018	43.8736 2917
2	475128 64583	145833 33333	145833 33333	475128 64583	15.595	3.383	18.977	15.3181 7363	34.8538 3516
1	475128 64583	145833 33333	145833 33333	475128 64583	16.512	3.582	20.094	19.5356 6153	19.5356 6153

The equations and tables for the following calculations of lateral flexural drifts are shown below:

$$M_i = h_i * V_i \quad \text{for floor 8}$$

$$M_i = M_{i+1} * h_i * V_i \quad \text{for floor 1-7}$$

$$\phi_i = \frac{2M_i}{E(mL)^2 * Ac}$$

$$\Delta\theta = \phi_i * h_i$$

$$\theta_i = \sum_i^n \Delta\theta_i = \Delta\theta_i + \theta_{i-1}$$

$$\delta_i = \theta_i * h_i$$

$$\Delta_i = \sum_i^n \delta_i$$

$$drift\% = \frac{\delta_i}{H_i} * 100$$

Table 3.4.9. Lateral Flexural Drifts due to Wind Load for Frame 4.

Floor	Column size	Beam size	hi (mm)	Hi (mm)	Fi (kN)	Vi (kN)
8	800x800	600x300	3300	32400	11.383	11.383
7	800x800	600x300	3300	29100	22.421	33.804
6	800x800	600x300	3300	25800	21.705	55.509
5	800x800	600x300	3300	22500	20.923	76.432
4	800x800	600x300	3300	19200	20.059	96.491
3	950x950	900x450	5300	15900	25.814	122.305
2	950x950	900x450	5300	10600	60.703	183.008
1	950x950	900x450	5300	5300	52.052	235.06

Table 3.4.10. Lateral Flexural Drifts due to Wind Load for Frame 4 (Results)

Floor	Mi (k-m)	Aci (mm ²)	fi (1/mm)	Dqi (rad)	qi (rad)	di (mm)	Di (mm)
8	37.564	422500	1.92E-12	6.33E-09	1.33E-06	4.39E-03	3.30E-02
7	149.117	422500	7.62E-12	2.51E-08	1.33E-06	4.37E-03	2.86E-02
6	332.297	422500	1.70E-11	5.60E-08	1.30E-06	4.29E-03	2.42E-02
5	584.522	422500	2.99E-11	9.85E-08	1.24E-06	4.11E-03	1.99E-02
4	902.943	422500	4.61E-11	1.52E-07	1.15E-06	3.78E-03	1.58E-02
3	1551.159	902500	3.71E-11	1.97E-07	9.93E-07	5.27E-03	1.20E-02
2	2521.102	902500	6.03E-11	3.20E-07	7.97E-07	4.22E-03	6.75E-03
1	3766.920	902500	9.01E-11	4.77E-07	4.77E-07	2.53E-03	2.53E-03

Table 3.4.11. Lateral Flexural Drifts due to Seismic Load for Frame 4

Floor	Column size	Beam size	hi (mm)	Hi (mm)	Fi (kN)	Vi (kN)
Roof	800x800	600x300	2500	34900	0.000	0.000
8	800x800	600x300	3300	32400	107.357	107.357
7	800x800	600x300	3300	29100	89.571	196.929
6	800x800	600x300	3300	25800	77.357	274.286
5	800x800	600x300	3300	22500	65.464	339.750
4	800x800	600x300	3300	19200	53.893	393.643
3	950x950	900x450	5300	15900	46.481	440.124
2	950x950	900x450	5300	10600	137.500	577.624
1	950x950	900x450	5300	5300	53.250	630.874

Table 3.4.12. Lateral Flexural Drifts due to Seismic Load for Frame 4 (Results)

Floor	Mi (k-m)	Aci (mm ²)	fi (1/mm)	Dqi (rad)	qi (rad)	di (mm)	Di (mm)
Roof	0	422500	0.0000000 00000000 0	0	0.0000029 75608285	0	0.0525461 8221
8	354.27857 14	422500	0.0000000 00046326 2	0.0000001 52876388 6	0.0000029 75608285	0.0098195 07341	0.0525461 8221
7	649.86428 57	422500	0.0000000 00084977 6	0.0000002 80425950 4	0.0000028 22731896	0.0093150 15258	0.0427266 7487
6	905.14285 71	422500	0.0000000 00118358 3	0.0000003 90582390 1	0.0000025 42305946	0.0083896 09622	0.0334116 5962
5	1121.175	422500	0.0000000 00146607 1	0.0000004 83803421 5	0.0000021 51723556	0.0071006 87735	0.0250220 4999
4	1299.0214 29	422500	0.0000000 00169862 7	0.0000005 60546758 3	0.0000016 67920134	0.0055041 36444	0.0179213 6226
3	2332.6580 05	902500	0.0000000 00055779 2	0.0000002 95629703 7	0.0000011 07373376	0.0058690 78894	0.0124172 2581
2	3061.4080 05	902500	0.0000000 00073205 3	0.0000003 87987925 9	0.0000008 11743672 5	0.0043022 41464	0.0065481 46921
1	3343.6330 05	902500	0.0000000 00079953 9	0.0000004 23755746 6	0.0000004 23755746 6	0.0022459 05457	0.0022459 05457

Table 3.4.13. Lateral Flexural Drifts due to Wind Load for Frame C

Floor	Column size	Beam size	hi (mm)	Hi	Fi	Vi
Roof	800x800	600x300	2500	34900	6.895	6.895
8	800x800	600x300	3300	32400	12.640	19.535

7	800x800	600x300	3300	29100	24.898	44.433
6	800x800	600x300	3300	25800	24.103	68.536
5	800x800	600x300	3300	22500	23.234	91.770
4	800x800	600x300	3300	19200	22.275	114.045
3	800x800	900x450	5300	15900	29.107	143.152
2	800x800	900x450	5300	10600	33.077	176.229
1	800x800	900x450	5300	5300	28.363	204.592

Table 3.4.14. Lateral Flexural Drifts due to Wind Load for Frame C (Results)

Floor	Mi	Aci	fi (1/mm)	Dqi (rad)	qi (rad)	di (mm)	Di (mm)
Roof	17.236	422500.00 0	0.0000000 00052167	0.00000001 30416261 5	0.0000171 7559208	0.0429389 802	0.2964852 749
8	64.464	422500.00 0	0.0000000 002469	0.00000008 14869162 5	0.0000170 4517582	0.0562490 8021	0.2535462 947
7	146.629	422500.00 0	0.0000000 005617	0.0000018 53478337	0.0000162 3030666	0.0535600 1197	0.1972972 145
6	226.168	422500.00 0	0.0000000 008663	0.0000028 5889707	0.0000143 7682832	0.0474435 3346	0.1437372 025
5	302.840	422500.00 0	0.0000000 011600	0.0000038 28080238	0.0000115 1793125	0.0380091 7312	0.0962936 6904
4	376.347	422500.00 0	0.0000000 014416	0.0000047 57259676	0.0000076 89851012	0.0253765 0834	0.0582844 9592
3	758.704	902500	0.0000000 001512	0.0000008 01197298 9	0.0000029 32591336	0.0155427 3408	0.0329079 8758
2	934.014	902500	0.0000000 001861	0.0000009 86325150 7	0.0000021 31394037	0.0112963 884	0.0173652 535
1	1084.338	902500	0.0000000 002161	0.0000011 45068887	0.0000011 45068887	0.0060688 65099	0.0060688 65099

Table 3.4.15. Lateral Flexural Drifts due to Seismic Load for Frame C

Floor	Column size	Beam size	hi (mm)	Hi (mm)	Fi (kN)	Vi (kN)
Roof	650x650	750x375	2500	34900	33.500	33.500
8	650x650	750x375	3300	32400	167.000	200.500
7	650x650	750x375	3300	29100	139.333	339.833
6	650x650	750x375	3300	25800	120.333	460.167
5	650x650	750x375	3300	22500	101.833	562.000
4	650x650	750x375	3300	19200	83.833	645.833
3	950x950	1000x500	5300	15900	44.000	689.833
2	950x950	1000x500	5300	10600	61.111	750.944
1	950x950	1000x500	5300	5300	23.667	774.611

Table 3.4.16. Lateral Flexural Drifts due to Seismic Load for Frame C (Results)

Floor	Mi (k-m)	Aci (mm ²)	fi (1/mm)	Dqi (rad)	qi (rad)	di (mm)	Di (mm)
Roof	83.75	422500	0.0000000 00063369	0.0000001 58421377 9	0.0000058 17059666	0.0145426 4916	0.1287635 401
8	661.65	422500	0.0000000 00070401	0.0000002 32323827 6	0.0000056 58638288	0.0186735 0635	0.1142208 909
7	1121.45	422500	0.0000000 00119325	0.0000003 93772472 5	0.0000054 2631446	0.0179068 3772	0.0955473 8454
6	1518.55	422500	0.0000000 00161577	0.0000005 33205393 1	0.0000050 32541988	0.0166073 8856	0.0776405 4682
5	1854.6	422500	0.0000000 00197334	0.0000006 51201950 5	0.0000044 99336594	0.0148478 1076	0.0610331 5826
4	2131.25	422500	0.0000000 00226770	0.0000007 48341506	0.0000038 48134644	0.0126988 4433	0.0461853 475
3	3656.1166	902500	0.0000000	0.0000009	0.0000030	0.0164289	0.0334865

	67		00182117	65221340 5	99793138	0363	0318
2	3980.0055 56	902500	0.0000000 00198251	0.0000010 50728587	0.0000021 34571797	0.0113132 3053	0.0170575 9954
1	4105.4388 89	902500	0.0000000 00204499	0.0000010 83843211	0.0000010 83843211	0.0057443 69018	0.0057443 69018

The ACI Code requires that the drifts induced by seismic loads be amplified to achieve a more accurate response. Amplified interstory drifts are determined using the formula below:

$$\delta_x = \frac{C_d * \delta}{I_e}$$

Subsequently, the drifts are compared to the allowable interstory drift (2% of the story height). The allowable interstory drift for a typical floor is calculated below.

$$\delta_{1-3, allow} = 0.02 * 5300 = 106 \text{ mm}$$

$$\delta_{4-8, allow} = 0.02 * 3300 = 66 \text{ mm}$$

3.4.2. Comparison of drifts under wind loads

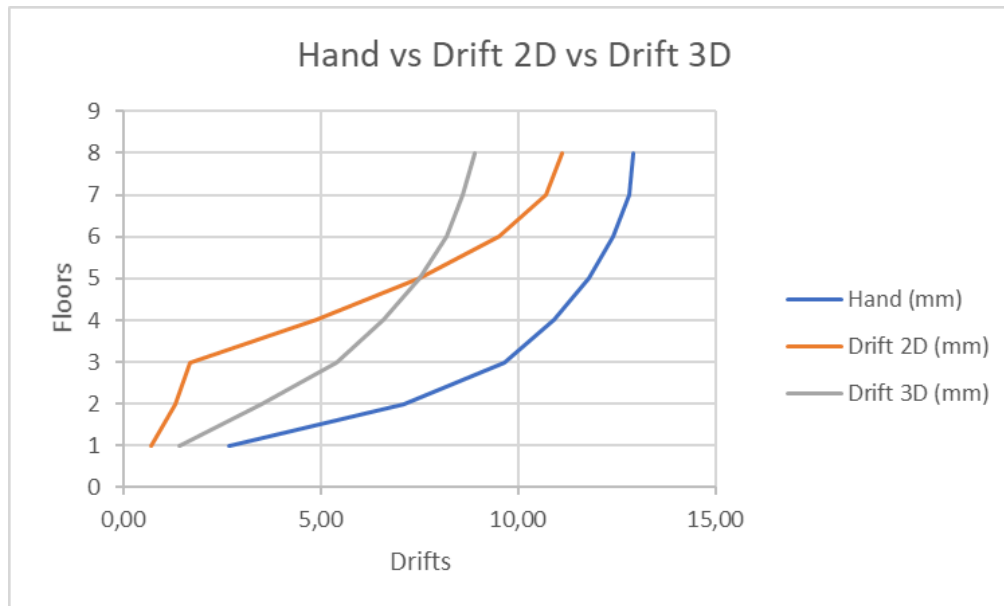


Figure 3.4.3. Drift comparison under Wind loads for Frame 4.

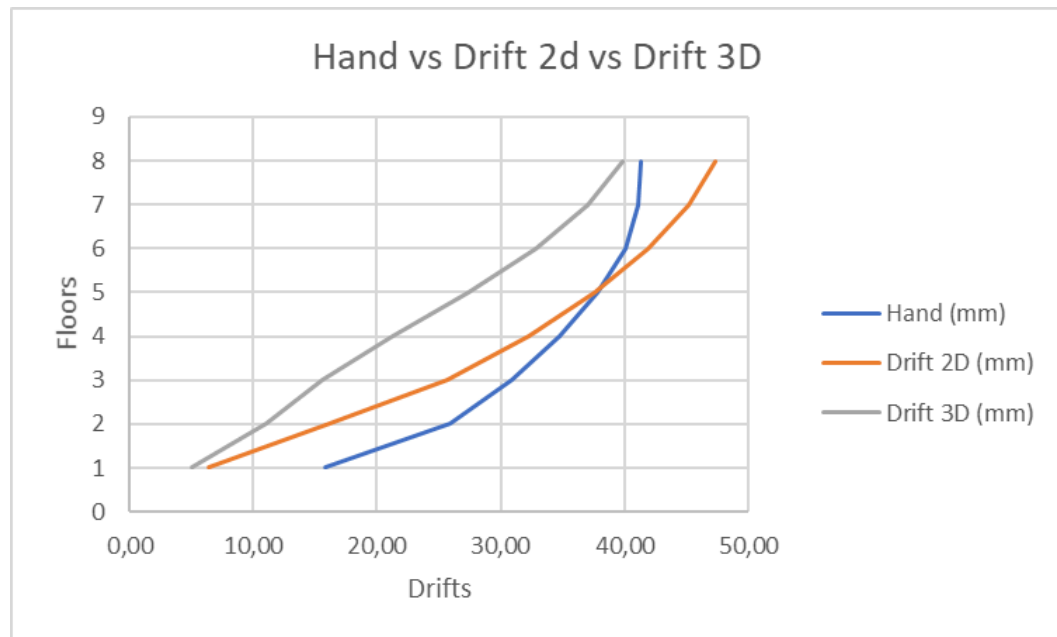


Figure 3.4.4. Drift comparison under Seismic loads for Frame 4.

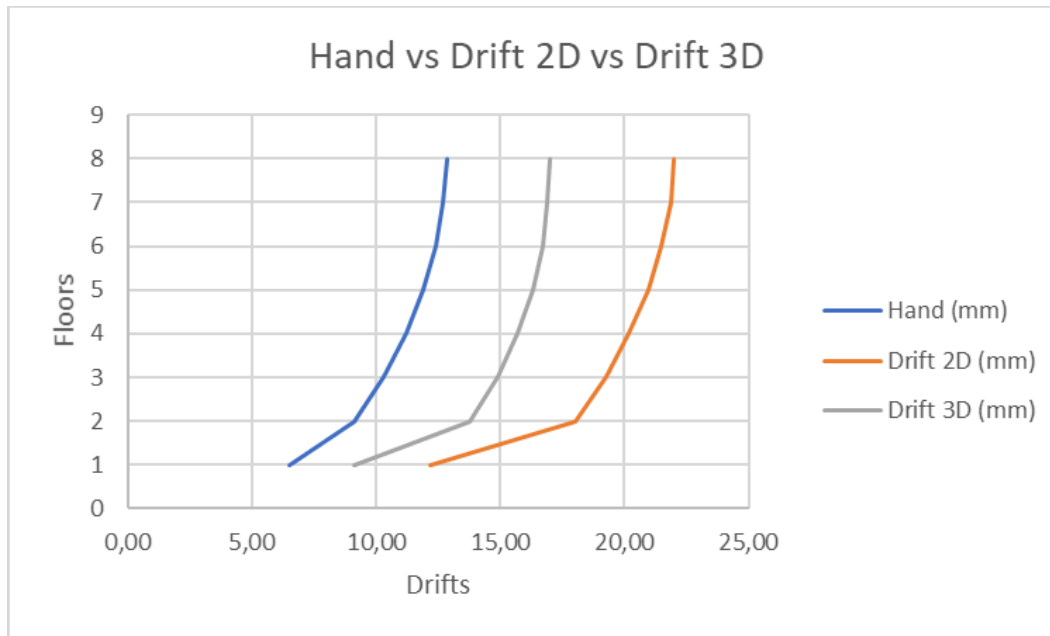


Figure 3.4.5. Drift comparison under Wind loads for Frame C

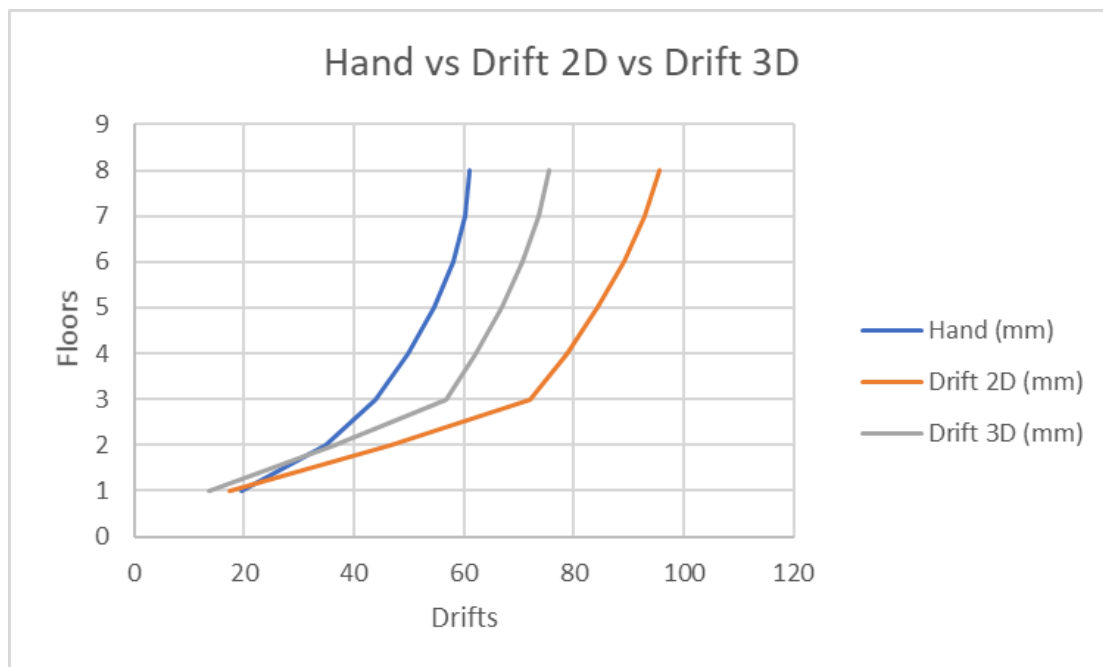


Figure 3.4.6. Drift comparison under Seismic loads for Frame C

3.5. Internal force analysis

3.5.1. Dead load internal force calculations.

Internal forces under dead loads were determined using the approximate analysis method. The inflection point is normally assumed to be located at $0.1L$; it is changed to $0.2L$ based on the SAP2000 results for better accuracy. Given the different lengths and dead loads on the beams, different load cases and frame analyses were evaluated, as shown below.

Table 3.5.1.1 Comparison of axial forces under dead load for Frame D for exterior column.

Floor	Axial Hand	Axial 2D	Axial 3D
Roof	-217.69	-267.95	-314.98
8	-745.03	-626.50	-849.01
7	-1258.48	-1051.37	-1367.82
6	-1515.21	-1472	-1888
5	-1769.01	-1892	-2407.23
4	-2748.51	-2321.62	-2984.89
3	-3630.36	-3218.8	-3964.72
2	-4651.71	-4119.03	-4937.37
1	-5504.46	-5018.52	-5813.29

Table 3.5.1.2 Comparison of shear forces under dead load for Frame D for exterior column.

Floor	Shear Hand	Shear 2D	Shear 3D
Roof	-217.69	-242.6	206.47
8	-295.01	-325.09	292.20
7	-294.49	-391.5	283.81
6	-294.49	-387.27	284.69
5	-294.49	-386.9	284.03
4	-294.49	-395.7	288.95
3	-1032.08	-782.39	-1035.83

2	-909.25	-785.43	-977.03
1	-984.72	-784.69	-878.96

Table 3.5.1.3 Comparison of moments under dead load for Frame D for exterior column.

Floor	Moment Hand	Moment 2D	Moment 3D
Roof	544.22	565.67	426.06
8	1014.81	1005.35	1023.8
7	962.73	1112.75	1216.48
6	962.73	1103.63	1184
5	962.73	1108.15	1120
4	962.73	1135.23	1004.295
3	1632.5	1587.26	1463.2
2	1469.5	1256.9	1404.55
1	1421	1259.31	1961.11

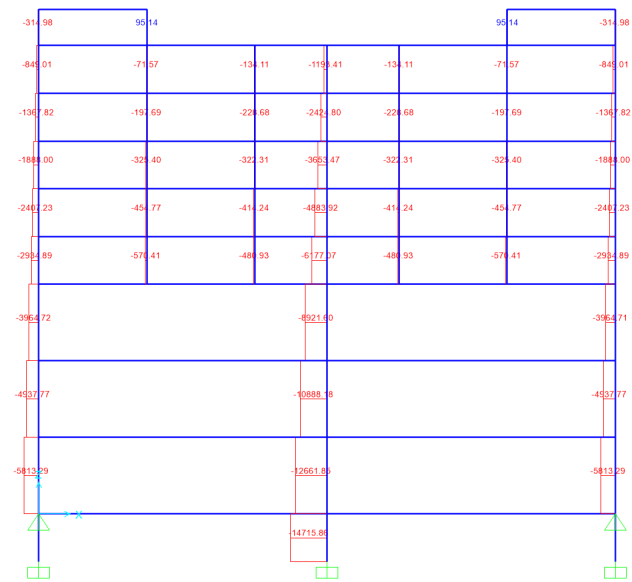


Figure 3.5.1.1. Axial force diagram from dead loads SAP2000 3D model.

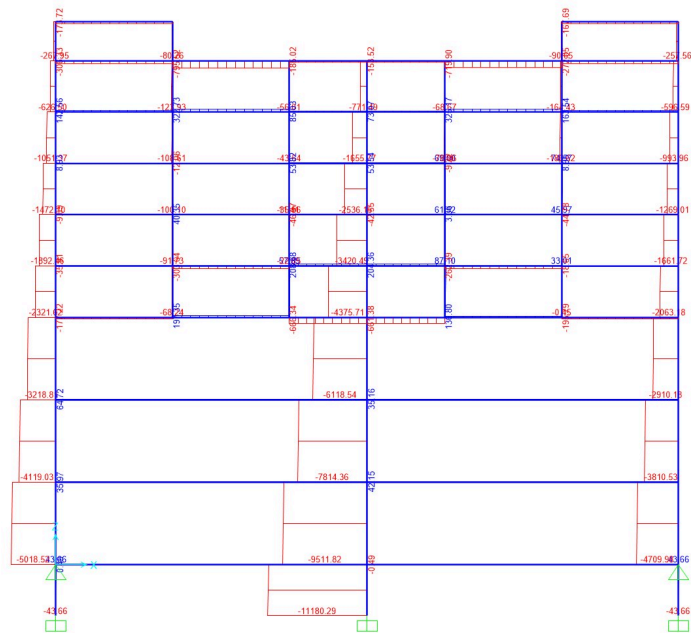


Figure 3.5.1.2. Axial force diagram from dead loads SAP2000 2D model.

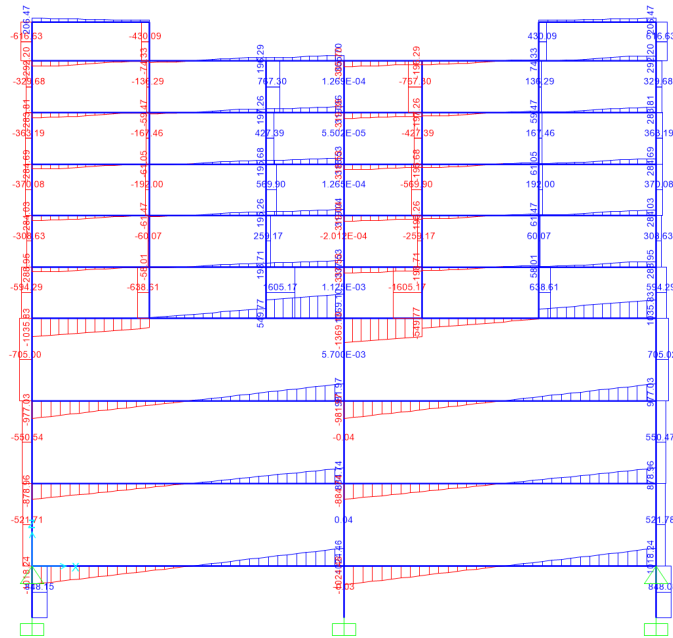


Figure 3.5.1.3 Shear force diagram from dead loads SAP2000 3D model.

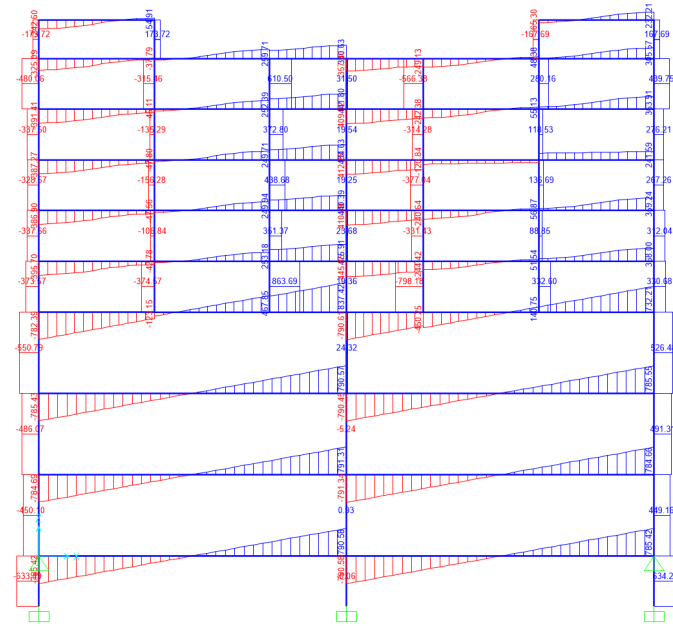


Figure 3.5.1.4 Shear force diagram from dead loads SAP2000 2D model.

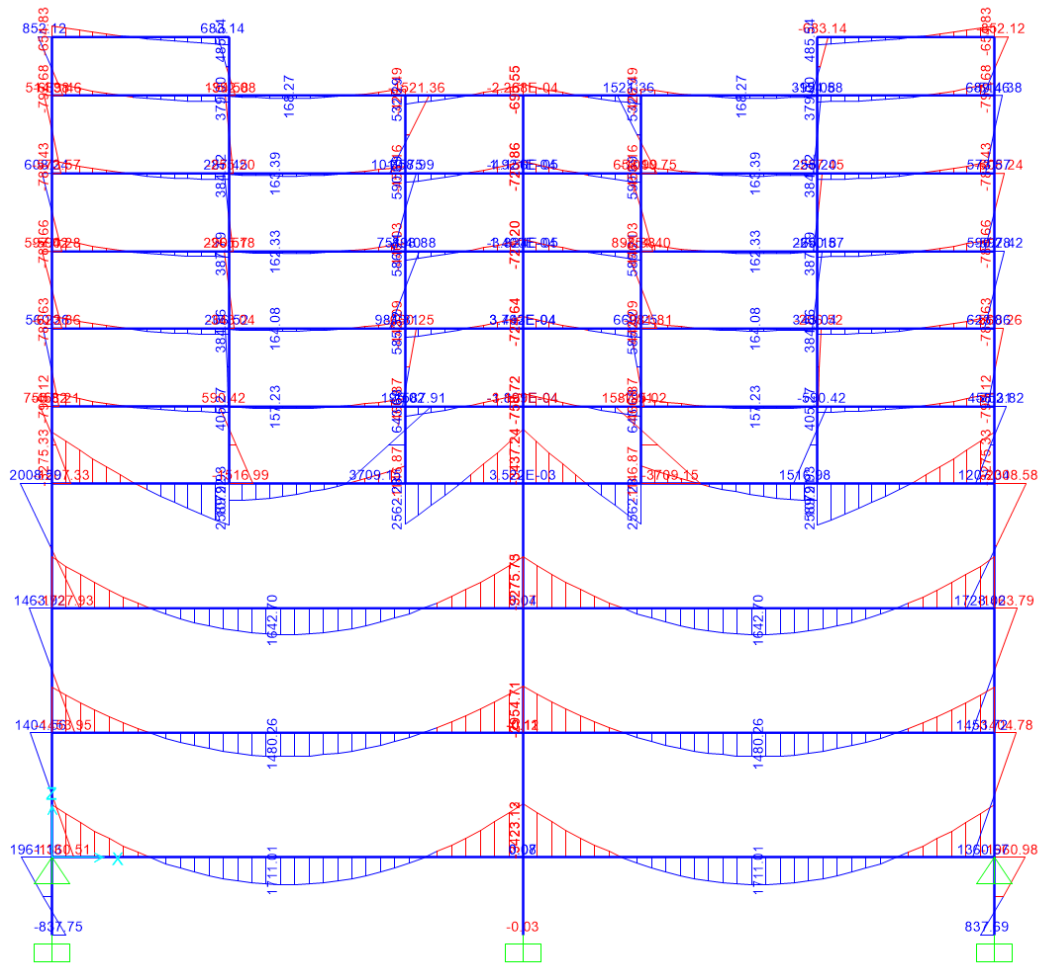


Figure 3.5.1.5. Moment diagram from dead loads SAP2000 3D model.

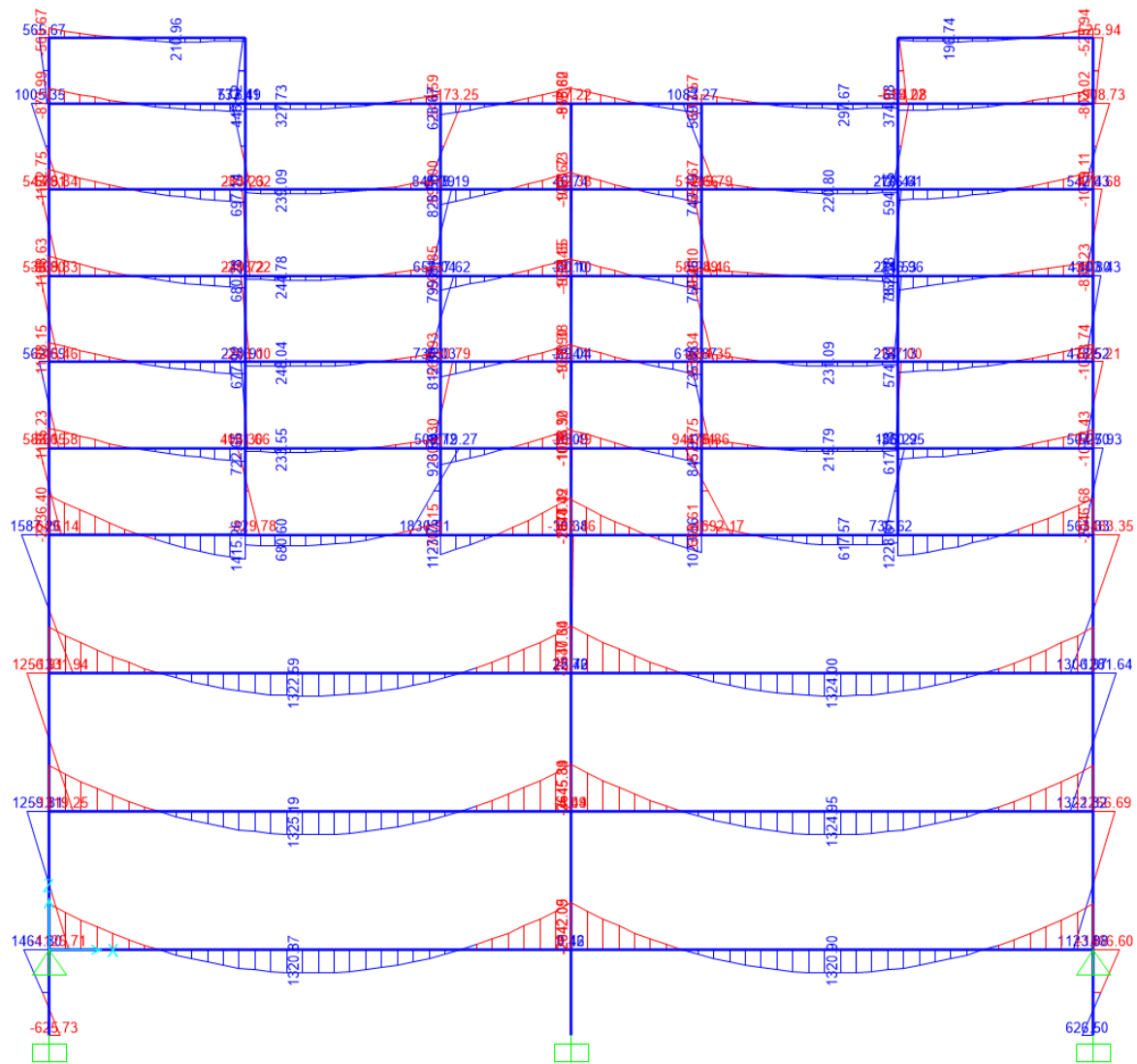


Figure 3.5.1.6. Moment diagram from dead loads SAP2000 2D model.

3.5.2. Wind Load Internal Force Calculations

The portal frame method is used to calculate internal forces due to wind load. The design steps are explained below, while the comparison between the manual and SAP2000 results are shown in Tables 3.6.2.1-3.6.2.3

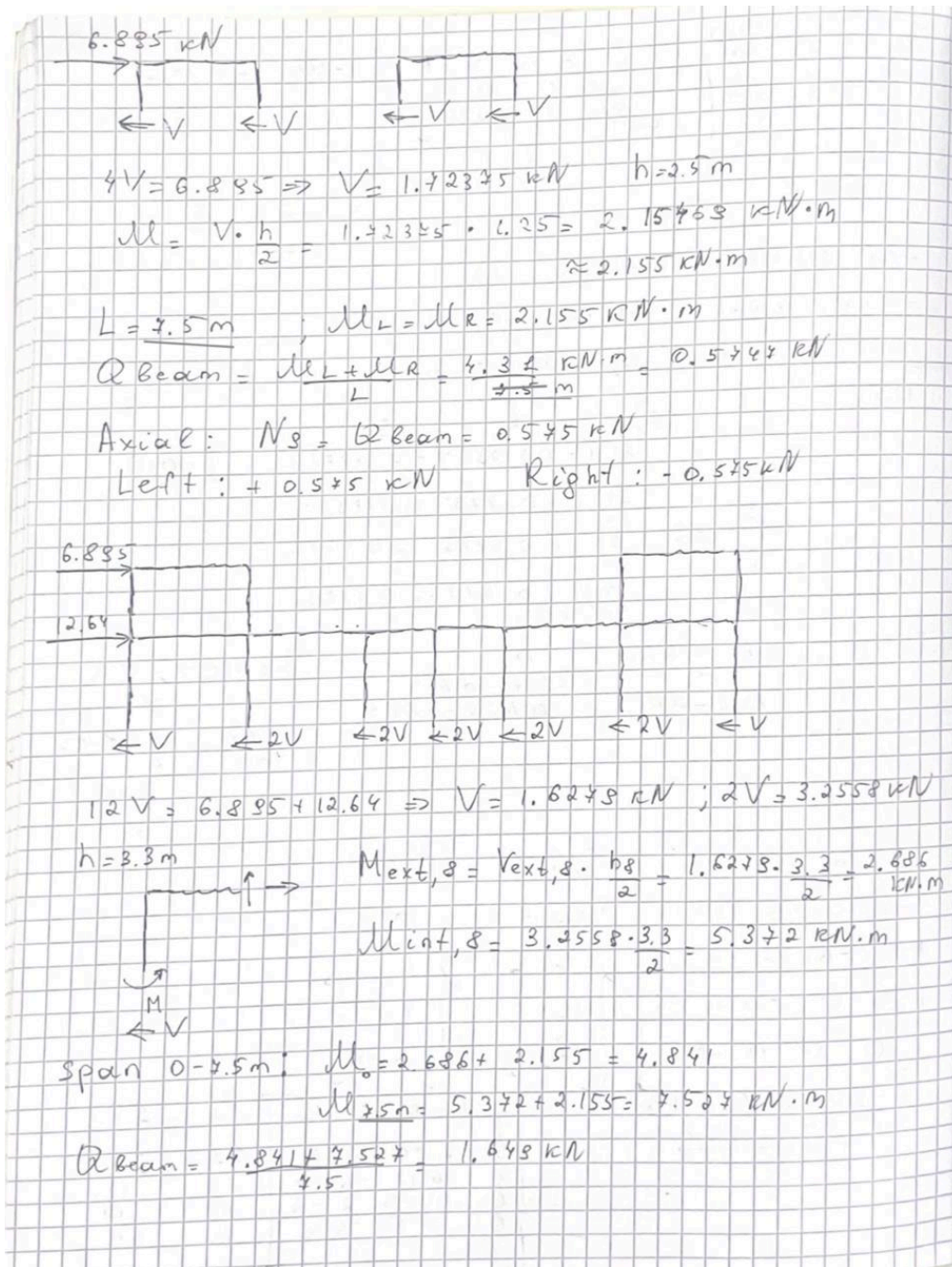


Figure 3.5.2.1. Hand calculations of internal forces part 1

Span 7.5-25m: $M_{7.5} = 2.527 \text{ kN}\cdot\text{m}$ $M_{15} = 5.372 \text{ kN}\cdot\text{m}$ $\left. \vphantom{\begin{matrix} M_{7.5} \\ M_{15} \end{matrix}} \right\} Q_B = \frac{2.527 + 5.372}{7.5} = 1.120 \text{ kN}$

Span 15-20m: $Q_B = \frac{5.372 + 5.372}{5 \text{ m}} = 2.148 \text{ kN}$

Axial force in columns of 8-th floor:

$Q_1 = 1.648$ $Q_2 = 1.120$ $Q_3 = 2.148$ $Q_4 = 2.148$

$Q_5 = 1.120$ $Q_6 = 1.648$

Axial from 8-th floor in combine:

$N_{8, x=0} = 0.575 + 1.648 = 2.224 \text{ kN}$

$N_{8, x=7.5} = -0.575 + (1.120 - 1.648) = -0.509 \text{ kN}$

$N_{8, x=15} = 2.148 - 1.120 = 0.428 \text{ kN}$

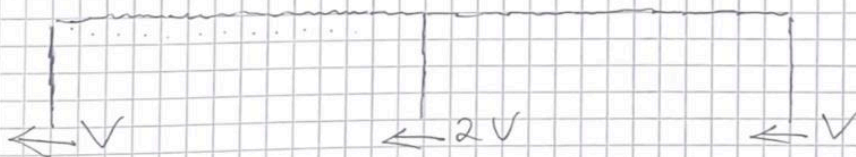
$N_{8, x=20} = 2.148 - 2.148 = 0$

$N_{8, x=25} = 1.120 - 2.148 = -0.428 \text{ kN}$

$N_{8, x=32.5} = 0.509 \text{ kN}$ $N_{8, x=40} = -2.224 \text{ kN}$

First Floor:

$V_i = 28.363 + 33.07 + 28.1 + 22.27 + 23.23 + 24.103 + 24.88 + 12.64 + 6.895 = 204.561 \text{ kN}$



$4V = 204.561 \text{ kN} \Rightarrow V = 51.140 \text{ kN}$ $2V = 102.281 \text{ kN}$

$M_{L, 1} = 51.140 \cdot \frac{5.3}{2} = 135.52 \text{ kN}$

$M_{middle, 1} = 102.281 \cdot \frac{5.3}{2} = 271.04 \text{ kN}$

Figure 3.5.2.2. Hand calculations of internal forces part 2

Handwritten calculations on graph paper:

$$Q_{B, \pm} (\text{left}) = \frac{252.253 + 504.506}{2} = 37.832 \text{ kN}$$

$$Q_{B, \pm} (\text{right}) = \underline{37.832}$$

$$M_{\text{overturning}} = \sum F_i \cdot z_i = 3910.143 \text{ kN}\cdot\text{m}$$

Figure 3.5.2.3. Hand calculations of internal forces part 3

Table 3.5.2.1 Comparison of axial forces under wind load for Frame D for exterior column.

Floor	Axial Hand	Axial 2D	Axial 3D
Roof	0.575	1.32	1.55
8	2.224	2.80	3.8
7	5.742	5.37	6.9
6	11.954	9.51	11.06
5	20.770	15.25	16.29
4	32.088	22.29	21.98
3	47.303	47.04	43.25
2	70.649	57.65	54.93
1	97.754	67.44	66.10

Table 3.5.2.2. Comparison of shear forces under wind load for Frame D for exterior column.

Floor	Shear Hand	Shear 2D	Shear 3D
Roof	1.723	2.10	0.74
8	1.628	0.99	1.96
7	3.702	4.07	4.03
6	5.711	4.88	4.28
5	15.294	13.97	17.09

4	28.506	28.49	24.15
3	35.782	37.75	36.31
2	44.049	50.81	42.53
1	51.140	70.40	56.50

Table 3.5.2.3. Comparison of moments under wind load for Frame D for exterior column.

Floor	Moment Hand	Moment 2D	Moment 3D
Roof	2.155	3.54	3.54
8	2.686	4.01	4.01
7	6.108	7.72	7.72
6	9.423	9.45	9.45
5	12.617	39.36	23.1
4	15.679	28.1	35.53
3	94.822	112.23	112.12
2	116.731	115.54	115.54
1	135.522	154.67	145.32

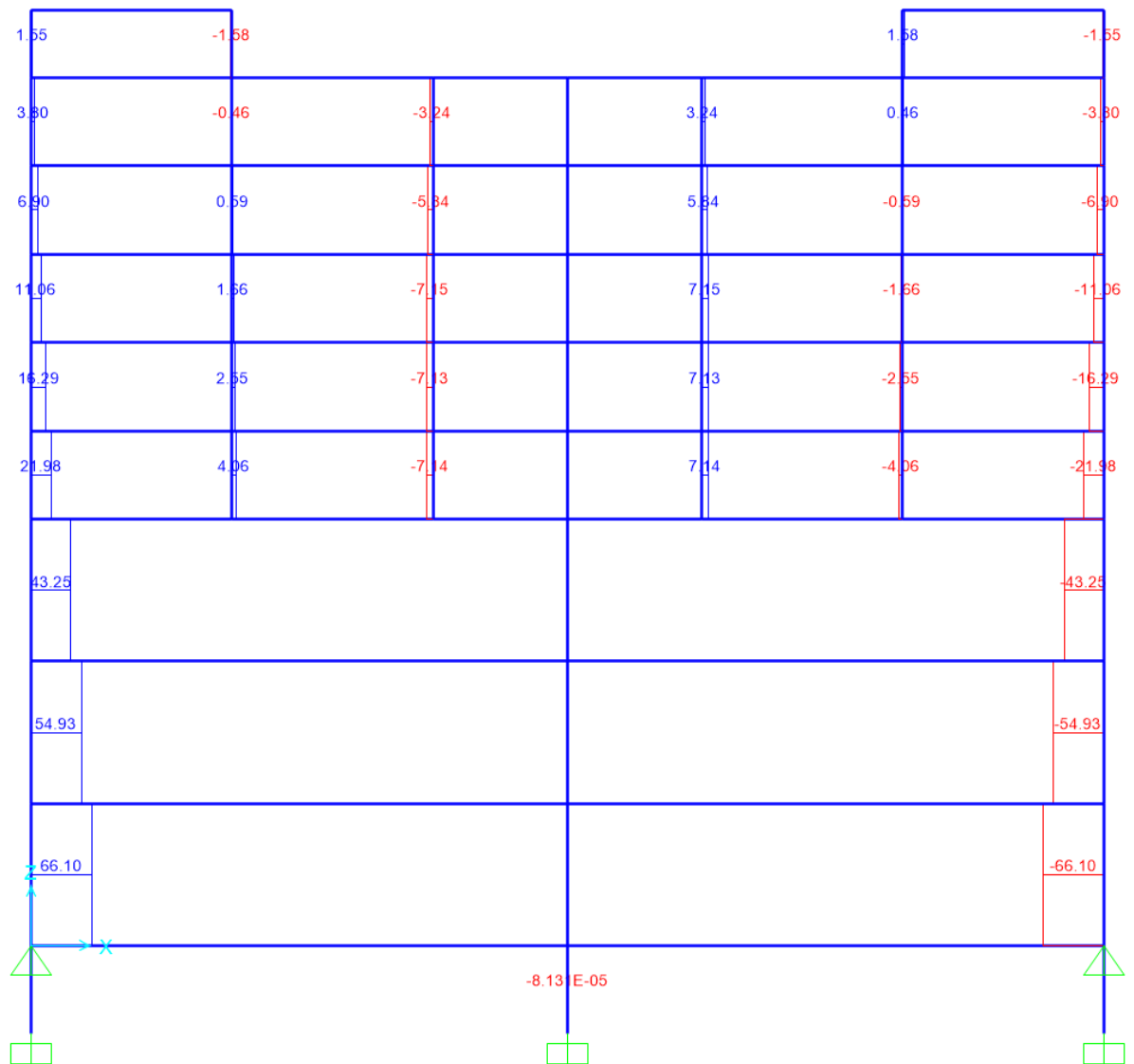


Figure 3.5.2.4. Axial Force diagram from Wind loads SAP2000 3D model.

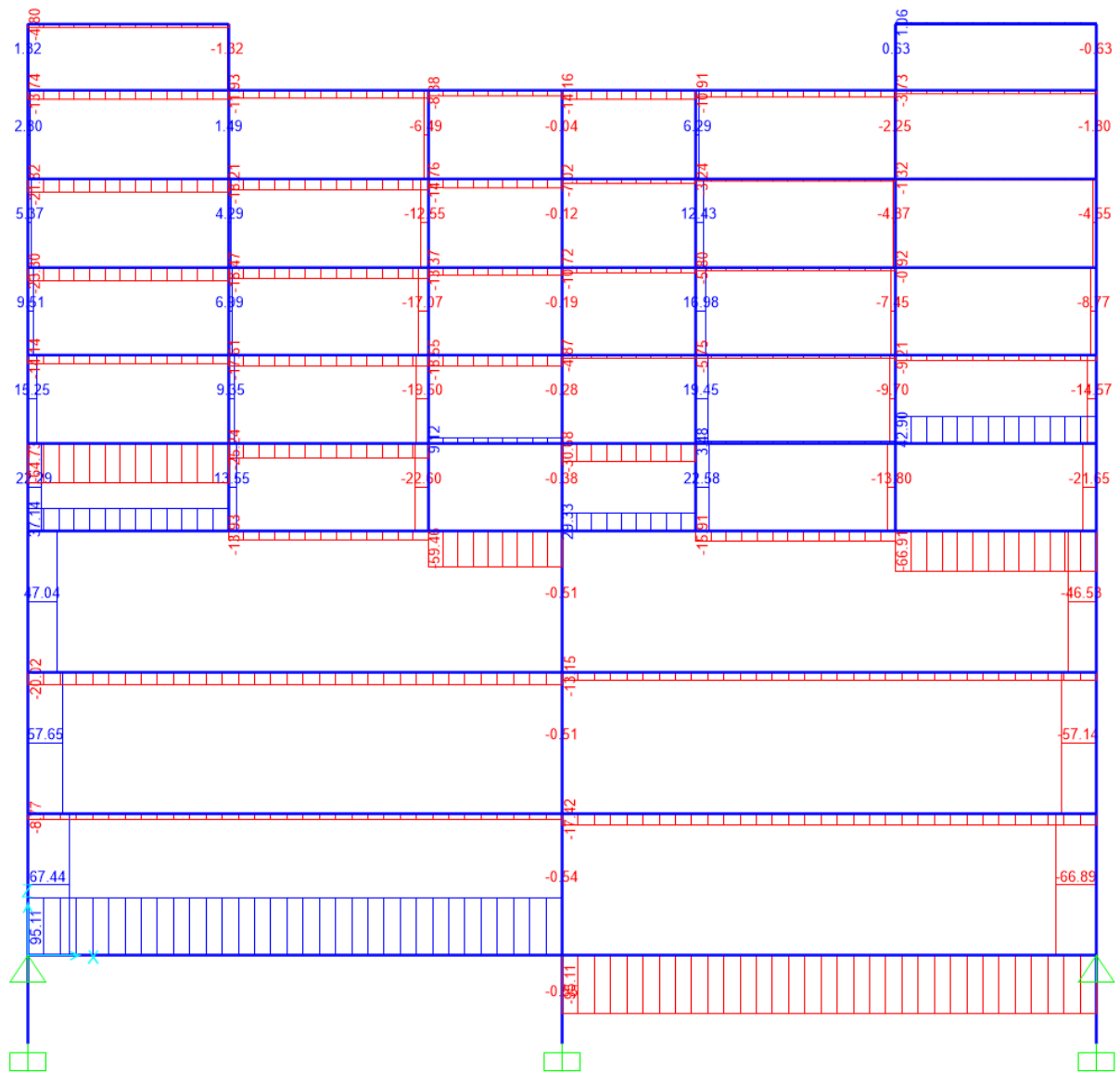


Figure 3.5.2.5. Axial Force diagram from Wind loads SAP2000 2D model.

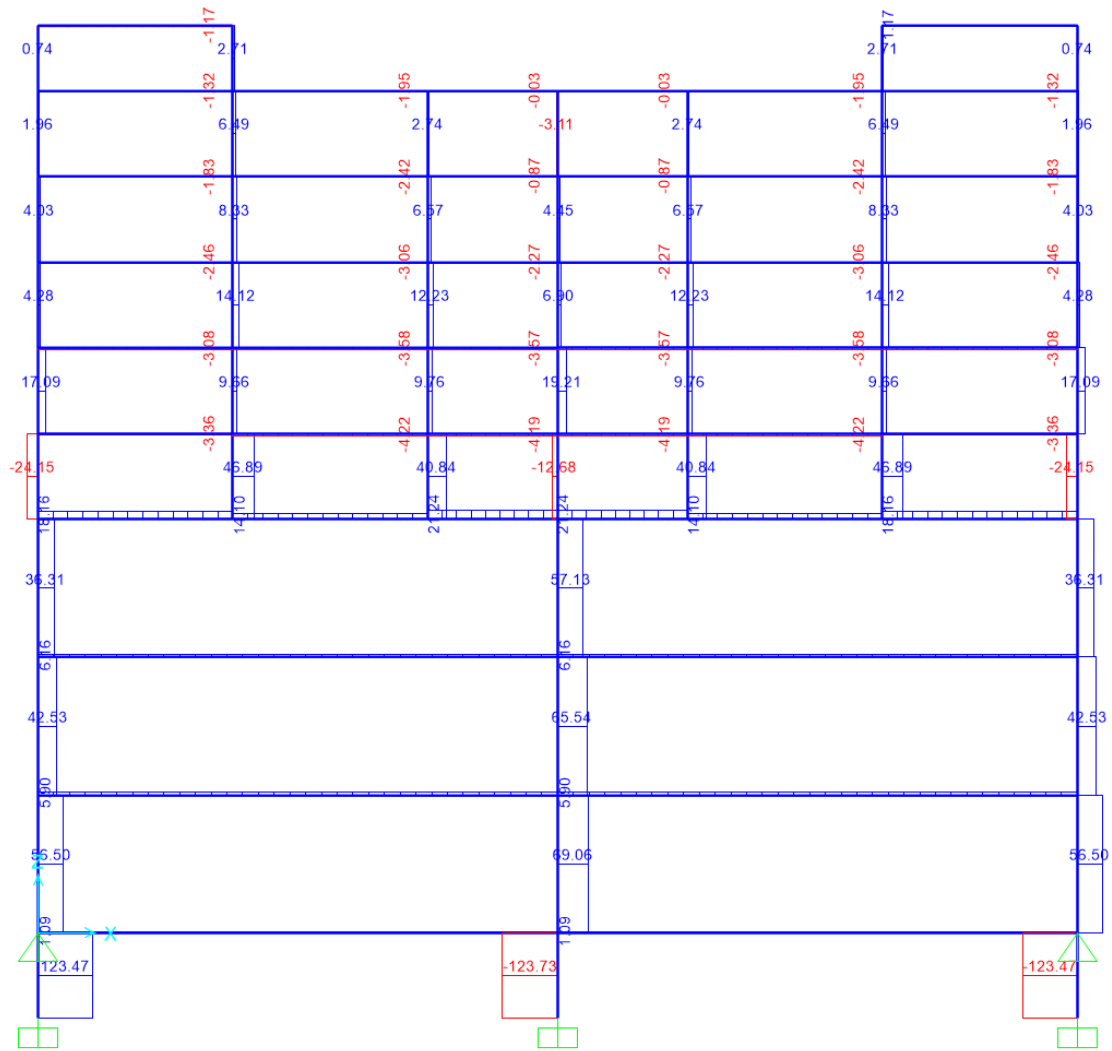


Figure 3.5.2.6. Shear Force diagram from Wind loads SAP2000 3D model.

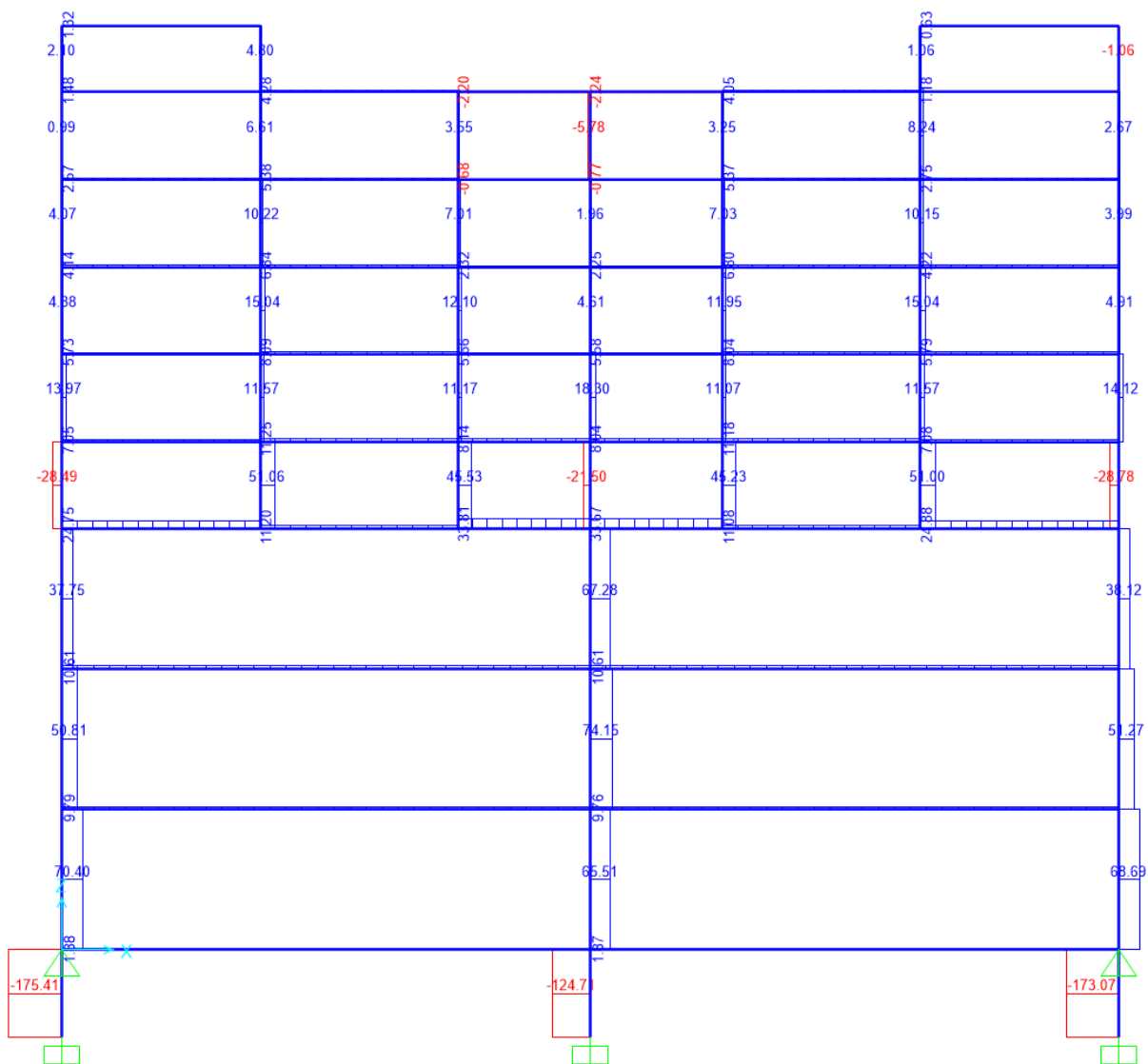


Figure 3.5.2.7. Shear Force diagram from Wind loads SAP2000 2D model.

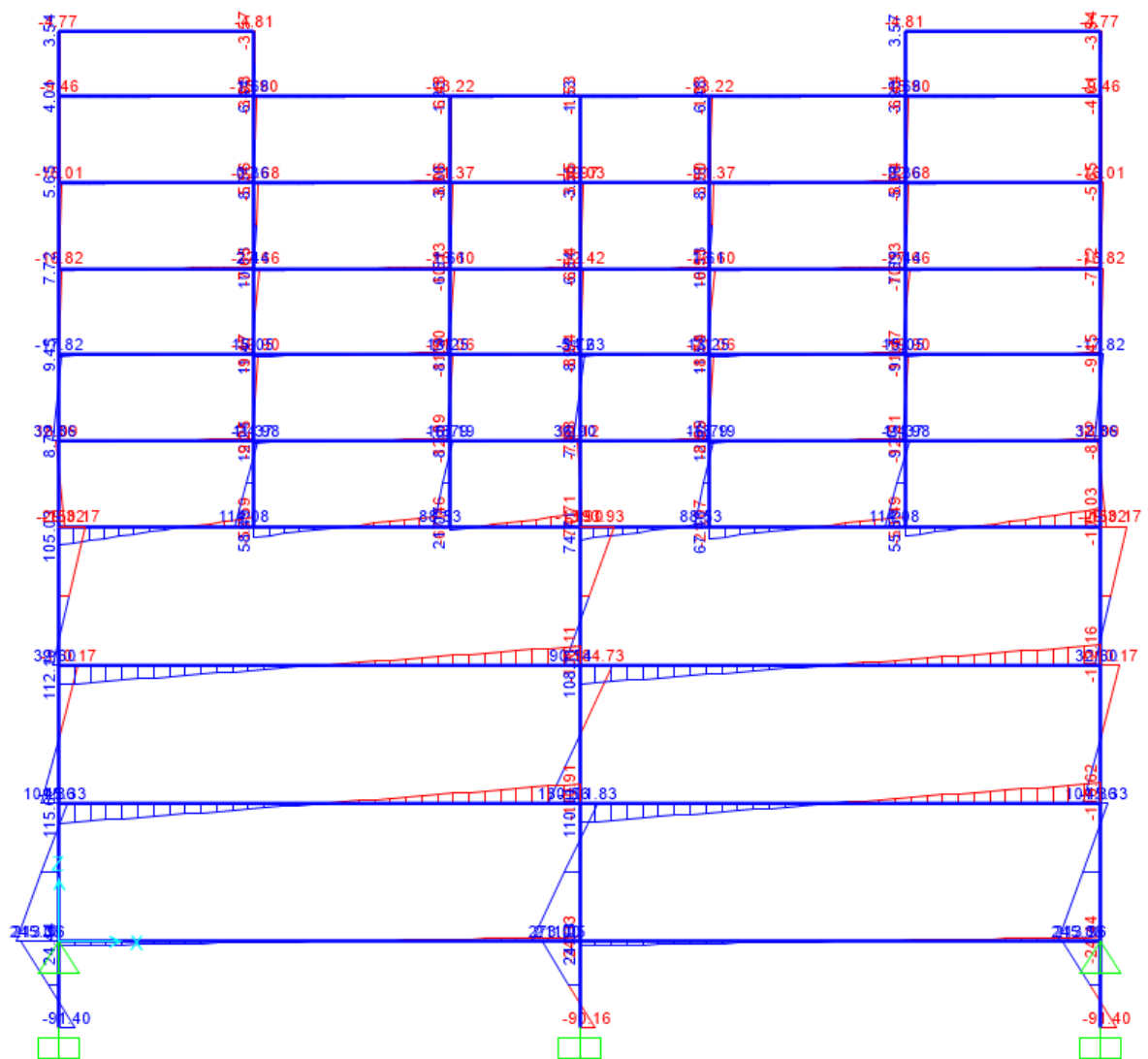


Figure 3.5.2.8. Moment diagram from Wind loads SAP2000 3D model.

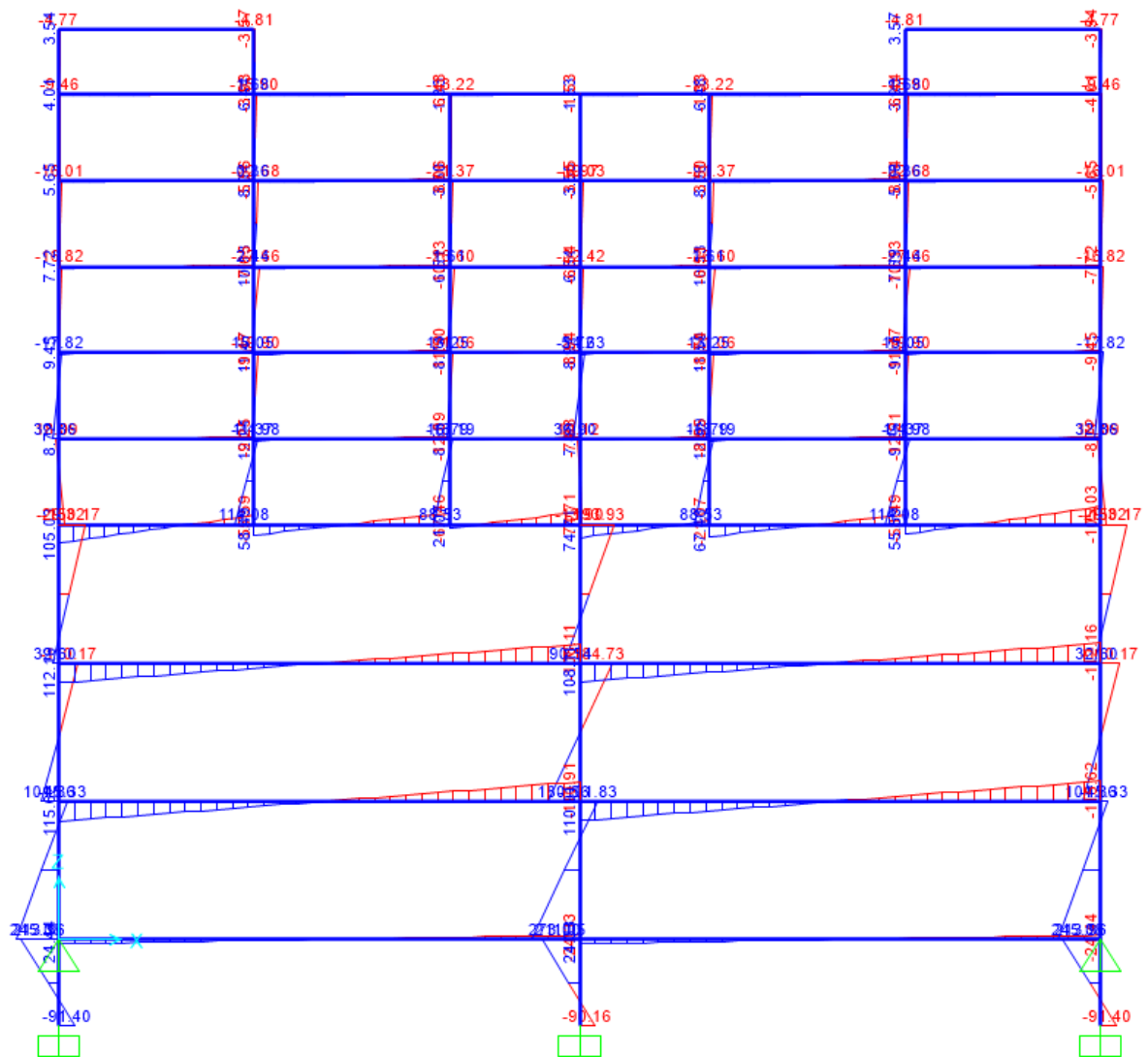


Figure 3.5.2.9. Moment diagram from Wind loads SAP2000 2D model.

3.6. Structural member design

3.6.1. Prestressed Beam design

The design example for the prestressed beam was conducted in a similar step-by-step logic as used in the example of post-tensioned beam design, which starts from geometry, material, load, serviceability, flexural, shear and torsional checks. The example also explains that

effective flange width should be considered for flexure, and the entire tributary area for the axial prestressing effects.

3.6.1.1. Step 1. Geometry and Section Properties

The beam was designed as a T-section. For bending effects, the effective flange width was taken as: $b_{eff} = 5000 \text{ mm}$

For axial prestressing effects, the full tributary width was used: $b_{eff} = 7500 \text{ mm}$

The main geometric properties are: $h = 900 \text{ mm}$, $h_f = 375 \text{ mm}$, $b_w = 450 \text{ mm}$

For bending: $A_f = 1875000 \text{ mm}^2$, $A_w = 405000 \text{ mm}^2$, $A = 2280000 \text{ mm}^2$

The section centroid and section modulus were calculated as:
 $Y_t = 300.74 \text{ mm}$, $Y_b = 974.26 \text{ mm}$, $I = 184667501027.96 \text{ mm}^4$, $S_{Top} = 614043426.99 \text{ mm}^3$

$S_{Bottom} = 189546451.63 \text{ mm}^3$

Axial effects:
 $A = 3217500 \text{ mm}^2$, $Y_t = 267.74 \text{ mm}$, $Y_b = 1007.26 \text{ mm}$, $I = 204172822880.25 \text{ mm}^4$

3.6.1.2. Step 2. Material Properties

Concrete: $f'_c = 41.37 \text{ MPa}$, $\gamma_c = 24 \text{ kN/m}^3$, $E_c = 30230.17 \text{ MPa}$.

Passive Reinforcement: $f_y = 517.107 \text{ MPa}$, $E_c = 200000 \text{ MPa}$, $\phi = 0.9$

Prestressing steel: $f_{pu} = 1860 \text{ MPa}$, $E_p = 200000 \text{ MPa}$, $A_{strand} = 140 \text{ mm}^2$, $f_{se} = 1302 \text{ MPa}$

The tendon system is unbonded, with: $\mu = 0.07$, $K = 0.003$, anchor set = 6 mm.

3.6.1.3. Step 3. Loads

$w_{beam} = 10.125 \text{ kN/m}$, $w_{slab} = 42.183 \text{ kN/m}$, $w_D = 88.185 \text{ kN/m}$

$P_D = 1046.16 \text{ kN/m}$, $P = 1052.84928 \text{ kN/m}$

3.6.1.4. Step 4. Design Parameters and Allowable Stresses

The allowable tensile stresses were calculated as: $f_t(U) = 3.9878 \text{ MPa}$, $f_t(T) = 4.824 \text{ MPa}$,

$$f_t(C) = 6.4319 \text{ MPa}$$

The allowable compressive stresses were: $f_c(\text{sustained}) = -18.6165 \text{ MPa}$

$$f_c(\text{total}) = -24.822 \text{ MPa}.$$

For service condition:: $f_t(\text{bottom}) = 6.432 \text{ MPa}$, $f_t(\text{Top}) = 4.824 \text{ MPa}$.

For initial condition:: $f_t = 1.345 \text{ MPa}$, $f_c = 17.375 \text{ MPa}$.

3.6.1.5. Step 5. Actions Due to Dead and Live Loads

The bending moments due to dead and live loads were obtained for Span 1 and Span 2.

Table 3.6.1.1. For Span 1

Location	MD, kN·m	ML, kN·m	MD + 0.3ML, kN·m	MD + ML, kN·m
Left	-3356.08	-387.87	-3472.44	-3743.95
Mid	1711.44	197.05	1770.55	1908.49
Right	-3425.17	-394.45	-3543.5	-3819.61

Table 3.6.1.2. For Span 2

Location	MD, kN·m	ML, kN·m	MD + 0.3ML, kN·m	MD + ML, kN·m
Left	-3425.16	-394.45	-3543.5	-3819.61
Mid	1711.44	197.05	1770.55	1908.49
Right	-3356.08	-387.87	-3472.44	-3743.95

3.6.1.6. Step 6. Post-Tensioning Design

The target average precompression was selected between: $1 \text{ MPa} \leq f_{PC} \leq 2 \text{ MPa}$

The target balanced dead load was: 60 % of Dead Load

The effective stress in the unbonded tendon was: $f_{SE} = 1302 \text{ MPa}$

Force Per Strand $P_{Strand} = f_{SE} * A_{Strand} = 1302 * 140/1000 = 182.28 \text{ kN}$

The required prestressing force was: $P = 3217.5 \text{ kN}$

Required number of strands: $n = \frac{3217.5 \text{ kN}}{182.28 \text{ kN}} = 17.65$

Therefore, 19 strands were selected. $P_{19} = 19 * (182.28) = 3463.32 \text{ kN}$

For the main tendon, the tendon eccentricity was: $e_{max} = 830 \text{ mm}$, $a = b = 760 \text{ mm}$

Span Length $L = 20 \text{ m}$

The balanced load from the tendon was: $w_b = \frac{8P^*e}{L^2} = 52.642464 \text{ kN/m}$

The percentage of dead load balanced was: $\frac{52.642464}{88.185} * 100\% = 60\%$

Therefore, the selected prestressing force balanced approximately 60% of the dead load.

The balanced reactions were: $R_{Left} = 526.42464 \text{ kN}$, $R_{Right} = 526.42464 \text{ kN}$

Table 3.6.1.3. The post-tensioning moments from balanced loads

Location	Moment, kN·m
Left End	1761.09
Right Mid	-898.3
Mid Support	1799.39
Mid Support	1761.09
Right Mid	-898.3
Right End	1799.39

3.6.1.6. Step 7. Serviceability Check

The deflection check gave: $\Delta DL = 50.3 \text{ mm}$, $\Delta PT = -25.9 \text{ mm}$

$\Delta DL + PT = 24.4 \text{ mm}$, $\Delta LL = 5.8 \text{ mm}$

Long term deflection was calculated using creep factor: $\lambda = 2$

$\Delta \text{Long term} = 78.42 \text{ mm}$, The allowable deflection was: $\Delta_{allow} = 83.33 \text{ mm}$

The deflection requirement is satisfied.

Service stress checks were performed for: $DL + 0.3LL + PT$ and $DL + LL + PT$

The stress formula used was: $f = A/P \pm S/M$

Table 3.6.1.4. The Serviceability Check

Load Combination	ACI 318	Point A	Point B	Point C	Point D	Point E	Point F
DL+0.3LL+PT	ft, MPa	1.268	-2.940	-4.359	1.384	-2.940	1.206
	fb, MPa	-10.548	3.083	7.682	-10.923	3.083	-10.346
	Ft, MPa	4.824	-18.617	4.824	4.824	-18.617	4.824
	Fb, MPa	-18.617	4.824	-18.617	-18.617	4.824	-18.617
		OK	OK	OK	OK	OK	OK
DL+LL+PT	ft, MPa	1.710	-3.164	1.771	1.833	-3.164	1.648
	fb, MPa	-11.980	3.811	-12.177	-12.379	3.811	-11.778
	Ft, MPa	4.824	-18.617	4.824	4.824	-24.822	4.824
	Fb, MPa	-24.822	6.432	-24.822	-24.822	6.432	-24.822
		OK	OK	OK	OK	OK	OK

3.6.1.7. Step 8. Flexural Strength Design

The design moments were calculated by combining dead load, live load, and secondary post-tensioning moments.

The flexural design moment was taken as: $Mu = MD + ML + Mhyp$

Table 3.6.1.5. obtained factored moments

Point	Mu, kN·m
A	-6408.978
B	5861.7515
C	4005.253
D	4005.265
E	5861.7515
F	-6408.978

The flexural capacity provided by the tendon alone was:

$$\Phi M_n = 4829.10498 \text{ kN} \cdot \text{m}$$

The cracking moment was checked using:

$$M_{Cr} = f_R \cdot S, \text{ and the design requirement was checked against: } 1.2 \cdot M_{Cr}$$

All checked points satisfied the flexural capacity requirement.

Additional flexural reinforcement was provided as follows:

Table 3.6.1.6. Additional flexural reinforcement

Location	Required reinforcement
Points A + F, top tension zone	12#9 bars
Points B + E, bottom tension zone	4#9 bars
Points C + D, top tension zone	12#9 bars

The provided reinforcement areas were: $A_s = 7740 \text{ mm}^2$ for top reinforcement zones, and: $A_s = 2580 \text{ mm}^2$ for bottom reinforcement zones.

3.6.1.8. Step 9. Shear Design

The maximum design shear force from SAP2000 was: $V_u = 895.247 \text{ kN} = 201.2515 \text{ kip}$

The effective depth was: $d = 43.7008 \text{ in}$. Concrete strength: $f'_c = 5998.65 \text{ psi}$.

The beam web width was: $b_w = 17.7165 \text{ in}$. The concrete shear capacity was: $\Phi V_c = 89.9469 \text{ kip}$. Since $V_u > \Phi V_c$ shear reinforcement was required.

The required shear contribution from stirrups was: $V_c = 148.4062 \text{ kip}$. Using #4 bars, the required spacing was: $s = 0.1795 \text{ m}$. Therefore, the selected shear reinforcement was: #4 bars @ 0.1795 m

3.6.1.9. Step 10. Torsion Design

The maximum design torsion from SAP2000 was: $T_u = 115.873 \text{ kN} * \text{m} = 1025.563 \text{ kip} * \text{in}$

The corresponding shear and moment were: $V_u = 895.247 \text{ kN}$, $M = 4986.692 \text{ kip} * \text{in}$

The effective section dimensions were: $b_w = 19.685 \text{ in}$, $h = 45.2756 \text{ in}$

The closed stirrup geometry was based on: $x_0 = 16.0354 \text{ in}$, $y_0 = 41.6260 \text{ in}$, $A_{oh} = 667.491 \text{ in}^2$, $A_o = 567.367 \text{ in}^2$, $p_h = 115.3228 \text{ in}$

The required torsional transverse reinforcement was:

$$A_t/s = 0.0201 \text{ in}^2/\text{in} = 0.000510 \text{ m}^2/\text{m}$$

The required longitudinal torsion reinforcement was:

$$A_L = 2.3162 \text{ in}^2 = 0.0015 \text{ m}^2$$

3.6.2. Beam design

3.6.2.1. Longitudinal reinforcement design

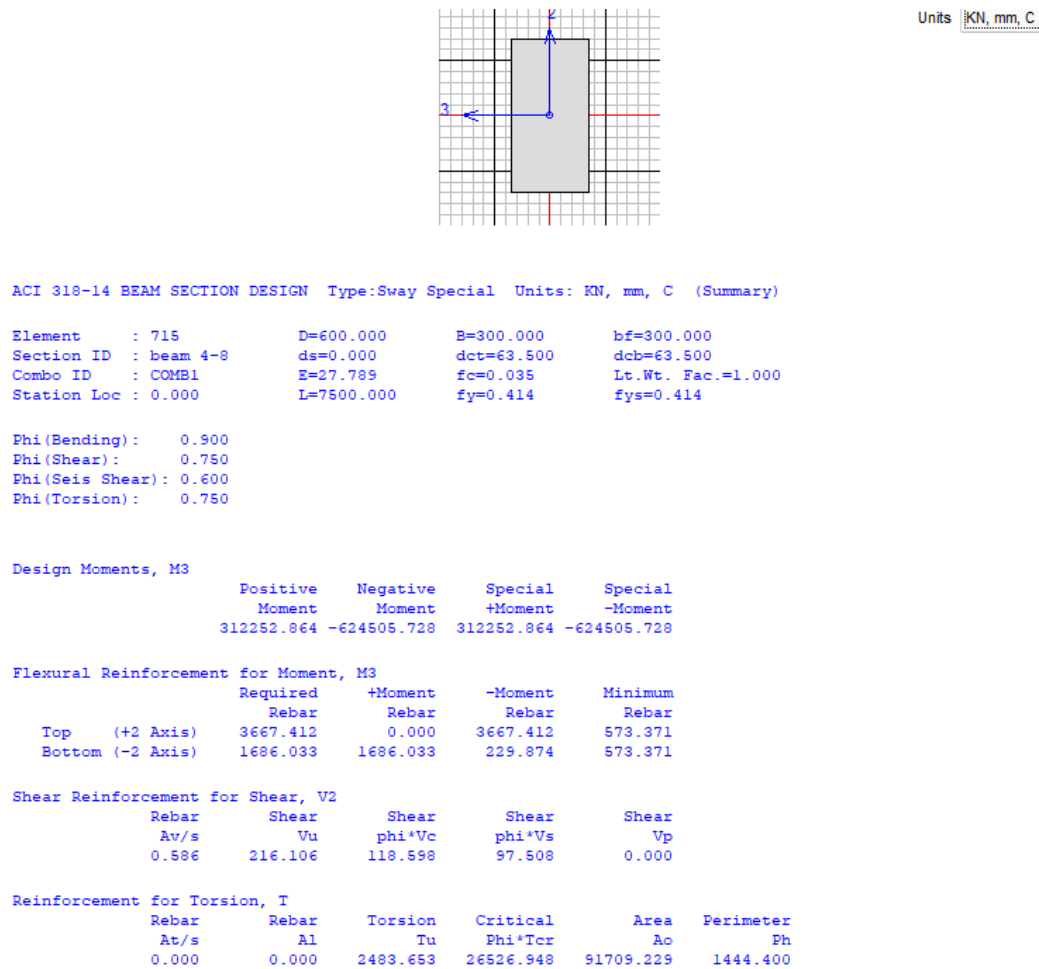


Figure 3.6.2.1. Beam reinforcement design in SAP2000

For a negative moment:

$$R_n = \frac{M_u}{\Phi * b * d^2} = \frac{624.506}{0.9 * 0.3 * 0.55^2} = 6995.8$$

$$\rho = \frac{0.85f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right] = 0.0196$$

$$A_s = \rho * b * d = 0.0196 * 0.3 * 0.55 = 3.381 * 10^{-3} m^2$$

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 2.34 * 10^{-4} m^2 \geq 8.34 * 10^{-5} m^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.381 * 10^{-3} * 413685}{0.85 * 35000 * 0.3} = 0.157m$$

$$\beta_1 = 0.85 - \left(\frac{f'_c - 4000psi}{1000}\right)(0.05) = 0.85 - \left(\frac{5000 - 4000psi}{1000}\right)(0.05) = 0.8$$

$$c = \frac{a}{\beta_1} = \frac{0.157}{0.8} = 0.196m$$

$$\epsilon_t = \frac{d-c}{c} (0.003) = \frac{0.55-0.196}{0.196} (0.003) = 0.0058 > 0.005$$

Tension controlled and ductile

$$Error = \frac{3.667 * 10^{-3} - 3.381 * 10^{-3}}{3.667 * 10^{-3}} = 7.8\%$$

According to SAP2000 the required rebar area is $3.667 * 10^{-3} m^2$. 7.8% error is not significant and $A_s > A_{s,min}$, which shows the correctness of hand calculations. Therefore, $3.667 * 10^{-3} m^2$ can be used to select bar size. 6#9 bar size is optimal for bottom reinforcement.

For a positive moment:

$$R_n = \frac{M_u}{\Phi * b * d^2} = \frac{312.258}{0.9 * 0.3 * 0.575^2} = 3497.95 kN/m^2$$

$$\rho = \frac{0.85 f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}} \right] = 0.009$$

$$A_s = \rho * b * d = 0.009 * 0.3 * 0.575 = 1.565 * 10^{-3} m^2$$

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 2.34 * 10^{-4} m^2 \geq 8.34 * 10^{-5} m^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.565 * 10^{-3} * 413685}{0.85 * 35000 * 0.3} = 0.073m$$

$$\beta_1 = 0.85 - \left(\frac{f'_c - 4000 \text{ psi}}{1000} \right) (0.05) = 0.85 - \left(\frac{5000 - 4000 \text{ psi}}{1000} \right) (0.05) = 0.8$$

$$c = \frac{a}{\beta_1} = \frac{0.073}{0.8} = 0.091m$$

$$\epsilon_t = \frac{d-c}{c} (0.003) = \frac{0.575 - 0.091}{0.091} (0.003) = 0.016 > 0.005$$

Tension controlled and ductile.

$$Error = \frac{1.686 * 10^{-3} - 1.565 * 10^{-3}}{1.686 * 10^{-3}} = 7.18\%$$

According to SAP2000 the required rebar area is $1.686 * 10^{-3} m^2$. 7.18% error is not significant and $A_s > A_{s,min}$, which shows the correctness of hand calculations. Therefore,

$1.686 * 10^{-3} m^2$ can be used to select bar size. 4#8 bar size is optimal for bottom reinforcement.

3.6.2.2. Shear reinforcement design

Maximum shear for the beam is 216.106 kN

$$\phi V_c = 2\phi\lambda\sqrt{f'_c}bd = 0.75 * 2 * 1.0 * \sqrt{35000} * 0.3 * 0.55 = 46.303 kN$$

Since V_u is larger than $\frac{1}{2}\phi V_c$ stirrups are needed.

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{216.106 - 46.303}{0.75} = 223.6 kN$$

$$V_{c1} = 4\sqrt{f'_c}bd = 4\sqrt{35000} * 0.3 * 0.55 = 123.475 kN$$

$$V_{c2} = 8\sqrt{f'_c}bd = 8\sqrt{35000} * 0.3 * 0.55 = 246.95 kN$$

Spacing is calculated assuming #3 stirrup size

$$S_1 = A_v f_{yt} d / V_s = \frac{1.42 \cdot 10^{-4} \cdot 420000 \cdot 0.550}{223.6} = 0.144m$$

Since, V_s less than V_{c1} , then S_2 is calculated using the following formula:

$$S_2 = d/2 = 0.550/2 = 0.275m$$

$$S_3 = \frac{A_v f_{yt}}{50b} = \frac{2.42 \cdot 10^{-4} \cdot 420000}{50 \cdot 0.3} = 3.976m \geq \frac{A_v f_{yt}}{0.75 \sqrt{f'_c} b} = \frac{1.42 \cdot 10^{-4} \cdot 420000}{0.75 \sqrt{35000} \cdot 0.3} = 1.42m$$

$$S_{max} = 0.275m$$

Since S_1 is less than S_{max} , then S is 0.15 m

V_s for maximum spacing:

$$V_s = A_v f_{yt} d / s = \frac{1.42 \cdot 10^{-4} \cdot 420000 \cdot 0.55}{0.275} = 119.28kN$$

$$\phi V_s = 89.46kN$$

$$\phi V_s + \phi V_c = 137.87kN$$

$$\frac{V_u - (\phi V_s + \phi V_c)}{x} = \frac{V_u}{L/2} \Rightarrow \frac{216.106 - 137.87}{x} = \frac{216.106}{3.75} \Rightarrow 1.36m$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{1.42 \cdot 10^{-4}}{0.2} = 0.00071m^2/m$$

$$Error = \frac{0.000586 - 0.00071}{0.000586} = 21\%$$

Table 3.6.2.1. Spacing for 7.5 m beam in Floor 8

Number of stirrups	Spacing	Distance from edge
9	0.150m	0-1.35
17	0.275m	1.35-6.15
9	0.150m	6.15-7.5

3.6.2.3. Torsional reinforcement design

$$\phi T_n = \phi * \lambda * \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) = 0.75 * 1.0 * \sqrt{35000} * \left(\frac{(0.3*0.575)^2}{2(0.3+0.575)} \right) = 2.39 \text{ kN} * \text{m}$$

$$T_u = 2.39 \text{ m} \leq 2.464 \text{ m}$$

Therefore, torsional reinforcement can be neglected in this case.

3.6.2.4. Reinforcement length

In order to make sure that top reinforcement length extends so that it can resist moment changes and make more economical design, the cutting length is calculated using the following formula from ACI 318 Section 9.7.3.8.4:

$$l_{top} = x_{inflection} + \text{MAX}(d; 12d_b; l_n/16)$$

The point of inflection for the edge beam from SAP2000 is 1680mm.

$$l_{top} = 680 + \text{MAX}(550; 12 * 28.65; 6700/16) = 1230 \text{ mm}$$

3.6.2.5. Detailed final beam reinforcement design

Longitudinal and reinforcement design hand and SAP2000 calculations are almost the same. Therefore, SAP2000 results are verified and can be used for other reinforcement designs. The beam reinforcement is designed for three sections of the beam and presented in the tables below.

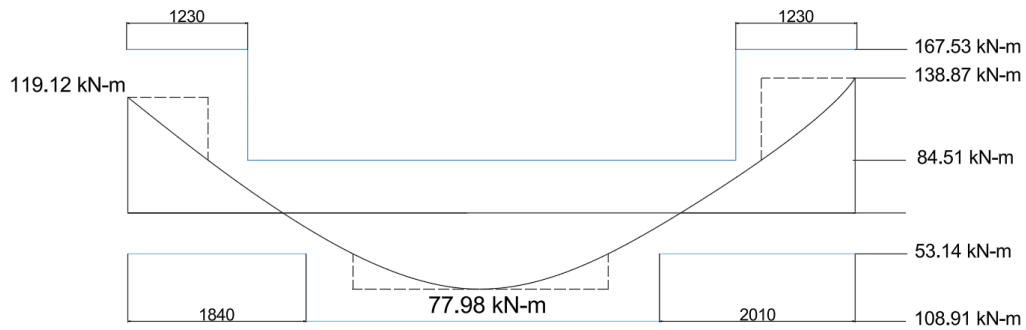


Figure 3.6.2.2. Moment demand vs strength for the edge beam

Table 3.6.2.2. Beam reinforcement design for edge beam of Floor 8

	End	Mid	End
Top reinforcement bars	6#9	2#8	2#8
Area, in²	6	1.57	1.57
Bottom reinforcement bars	4#8	4#8	4#8
Area, in²	3.14	3.14	3.14
Reinforcement ratio, %	3.28	1.69	1.69
Shear reinforcement	#3@150	#3@275	#3@150

3.6.3. Column design

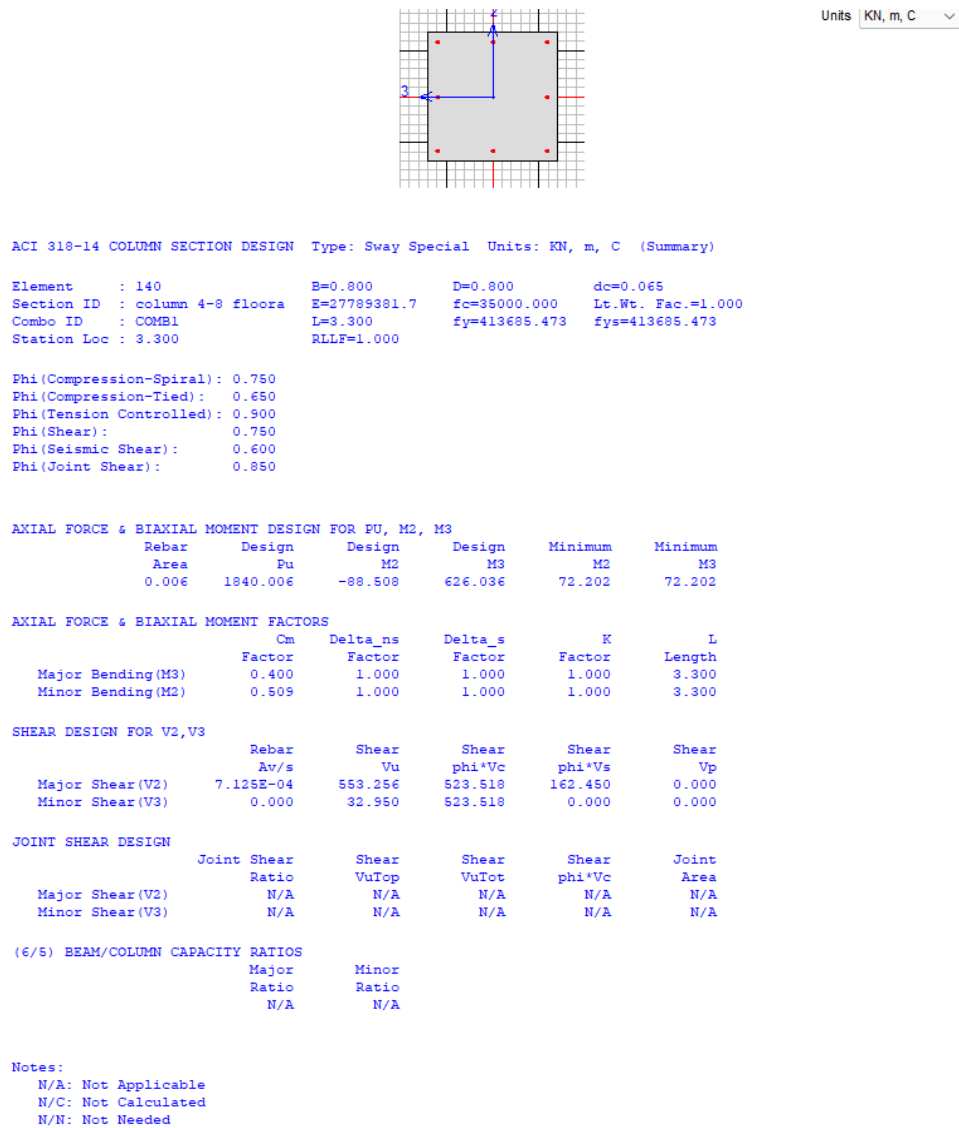


Figure 3.6.3.1. Column reinforcement design in SAP2000

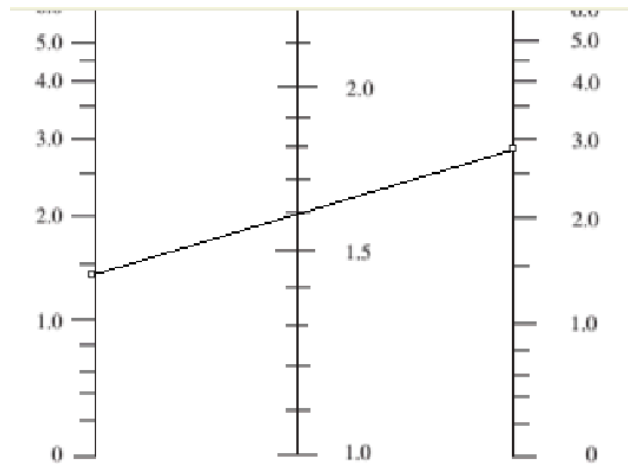
3.6.3.1. Slenderness check for columns

$$I_c = 0.7 * \frac{0.8*0.8^3}{12} = 0.0024m^4$$

$$I_b = 0.35 * \frac{0.3*0.6^3}{12} = 0.00189m^4$$

$$\psi = \frac{\Sigma(EI/l) \text{ of compression members}}{\Sigma(EI/l) \text{ of flexural members}} = \frac{0.0024/3.3}{(0.00189/7.5)+(0.00189/7.5)} = 1.443$$

$$\psi = \frac{\Sigma(EI/l) \text{ of compression members}}{\Sigma(EI/l) \text{ of flexural members}} = \frac{2*(0.0024/3.3)}{(0.00189/7.5)+(0.00189/7.5)} = 2.886$$



(b) Unbraced frames

3.6.3.2. Value of effective length factor of column in unbraced frame

$$l_u = 3.3 - (0.3 + 0.3) = 2.7m$$

$$\frac{kl_u}{r} = \frac{1.6*2.7}{0.3*0.8} = 18 < 22 \text{ Not Slender Column}$$

$$\gamma = \frac{0.523}{0.65} = \frac{0.7}{0.8} = 0.875$$

$$P_n = \frac{P_u}{\phi} = \frac{1840}{0.65} = 2830.77 \text{ kN}$$

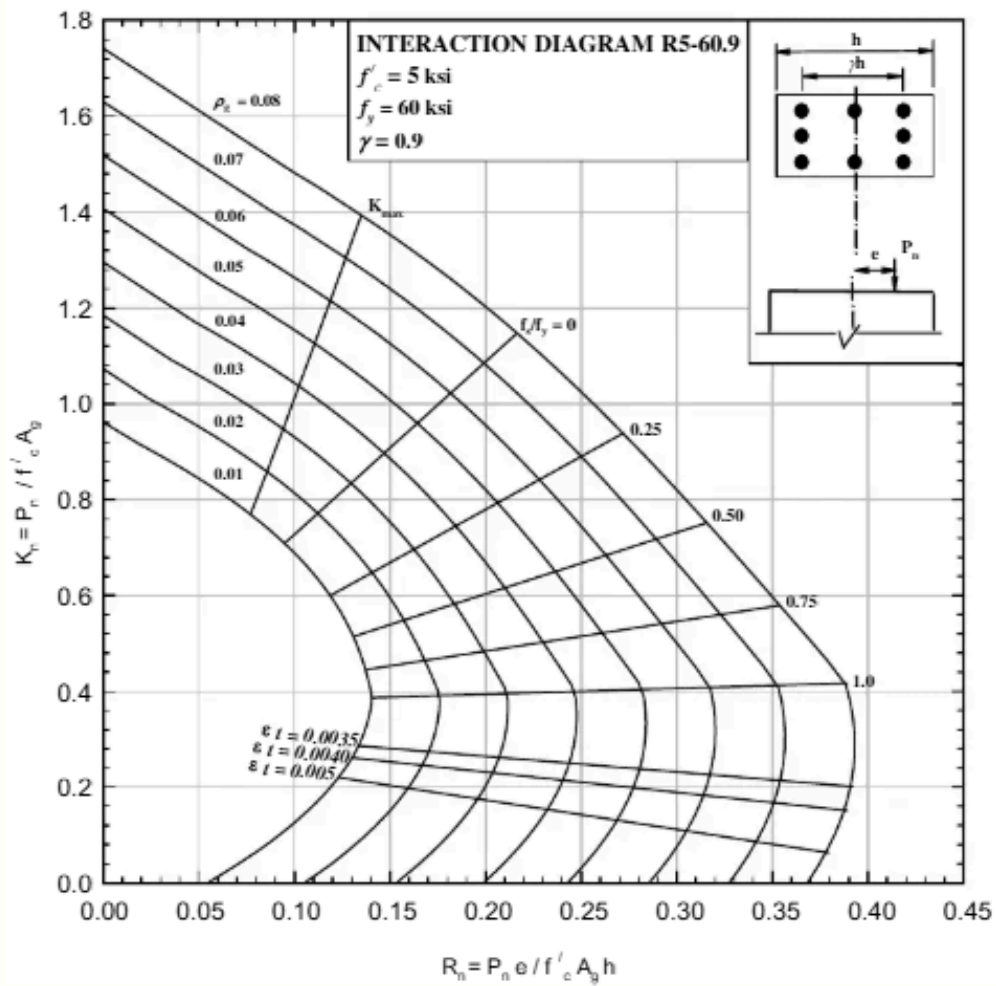
$$M_n = \frac{M_u}{\phi} = \frac{626.03}{0.65} = 963.12 \text{ kN}$$

$$e = \frac{M_n}{P_n} = 0.34 \text{ m}$$

$$K_n = \frac{P_n}{f'_c A_g} = 0.252$$

$$R_n = \frac{P_n}{f'_c A_g} \frac{e}{h} = 0.136$$

COLUMNS 3.3.4 - Nominal load-moment strength interaction diagram, R5-60.9



3.6.3.3. Column interaction diagram

From Linear Interpolation:

$$P_g = 0.01$$

$$A_s = 0.01 * 0.8 * 0.8 = 6.4 * 10^{-3} m^2 (10\#9 \text{ bars})$$

3.6.3.2. Biaxial bending check

Table 3.6.3.1. Moment and loads applied to columns from SAP 2000

Floor	Dimension	P_n , kip	M_{nx}	M_{ny}
4-8	31.5x31.5	413.65	5540.885	783.365
1-3	37.4x37.4	843.268	7711.101	160.773
Basement	41.34x41.34	0.000	3940.955	5.246

$$\gamma_x = \gamma_y = 0.875$$

$$e_x = \frac{M_{ny}}{P_n} = 1.894$$

$$e_y = \frac{M_{nx}}{P_n} = 13.395$$

$$\frac{e_x}{h} = \frac{1.894}{31.5} = 0.06$$

$$\frac{e_y}{h} = \frac{13.395}{31.5} = 0.425$$

$$\frac{\phi P_{nx}}{bh} = 2.95$$

$$P_{nx} = \frac{3.5bh}{\phi} = 4503.29 \text{ psi}$$

$$\frac{\phi P_{ny}}{bh} = 1.3$$

$$P_{nx} = \frac{1.3bh}{\phi} = 1984.5 \text{ psi}$$

$$P_{n0} = 0.85 * 5 * 31.5^2 + 10.0(60 - 0.85 * 5) = 4774.56 \text{ psi}$$

$$\frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_{n0}} = \frac{1}{3526.1} + \frac{1}{1813.43} - \frac{1}{4774.56} \Rightarrow P_n = 1936.03 \text{ psi}$$

$$P_n = 1936.03 \text{ psi} \geq 0.1P_{n0} = 477.456 \text{ psi}$$

$$P_u = 368.2 \text{ psi}$$

$$\phi P_n = 1936.03 * 0.65 = 1258.42 \geq P_u = 368.2 \Rightarrow \text{SAFE}$$

3.6.2.3. Shear reinforcement design.

$$V_{u1} = 625.525 \text{ kN}$$

$$V_{u2} = 38.171 \text{ kN}$$

$$\phi V_c = 0.75 * 0.17 * (1 + \frac{1444.79}{14 * 850^2}) * 1.0 * \sqrt{35000} * 850 * 850 = 17236 \text{ kN}$$

$$V_u < 0.5\phi V_c, \text{ so minimum reinforcement is required}$$

$$16d_b = 16 * 0.02865 = 0.4584 \text{ m}$$

$$48d_{bt} = 48 * 0.009525 = 0.4572 \text{ m}$$

$$\text{least column dimension} = 0.8 \text{ m}$$

Lesser of three conditions are chosen, therefore #4 ties with 450 mm spacing are selected for columns.

Table 3.6.3.2. Detailed reinforcement design of columns

	Basement	Floor 1-3	Floor 4-8
Top reinforcement bars	10#9	14#10	14#10
Area, in²	10	17.78	17.78
Reinforcement ratio, %	1.01	1.27	1.04
Shear reinforcement	#4@450	#4@450	#4@450

3.6.4. One-way slab design

3.6.4.1. Calculation of main reinforcement

One-way slab design for slabs for the first three floors are shown below. Minimum thickness are calculated following the formula:

$$d = \frac{L}{24} \left(0.4 + \frac{f_y}{100000} \right) = 359.375 \text{ mm} \Rightarrow 375 \text{ mm will be taken}$$

Moment from Load Combination is 13157 k*in. The new moment is calculated according to moment coefficients.

$$R_u = \frac{M_u}{bd^2}$$

$$\rho = \frac{0.85f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2R_u}{0.85f'_c}} \right]$$

$$A_s = \rho bd$$

$$S = 12A_b/A_s$$

Table 3.6.4.1. One-way slab reinforcement design

	Beam 1			Beam 2		
	End	Mid	End	End	Mid	End
Moment coefficient	$-\frac{1}{16}$	$+\frac{1}{14}$	$-\frac{1}{9}$	$-\frac{1}{9}$	$+\frac{1}{14}$	$-\frac{1}{16}$
Mu, k*in	822.3615	939.47	1461.83	1461.83	939.47	822.36
Ru, psi	575.25	657.16	1022.56	1022.56	657.16	575.25
Steel percentage	0.013	0.015	0.0259	0.0259	0.015	0.013
As, in^2	1.51	1.74	3.01	3.01	1.74	1.51
Selected Bars	#11	#14	#18	#18	#14	#11
New As, in^2	1.56	2.25	4.00	4.00	2.25	1.56
Spacing, mm	300	400	400	400	400	300

3.6.4.2. Calculation of temperature and shrinkage reinforcement

Reinforcement percentage must be taken as $\rho = 0.18\%$ of the gross area.

$$A_s = 0.0018bd = 0.209 \text{ in}^2 \Rightarrow \#4 \text{ bar size is selected}(0.2 \text{ in}^2)$$

$$S = 12A_b/A_s = 11.48 \text{ in} = 291.7 \text{ mm} \Rightarrow 300 \text{ mm will be taken as spacing between shrinkage}$$

3.6.4.3. Shear strength check

$$V_u = \frac{1.15\omega L}{2} = 14.58 \text{ k/ft}$$

$$\lambda_s = \sqrt{\frac{2}{(1+\frac{d}{10})}} = 0.899$$

$$\phi V_c = \phi \left[8\lambda_s \lambda_p^{1/3} \sqrt{f_c} + \frac{N_u}{6A_g} \right] b_u d = 0.75 \left(\frac{(8*0.899*1*0.009^{1/3} \sqrt{6000} + 0) 14.76 * 65.62}{1000} \right) = 84.18 \text{ kips} > V_u$$

No shear reinforcement is needed

3.6.5. Two-way slab design

3.6.5.1. Interior panel reinforcement design:

$$M_{0l} = M_{0s} = \frac{0.256 \text{ ksf} * 24.61 * (24.61 - \frac{25.59}{12})^2}{8} = 397.89 \text{ ft} - k$$

$$M_n = 0.65 * M_0 = 0.65 * 397.89 = 258.63 \text{ ft} - k$$

$$M_p = 0.35 * M_0 = 0.35 * 397.89 = 139.26 \text{ ft} - k$$

$$\text{Area of flange} = 0.2 * 1 = 0.2 \text{ m}$$

$$\text{Area of web} = 0.375 * 0.75 = 0.28 \text{ m}$$

$$0.2 * 0.1 + 0.28 * (0.2 + 0.375) = 0.48$$

$$\bar{y} = 0.377 \text{ m}$$

$$I_b = \left[\frac{1}{12} * 1 * 0.2^3 + 0.2 * 0.277^2 \right] + \left[\frac{1}{12} * 0.375 * 0.75^3 + 0.28 * 0.198^2 \right] = 0.04 \text{ m}^4$$

$$I_s = \left[\frac{1}{12} * 7.5 * 0.2^3 \right] = 0.005 \text{ m}^4$$

$$\alpha_{fm} = 8 > 2$$

$$h = \frac{l_n (0.8 + \frac{f_y}{200000})}{36 + 9\beta} = \frac{7.5 (0.8 + \frac{75000}{200000})}{36 + 9} = 0.196 \text{ m}$$

Since $\frac{l_2}{l_1} = 1$, distribution of negative moment:

$$\text{Column strip} = 0.75 M_n = 258.63 * 0.75 = -193.97 \text{ ft} - k$$

$$\text{Middle strip} = 0.25 M_n = 258.63 * 0.25 = -64.66 \text{ ft} - k$$

According to ACI 314-14 code, if $\alpha_{f1} \frac{l_2}{l_1} \geq 1$, then 85% of moment is carried by beam

$$\text{Column strip} = 0.15M_n = 193.97 * 0.15 = -29.1 \text{ ft} - k$$

Since $\frac{l_2}{l_1} = 1$, distribution of positive moment:

$$\text{Column strip} = 0.75M_n = 139.26 * 0.75 = 104.445 \text{ ft} - k$$

$$\text{Middle strip} = 0.25M_n = 193.26 * 0.25 = 34.815 \text{ ft} - k$$

According to ACI 314-14 code, if $\alpha \frac{l_2}{l_1} \geq 1$, then 85% of moment is carried by beam

$$\text{Column strip} = 0.15M_n = 104.445 * 0.15 = 15.67 \text{ kN} * m$$

$$\rho = \frac{0.85f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_u}{\phi 0.85f'_c}} \right)$$

Table 3.6.5.1. Interior panel reinforcement design

	Column Strip		Middle Strip	
Moment, ft-k	-29.1	15.67	-64.66	34.815
Moment, kN*m	-39.46	21.25	-87.68	47.21
b, m	3.75	3.75	3.75	3.75
d, m	0.175	0.175	0.175	0.175
$R_u = \frac{M_u}{bd^2}$	-343.60	185.03	-763.47	411.08
ρ	0.000738	0.000397	0.00165	0.000884
A_s, m^2	0.0004843125	0.00026053125	0.0010828125	0.000580125
$A_{s,min}, m^2$	0.00118125	0.00118125	0.00118125	0.00118125
$A_{s,min}, in^2$	1.8309375	1.8309375	1.8309375	1.8309375
Selected bars	6#5	6#5	6#5	6#5
A_s, in^2	1.84	1.84	1.84	1.84
Spacing, mm	350	350	350	350

3.6.5.2. Exterior panel reinforcement design:

$$\text{Area of flange} = 0.2 * 0.6875 = 0.1375m$$

$$\text{Area of web} = 0.375 * 0.75 = 0.28 m$$

$$0.1375 * 0.1 + 0.28 * (0.2 + 0.375)0.175 = 0.4175\bar{y}$$

$$\bar{y} = 0.419 m$$

$$I_b(\text{edge beam}) = \left[\frac{1}{12} * 0.6875 * 0.2^3 + 0.1375 * 0.2175^2 \right]$$

$$+ \left[\frac{1}{12} * 0.375 * 0.75^3 + 0.28 * 0.1575^2 \right] = 0.02$$

$$I_s(\text{edge slab}) = \left[\frac{1}{12} * 3.75 * 0.2^3 \right] = 0.0025m^4$$

$$\alpha_{fm} = 10.8 > 2$$

$$h = \frac{l_n(0.8 + \frac{f_y}{200000})}{36 + 9\beta} = \frac{7.5(0.8 + \frac{75000}{200000})}{36 + 9} = 0.196m$$

Since $\frac{l_2}{l_1} = 1$, distribution of negative moment:

$$\text{Exterior negative factored moment} = 0.16 * 397.89 = 63.66 ft - k$$

$$\text{Positive moment within span} = 0.57 * 397.89 = 226.8 ft - k$$

$$\text{Interior negative factored moment} = 0.7 * 397.89 = 278.5 ft - k$$

$$C = (1 - 0.63 \frac{0.375}{0.95}) (\frac{0.95 * 0.375^3}{3}) + (1 - 0.63 \frac{0.2}{0.75}) (\frac{0.75 * 0.2^3}{3}) = 0.0139m^4$$

$$\beta_t = \frac{E_b C}{2E_b I_s} = \frac{0.0139}{2 * 0.005} = 1.39$$

Distribution of negative moment:

$$\text{Column strip} = 0.75M_n = 278.5 * 0.75 = -208.9 ft - k$$

$$\text{Middle strip} = 0.25M_n = 278.5 * 0.25 = -69.6 ft - k$$

According to ACI 314-14 code, if $\alpha_{f1} \frac{l_2}{l_1} \geq 1$, then 85% of moment is carried by beam

$$\text{Column strip} = 0.15M_n = 208.9 * 0.15 = -31.3 ft - k$$

Distribution of positive moment:

$$\text{Column strip} = 0.15(0.75 * M_n) = 226.8 * 0.75 * 0.15 = 25.52 ft - k$$

$$\text{Middle strip} = 0.25M_n = 226.8 * 0.25 = 56.7 ft - k$$

Distribution of exterior negative moment:

$$\text{Column strip} = 0.15(0.862 * M_n) = 63.66 * 0.862 * 0.15 = -8.23 \text{ ft} - k$$

$$\text{Middle strip} = 0.138M_n = 63.66 * 0.138 = -8.78 \text{ ft} - k$$

Table 3.6.5.2. Exterior panel reinforcement design

	Column Strip			Middle strip		
	Negative	Positive	Negative	Negative	Positive	Negative
Moment, ft-k	8.23	25.52	31.3	8.78	56.7	69.6
Moment, kN*m	11.16	34.61	42.44	11.9	76.89	94.38
b, m	2.5	2.5	2.5	2.5	2.5	2.5
d, m	0.175	0.175	0.175	0.175	0.175	0.175
$R_u = \frac{M_u}{bd^2}$	145.76	452.05	554.32	155.43	1004.28	1232.72
ρ	0.000312	0.000973	0.001195	0.000333	0.002181	0.002687
$A_{s, req}, m^2$	0.000137	0.000426	0.000523	0.000146	0.000954	0.001176
$A_{s, min}, m^2$	0.000788	0.000788	0.000788	0.000788	0.000788	0.000788
$A_{s, min}, in^2$	1.2206250	1.2206250	1.2206250	1.2206250	1.2206250	1.2206250
$A_{s, selected}, in^2$	1.2206250	1.2206250	1.2206250	1.2206250	1.4790347	1.8224021
Selected bars	4#5	4#5	4#5	4#5	5#5	6#5
$A_{s, new}, in^2$	1.23	1.23	1.23	1.23	1.53	1.84

Shear strength check:

$$d = 6.9 - 0.75 - 0.625 = 5.525 \text{ in} = 140.3 \text{ mm}$$

$$V_u = q_u * [l_1 - d - h]$$

$$V_u = 12.24 * [2.5 - 0.4 - 0.1403] = 23.99 \text{ kN}$$

$$\phi V_c = 0.75 * 1.0 * 2\sqrt{35 * 10^3} * 2.5 * 0.1403 = 98.43 \text{ kN} > V_u \Rightarrow OK$$

No shear reinforcement is needed.

3.6.6. Joint detailing

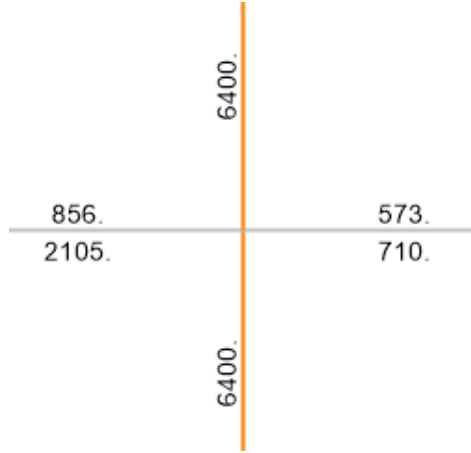


Figure 3.6.6.1. Reinforcement areas of beam and column between Floor 7 and 8

$$V_{u,joint} = T_{pr1} + C_{pr2} - V_{col}$$

$$T_{pr1} = \alpha A_s f_y = 1.25 * 573 * 0.413 = 295.81 kN$$

$$C_{pr2} = \alpha A_s f_y = 1.25 * 2105 * 0.413 = 1086.71 kN$$

$$V_{col} = 264.21 kN$$

$$V_{u,joint} = 295.81 + 1086.71 - 264.21 = 1118.31 kN$$

$$b_b + b_{col} = 300 + 800 = 1100 mm$$

$$h_b + 2x = 300 + 2 * 150 = 600 mm$$

$$V_n = \gamma \sqrt{f'_c} * A_j = 1.7 * \sqrt{34.47} * 600 * 800 = 4790.83 kN$$

$$\phi V_n \geq V_{u,joint}$$

$$0.85 * 4790.83 = 4072.21 kN \geq 1118.31 kN$$

$$A_v = \max\left(\frac{0.062 \sqrt{f'_c} b_c s}{f_{yt}}, \frac{0.35 b_c s}{f_{yt}}\right)$$

$$s = \min\left(\frac{A_v f_{yt}}{0.062 \sqrt{f'_c} b_c}, \frac{A_v f_{yt}}{0.35 b_c}\right)$$

$$s = \min\left(\frac{142 * 420}{0.062 \sqrt{34.47} * 800}, \frac{142 * 420}{0.35 * 800}\right) = \min\left(\frac{142 * 420}{0.062 \sqrt{34.47} * 800}, \frac{142 * 420}{0.35 * 800}\right)$$

$$s = \min(204.8; 213) = 204.8 mm$$

2 #3 ties with spacing of 200 mm will be used

$$\Sigma M_{nc} \geq \frac{6}{5} \Sigma M_{nb}$$

$$527.553 + 367.783 = 895.336 kN - m \geq \frac{6}{5} (329.25 + 188.308) = 621.07 kN - m$$

The moment capacity of the column is greater than the moment capacity of the beam by 20 percent.

3.6.7. Lap splices

According to ACI 318, lap splices for beams in tension were calculated using the following formula:

$$l_{st} = \text{MAX}(1.3l_d; 12in) = 855.4 \text{ mm}$$

Calculation for compression bars and columns were calculated using the following formula:

$$l_{sc} = 0.0005f_y d_b = 0.0005f_y d_b$$

Table 3.6.7.1. Lap splices design of columns

Member	Lap splice, mm
Column	
Basement - Floor 3	1210
Floor 4	968
Floor 5-8	762
Beams in tension	
Basement - Floor 3	1023
Floor 4-8	855
Beams in compression	
Basement - Floor 3	849
Floor 4-8	586

3.6.8. Development length

Development length for tension bars:

$$\frac{l_d}{d_b} = \frac{1}{1.1} * \frac{f_y}{\lambda \sqrt{f'_c}} * \frac{\psi_t \psi_e \psi_s}{(\frac{c_b + K_{tr}}{d_b})} = \frac{1}{1.1} * \frac{420}{1.0 \sqrt{35}} * \frac{1.3}{1.54} = 54.28 \text{ diameters} = 1561 \text{ mm}$$

$$\psi_s = 1.0$$

$$\psi_t = 1.3$$

$$\psi_e = 1.0$$

$$K_{tr} = \frac{40 A_n}{s n} = \frac{40 * 1.42 * 10^{-4}}{6 * 0.05} = 0.019$$

$$\frac{c_b + K_{tr}}{d_b} = \frac{0.025 + 0.019}{0.02865} = 1.54 < 2.5$$

$$d_b = 28.65 \text{ mm}$$

$$l_d = \frac{f_y \psi_t \psi_e}{20 \lambda \sqrt{f'_c}} \times d_b$$

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{4 \lambda \sqrt{f'_c}} \right) * d_b = 658 \text{ mm}$$

$$D = 6d_b = 171 \text{ mm}$$

$$r = D/2 = 85.5 \text{ mm}$$

$$\text{Hook distance} = 12d_b = 342 \text{ mm}$$

Development length for compression bars:

$$l_{dc} = \left(\frac{f_y \psi_r}{4 \lambda \sqrt{f'_c}} \right) * d_b \geq 0.043 f_y \psi_r d_b \Rightarrow l_{dc} = 450.8 \text{ mm}$$

3.6.9. Structural serviceability design

3.6.9.1. Minimum deflection

According to ACI 318-14, minimum thickness of structural member is required to be checked to pass serviceability design requirements. If calculated values are greater than the depth of the beam must be calculated, in other cases, it might be ignored. For Grade 75 reinforcement, the following formula is applicable:

$$h_b = \frac{L}{21} = \frac{7500}{21} = 357.14 \text{ mm} < \text{depth of beam} = 600 \text{ mm} \Rightarrow \text{ok}$$

$$h_b = \frac{L}{21} = \frac{5000}{21} = 238.09 \text{ mm} < \text{depth of beam} = 600 \text{ mm} \Rightarrow \text{ok}$$

h_b is less than the depth of the beam, therefore deflection calculations are not required.

3.6.9.2. Crack width

Crack control is necessary to ensure that structure is safe. Crack width of the structural members must be in the allowable range specified in ACI 318-14. The estimated crack width of the structure can be calculated using the following formula:

$$w = 0.076 \beta_h f_s \sqrt[3]{d_c^* A} = 0.0121 \text{ in} < 0.016 \text{ in} \Rightarrow OK$$

The estimated crack width does not exceed the allowable crack limit of 0.016 in. Spacing for longitudinal reinforcement should not be greater than the calculated s value.

$$s = (15(\frac{40}{f_s}) - 2.5C_c) = 8.91 \text{ in} = 226.314 \text{ mm}$$

4. Geotechnical Engineering

4.1 Site description

As mentioned above, the site was selected in Humble, Texas, on 6819 Manor Way st. to build an 8-storey hotel. Currently, there are no existing buildings or constructions that would need to be removed, so the site is unoccupied and suitable for construction. You can see the site location in Figure 4.1.1.

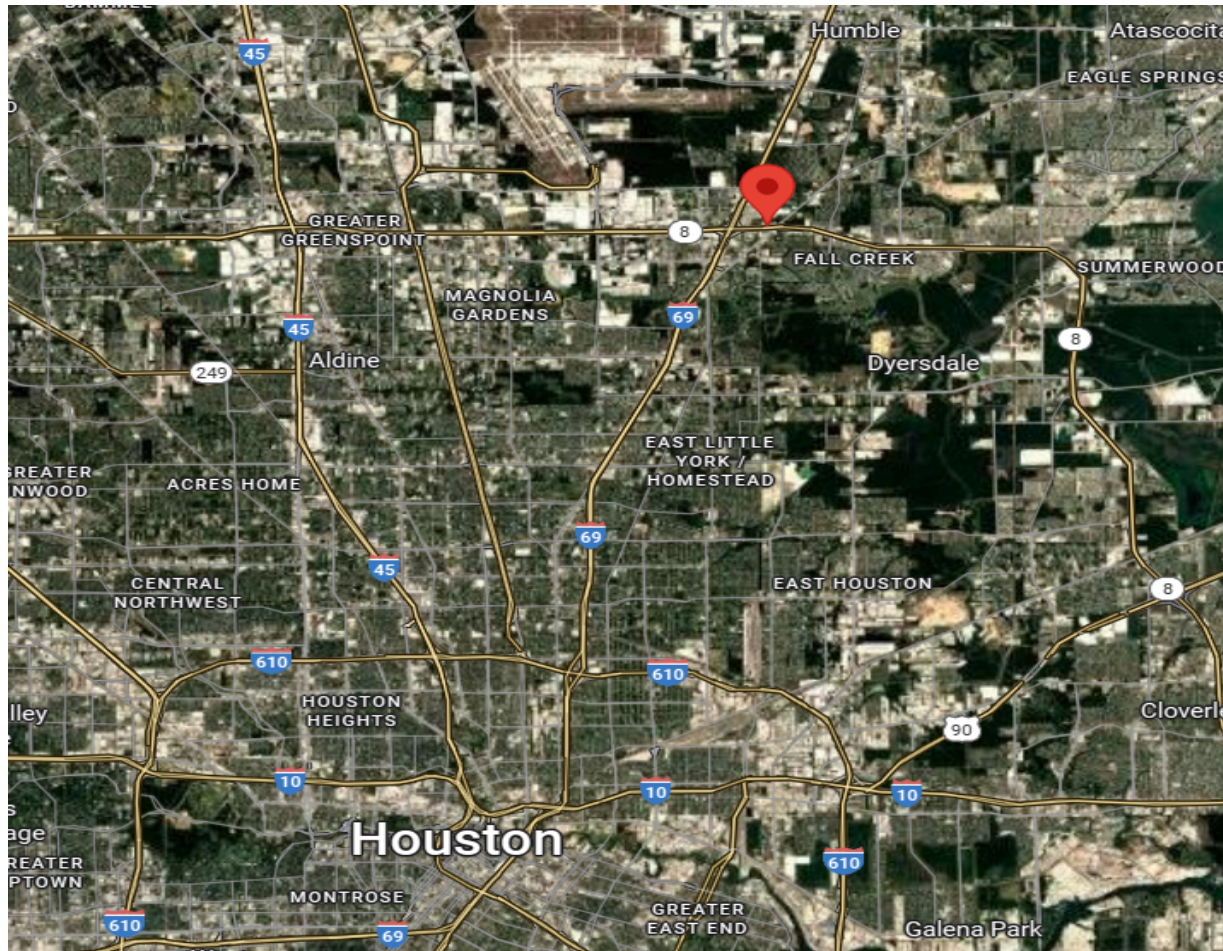


Figure 4.1.1 Site location

The nearest borehole log report was found at a distance of approximately 30 meters from the site, and there are also 8 additional boreholes ranging at a distance of 100 to 300 meters from the site. All of them share relatively the same soil lithology, which primarily consists of deep clay layers. The Texas Water Development Board was responsible for the borings, with the closest one of them being drilled up to 60 meters. The location of boreholes can be observed in

Figure 4.1.2. The red line trapezoid represents the territorial borders of the site, while the dots in orange () represent the borehole on the map.



Figure 4.1.2. Borehole Location.

4.2 Field Investigation

For the 6819 Manor Way site in Humble, Texas, the subsurface characterization and geotechnical data were obtained directly from nearby well logs and geotechnical investigation reports located within a 200–300 meter radius of the project site. The primary sources included detailed borings and test data archived in the Texas Water Development Board (TWDB) database (Texas Department of Licensing and Regulation, 2009) and additional engineering reports available for adjacent parcels by Gorrondona & Associates, Inc.(2014) and HVJ ASSOCIATES, INC. (2023).

Six boreholes with depths extending up to approximately 60 meters were used to establish the stratigraphic profile. Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586, and undisturbed samples were collected following ASTM D1587 for laboratory analysis. The laboratory testing included natural water content (ASTM D2216), Atterberg limits (ASTM D4318), and unit weight determination. Consolidation test results (ASTM D2435) from these reports were used to evaluate compressibility characteristics, calculate settlement parameters, and define consolidation curves for the soft and medium clays.

Seismicity

Regional seismicity in Houston is extremely low, placing the project in Seismic Design Category A. The design spectral accelerations remain small (SDs on the order of 0.10 g and SD1 around 0.05 g), and local peak ground acceleration is roughly 0.04-0.08 g. Wind loading, therefore, controls structural lateral design rather than seismic effects. Because the groundwater table is far below the sand layers, the site is not susceptible to liquefaction.

4.3 Soil Profile

Table 4.3.1 Main Soil Profile

Top depth (m)	Base depth (m)	Height (m)	Soil Type	USC S	Nspt	γ , kPa	c_u , kPa	ϕ'	σ' , kPa	E_s , Mpa	μ_s
0 - 2	2	2	Silty CLAY (fill/very soft)	CL	3	18.5	18	0	18.5	9	0.47
2 - 8	8	6	Very soft to soft CLAY	CL/CH	2	18.8	20	0	93.4	10	0.47
8 - 16	16	8	Soft CLAY	CL-CH	3	19	25	0	225.8	12.5	0.45
16 - 24	24	8	Soft-medium CLAY	CL	5	19.2	35	0	378.6	17.5	0.44
24 - 33.5	33.5	9.5	Medium CLAY	CL	6	19.2	45	0	546.6	22.5	0.44
33.5 - 38	38	4.5	Well-graded SAND with gravel	SW-SM	15	20	0	33	682.8	14.35	0.3
38 - 42	42	4	Silty SAND	SM	18	19.	0	33	767.4	16.25	0.32

						8					
42 - 45.7	45.7	3.7	Well-graded SAND & gravel	SW-SM	25	20.5	0	34	844.93	21.5	0.3
45.7 - 52	52	6.3	Medium-stiff CLAY	CL	8	19.5	60	0	958.5	30	0.4
52 - 60	60	8	Stiff CLAY	CL	12	19.5	80	0	1027.78	40	0.36

Table 4.3.2 Consolidation data

Layer (Top–Base, m)	USCS	σ'_0	C_c	C_r	e_0	OCR
0.0–2.0	Silty CLAY (fill/very soft)	18.5	0.4	0.08	1.05	1.1
2.0–8.0	Very soft to soft CLAY	93.4	0.4	0.08	1.05	1.1
8.0–16.0	Soft CLAY	225.8	0.35	0.07	1	1.1
16.0–24.0	Soft-medium CLAY	378.6	0.3	0.06	0.95	1.1
24.0–33.5	Medium CLAY	546.6	0.28	0.056	0.9	1.1
45.7–52.0	Medium-stiff CLAY	958.5	0.28	0.056	0.9	1.1
52.0–60.0	Stiff CLAY	1027.78	0.2	0.04	0.75	1.1

The soil borings revealed a typical Gulf Coastal Plain stratigraphy dominated by soft to medium plastic clays interbedded with thin layers of silty sands and gravels. The ground surface generally consists of a thin cover of silty clay and natural topsoil extending to depths of about 2

meters. Beneath this, very soft to soft clay layers were encountered between approximately 2 and 16 meters, followed by soft to medium clays extending down to about 24 meters. From about 24 to 33.5 meters, medium clays with higher strength and lower compressibility were observed, representing a transitional zone between the soft upper deposits and the dense sand layers below. Between approximately 33.5 and 45.7 meters, well-graded sands and silty sands with gravel inclusions were encountered. These layers are dense, freely draining, and contribute significant bearing capacity and stability to the profile. Below this, from about 45.7 meters to the termination depth of 60 meters, medium-stiff to stiff clay was observed, representing the strongest cohesive unit in the boreholes.

Groundwater was encountered at a depth of approximately 50.3 meters below ground surface. This deep groundwater level classifies the site as having a “deep” water table according to the definition by Hare et al. (2021), where groundwater below 5-6 meters is considered deep. The depth is consistent with regional hydrogeologic data for the Humble area, and minor fluctuations may occur seasonally depending on precipitation and surface runoff conditions.

Index properties, including natural water content, Atterberg limits, and unit weights, were obtained from the referenced reports. Strength parameters were estimated using a $\phi = 0$ total-stress approach for clays and drained friction angles for sands. Clay cohesion values were derived from unconfined compression tests or empirical correlations and fall in the approximate range of 18–50 kPa, depending on depth. The dense sand and gravel layers were assigned a drained friction angle of about 33–34 degrees, consistent with their $(N_1)_{60}$ values and standard SPT-based correlations. Elastic moduli were derived using common empirical relationships where E_s for clays is taken as roughly 500 times the undrained shear strength, giving values from about 8–10 MPa near surface to 25–40 MPa at depth. For sands, E_s was estimated from correlations of the form $E_s = \alpha(N_1)_{60}$, yielding values in the 20–30 MPa range for the deeper dense layer.

4.4 Shallow Foundation

A shallow foundation is a type of foundation that transfers loads from the structure to the soil very close to the surface. It is typically used when competent soil with sufficient bearing capacity exists at a shallow depth. Common forms include spread footings, strip footings, and mat (raft) foundations.

4.4.1 Bearing Capacity for Shallow Foundations

To determine the bearing capacity of the soil supporting a foundation, two commonly accepted methods of analysis in international practice have been chosen, which are the Meyerhof method and the Hansen method. These two techniques are both developments of the classical Terzaghi bearing-capacity theory, and as such, both take an interest in foundation shape, depth, inclination of the load to the foundation, and base slope. Their use allows a reliable and conservative assessment of bearing capacity to be used during design.

Meyerhof formula:

$$q_{ult} = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + 0.5 \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (1)$$

c' - cohesion

q - effective stress at the level of the bottom of the foundation

γ - unit weight of soil

B - width of foundation

$F_{cs} F_{qs} F_{\gamma s}$ - shape factors

$F_{cd} F_{qd} F_{\gamma d}$ - depth factors

$F_{ci} F_{qi} F_{\gamma i}$ - load inclination factors

$N_c N_q N_\gamma$ - bearing capacity factors

This formula combines three elements of bearing capacity: bearing capacity caused by adhesion, bearing capacity caused by the weight of the soil over the footing, and bearing capacity caused by the weight of the soil below the footing.

Hansen formula:

Here is a simplified version of the general formula used by Hansen to the circumstances of our project, which includes clay with $\phi=0$, vertical load, and horizontal base.

$$q_{ult} = 5.14 c' (1 + s_c^a + d_c^a - i_c^a - b_c^a - g_c^a) + q \quad (2)$$

s_c^a - shape factor

d_c^a - depth factor

i_c^a - inclination factor

b_c^a - slope factor

g_c^a - tilted Base Factor

The Hansen method is more general and offers more detailed correction factors for different conditions.

Calculation Results

Table 4.4.1.1 Calculated Ultimate Bearing Capacity vs. Footing Base Dimensions by Meyerhof method

Base Dimension B x L (m)	Depth, d _f (m)	1.5	3	4.5
2 x 2		187.84 kN/m ²	227.1 kN/m ²	308.95 kN/m ²
3 x 3		175.6 kN/m ²	228.3 kN/m ²	298.58 kN/m ²
1.5 x 3		186 kN/m ²	219.1 kN/m ²	296.08 kN/m ²

Table 4.4.1.2 Calculated Ultimate Bearing Capacity vs. Footing Base Dimensions by Hansen method.

Base Dimension B x L (m)	Depth, d_f (m)	1.5	3	4.5
2 x 2		182.4 kN/m ²	220.2 kN/m ²	298.08 kN/m ²
3 x 3		172.13 kN/m ²	220.8 kN/m ²	289.45 kN/m ²
1.5 x 3		182.4 kN/m ²	215.01 kN/m ²	290.2 kN/m ²

Interpretation of the results

Table 4.4.1.1 and Table 4.4.1.2 illustrate the findings that the bearing capacity between the Meyerhof and Hansen methods prove to have a great level of agreement. In this regard therefore, it can be stated that the calculations are accurate and that the data that is obtained can be confidently used.

Effect of foundation depth: the bearing capacity is directly proportional to the depth of the foundation. For example, for a 2x2 m foundation:

- When $d_f = 1.5$ m, $q_{ult} \approx 187.84$ kN/m²
- When $d_f = 4.5$ m, $q_{ult} \approx 308.95$ kN/m²

This happens because of the added pressure of the weight of the soil above the footings, which creates more retaining pressure. When there is $\phi=0$ in the clay, an increase in the width of the foundation that is to be enlarged does not significantly increase the bearing capacity since the

term the $0.5\gamma BN_{\gamma}$ becomes zero. The key input is contributed by the adhesion and embedment depth.

Allowable net bearing capacity formula:

$$q_{all} = (q_{ult} - q)/FS \quad (3)$$

q_{ult} - Ultimate Bearing Capacity

q - effective stress at the level of the bottom of the foundation

FS - Factor of safety

Table 4.4.1.3 Calculated Allowable Net Bearing Capacity vs. Footing Base Dimensions by Meyerhof method

B x L (m) / Depth, d_f (m)	1.5	3	4.5
2x2	53.21 kN/m ²	56.9 kN/m ²	74.75 kN/m ²
3x3	49.13 kN/m ²	57.3 kN/m ²	71.29 kN/m ²
1.5x3	52.6 kN/m ²	54.23 kN/m ²	70.46 kN/m ²

Table 4.4.1.4 Calculated Allowable Net Bearing Capacity vs. Footing Base Dimensions by the Hansen method

B x L (m) / Depth, d_f (m)	1.5	3	4.5
2x2	51.4 kN/m ²	54.6 kN/m ²	71.13 kN/m ²
3x3	47.98 kN/m ²	54.8 kN/m ²	68.25 kN/m ²
1.5x3	51.4	52.87	68.5

	kN/m ²	kN/m ²	kN/m ²
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The net ultimate bearing capacity of the mat foundation was determined using this formula:

$$q_{ult} = 5.14c_u(1 + 0.195 * B/L)(1 + 0.4 * D_f/B) \quad (4)$$

$$q_{ult} = 5.14 * 20 * (1 + 0.195 * 40/60)(1 + 0.4 * 6/40) = 123.13$$

The net allowable bearing capacity of the mat foundation:

$$q_{all\ net} = q_{ult}/FS = 123.13 / 3 = 41.04 \text{ kN/m}^2$$

It was determined that the bearing capacity of different alternatives of foundation indicates that shallow foundations cannot be used on the soft clays of our site.

The acceptable amounts of pressure deviate between 47 and 75 kN/m^2 . These values will be utilized to verify the pressures that will be transferred between the columns of the building to the ground. However, it can already be concluded that the permissible bearing capacity of the upper soil layers is very low and will not be able to withstand the load from the columns. In this building, the column loads will be quite high. Pile foundations are likely to be required for most of the columns. The reason is that piles will enable penetration to deeper layers of soil, whose bearing capacity is an order of magnitude better.

4.4.2 Settlement calculation for Shallow Foundations

For settlement calculations, we considered immediate, consolidation, and secondary settlement. For applied contact pressure q , it was taken as the net bearing capacity of the shallow foundation, calculated by the Hansen Formula. The settlement was calculated for 3 different footing dimensions, with D_f as 4.5 meters.

For a footing $B \times L$ at base depth D_f , the vertical effective stress increment at a point z :

$$\Delta\sigma'(z) = q \frac{B \times L}{(B+z)(L+z)} \quad (5)$$

q - applied contact pressure

B - width of foundation

L - length of foundation

z - middle depth of the layer minus embedment D_f

Immediate Settlement

Immediate (elastic) settlement was evaluated using the elastic modulus and Poisson's ratio values derived from field and laboratory data correlations. For clayey soils, the modulus of elasticity was estimated as approximately 0.5 times the undrained shear strength (in MPa), while for granular soils, empirical relationships with SPT N-values were applied. This component of settlement was found to be relatively small compared to the consolidation component, as the soft clays dominated the deformation behavior.

Immediate Elastic settlement was calculated using this formula:

$$S_{i,layer} = \frac{\Delta\sigma'(1-\nu^2)}{E} H \quad (6)$$

$\Delta\sigma'$ - vertical stress increment in layer i due to foundation load (kPa)

ν - Poisson's ratio of the soil (dimensionless)

E - modulus of elasticity of the soil (kPa)

H - thickness of the layer (m)

Consolidation Settlement

Primary consolidation settlement was calculated using the classical one-dimensional consolidation theory. Compression and recompression indices (C_c and C_r) and the initial void ratio (e_0) were obtained from laboratory consolidation test data of nearby boreholes. The overconsolidation ratio (OCR) was also used to differentiate between recompression and virgin

compression zones. The analysis indicated that most layers up to 24 meters were normally consolidated or lightly overconsolidated, leading to significant primary settlement.

Preconsolidation:

$$\sigma_p' = OCR \times \sigma_o' \quad (7)$$

$$C_a = 0.03C_c \quad (8)$$

If $\sigma_o' + \Delta\sigma' \leq \sigma_p'$, then:

$$S_{c,layer} = H \frac{C_r}{1+e_0} \log_{10} \left(\frac{\sigma_o' + \Delta\sigma'}{\sigma_o'} \right) \quad (9)$$

If it **crosses** σ_p' If $\sigma_o' + \Delta\sigma' \geq \sigma_p'$:

$$S_{c,layer} = H \frac{C_a}{1+e_0} \log_{10} \frac{\sigma_p'}{\sigma_o'} + H \frac{C_c}{1+e_0} \log_{10} \frac{\sigma_o' + \Delta\sigma'}{\sigma_p'} \quad (10)$$

H - thickness of the compressible layer (m)

e_0 - initial void ratio of the soil

C_r - recompression index (for overconsolidated range)

C_c - compression index (for normally consolidated range)

σ_o' - initial effective overburden pressure (kPa)

σ_p' - preconsolidation pressure (kPa)

σ_f' - final effective pressure after loading (kPa)

Secondary Settlement

Secondary (creep) settlement was included for the layers undergoing virgin compression, using a secondary compression index (C_{α}) estimated as $0.03 \cdot C_c$. This component represented the

long-term deformation that may occur after the dissipation of excess pore pressures. The total settlement after a 10-year service period was obtained by summing the immediate, primary, and secondary components across all contributing layers.

Secondary settlement was calculated using this formula:

$$S_s = H \frac{C_a}{1+e_0} \log_{10} \left(\frac{t}{t_p} \right) \quad (11)$$

C_{a_i} - secondary compression index ($\approx 0.03 \cdot C_c$ for inorganic clays)

e_{0_i} - initial void ratio

H_i - thickness of the layer (m)

t - elapsed time after end of primary consolidation (years)

t_p - time at end of primary consolidation (years)

Table 4.4.2.1 Settlement Calculation Results for Shallow Foundation

Footing	q (kPa)	S_immediate (mm)	S_consolidation (mm)	S_secondary (mm)	TOTAL (mm)
2x2 m	71.13	10.64	29.32	46.83	86.78
3x3 m	68.25	16.9	75.19	46.83	138.92
1.5x3 m	68.5	10.78	30.63	46.83	88.24

The results showed that the total settlements are primarily governed by the soft clay strata from 2 to 24 meters in depth. The analysis confirmed that most of the settlement would occur gradually over time as consolidation proceeds, with minimal risk of differential settlement if the foundation system is properly designed and uniformly loaded.

In summary, the settlement behavior at the Manor Way site is typical for deep, compressible Gulf Coast clays. The upper soft clays are responsible for the majority of total settlement, while the underlying sands and stiff clays act as supporting and drainage layers. The final design recommendations, including allowable bearing pressure and estimated total settlement, are based on the calculated results using site-specific parameters and verified against

regional geotechnical data from nearby reports. However, it is recommended to explore other foundation designs to compare and find the most suitable one.

4.5 Pile Foundation

To ensure a reliable, functional foundation for the project under development, it was decided to use piles, as they are the most suitable foundation type in this case. The fact is that, according to the results of geological surveys in the selected area, the first five layers of soil consist of soft clay, which lacks a sufficiently strong structure to withstand the load from the building above, even if the upper layers are compacted or replaced. These layers extend to a depth of almost 34 metres, so it was necessary to develop piles that would allow the building to rest on more solid soil layers.

Pre-stressed, drilled reinforced concrete piles were used as the basis. The technology for installing such piles consists of first drilling a borehole, then placing a reinforcement cage in it, and then filling it with concrete. For this project, a reinforced concrete pile with a diameter of 610 mm and a length of 36 meters was designed. Inside this pile, there will be 18 reinforcing bars with a diameter of 12.7 mm.

4.5.1 Bearing Capacity for Pile Foundation

4.5.1.1 Bearing Capacity for Single Pile

The Ultimate load-bearing capacity for a single pile foundation consists of two parts: the load-carrying capacity of the pile point Q_p and frictional resistance (skin friction) Q_s . We will use various methods and compare them to find the best fit for our case.

For each iteration, the pile length was varied from 36 to 42 m to keep the pile tip inside the dense sand strata (33.5–45.7 m), where point resistance in sand is highest. A 44 m case is also evaluated in the summary tables at the end of this section: at 44 m, the tip drops into Layer 9 (medium-stiff clay), and although the resulting settlement is reduced, point capacity in clay is small, so that case is used only to demonstrate that a single pile remains inadequate at every length considered.

Meyerhof's Method for Estimating Q_p

$$Q_p = A_p q' N_q^* \leq A_p q_l \quad (12)$$

$$q_l = 0.5 p_a N_q^* \tan \varphi' \quad (13)$$

Where

Q_p - point load capacity

A_p - area of the pile tip

q' - effective vertical stress at the level of the pile tip

N_q^* - the bearing capacity factor for piles

q_l - limiting point resistance

p_a - atmospheric pressure = 100

φ' - effective soil friction angle

$$N_q^* = 96, \text{ for 36 meters, according to Das (2019)}$$

$$A_p = \frac{\pi * 0.61^2}{4} = 0.292 \text{ m}^2$$

$$q' = 18.5 * 2 + 18.8 * 6 + 19 * 8 + 19.2 * 8 + 19.2 * 9.5 + 2.5 * 20 = 692.1 \text{ kN/m}^2$$

$$Q_{p(1)} = 692.1 * 96 * 0.292 = 19400.95 \text{ kN}$$

$$q_l = 0.5 * 100 * 96 * \tan 33^\circ = 3117.16 \text{ kN/m}^2$$

$$Q_{p(2)} = 3117.16 * 0.292 = 910.21 \text{ kN}$$

$$Q_{p(2)} < Q_{p(1)}, \text{ therefore } Q_p = 910.21 \text{ kN}$$

Other values for the load-carrying capacity of the pile point can be observed in the Table below:

Table 4.5.1.1.1 Load-carrying capacity of the pile point by Meyerhof's method

L, m	q'	A_p, m^2	N_q	Q_p , kN	q_l	$A_p^* q_l$
36	692.1	0.292	96	19400.95	3117.16	910.21
38	731.7	0.292	96	20511.01	3117.16	910.21
40	772.21	0.292	115	25930.81	3878.43	1132.50
42	813.21	0.292	115	27307.59	3878.43	1132.50

Vesic's Method for Estimating Q_p

Vesic (1977) proposed a method for estimating the pile point bearing capacity based on the theory of expansion of cavities. It can be found using the following formula,

$$Q_p = A_p \sigma_0' N_\sigma^* \quad (14)$$

Where

$$\sigma_0' - \text{mean effective normal ground stress} = \frac{(1+2K_0)}{3} q'$$

$$K_0 - \text{earth pressure coefficient at rest} = 1 - \sin\phi'$$

$$N_\sigma^* - \text{bearing capacity factor} = \frac{N_q^*}{(1+2K_0)}$$

According to Vesic's theory,

$$N_\sigma^* = f(I_{rr}) \quad (15)$$

$$I_{rr} = \frac{I_r}{1+I_r \Delta} \quad (16)$$

$$I_r = \frac{E_s}{2(1+\mu_s)*q'*\tan\phi'} \quad (17)$$

$$\Delta = 0.05(1 - \frac{\phi'-25}{20})\frac{q'}{p_a} \quad (18)$$

Where,

I_r = rigidity index

I_{rr} = reduced rigidity index

Δ = average volumetric strain in the plastic zone below the pile point

μ_s = Poisson's ratio of soil

For a 36-meter-long pile:

$$K = 1 - \sin 33^\circ = 0.4554$$

$$\Delta = 0.05(1 - \frac{33-25}{20})\frac{692.1}{100} = 0.021$$

$$I_r = \frac{16250}{2(1+0.3)*692.1 * \tan 33} = 13.906$$

$$I_{rr} = \frac{13.906}{1+13.906*0.021} = 10.790$$

$$N_\sigma^* = 24.538, \text{ according to Das (2019)}$$

$$\sigma_0' = \frac{1+2*0.455}{3} * 692.1 = 440.80$$

$$Q_p = 0.292 * 440.80 * 24.54 = 3158.40 \text{ kN}$$

Other values for the load-carrying capacity of the pile point can be observed in the Table below:

Table 4.5.1.1.2 Load-carrying capacity of the pile point by Vesic's method

L, m	ϕ'	K	q' , kPa	E_s , kPa	I_{rr}	A_p , m^2	N_σ	σ'_0	Q_p , kN
36	33	0.4554	692.1	16250	11.331	0.292	24.538	440.80	3158.40
38	33	0.4554	731.7	16250	10.718	0.292	23.986	466.03	3264.00
40	34	0.4408	772.21	21000	12.545	0.292	27.598	484.33	3903.06
42	34	0.4408	813.21	21000	11.912	0.292	26.980	510.05	4018.25

Coyle and Castello's Method

Coyle and Castello (1981) compared 24 field load tests of driven piles done in sand of large size. On the test results, they proposed that in sand,

$$Q_p = A_p q' N_q^* \quad (19)$$

For 36 meters, we calculated the bearing capacity factor using the embedment ratio $\frac{L}{D}$,

$$\frac{L}{D} = \frac{36}{0.61} = 59.02 \quad (20)$$

N_q^* was then obtained using the graph below, using the embedding ratio and the friction angle of 33 degrees:

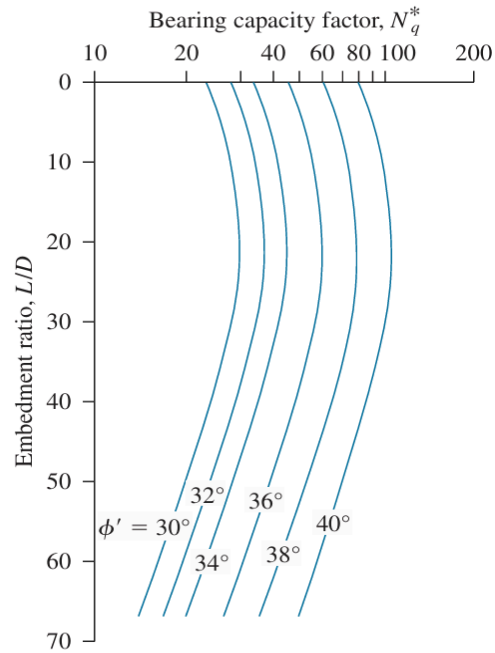


Figure 4.5.1.1.1 Variation of N_q^* with $\frac{L}{D}$

From Figure 4.5.1.1.1, we can see that $N_q = 27$.

Now, by plugging in the values, we get:

$$Q_p = (18.5 * 2 + 18.8 * 6 + 19 * 8 + 19.2 * 8 + 19.2 * 9.5 + 2.5 * 20) * 27 * \left(\frac{\pi * 0.61^2}{4}\right) = 5422.62 \text{ kN}$$

The results of calculating Q_p for different pile lengths can be seen in the table below.

Table 4.5.1.1.3 Load-carrying capacity of the pile point by Coyle and Castello's method

L, m	γ , kN/m ³	q'	A_p , m ²	N_q	Q_p , KN
36	20	687.8	0.292	27	5422.62
38	20	727.8	0.292	25	5312.94
40	19.8	767.4	0.292	24	5377.94
42	19.8	807	0.292	22	5184.17

The final values for the Q_p can be seen in the table below. It was determined to move with Vesic's method as it was the most consistent one and showed variation with depth. For completeness, a 44 m pile is also considered. At this length the tip reaches 49.30 m, which lies in Layer 9 — a medium-stiff clay with $c_u = 60$ kPa, $E_s = 30$ MPa. Because the tip material is cohesive, the Meyerhof and Coyle & Castello formulations (both based on a drained friction angle ϕ') are not applicable to this depth. Vesic's method is retained in its cohesive form:

$$Q_p = A_p \cdot c_u \cdot N_c^*$$

The rigidity index for an undrained clay is approximated by $I_r = E_s / (3 \cdot c_u) = 30,000 / (3 \cdot 60) = 166.67$. Reading Das (2019), Table 12.4, this gives $N_c^* \approx 10.66$. With $A_p = \pi \cdot (0.61)^2 / 4 = 0.2922$ m²:

$$Q_p = 0.2922 \times 60 \times 10.66 = 186.87 \text{ kN}$$

This is roughly 5% of the Vesic value at 42 m (4,018 kN). The sharp drop is physically expected — the tip has left the dense sand and entered a cohesive layer — and is recorded in the summary table below.

Table 4.5.1.1.4 Final Values for load-carrying capacity of the pile point

L, m	Q_p (Meyerhof, kN)	Q_p (Vesic, kN)	Q_p (Coyle & Castello, kN)
36	910.21	3158.40	5422.62
38	910.21	3264.00	5312.94
40	1132.50	3903.06	5377.94
42	1132.50	4018.25	5184.17
44	-	186.87	-

Frictional Resistance (Q_s) in Sand

Frictional Resistance of the pile in sand can be calculated with,

$$Q_s = pLf \quad (21)$$

Where

p - pile perimeter

L - pile length

f - unit friction resistance, for sand it is calculated as follows:

$$f = K\sigma'_0 \tan \delta' \quad (22)$$

For bored piles, the coefficient K is calculated using the formula below:

$$K = 1 - \sin \phi' \quad (23)$$

δ' is the soil-pile friction angle and equals $0.8\phi'$, whereas σ'_0 is the average effective overburden pressure taken from the soil profile table.

λ Method:

According to Vijayvergiya and Focht (1972), the total stress method was proposed and can be summed up as follows: it is grounded on the fact that the movement to the side of soil resulting in pile driving produces a passive lateral pressure at any depth, and the average unitary skin resistance is. This method was used as the most optimal method to calculate the friction resistance in clay.

$$Q_s = pLf_{av} \quad (24)$$

Where f_{av} is unit friction resistance, and according to the method, it equals:

$$f_{av} = \lambda(\sigma'_0 + 2c_u) \quad (25)$$

Where

σ_0' - mean effective vertical stress for the entire embedment length

c_u - mean undrained shear strength

The coefficient λ can be found from the table below in accordance with the pile depth:

Table 4.5.1.1.5 Variation of λ with pile depth

Embedment length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

In the case of a single pile whose embedment in clay is less than 35 m, λ is equal to 0.132.

Table 4.5.1.1.6 Calculation Q_s for every soil layer

Layer	Depth (m)	σ'_v (kPa)	λ	P (m)	K	δ'	$\tan\delta'$	f_avg (kPa)	Qs (kN)
Layer 2	3.3-8.0	93.4	0.132	1.9164	0	0	0	17.61	158.61
Layer 3	8.0-16.0	225.8	0.132	1.9164	0	0	0	36.41	558.14
Layer 4	16.0-24.0	378.6	0.132	1.9164	0	0	0	59.21	907.84
Layer 5	24.0-33.5	546.6	0.132	1.9164	0	0	0	84.031	1529.85
Layer 6	33.5-38	682.8	0.132	1.9164	0.4554	26.4	0.4964	154.35	1331.12
Layer 7	38.0-42.0	767.4	0	1.9164	0.4554	26.4	0.4964	173.47	1329.82
Layer 8	42.0-45.7	844.93	0	1.9164	0.4408	27.2	0.5139	191.39	1357.15

Table 4.5.1.1.7 Frictional Resistance results

L, m	D, m	Q_s
36	0.61	4917.75
38	0.61	5582.66
40	0.61	6292.22
42	0.61	7025.81
44	0.61	7,569.10

Now we can calculate the total bearing capacity of the 36-meter-long pile by adding together the point load capacity and the frictional resistance with a Factor of safety of 3.

$$Q_u = Q_p + Q_s = 3158.40 + 4917.75 = 8076.15 \text{ kN}$$

$$Q_{all} = \frac{Q_u}{FS} = \frac{8076.15}{3} = 2692.05 \text{ kN}$$

The rest the Q_u are listed in the table below,

Table 4.5.1.1.8 Final Bearing Capacity values for single pile foundation

L, m	D, m	Q_p (Meyerhof, kN)	Q_p (Vesic, kN)	Q_p (Coyle & Castello, kN)	Q_s , kN	Q_u , kN	FoS	Q_{allow} , kN
36	0.61	910.21	3158.40	5422.62	4917.75	8076.15	3	2692.05
38	0.61	910.21	3264.00	5312.94	5582.66	8846.66	3	2948.89
40	0.61	1132.50	3903.06	5377.94	6292.22	10195.2 8	3	3398.43
42	0.61	1132.50	4018.25	5184.17	7025.81	11044.0 6	3	3681.35
44	0.61	-	186.87	-	7,569.10	7,755.97	3	2,585.32

Across the full range considered (36–44 m), Q_{allow} for a single Ø610 mm pile remains in the 2,585–3,681 kN band, an order of magnitude below the $\approx 22,000$ kN column demand. Increasing the length beyond 42 m does not help, since once the tip crosses into Layer 9 clay, the lost point resistance more than offsets the added skin friction. A single-pile foundation is therefore rejected, and the analysis proceeds to pile groups of smaller-diameter piles in Section 4.5.1.2.

4.5.1.2 Bearing Capacity of a Group of Piles

A single pile-group configuration was designed and applied uniformly beneath every column in the hotel, rather than differentiating between corner, edge, and interior columns. This approach follows directly from the structural analysis in Section 3, where the factored axial load was distributed evenly across all columns, so that each column transfers the same design load of

approximately 21,920 kN to its foundation. Under this assumption, there is no tributary-area variation to justify separate foundation layouts, and designing a single governing pile group ensures that the most heavily loaded condition is satisfied everywhere in plan. This simplification also aids constructability, since identical pile caps, reinforcement details, and installation sequences can be used throughout the site without column-specific variations.

The bearing capacity of the pile group is evaluated using the summation of individual pile capacities, which is a common and practical method when piles are spaced sufficiently apart and soil-pile interaction effects are limited. In this approach, each pile is assumed to carry its share of load independently, and the total group capacity is obtained by adding the ultimate axial capacity of each pile in the group. It will be calculated as follows,

$$Q_{group} = \eta * n * Q_{single} \quad (26)$$

Where,

Q_{single} - ultimate capacity of one pile

n - total number of piles in the group,

η - group efficiency corresponding to the selected spacing.

Negative skin friction

Negative skin friction (also called downdrag) arises when soft or compressible soil surrounding a pile consolidates or settles more than the pile itself, causing downward shear along the pile shaft (Victor, 2025). In that case, the soil “drags” the pile downward rather than the pile supporting the soil, and the additional shear force adds to the axial load on the pile or pile group, and must be treated as an extra permanent downward load in design rather than as contributing to pile capacity. It can be calculated as follows:

$$Q_n = \frac{pK'y_f H_f^2 \tan \delta'}{2} \quad (27)$$

Where,

K' - earth pressure coefficient

σ'_0 - vertical effective stress at any depth z

γ'_f - effective unit weight of fill

H_f - height of the clay fill

Table 4.5.1.2.1 Negative skin friction for various diameter values

	Depth, m	D = 0.4	D = 0.5
Layer 2	3.3-8.0	34.38 kN	43.00 kN
Layer 3	8.0-16.0	106.81 kN	133.59 kN
Layer 4	16.0-24.0	119.39 kN	149.33 kN
Layer 5	24.0-33.5	119.39 kN	215.91 kN
Layer 6	45.7-49.3	25.18 kN	25.18 kN
Total Q_n		458.37 kN	567.01 kN

Group efficiency

To evaluate the effectiveness of each pile layout, we must calculate the group efficiency. We will pick a diameter of 0.4 and 0.5 meters for evaluation.

Table 4.5.1.2.2 Group efficiency at different spacings

D, m	n_1	n_2	d, m	p, m	η	d taken as
0.4	4	5	1.2	1.2566	0.73	3D
0.5	4	4	1.5	1.5708	0.79	3D
0.4	4	5	1.4	1.2566	0.84	3.5D
0.5	3	4	1.75	1.5708	1.00	3.5D

For the final design, the layouts with a spacing of 3.5D were selected for both 0.4 m and 0.5 m diameter piles. This spacing keeps the piles sufficiently far apart, minimizing interaction between stress zones and allowing the summation of individual pile capacities to remain a valid and straightforward method. At 3.5D spacing, the group efficiencies obtained from the calculations are $\eta = 0.84$ for the 0.4 m piles and $\eta = 1.00$ for the 0.5 m piles, which indicates that each pile can develop most of its axial resistance without significant reduction from neighbouring piles.

To evaluate these layouts, the piles are extended to 44 m. With the 3.3 m basement excavation, the 0.35 m concrete basement slab and the 1.65 m pile cap above the piles, the pile head sits at 5.30 m and the pile tip at 49.30 m — inside Layer 9, a medium-stiff clay. Embedding the tip in clay rather than sand reduces point capacity but is still acceptable: the single-pile capacities reported below combine a modest Vesic clay-tip Q_p with a substantially larger skin-friction Q_s developed along the overlying sand and clay layers, and the consolidation settlement is limited because the stresses imposed on the deep clay layers remain below their preconsolidation pressure.

First, we will determine the capacities of 44-meter-long piles with diameters of 0.4 and 0.5 meters, each treated as a single pile. Vesic's method for Q_p will be used, and due to the pile tip being placed in a clay layer, the following formula was used to estimate the carrying capacity of the pile point:

$$Q_p = A_p * c_u * N_c^* \quad (28)$$

N_c^* is estimated through $I_{rr} = \frac{E_s}{3c_u} = \frac{30000}{3*60} = 166.6$, which gives us a value of 10.66 according to Das (2019).

$$Q_p = \frac{\pi * 0.4^2}{4} * 166.6 * 60 = 80.37 \text{ kN}$$

For the new value of Q_s , an additional clay layer was added to the previously calculated value for the 44-meter pile, giving us a value of 5308.8 kN.

The final capacities as a single pile can be seen in the table below

Table 4.5.1.2.3 Capacities as a single pile

L, m	D, m	Q_p , kN	Q_s , kN	Q_u , kN	Q_{allow} , kN
44	0.4	80.37	5308.8	5,389	1796.39
44	0.5	125.58	6636	6,762	2253.86

Now we calculate Capacities by multiplying the single pile Q_{allow} by the group efficiency and the number of piles.

For a 4x5 pile and a 0.4 m diameter bearing capacity is

$$Q_g = 1796.39 * 20 * 0.84 = 30179.4 \text{ kN}$$

Table 4.5.1.2.4 Final Bearing capacity values for the Group of piles

D, m	η	Q_{allow}	n	$Q_{g(allow)}$, kN	Q_{demand} , kN	Spacing taken as
0.4	0.84	1796.39	20	30179.4	26456	3.5D
0.5	1.00	2253.86	12	27046.3	26456	3.5D

The results shown at the Table 4.5.1.2.4, indicate that both pile group configurations, using 0.4 m and 0.5 m diameter piles at 3.5D spacing, provide sufficient allowable group bearing capacity to safely support the applied load, with both layouts meeting the required demand with an adequate margin.

4.5.2 Settlement of the Pile Foundation

4.5.2.1 Settlement of Single Pile

The settlement of the single pile was calculated using Vesic's (1977) method, as he separated the elastic settlement of the pile head into three components:

1. Elastic shortening of the pile
2. Settlement of the pile due to the working load at the pile point

3. Settlement of the pile due to the working load along the pile shaft

Elastic shortening of pile due to load transfer along the shaft:

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \quad (29)$$

Where,

Q_{wp} - working load at the pile point

Q_{ws} - working load along the pile shaft

A_p - cross-sectional area of the pile point

E_p - modulus of elasticity of the pile material = 33 GPa

L - pile length

ξ - 0.5

Elastic shortening for the 36 m long pile:

$$Q_{wp} = \frac{3158.40}{3} = 1052.80 \text{ kN}$$

$$Q_{ws} = \frac{4917.75}{3} = 1639.25 \text{ kN}$$

$$s_{e(1)} = \frac{(1052.80 + 0.5 \cdot 1639.25) \cdot 36}{0.292 \cdot 33000000} = 0.006995 \text{ m} = 6.995 \text{ mm}$$

Table 4.5.2.1.1 Elastic shortening results

L, m	Q_{wp} , kN	Q_{ws} , kN	A_p , m^2	E_p , MPa	$s_{e(1)}$, m	$s_{e(1)}$, mm
36	1052.800	1639.250	0.292	33000	0.00700	6.995
38	1088.000	1860.887	0.292	33000	0.00796	7.960

40	1301.020	2097.406	0.292	33000	0.00975	9.754
42	1339.416	2341.938	0.292	33000	0.01094	10.942
44	62.29	2,523.03	0.2922	33000	0.00604	6.04

The settlement of a pile caused by the load carried at the pile point:

$$s_{e(2)} = \frac{q_{wp} D}{E_s} (1 - \mu_s^2) I_{wp} \quad (30)$$

Where

D - width or diameter of pile

q_{wp} - point load per unit area at the pile point

E_s - modulus of elasticity of soil at or below the pile point

μ_s - Poisson's ratio of soil

I_{wp} - influence factor

Settlement of the 36-meter-long pile due to the working load at the pile point:

$$s_{e(2)} = \frac{1052.80 \cdot 0.61}{16250} (1 - 0.32^2) \cdot 0.85 = 0.10318 \text{ m} = 103.18 \text{ mm}$$

Table 4.5.2.1.2 Settlement caused by the load carried at the pile point results

L, m	Q_{wp} , kN	A_p , m^2	q_{wp} , kN/m^2	μ_s'	E_s , kPa	I_{ws}	$s_{e(2)}$, m	$s_{e(2)}$, mm
36	1052.80	0.292	3602.436	0.32	16250	0.85	0.1032	103.18
38	1088.00	0.292	3726.026	0.32	16250	0.85	0.1067	106.71
40	1301.02	0.292	4455.547	0.3	21500	0.85	0.0978	97.78
42	1339.41	0.292	4587.042	0.3	21500	0.85	0.1007	100.67

44	62.29	0.292	213.172	0.4	30,000	0.85	0.00310	3.10
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Settlement of the pile due to the working load along the pile shaft:

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \quad (31)$$

Where

p - perimeter of pile

L - embedded length of pile

I_{ws} - influence factor = $2 + 0.35\sqrt{\frac{L}{D}}$

Settlement of the 36-meter-long pile due to the working load along the pile shaft:

$$s_{e(3)} = \left(\frac{1639.25}{1.9164 \cdot 36}\right) \frac{0.61}{16250} (1 - 0.32^2) * (2 + 0.35\sqrt{\frac{36}{0.61}}) = 0.003754 \text{ m} = 3.754 \text{ mm}$$

Table 4.5.2.1.3 Settlement due to the working load along the pile shaft

L, m	Q_{ws} , kN	p, m	D, m	μ_s	E_s , kPa	I_{ws}	$s_{e(3)}$, m	$s_{e(3)}$, m
36	1639.25	1.9164	0.61	0.32	16250	4.69	0.00375	3.754
38	1860.88	1.9164	0.61	0.32	16250	4.76	0.00410	4.101
40	2097.40	1.9164	0.61	0.3	21500	4.83	0.00342	3.415
42	2341.93	1.9164	0.61	0.3	21500	4.90	0.00368	3.684
44	2,523.03	1.9164	0.61	0.4	30,000	4.97	0.00254	2.54

ASTM D1143 suggests that settlement should not exceed 25 mm. At 36–42 m, the computed settlements (≈ 111 – 119 mm) all exceed the 25 mm ASTM D1143 limit and are therefore rejected. At 44 m the settlement falls to ≈ 11.7 mm (because Q_p , and with it q_{wp} , drops when the tip enters clay), which satisfies the settlement criterion; however, the allowable

single-pile capacity at 44 m is only 2,585 kN, so a single-pile solution is ruled out on capacity grounds regardless. A pile group is therefore adopted in Section 4.5.1.2.

Table 4.5.2.1.4 Final Settlement values

L, m	$s_{e(1)}, \text{ m}$	$s_{e(2)}, \text{ m}$	$s_{e(3)}, \text{ m}$	$s_{e(total)}, \text{ m}$	$s_{e(total)}, \text{ mm}$
36	0.0070	0.103	0.00375	0.114	113.92
38	0.0080	0.107	0.00410	0.119	118.78
40	0.0098	0.098	0.00342	0.111	110.95
42	0.0109	0.101	0.00368	0.115	115.29
44	6.04	3.10	2.54	0.01168	11.68

4.5.2.2 Settlement of the Group of Piles

To evaluate the immediate (elastic) settlement of the pile group, the group is treated as a single block transmitting load to the soil beneath the pile tips. The applied group load is distributed over the equivalent group area, and the resulting stress increase is used together with the soil's elastic parameters to estimate settlement. Following the formula, suggested by Vesic (1969), to obtain the settlement of group piles:

$$s_{g(e)} = \sqrt{\frac{B_g}{D}} * s_e \quad (32)$$

Where

$s_{g(e)}$ - elastic settlement of group piles

B_g - width of group pile section

D - width or diameter of each pile in the group

s_e - elastic settlement of each pile at a comparable working load

First, the width of the pile sections was calculated:

$$L_{g(0.4)} = (n_1 - 1) * d + 2(\frac{D}{2}) = (5 - 1) * 1.4 + 2(\frac{0.4}{2}) = 6 \text{ m}$$

$$B_{g(0.4)} = (n_2 - 1) * d + 2(\frac{D}{2}) = (4 - 1) * 1.4 + 2(\frac{0.4}{2}) = 4.6 \text{ m}$$

$$L_{g(0.5)} = (n_1 - 1) * d + 2(\frac{D}{2}) = (4 - 1) * 1.75 + 2(\frac{0.5}{2}) = 5.75 \text{ m}$$

$$B_{g(0.5)} = (n_2 - 1) * d + 2(\frac{D}{2}) = (3 - 1) * 1.75 + 2(\frac{0.5}{2}) = 4 \text{ m}$$

For the pile groups with a diameter of 0.4 and 0.5 meters, the settlement of each pile was calculated using equation x. Final values are in the table below.

Table 4.5.2.2.1 Elastic Settlement as a single pile

L, m	D, m	$s_{e(1)}$, m	$s_{e(2)}$, m	$s_{e(3)}$, m	s_e , m	s_e , mm
44	0.4	0.00005	0.00011	0.000734	0.00089	0.89
44	0.5	0.00007	0.00020	0.000886	0.00115	1.15

Final elastic settlement values were calculated using equation x

$$s_{g(e)_1} = \sqrt{\frac{4.6}{0.4}} * 0.89 = 3.018 \text{ mm}$$

$$s_{g(e)_2} = \sqrt{\frac{4}{0.5}} * 1.15 = 3.25 \text{ mm}$$

Consolidation settlement

Consolidation settlement is estimated by treating the pile group as a rigid block that increases vertical stress in the compressible clay layers beneath the pile tips. The stress increment caused by the group load is applied to each clay layer, and settlement is computed

using the classical one-dimensional consolidation relationship with the layer's compression index, recompression index, initial void ratio, and effective overburden pressure. By summing the settlements of all contributing layers, the total long-term consolidation settlement of the pile group is obtained. The 2:1 stress distribution method suggests that stress will start from the 33.5 meters up to the last two clay layers, so they will be the main contributors to the consolidation settlement. The following formulas were used in the calculation:

$$\Delta\sigma'_i = \frac{Q_g}{(B_g + z_i)(L_g + z_i)} \quad (33)$$

Where

$\Delta\sigma'_i$ - increase in effective stress at the middle of layer i

L_g, B_g - length and width, respectively, of the planned group piles

z_i - distance from $z = 0$ to the middle of the clay layer i

$$\text{If } \sigma'_0 + \Delta\sigma'_i < \sigma'_p, \text{ then } S_c = \frac{C_r * H}{1 + e_0} * \log\left(\frac{\sigma'_0 + \Delta\sigma'_i}{\sigma'_0}\right) \quad (34)$$

Where

σ'_p - preconsolidation stress = $\sigma'_o * OCR$

C_r - recompression index of soil

H - height of clay layer

e_0 - void ratio of soil

The settlement was then calculated layer by layer for group piles with different diameters to determine how much this parameter can affect settlement.

Consolidation settlement for Layer 9 (45.7 - 52):

$$\sigma'_0 = 958.5 \text{ kPa}$$

$$C_r = 0.056$$

$$e_0 = 0.9$$

$$z_1 = \frac{45.7+52}{2} - 33.5 = 15.35 \text{ m}$$

For D=0.4 m:

$$\Delta\sigma'_i = \frac{24349.4}{(4.6+15.35)(6+15.35)} = 57.17 \text{ kPa}$$

$$\sigma'_p = 958.5 * 1.1 > 958.5 + 57.17 \Rightarrow 1054.35 > 1015.67$$

$$S_{c_1} = \frac{0.056*6.3}{1+0.9} * \log\left(\frac{1015.67}{958.5}\right) = 0.00467 \text{ m} = 4.67 \text{ mm}$$

For D=0.5 m:

$$\Delta\sigma'_i = \frac{21986.1}{(4+15.35)(5.75+15.35)} = 53.85 \text{ kPa}$$

$$\sigma'_p = 958.5 * 1.1 > 958.5 + 53.85 \Rightarrow 1054.35 > 1012.35$$

$$S_{c_1} = \frac{0.056*6.3}{1+0.9} * \log\left(\frac{1012.35}{958.5}\right) = 0.00441 \text{ m} = 4.41 \text{ mm}$$

Consolidation settlement for Layer 10 (52 - 60):

$$\sigma'_0 = 1027.8 \text{ kPa}$$

$$C_r = 0.04$$

$$e_0 = 0.75$$

$$z_1 = \frac{60+52}{2} - 33.5 = 22.5 \text{ m}$$

For D=0.4 m:

$$\Delta\sigma'_i = \frac{24349.4}{(4.6+22.5)(6+22.5)} = 31.53 \text{ kPa}$$

$$\sigma'_p = 1027.8 * 1.1 > 1027.8 + 31.53 \Rightarrow 1130 > 1059.33$$

$$S_{c_2} = \frac{0.04*8}{1+0.75} * \log\left(\frac{1059.33}{1027.8}\right) = 0.0024 \text{ m} = 2.4 \text{ mm}$$

For D=0.5 m:

$$\Delta\sigma'_i = \frac{21986.1}{(4+22.5)(5.75+22.5)} = 29.37 \text{ kPa}$$

$$\sigma'_p = 1027.8 * 1.1 > 1027.8 + 29.37 \Rightarrow 1130 > 1057.17$$

$$S_{c_2} = \frac{0.04*8}{1+0.75} * \log\left(\frac{1057.17}{1027.8}\right) = 0.00224 \text{ m} = 2.24 \text{ mm}$$

Total settlement of Group piles with D=0.4m: $S_{total} = 3.018 + 4.67 + 2.4 = 10.09 \text{ mm}$

Total settlement of Group piles with D=0.5m: $S_{total} = 3.25 + 4.41 + 2.24 = 9.90 \text{ mm}$

The results indicate that the total settlement is 10.09 mm for the 0.4 m piles and 9.90 mm for the 0.5 m piles. These values are very small and fall well within acceptable limits for a pile-supported structure, confirming that long-term consolidation effects will not govern the performance of the selected pile group configurations

4.5.3 Laterally Loaded Piles

4.5.3.1 Determination of Lateral Load

The structural analysis (Section 3.4.1, Table 3.4.1.11) provided the horizontal wind forces that occur on the building at the first-floor level. The forces are:

$$F_x = 255.268 \text{ kN}, F_y = 156.155 \text{ kN}$$

To obtain a conservative estimate, the total resulting force is considered to be on a single column. The horizontal force per column is:

$$F_{col} = (F_x^2 + F_y^2)^{1/2} = 299.24 \text{ kN} \quad (35)$$

A column is supported by a rigid pile cap that is on 12 piles (4×3 group). Assuming the pile cap distributes the horizontal load equally among all piles, the lateral load per individual pile is:

$$Q_g = F_{col}/12 = 24.94 \text{ kN} \approx 25 \text{ kN}$$

4.5.3.2 Elastic Solution for Pile Head Deflection

Two methods are used to find the lateral bearing capacity and head deflection of a pile: the first is the elastic solution method, and the second is the method of Broms. Initially, we will calculate using the first algorithm.

Elastic Solution:

First, the moment of inertia for the pile must be calculated.

$$I_p = \frac{\pi D^4}{64} = 0.003068 \text{ m}^4 \quad (36)$$

It is assumed that $n_h = 16500$ and $E_p = 4700\sqrt{f_{ck}} = 2.7806 \times 10^7 \text{ kN/m}^2$, this information is needed to calculate T:

$$T = \left(\frac{E_p I_p}{n_h} \right)^{1/5} = 1.389 \text{ m} \quad (37)$$

$$\frac{L}{T} = \frac{44}{1.389} = 31.78 > 5: \text{ long pile}$$

Given this fact, and since the applied moment at the pile head, M_g , is zero, the deflection can be calculated using the formula below:

$$x_0 = \frac{Q_g A_x T^3}{E_p I_p} \quad (38)$$

At a depth equal to zero, $A_x = 2.435$ (from Table 12.15, Das, 2019), in this case deflection is:

$$x_0 = \frac{2.435 \cdot 25 \cdot 2.678}{85300} = 1.91 \text{ mm}$$

Lateral deflection at the head of the pile is calculated at 1.91 mm, and this is much lower than the normal serviceability limit of 8 mm that is suggested in the case of laterally loaded piles. Therefore, the pile meets the deflection requirements. In addition, it is necessary to compare the actual lateral load with the maximum allowable lateral resistance. But to do this, we must first calculate the ultimate moment for a pile with a circular cross-section and symmetrical reinforcement.

$$M_{z(max)} = R_s A_s (h_0 - 0.5x) + R_{sc} A'_s (h_0 - a') \quad (39)$$

Where:

R_s - tensile strength of steel.

A_s - area of the stretched reinforcement.

h_0 - design height, $500 - 75 = 425$ mm.

R_{sc} - design shear strength of steel.

A'_s - the area of the compressed reinforcement is taken as half of the total area of the reinforcement.

a' - protective layer to the center of mass of compressed reinforcement, 75 mm.

x - the height of the compressed zone is calculated using the formula below:

$$x = \frac{Q_{pile} - R_{sc} * A_{s,total}^{*0.5} + R_s * A_{s,total}^{*0.5}}{R_b d} = 155 \text{ mm} \quad (40)$$

In that case, the maximum moment is:

$$M_{z(max)} = 450 * 616 * (425 - 0,5 * 155) + 500 * 616 * (425 - 75) = 204 \text{ kN} * \text{m}$$

Now we can find the maximum lateral resistance:

$$Q_{g(max)} = \frac{M_{z(max)}}{A_m T} = \frac{204}{0.772 * 1.389} = 190.24 \text{ kN} > Q_g, \text{ the deflection criteria apply.} \quad (41)$$

4.5.3.3 Deflection Profile along the Pile Shaft

It is possible to determine the deflection of a pile at different depths using the same elastic method. To do this, the coefficient A_x must be adjusted for each new depth. Table 4.5.3.3.1 shows the pile deflections for different depths.

Table 4.5.3.3.1 Lateral deflection along the pile

Depth z (m)	Z = z/T	A_x	Deflection x_z (mm)
0.00	0.0	2.435	1.91
0.28	0.2	2.112	1.66
0.56	0.4	1.796	1.41
0.83	0.6	1.496	1.17
1.11	0.8	1.216	0.95
1.39	1.0	0.962	0.76

1.67	1.2	0.738	0.58
1.94	1.4	0.544	0.43
2.22	1.6	0.381	0.30
2.50	1.8	0.247	0.19
2.78	2.0	0.142	0.11
4.17	3.0	-0.075	-0.059
5.56	4.0	-0.050	-0.039
6.94	5.0	-0.009	-0.007

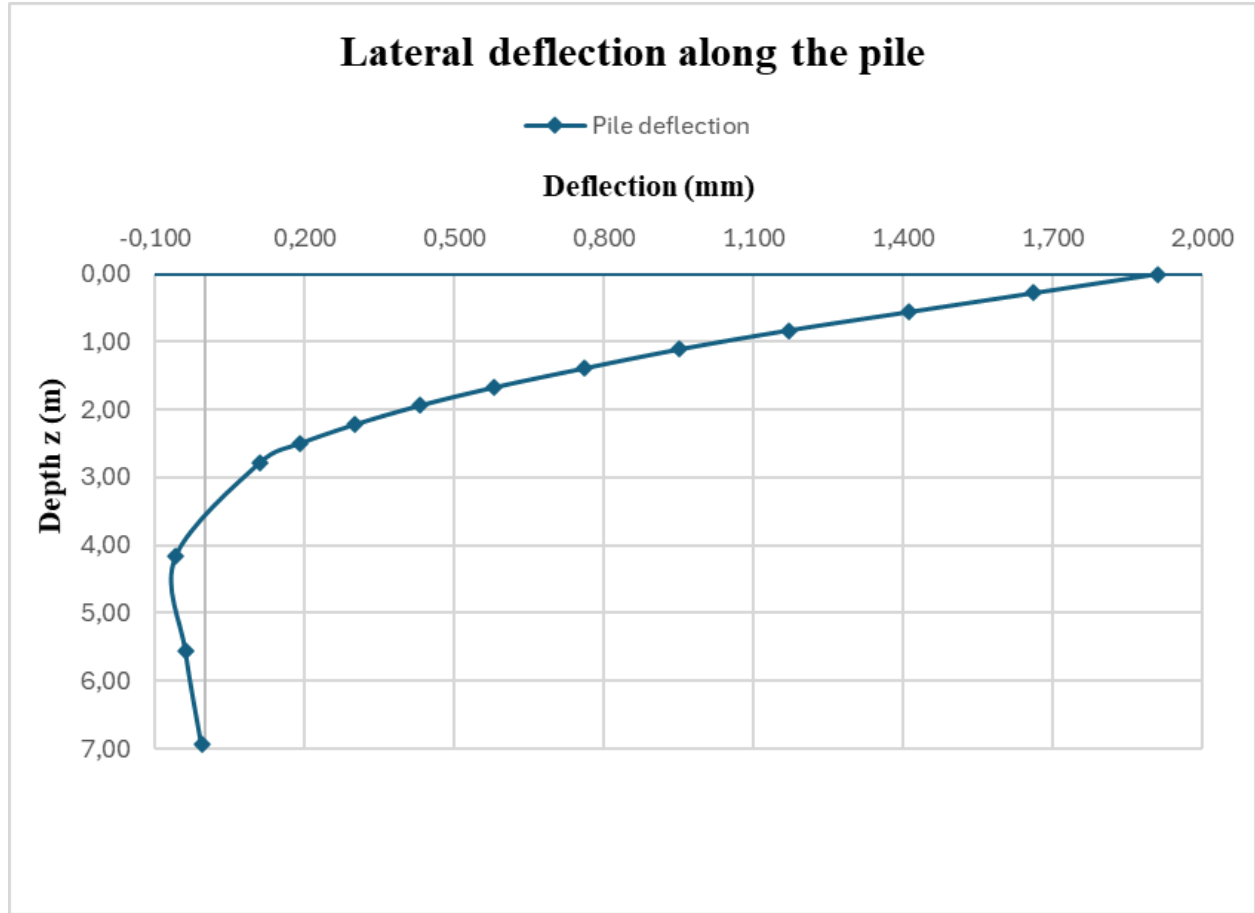


Figure 4.5.3.3.1 Graph of pile deflection

4.5.3.4 Method of Broms for Pile Head Deflection

The main equation for this method is as follows:

$$\frac{M_{z(max)}}{D^4 \gamma K_p} = \frac{M_{z(max)}}{D^4 \gamma \tan^2(45 + \phi'/2)} = \frac{204}{0.5^4 * 20.5 * \tan^2(45 + 34/2)} = 45.01 \quad (42)$$

Then we found the magnitude of $Q_{u(g)}/K_p D^3 \gamma$, which is about 23 according to the figure below:

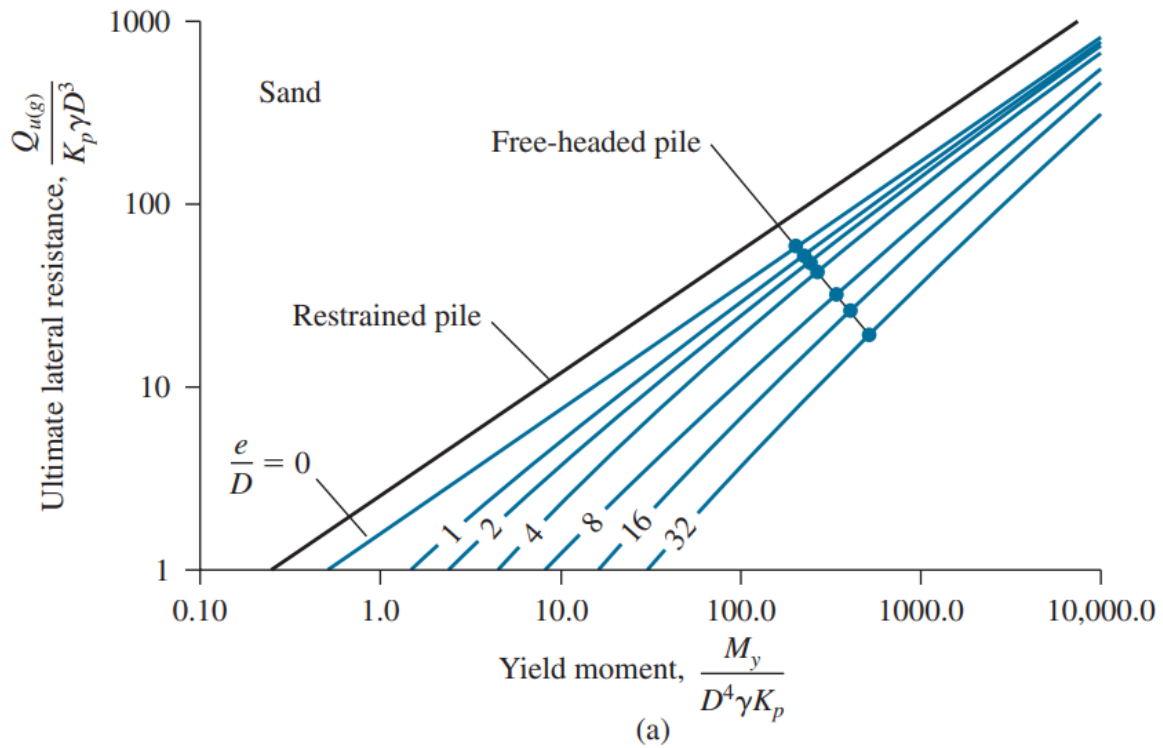


Figure 4.5.3.4.1 The solution of Broms for the ultimate lateral resistance of long piles in sand

Applying the magnitude from the figure above maximum lateral resistance now can be calculated using the following formula:

$$Q_{u(g)} = 23K_p D^3 \gamma = 23 * \tan^2(45 + 34/2) * 0.5^3 * 20.5 = 208.47 \text{ kN} > Q_g = 25 \text{ kN}$$

It can be seen that the deflection criteria apply. The next step is to check the deflection of the pile head.

$$n = \left(\frac{n_h}{E_p I_p} \right)^{1/5} = 0.719 \quad (43)$$

$$nL = 0.719 * 44 = 31.743$$

In this case when $nL = 31.743$ and $e/L = 0$ (free-headed pile), from Figure 4.5.3.4.2:

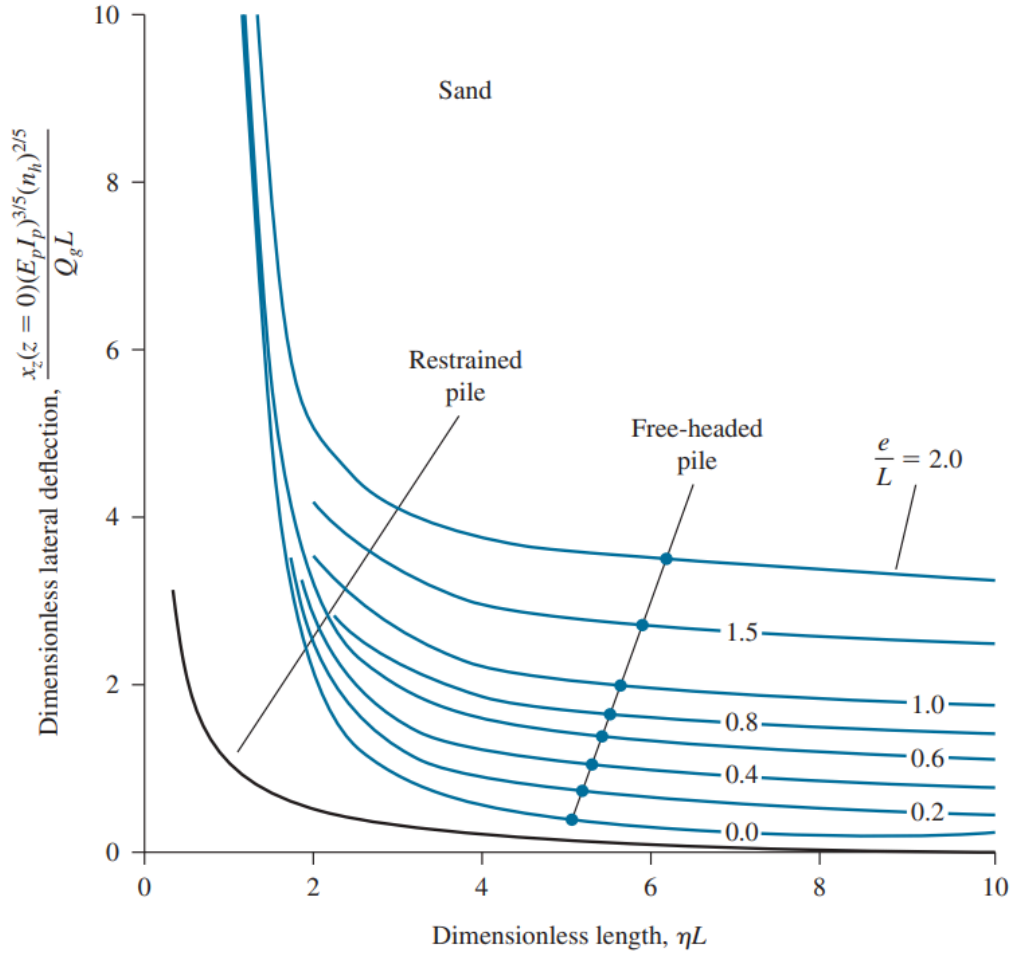


Figure 4.5.3.4.2 The solution of Broms for pile deflection in sand

$$\frac{x_0(E_p I_p)^{3/5}(n_h)^{2/5}}{Q_g L} = 0.095 \quad \text{by interpolation} \quad (44)$$

it follows that the pile deflection ($z=0$) according to the Broms method is equal to:

$$x_0 = \frac{0.095 Q_g L}{(E_p I_p)^{3/5}(n_h)^{2/5}} = \frac{0.095 \cdot 25 \cdot 10^3 \cdot 44}{(2.7806 \cdot 10^7 \cdot 0.003068)^{3/5} (16500)^{2/5}} = 2.47 \text{ mm} \quad (45)$$

The lateral deflection at the top of the pile was 2.47 mm, which is significantly lower than the limit value of 8 mm recommended for piles subjected to lateral loads. Thus, the pile meets the deflection requirements. The Broms method yields slightly higher deflection values than the elastic method, which produced a deflection of 1.91 mm.

As an independent cross-check on the two hand methods above, the same pile was analysed in GEO5 with the modulus of subgrade reaction K_h held piecewise constant within each soil layer. Figure 4.5.3.4.3 shows, from left to right, the pile geometry, the assigned K_h profile, and the computed distributions of lateral displacement, shear force, and bending moment along the full 44 m pile length.

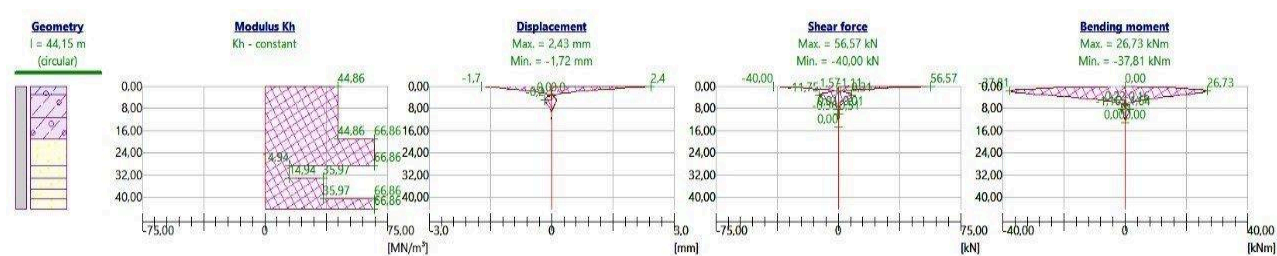


Figure 4.5.3.4.3 GEO5 distributions along the 44 m pile: subgrade reaction modulus K_h , lateral displacement, shear force, and bending moment

The GEO5 results confirm the earlier hand analyses. The maximum lateral displacement at the pile head is 2.43 mm, which sits between the elastic-solution value of 1.91 mm and the Broms value of 2.47 mm and is well below the 8 mm serviceability limit. The peak shear force (56.57 kN, at the pile head) occurs where the applied lateral load is introduced, and the peak bending moment (37.81 kNm) develops in the upper third of the pile where the lateral deflection is largest; both peaks are consistent with the expected response of a long flexible pile in a soft surface clay overlying stiffer layers. Below approximately 10 m depth, the displacement, shear, and moment distributions all decay to essentially zero, confirming that the lateral load is fully dissipated in the upper pile shaft and that the long-pile assumption used by the elastic and Broms methods ($L/T \gg 5$) is appropriate.

The summary table below clearly shows the results of pile cap deflection when using two different calculation methods and software.

Table 4.5.3.4.1 Comparison of deflections at the top of the pile.

Method of Calculation	Deflection x_z (mm)
-----------------------	---------------------

Elastic Solution Method	1.91
Method of Broms	2.47
Geo5 Software	2.43

There is also a summary table of the pile's lateral bearing capacity, which was calculated using two methods. Both checks showed that the lateral force acting on the pile does not exceed its limit value. The Broms method again yields slightly higher results

Table 4.5.3.4.2 Lateral load-bearing capacity according to two methods

Lateral Bearing Capacity	Elastic Solution Method	Method of Broms
Qg (kN)	25	25
Qg_max (kN)	190.24	208.47

4.5.4 Final Design of Pile Foundation

Table 4.5.4.1 Pile design comparison

L, m	D, m	Layout	Number of piles	$Q_{(allow)}$	S_{total}
36	0.61	Single	1	2692.05	113.92
38	0.61	Single	1	2948.89	118.78
40	0.61	Single	1	3398.43	110.95
42	0.61	Single	1	3681.35	115.29
44	0.4	5x4	20	30179.4	10.09
44	0.5	4x3	12	27046.3	9.90

The final pile diameter and layout were selected by directly comparing feasible pile group configurations that met the required bearing capacity and serviceability criteria. For the selected pile length of 44 m and spacing of 3.5D, two alternatives were considered: 0.40 m diameter piles arranged in a 5×4 group (20 piles) and 0.50 m diameter piles arranged in a 4×3 group (12 piles).

Both configurations provide group capacities that exceed the design demand. The 0.40 m option yields an allowable group capacity of 30,179.4 kN, while the 0.50 m option provides 27,046.3 kN. Since the required demand is satisfied in both cases, bearing capacity does not govern the final selection once adequacy is achieved.

The two layouts differ significantly in group performance. At a spacing of 3.5D, the group efficiency for 0.40 m piles is 0.84, indicating a reduction in mobilized resistance due to pile interaction. In contrast, the 0.50 m piles achieve a group efficiency of 1.00, meaning that the full individual pile capacity is effectively mobilized in group action. This indicates a more efficient and reliable transfer of axial resistance in the larger-diameter piles.

Serviceability considerations further support this selection. The calculated total settlements are small for both alternatives and well within acceptable limits. The 0.40 m pile group results in a total settlement of 10.09 mm, while the 0.50 m pile group results in a slightly lower settlement of 9.90 mm. Therefore, the increase in pile diameter does not lead to higher settlement and marginally improves serviceability performance.

Since both options satisfy capacity and settlement requirements, the final decision is governed by efficiency and constructability. The 0.50 m-diameter configuration requires only 12 piles, compared to 20 for the 0.40 m option. This reduction directly decreases drilling length, reinforcement quantity, concrete volume, and construction time, without compromising structural performance.

Based on adequate allowable capacity, full group efficiency, slightly lower settlement, and a substantial reduction in the number of piles, the final design adopts 44 m long, Ø500 mm drilled reinforced concrete piles arranged in a 4×3 group at 3.5D center-to-center spacing, the pile tip terminating at 49.30 m in the medium-stiff clay of Layer 9.. This configuration provides the most efficient and rational foundation solution for the project.

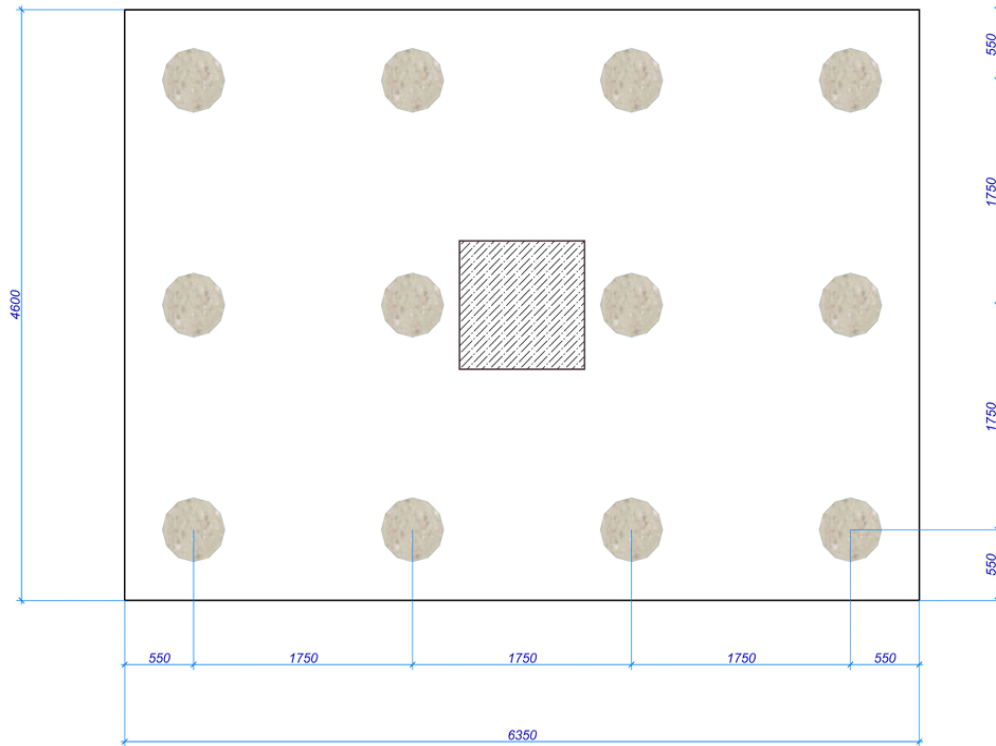


Figure 4.5.4.1 Final Group Pile layout



Figure 4.5.4.2 Cross-section of Foundation with connection to columns

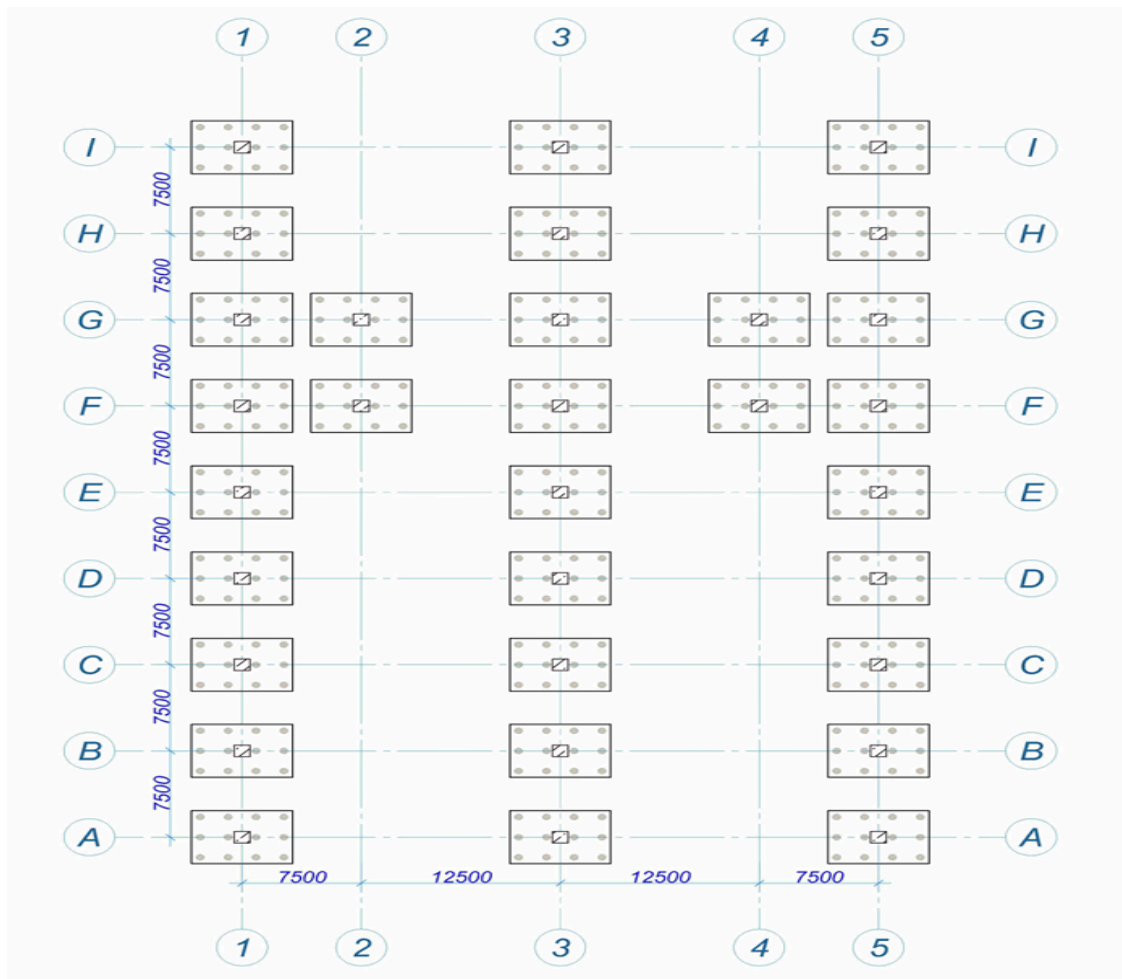


Figure 4.5.4.3 Layout of Pile Foundation

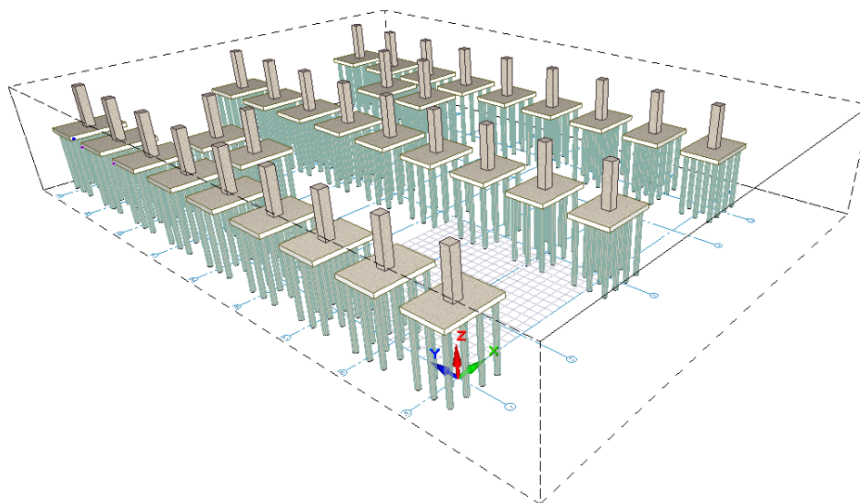


Figure 4.5.4.4 3D View on the Layout of Pile Foundation

4.5.5 Pile Cap Design

4.5.5.1 Individual Pile Cap

Table 4.5.5.1.1 General parameters and materials of pile cap

Parameter	Value
Concrete	C35/45 ($f_{ck} = 35$ MPa)
Reinforcement steel	A500C ($f_{yk} = 520$ MPa)
Pile cap dimensions	$6350 \times 4600 \times 1650$ mm
Bottom cover	75 mm
Top cover	50 mm
Pile diameter, D	500 mm
Column cross-section, A _c	1050×1050 mm

First, you need to know the vertical design load per column.

$$P_{col} = 21920 \text{ kN}$$

Next, we calculate the self-weight of the pile cap and then factored self-weight.

$$W_{cap} = LBh\gamma_{conc} = 6.35 * 4.60 * 1.65 * 25 = 1205 \text{ kN} \quad (46)$$

$$W_{cap,Ed} = 1.35 * 1205 = 1627 \text{ kN}$$

There is no soil layer above the slab, so the additional weight of the soil is not taken into account. In this case, the total vertical load acting on the pile group is equal to:

$$P_{tot} = P_{col} + W_{cap,Ed} = 21920 + 1627 = 23547 \text{ kN}$$

The next step is to determine the wind forces at the level of the foundation footing:

$$M_x = F_y h_{arm} = 182.109 * 5.3 = 965.18 \text{ kN} * m \quad (47)$$

$$M_y = F_x h_{arm} = 255.268 * 5.3 = 1352.92 \text{ kN} * m$$

Next, we calculate the second moments of area for the group of piles. To do this, we need to know the coordinates of the centers of the piles relative to the origin.

- x - coordinates: -2625 mm, -875 mm, + 875 mm, + 2625 mm
- y - coordinates: -1750 mm, 0 mm, + 1750 mm

$$I_{xx} = \sum y_i^2 = 4 * ((- 1.75)^2 + 0 + (1.75)^2) = 4 * 6.125 = 24.5 \text{ m}^4 \quad (48)$$

$$I_{yy} = \sum x_i^2 = 3 * ((- 2.625)^2 + (- 0.875)^2 + (0.875)^2 + (2.625)^2) = 3 * 15.31 = 45.94 \text{ m}^4$$

Then it's time to figure out the axial force on a single pile using the formula below:

$$Q_i = \frac{P_{tot}}{n} \pm \frac{M_x * y}{I_{xx}} \pm \frac{M_y * x}{I_{yy}} \quad (49)$$

Corner pile (x = +2.625, y = +1.75):

$$Q_1 = 1962.25 + 68.94 + 77.30 = 2108.49 \text{ kN}$$

Pile on the Y-axis (x=0, y=+1.75):

$$Q_2 = 1962.25 + 68.94 + 0 = 2031.19 \text{ kN}$$

Pile at the point (x=+2.625, y=0):

$$Q_3 = 1962.25 + 0 + 77.3 = 2039.55 \text{ kN}$$

Central pile (x=0, y=0):

$$Q_4 = 1962.25 \text{ kN}$$

Now we can find the moment relative to the cross-section along the edge of the column:

$$M_{xx}'' = \sum Q_i(x_i - 0.4) = 2832.93 + 13615.50 = 16448.43 \text{ kN} * m \quad (50)$$

$$M_{yy}'' = \sum Q_i * 1.35 = 8124.76 * 1.35 = 10968.43 \text{ kN} * m \quad (51)$$

After that, determine the moment due to the self-weight of the pile cap:

$$M_{xx}' = B * w_{cap} * L^2/2 = 4.6 * 55.69 * (2.7752)/2 = 986.2 \text{ kN} * m \quad (52)$$

$$M_{yy}' = B * w_{cap} * L^2/2 = 6.35 * 55.69 * (1.9)^2/2 = 638.36 \text{ kN} * m$$

In this case, the total design moments for the reinforcement in the X and Y directions are equal to:

$$M_{x,Ed} = M_{xx}'' + M_{xx}' = 16448.43 + 986.2 = 17434.6 \text{ kN} * m \quad (53)$$

$$M_{y,Ed} = M_{yy}'' + M_{yy}' = 10968.43 + 638.36 = 11606.79 \text{ kN} * m$$

Next, we need to calculate the minimum thickness of the pile cap based on the buckling conditions using the formula below:

$$\phi * 0.33 * \lambda * \sqrt{f_c} * b_0 * d \geq P_u \quad (54)$$

$$0.75 * 0.33 * 5.92 * 10^6 * 4(1.05 + d) * d \geq 21920 * 10^3$$

The required working height is: $d = 1.48 \text{ m}$. Then, assuming that the protective layer will be 75 mm thick and the lower reinforcement will have a diameter of 32 mm, the total height of the pile cap will be:

$$h = d + 0.075 + 0.016 = 1.48 + 0.091 = 1.57 \text{ m}.$$

However, to ensure the required reserve thickness and additional concrete strength, it was decided to adopt a pile cap thickness of 1.65 meters. In this case, we can calculate the effective depth for the bottom reinforcement in the X and Y directions.

$$d_x = h - 75 - \frac{d_{b,x}}{2} = 1650 - 75 - 16 = 1559 \text{ mm}$$

$$d_y = h - 75 - d_{b,x} - \frac{d_{b,y}}{2} = 1650 - 75 - 32 - 16 = 1527 \text{ mm}$$

After calculating the moments and effective depths, it is necessary to proceed with the selection of longitudinal reinforcement:

$$R_n = \frac{M_{x,Ed}}{\phi b d^2} = \frac{17435 \cdot 10^6}{0.9 \cdot 4600 \cdot (1559)^2} = 1.732 \text{ MPa} \quad (55)$$

$$\rho = \frac{0.85 \cdot f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85 \cdot f'_c}}\right) = \frac{29.75}{520} \left(1 - \sqrt{1 - \frac{2R_n}{0.85 \cdot f'_c}}\right) = 0.003433 \quad (56)$$

$$A_s = \rho b d = 0.003433 \cdot 4600 \cdot 1559 = 24620 \text{ mm}^2 \quad (57)$$

Then, minimum reinforcement must also be calculated:

$$A_{min} = 0.0018 \cdot 4600 \cdot 1650 = 13662 \text{ mm}^2 < 24620 \text{ mm}^2$$

It has been decided to select reinforcement bars with a diameter of 32 mm and a cross-sectional area of 804 mm^2 . In this case, the number of bars will be:

$$\frac{24620}{804} = 31 \text{ bars}$$

The spacing between the rods will be: $\frac{4600}{31} = 148 \text{ mm}$. Accepting 31 $\varnothing$ 32@150 mm.

The following values were found in the Y direction:

$$R_n = \frac{11607 \cdot 10^6}{0.9 \cdot 6350 \cdot (1527)^2} = 0.871 \text{ MPa}$$

$$\rho = 0.0572 \cdot 0.0297 = 0.001699$$

$$A_s = 0.001699 \cdot 6350 \cdot 1527 = 16483 \text{ mm}^2$$

When checking the minimum reinforcement, the area value was:

$$0.0018 * 6350 * 1650 = 18860 \text{ mm}^2$$

Number of bars $\phi 32$: $\frac{18860}{804} = 24$. However, for more effective distribution, it is better to choose 26 bars. The spacing between the bars will be: $\frac{6350}{26} = 244 \text{ mm}$. Accepting $26\phi 32@244 \text{ mm}$.

All final decisions and reinforcement information of the pile cap have been compiled in the table below.

Table 4.5.5.1.2 Summary information on bottom reinforcement

Direction	Effective depth (mm)	A _s required (mm ²)	A _s minimum (mm ²)	Selected reinforcement	Spacing (mm)
X (along a width of 4600 mm)	1559	24620	13662	31 $\phi 32$	150
Y (along a length of 6350 mm)	1527	16483	18860	26 $\phi 32$	244

4.5.5.2 Combined Pile Cap

To avoid separate pile caps placed too close to each other, a single combined pile cap is designed to support two interior columns. The spacing of the columns is 7500 centre-to-centre. The same factored axial load is transferred by each column; and the same pile group supports it. The combined cap is regarded to be stiff and to redistribute the loads evenly to the piles.

Table 4.5.5.2.1 General parameters and materials of pile cap

Parameter	Value
Concrete	C35/45 ($f_{ck} = 35 \text{ MPa}$)
Reinforcement steel	A500C ($f_{yk} = 520 \text{ MPa}$)

Pile cap dimensions	13850 × 4600 × 1650 mm
Bottom cover	75 mm
Top cover	50 mm
Pile diameter, D	500 mm
Column cross-section, A _c	1050×1050 mm

First, it is necessary to calculate the self-weight of the pile cap and then factored self-weight.

$$W_{cap} = LBh\gamma_{conc} = 13.85 * 4.60 * 1.65 * 25 = 2628 \text{ kN}$$

$$W_{cap,Ed} = 1.35 * 2628 = 3548 \text{ kN}$$

In this case, the total vertical load acting on the pile groups is equal to:

$$P_{tot} = 2 * P_{col} + W_{cap,Ed} = 2 \times 21920 + 3548 = 47388 \text{ kN}$$

After this, the distributed self-weight per unit area and line load per metre length is found:

$$w_{cap} = 1.35 \times 25 \times 1.65 = 55.69 \text{ kN/m}^2$$

$$w_{line} = 55.69 \times 4.60 = 256.2 \text{ kN/m}^2$$

The combined cap is very stiff; therefore the total vertical load and the wind moments are distributed equally to the two pile groups. Each group receives:

$$P_{group} = P_{tot}/2 = 23694 \text{ kN}$$

$$M_x = 965.18/2 = 482.6 \text{ kN} * m$$

$$M_y = 1352.92/2 = 676.5 \text{ kN} * m$$

Using the same coordinates of the piles relative to the center, the maximum load will be determined:

$$Q_{max} = \frac{23964}{12} + \frac{676.5*2.625}{45.94} + \frac{482.6*1.75}{24.5} = 2047.7 \text{ kN}$$

This value is very close to that obtained for the individual cap, confirming that the transverse bending (across the width) is practically unchanged. Hence, the transverse reinforcement can be adopted from the individual cap design.

The combined cap is modeled as a continuous beam on two supports. The overhanging ends are 3175 mm long. The uniform load 256.2 kN/m acts over the whole length. The column loads are directly distributed over the supports which means that they do not provide any contribution to the bending moment in the span. Now the positive moment for calculating the bottom reinforcement will be determined:

$$M_{pos} = 256.2 * (7.5)^2 / 8 = 1801 \text{ kN} * \text{m}$$

A negative point to consider when calculating the top reinforcement:

$$M_{neg} = 1.2 * 256.2 * (3.175)^2 / 2 = 1550 \text{ kN} * \text{m}$$

Longitudinal Reinforcement Design

Now, the effective depth for the top and bottom reinforcement can be estimated.

$$d_{bottom} = h - 75 - \frac{25}{2} = 1650 - 75 - 12.5 = 1562.5 \text{ mm}$$

$$d_{top} = 1650 - 50 - 10 = 1590 \text{ mm}$$

After calculating the moments and effective depths, it is necessary to proceed with the selection of longitudinal reinforcement:

$$R_n = \frac{M_{pos}}{\phi b d^2} = \frac{1801*10^6}{0.9*4600*(1562.5)^2} = 0.178 \text{ MPa}$$

$$\rho = \frac{0.85*f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85*f'_c}}\right) = \frac{29.75}{520} \left(1 - \sqrt{1 - \frac{2R_n}{0.85*f'_c}}\right) = 0.000396$$

$$A_s = \rho b d = 0.000396 * 4600 * 1562.5 = 2846 \text{ mm}^2$$

Then, minimum reinforcement must also be calculated:

$$A_{min} = 0.0018 * 4600 * 1650 = 13662 \text{ mm}^2 > 2846 \text{ mm}^2$$

It has been decided to select reinforcement bars with a diameter of 25 mm and a cross-sectional area of 491 mm^2 . In this case, the number of bars will be:

$$\frac{13662}{491} = 28 \text{ bars}$$

The spacing between the rods will be: $\frac{4600}{28} = 164 \text{ mm}$. Accepting $28\varnothing 25@165 \text{ mm}$.

The following values were found for the bottom design:

$$R_n = \frac{1550*10^6}{0.9*4600*(1590)^2} = 0.148 \text{ MPa}$$

$$\rho = 0.04388 * 0.0075 = 0.000329$$

$$A_s = 0.000329 * 4600 * 1590 = 2406 \text{ mm}^2$$

When checking the minimum reinforcement, the area value was:

$$A_{min} = 13662 \text{ mm}^2$$

Provide reinforcement consisting of 28 bars with a diameter of 25 mm at the location above each support, with an overhang of $0.25 \times 7500 = 1875 \text{ mm}$ on each side of the support center

All final decisions and reinforcement information of the pile cap have been compiled in the table below.

Table 4.5.5.2.2 Summary information on bottom reinforcement

Direction	Effective depth (mm)	A _s required (mm ²)	A _s minimum (mm ²)	Selected reinforcement	Spacing (mm)
Bottom X (along a width of 4600 mm)	1559	24620	13662	31Ø32	150
Bottom Y (length 13850 mm)	1562.5	2846	13662	28Ø25	165
Top – longitudinal over supports	1590	2406	13662	28Ø25	165

4.5.6 Single Pile Reinforcement

To complement the geotechnical capacity and pile cap design presented in the preceding sections, a structural reinforcement design was carried out for each individual pile in the 4×3 group. The design follows ACI 318-19 provisions for cast-in-place drilled concrete piles subjected to combined axial compressive loading and negative skin friction (NSF) in low-seismicity conditions. The general design parameters are summarised in Table 4.5.6.1.

Table 4.5.6.1 Single pile design parameters

Parameter	Value
Pile type	Drilled cast-in-place reinforced concrete
Pile diameter (D)	500 mm
Pile length	44 m
Pile group layout	4×3 (12 piles total)
Column factored axial load (P _{col})	21,920 kN

Parameter	Value
Factored pile cap self-weight ($W_{cap,Ed}$)	1,627 kN
Negative skin friction per pile	567 kN
Design code	ACI 318-19
Concrete strength (f'_c)	35 MPa (C35/45)
Steel yield strength (f_y)	520 MPa (A500C)
Concrete cover (side)	75 mm
Seismicity	Low

Factored Design Load per Pile

The total structural load transferred to the pile group consists of the factored column load and the factored self-weight of the pile cap. This combined load is shared equally among the 12 piles of the 4×3 group:

$$P_{\text{structural/pile}} = (P_{\text{col}} + W_{\text{cap,Ed}}) / n = (21,920 + 1,627) / 12 = 1,962 \text{ kN per pile}$$

In addition to the structural load, negative skin friction (NSF) develops when surrounding soft soils consolidate and drag the pile downward. NSF acts as an additional downward force applied directly to each pile and must be combined with the structural load for the purposes of pile structural design (ACI 318-19). The NSF load on each pile is:

$$P_{\text{NSF}} = 567 \text{ kN per pile}$$

Therefore, the total factored design load acting on a single pile is:

$$P_{\text{pile}} = P_{\text{structural/pile}} + P_{\text{NSF}} = 1,962 + 567 = 2,529 \text{ kN per pile}$$

Cross-Sectional Properties

The gross cross-sectional area of the 500 mm diameter pile is:

$$A_g = \pi \times D^2 / 4 = \pi \times 500^2 / 4 = 196,350 \text{ mm}^2$$

With a concrete cover of 75 mm on all sides and a spiral wire diameter of 16 mm, the confined core diameter and corresponding core area are:

$$d_{\text{core}} = 500 - 2 \times 75 = 350 \text{ mm}$$

$$A_{\text{core}} = \pi \times 350^2 / 4 = 96,211 \text{ mm}^2$$

4.5.6.1 Longitudinal Reinforcement

Per ACI 318-19 §26.6.2.2, the minimum longitudinal steel ratio for piles primarily subjected to axial compression is $\rho = 0.005$ (0.5%). The minimum required steel area is therefore:

$$A_{s,\text{min}} = 0.005 \times A_g = 0.005 \times 196,350 = 982 \text{ mm}^2$$

A structural demand check is carried out using the ACI 318-19 nominal capacity equation for spiral-confined sections (§22.4.2), with a strength reduction factor $\phi = 0.75$:

$$A_{s,\text{req}} = (P_{\text{pile}} / \phi - 0.85 \cdot f'_c \cdot A_g) / (f_y - 0.85 \cdot f'_c) \quad (58)$$

$$A_{s,\text{req}} = (2,529,000 / 0.75 - 0.85 \times 35 \times 196,350) / (520 - 0.85 \times 35)$$

$$A_{s,\text{req}} = (3,372,000 - 5,851,413) / 490.25 = -5,057 \text{ mm}^2 \text{ [negative} \rightarrow \text{minimum governs]}$$

The negative result confirms that the concrete section alone is more than sufficient to sustain the factored axial load including NSF, and minimum reinforcement governs the design. Eight longitudinal bars of Ø35 mm are selected, providing a total area of 7,697 mm² and a steel ratio of 3.92%, comfortably satisfying the ACI 318-19 minimum of 0.50%. Eight bars are used to achieve symmetry and ensure compatibility with the spiral reinforcement cage; this selection is consistent with the structural analysis performed in GEO5, as shown in Table 4.5.6.1.1.

Table 4.5.6.1.1 Longitudinal reinforcement summary

Parameter	Value
Total factored design load (P _{pile})	2,529 kN
Structural component	1,962 kN (column + cap weight / 12 piles)
Negative skin friction	567 kN (per pile)
Gross area (A _g)	196,350 mm ²
Minimum steel area (A _{s,min})	982 mm ² (ACI 318-19 §26.6.2.2)
Structural demand (A _{s,req})	Negative → minimum governs
Selected longitudinal bars	8 × Ø35 mm
Area provided (A _{s,prov})	7,697 mm ²
Steel ratio (ρ)	3.92% ≥ 0.50% ✓

4.5.6.2 Transverse (Spiral) Reinforcement

Spiral reinforcement is adopted in preference to individual ties, as it provides superior ductility and confinement. This is particularly important for long slender piles in clay, where lateral deformation and installation stresses may induce bending. The required volumetric spiral ratio is calculated in accordance with ACI 318-19 §25.7.3:

$$\rho_s = 0.45 \times (A_g/A_{core} - 1) \times f_c / f_{yt} \quad (59)$$

$$\rho_s = 0.45 \times (196,350 / 96,211 - 1) \times 35 / 520$$

$$\rho_s = 0.45 \times 1.041 \times 0.0673 = 3.15\%$$

A Ø16 mm spiral wire is selected, consistent with the GEO5 software output. Its cross-sectional area is A_{sp} = 201 mm². The required spiral pitch to satisfy confinement is:

$$s = 4 \times A_{sp} / (\rho_s \times d_{core}) = 4 \times 201 / (0.0315 \times 350) = 73 \text{ mm} \quad (60)$$

A pitch of 70 mm is adopted in the critical confinement zones at the pile head and toe, providing a volumetric spiral ratio of:

$$p_{s,prov} = 4 \times A_{sp} / (s \times d_{core}) = 4 \times 201 / (70 \times 350) = 3.28\% \geq 3.15\%$$

In the mid-length zone, where moment and shear demands are significantly reduced, a relaxed pitch of 200 mm (Ø16@200) is adopted for constructability, matching the shear reinforcement configuration verified by GEO5. Pitch limits per ACI 318-19 §25.7.3.2 (clear spacing between 25 mm and 80 mm) are satisfied in the critical zones.

Table 4.5.6.2.1 Spiral reinforcement zones along pile length

Zone	Length	Spiral reinforcement	Rationale
Pile head (top)	1.5 m (3×D)	Ø16 @ 70 mm	Maximum moment; pile cap connection zone; confinement required
Mid-length	41 m	Ø16 @ 200 mm	Reduced shear and moment demand; constructability
Pile toe (bottom)	1.5 m (3×D)	Ø16 @ 70 mm	Installation and driving stresses; confinement required

4.5.6.3 Shear and Bending Capacity Check

The GEO5 lateral analysis of the embedded pile yields the following peak internal forces (Table 4.5.6.3.1):

Table 4.5.6.3.1 Peak internal forces from GEO5 lateral analysis

Internal force	Value
Maximum shear force	56.57 kN
Maximum bending moment	37.81 kNm
Maximum pile head displacement	2.4 mm

The concrete shear capacity is evaluated per ACI 318-19 §22.5.5. For a circular pile section, an effective shear width $b_w = 500$ mm and an effective depth $d \approx 0.8D = 400$ mm are used:

$$\phi V_c = \phi \times 0.17 \times \lambda \times \sqrt{f_c} \times b_w \times d \quad (61)$$

$$\phi V_c = 0.75 \times 0.17 \times 1.0 \times \sqrt{35} \times 500 \times 400 / 1,000 = 150.9 \text{ kN}$$

Since the maximum factored shear force (56.57 kN) is well below the concrete capacity (150.9 kN), the transverse spiral reinforcement provided for confinement is more than adequate to resist shear without additional stirrups. No supplementary shear reinforcement is required beyond the confinement spiral.

4.5.6.4 Pile Capacity Verification

The design axial capacity of the pile section is verified using the ACI 318-19 §22.4.2 strength equation for spiral-confined compression members:

$$\phi P_n = \phi \times [0.85 \cdot f'_c \cdot (A_g - A_s) + f_y \cdot A_s] \quad (62)$$

$$\phi P_n = 0.75 \times [0.85 \times 35 \times (196,350 - 7,697) + 520 \times 7,697]$$

$$\phi P_n = 0.75 \times [0.85 \times 35 \times 188,653 + 4,002,440]$$

$$\phi P_n = 0.75 \times [5,606,876 + 4,002,440] = 0.75 \times 9,609,316 = 7,211 \text{ kN}$$

The design capacity of 7,211 kN substantially exceeds the total factored pile load of 2,529 kN, yielding a utilisation ratio of 35.1%. This significant reserve reflects the fact that minimum reinforcement governs the design, which is consistent with typical practice for large-diameter bored piles where geotechnical rather than structural capacity controls pile sizing. Notably, even when negative skin friction is included, the concrete section remains more than adequate; the increased demand from NSF is accommodated without any change to the governing design criterion.

Table 4.5.6.4.1 Pile structural capacity verification

Check	Value	Limit
Factored pile load	2,529 kN	—
Structural component	1,962 kN	—
Negative skin friction	567 kN	—

Check	Value	Limit
Design capacity (ϕP_n)	7,211 kN	$\geq 2,529$ kN
Utilisation ratio	35.1%	$\leq 100\%$
Longitudinal steel ratio (ρ)	3.92%	$\geq 0.50\%$
Spiral ratio (ρ_s, ρ_{prov})	3.28%	$\geq 3.15\%$
Shear: ϕV_c	150.9 kN	≥ 56.57 kN
Max pile displacement	2.4 mm	Acceptable

The final reinforcement selection and all three ACI 318-19 structural checks (shear, bending + compression, and reinforcement ratio) are shown in Figure 4.5.6.4.1 as output by GEO5.

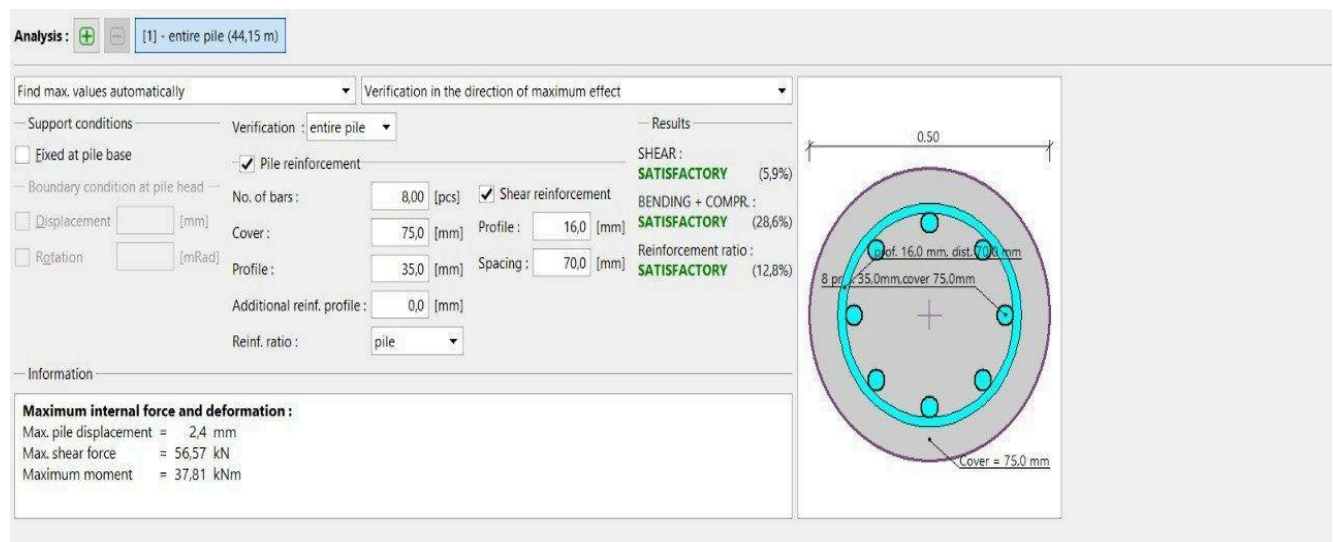


Figure 4.5.6.4.1 Geo5 single-pile reinforcement verification screen

The very low utilisation ratios confirm the conclusion reached in Sections 4.5.6.1 and 4.5.6.4: the pile section is dictated by geotechnical rather than structural demand, and minimum-reinforcement rules govern the design. The $8 \times \text{Ø}35$ mm arrangement is retained primarily to ensure symmetry and bar-spiral compatibility, not because structural capacity requires it.

4.5.6.5 Design Summary

The structural reinforcement design for the individual pile is summarised in Table 4.5.6.5.1. All ACI 318-19 requirements for minimum reinforcement, confinement, and axial capacity are satisfied. The negative skin friction of 567 kN per pile has been explicitly accounted for in the factored design load. The 4×3 pile group comprising 44 m long, 500 mm diameter drilled reinforced concrete piles is confirmed as structurally adequate under the combined applied loading.

Table 4.5.6.5.1 Final single pile reinforcement design summary

Item	Design Value
Pile dimensions	Ø500 mm × 44,000 mm
Concrete	$f_c = 35 \text{ MPa}$ (C35/45)
Steel	$f_y = 520 \text{ MPa}$ (A500C)
Total factored column load (P_{col})	21,920 kN
Factored pile cap self-weight (W_{cap} , E_d)	1,627 kN
Negative skin friction per pile (PNSF)	567 kN
Structural load per pile	1,962 kN
Total factored design load per pile (P_{pile})	2,529 kN
Longitudinal reinforcement	8 × Ø35 mm ($A_s = 7,697 \text{ mm}^2$)
Longitudinal steel ratio (ρ)	3.92% $\geq$ 0.50%
Confinement spiral (head /toe zones, 1.5 m each)	Ø16 @ 70 mm
Spiral (mid-length, 41 m)	Ø16 @ 200 mm
Volumetric spiral ratio (confinement zones)	3.28% $\geq$ 3.15% ✓
Concrete cover	75 mm

Item	Design Value
Design capacity (ϕP_n)	7,211 kN
Utilisation ratio	35.1%
Max shear / ϕV_c	56.57 kN / 150.9 kN ✓
Max moment	37.81 kNm
Max pile head displacement	2.4 mm

4.5.7 Software Analysis of Pile Groups

To verify the hand calculation and obtain a more realistic representation of near-real-world soil-structure interaction, a three-dimensional finite element model of the foundation system was developed and tested in Geo5 in Plaxis3D Ultimate Software. The main tasks of the research were to evaluate the settlement experienced by a single pile group under the predetermined design loading, to examine the distribution of the axial forces within the piles that constitute the pile group, and to confirm the fact that the selected length of pile and that of the cap are actually satisfactory.

4.5.7.1 Single Pile Group Model

Plaxis 3D

The computational model dimensions were deliberately large to reduce boundary artifacts: X ranged from -50 to +50 m, Y likewise from -50 to +50 m, and Z from -65 m (well below the tip of the piles) to +5 m above ground level.

The soil stratigraphy was divided into 10 homogeneous horizontal layers, with material properties derived from the site investigation. Moore-Coulomb constitutive modeling was used to simulate each clay layer, while the Hardening Soil model was used for sands. The clay layers were prescribed undrained behaviour, while the sand layers were assumed to exhibit overall drained behaviour. The groundwater level was pegged at -50.3m, significantly below the planned excavation level, and hence, seepage was not factored into the analysis.

The piles were modelled as embedded beam elements with a circular cross-section of 0.5m diameter and a length of 44 m, extending from the base of the pile cap at -5.30 m to the tip

at -49.3m. Their elastic modulus was modified to 33 GPa. A plate element with dimensions of 6.35m x 4.60m x 1.65m, an elastic modulus of 30 GPa, Poisson ratio 0.2, and unit weight of 24 kN/m³ were used to model the pile cap. The basement walls and slab, which are 0.33 m and 0.35 m thick, respectively, were also modeled as plate elements to capture their stiffness contributions.

General		Mechanical	
Property		Unit	Value
Properties			
Cross section type		Predefined ▼	
Predefined cross section type		Solid circular beam ▼	
Diameter	m	0,50000	
A	m ²	0,19635	
I ₂	m ⁴	3,0680E-3	
I ₃	m ⁴	3,0680E-3	
Stiffness			
E	kN/m ²	33,000E6	
Axial skin resistance			
Axial skin resistance		Linear ▼	
T _{skin, start, max}	kN/m	33,800	
T _{skin, end, max}	kN/m	3513,0	
Base resistance			
F _{max}	kN	0,0000	

Figure 4.5.7.1.1 Embedded beam material properties

General		
Mechanical		
Property	Unit	Value
Properties		
Isotropic		☒
Stiffness		
E_1	kN/m ²	33,000E6
E_2	kN/m ²	33,000E6
ν_{12}		0,20000
d	m	1,6500
G_{12}	kN/m ²	13,750E6
G_{13}	kN/m ²	13,750E6
G_{23}	kN/m ²	13,750E6

Figure 4.5.7.1.2 Plate material properties

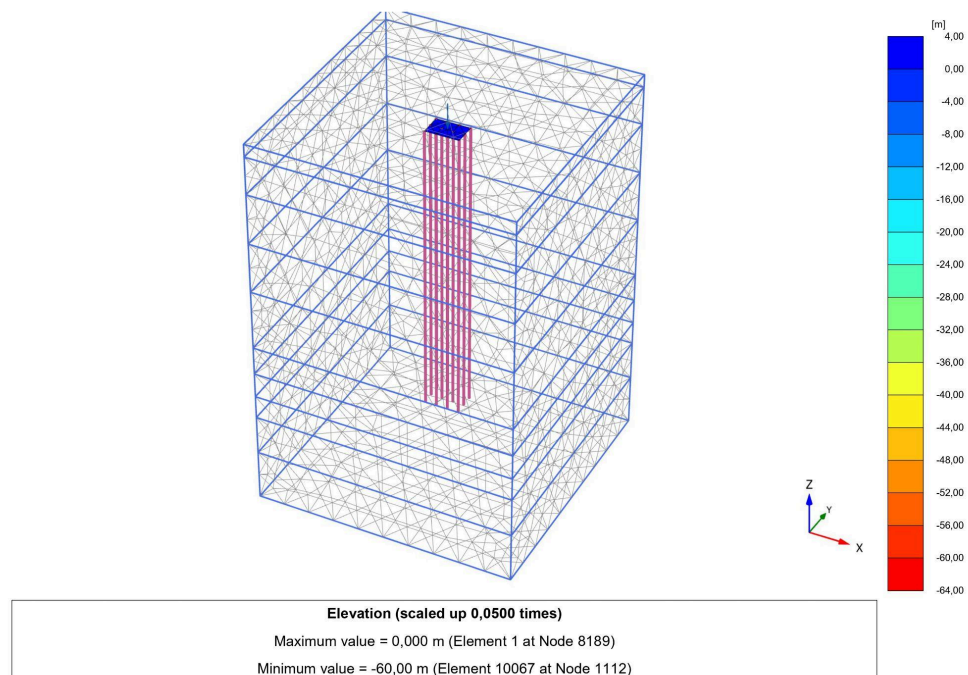


Figure 4.5.7.1.3 General view of a 3D model of a group of piles

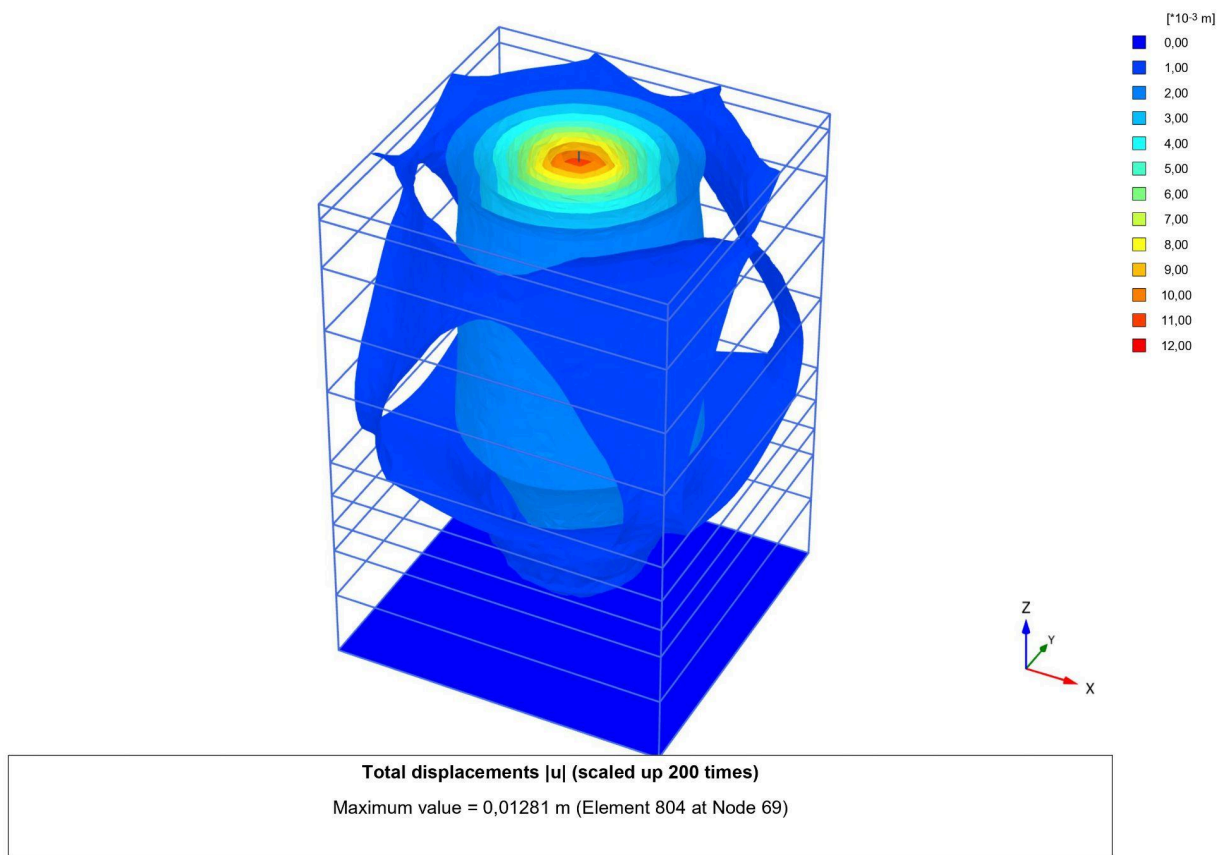


Figure 4.5.7.1.4 Settlement value of pile group

The computed settlement in the middle of the pile group under the given load was 12.8mm. This value is consistent with the hand-calculated estimate of 10mm, indicating that the operational capacity threshold, normally set at 25mm, is satisfied by a large margin.

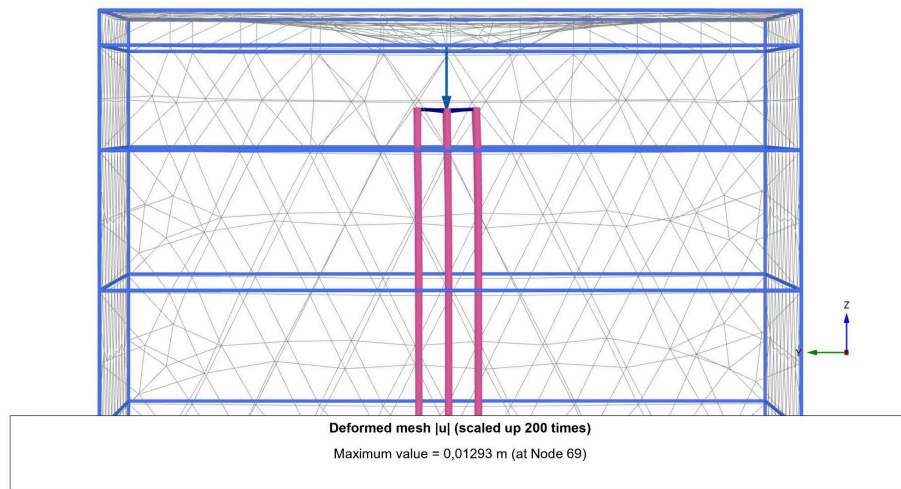


Figure 4.5.7.1.5 Deviation of pile heads along the Y axis

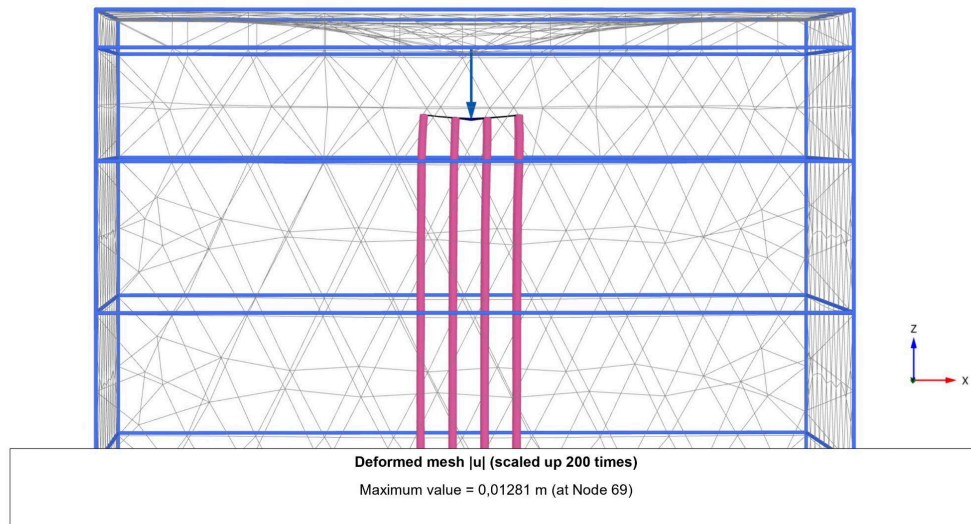


Figure 4.5.7.1.6 Deviation of pile heads along the X axis

Geo5

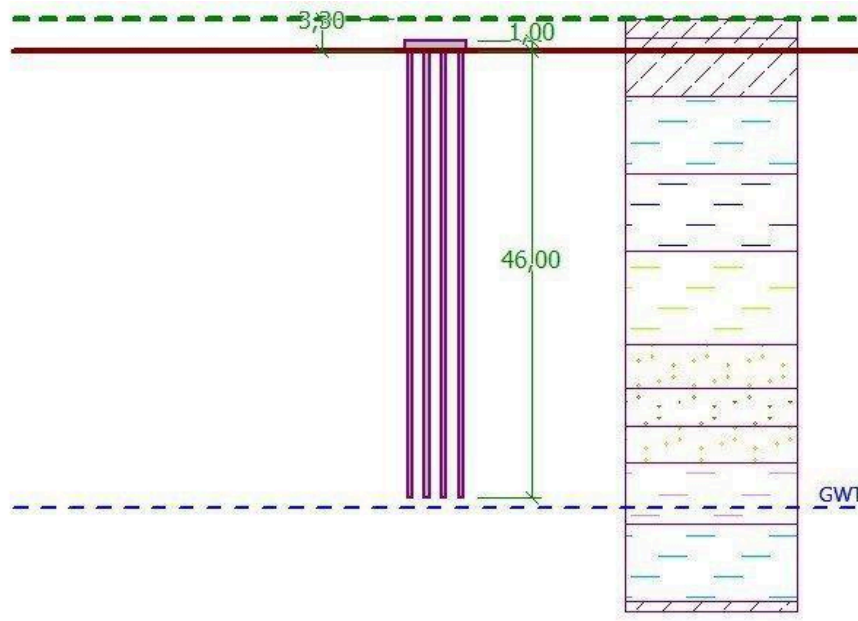


Figure 4.5.7.1.7 Geometry of the single-pile lateral-load model in GEO5Single Pile Group Model

The single pile group, comprising 12 piles (4x3 group) of 0.5 m diameter and 44 m length, was modeled in the GEO5 Pile Group analysis module. This model was used to verify the pile interaction and group performance under the combined axial and lateral loading derived from the structural analysis. The soil stratigraphy and parameters used were consistent with the hand calculations, including the use of an elastic subgrade reaction approach to model soil stiffness. The GWT dashed line at the bottom of the figure confirms that groundwater does not interact with the upper 50 m of the profile and therefore does not affect the lateral response in the zone of maximum pile deflection

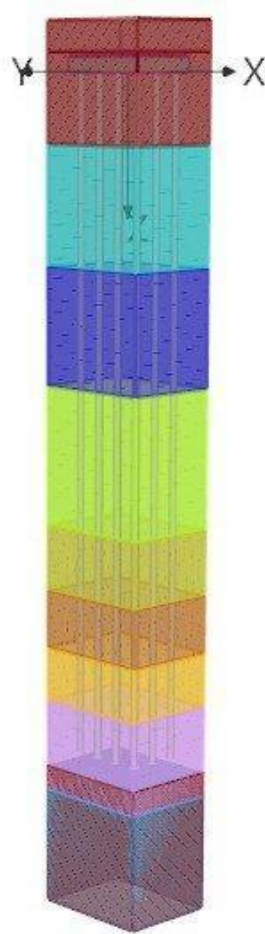


Figure 4.5.7.1.8 Three-dimensional view of the pile group and layered soil column

Analysis of bearing capacity of pile group in cohesive soils

Max. vertical force includes self-weight of pile cap.

Average undrained shear strength along the piles $c_{us} = 25,98 \text{ kPa}$

Undrained shear strength at base of pile group $c_{ub} = 60,00 \text{ kPa}$

Cohesion group bearing capacity factor $N_{cg} = 8,59$

Vertical bearing capacity of pile group $R_g = 43097,41 \text{ kN}$

Maximum vertical force $V_d = 25725,56 \text{ kN}$

$R_g = 43097,41 \text{ kN} > 25725,56 \text{ kN} = V_d$

Vertical bearing capacity of pile group is SATISFACTORY

Figure 4.5.7.1.10 Bearing Capacity of the pile group verification

4.5.7.2 Full Foundation System Model

Moreover, in addition to the analysis of a single group, the full foundation system, including all 35 pile groups, the basement walls, and the slab, was modeled (Figure 4.5.7.2.1). The global model ensures that the development of the boundary conditions and overall stiffness is well determined, even though, because the behaviors are identical across all groups, a systematic group-by-group analysis was not considered necessary. The short distance between the groups does not cause excessive stress on the ground, as the piles are long and the loads are well balanced, thereby stabilizing the situation.

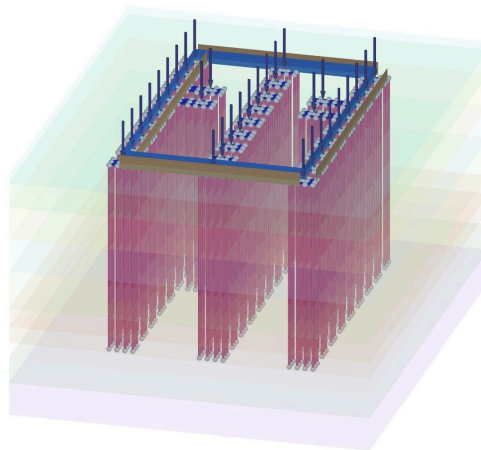


Figure 4.5.7.2.1 General 3D view of the pile field

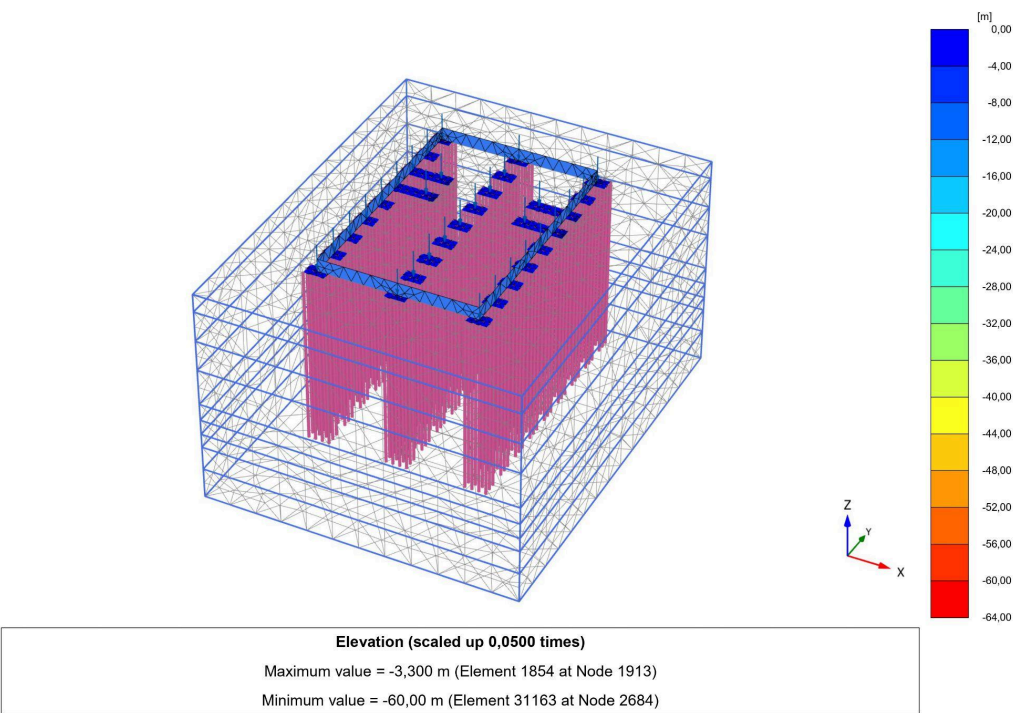


Figure 4.5.7.2.2 General 3D view of the mesh model showing elevation

ID		Calculation t...	Loading type...	Pore pressur...	Time interval	Estimated en...	Ignore undr...	Reset displa...	Updated me...	Max steps (D)	First step	Last step
Initial phase [InitialPhase]					0,0000 day	0,0000 day	☒	☐	☐	1000	0	0
Excavation of soil within the excavati					0,0000 day	0,0000 day	☐	☒	☐	1000	1	4
Pile installation [Phase_2]					0,0000 day	0,0000 day	☐	☐	☐	1000	5	7
Installation of pile caps [Phase_3]					0,0000 day	0,0000 day	☐	☐	☐	1000	8	16
Installation of retaining walls [Phase_					0,0000 day	0,0000 day	☐	☐	☐	1000	17	19
Loads from the building [Phase_5]					0,0000 day	0,0000 day	☐	☐	☐	1000	20	25

Figure 4.5.7.2.3 Construction Stages

Results of the full model:

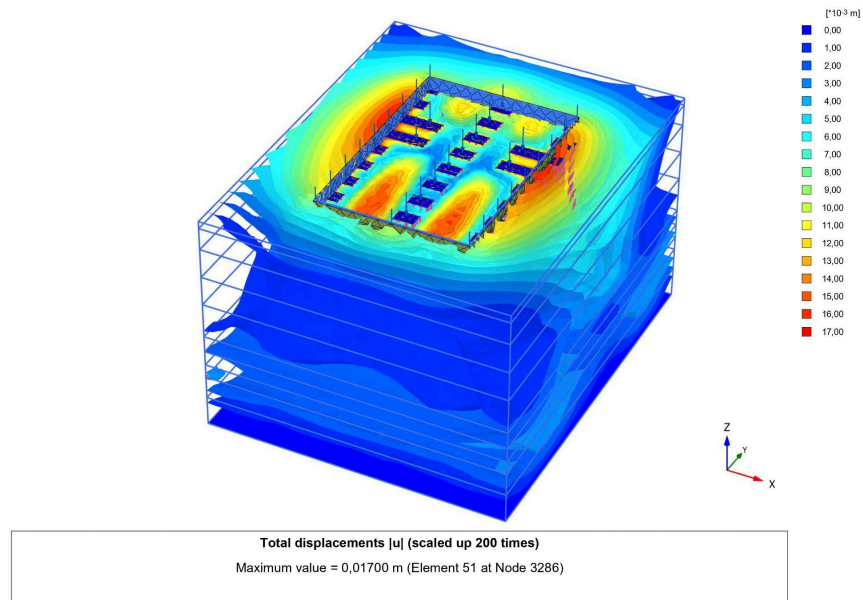


Figure 4.5.7.2.4 Settlement value of pile groups

The maximum settlement obtained from the computational model is observed at the points where the pile caps are joined, where this value reaches 17 mm. This value is slightly greater than the settlement of a single pile group (12.8 mm), since the pile groups are located closest to each other at this point and therefore exert an additional load on the soil. However, it can be noted that the increase in settlement is quite small, thanks to the significant length of the piles (44 m) and the number of piles.

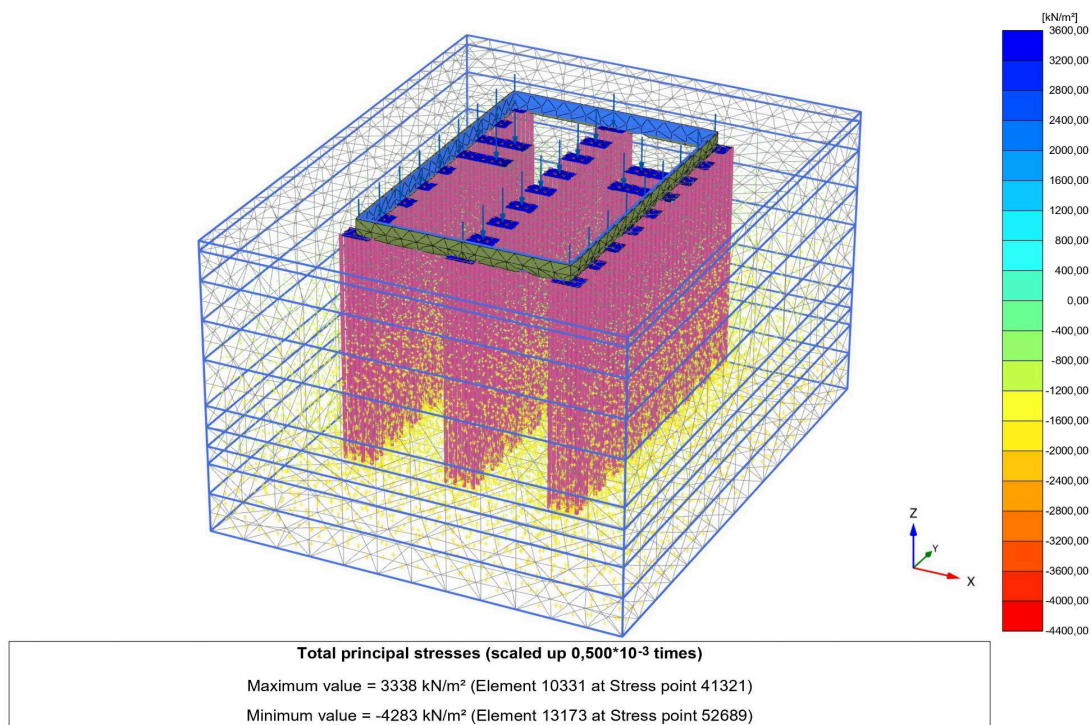


Figure 4.5.7.2.5 Total Principal Stresses

The largest compressive stress is -4283 kN/m^2 and is directly below the pile tips in the medium-stiff clay layer, a foreseeable consequence of the pile load being displaced to a comparatively limited bearing layer. Maximum tensile stress reaches 3338 kN/m^2 , and occurs locally on the pile cap and basement slab, where flexural action takes place. In general, the stress fields are localized; plastic areas in the soil are limited to the near pile shaft close surroundings and fail to form continuous surfaces of failure, suggesting stable foundation behaviour.

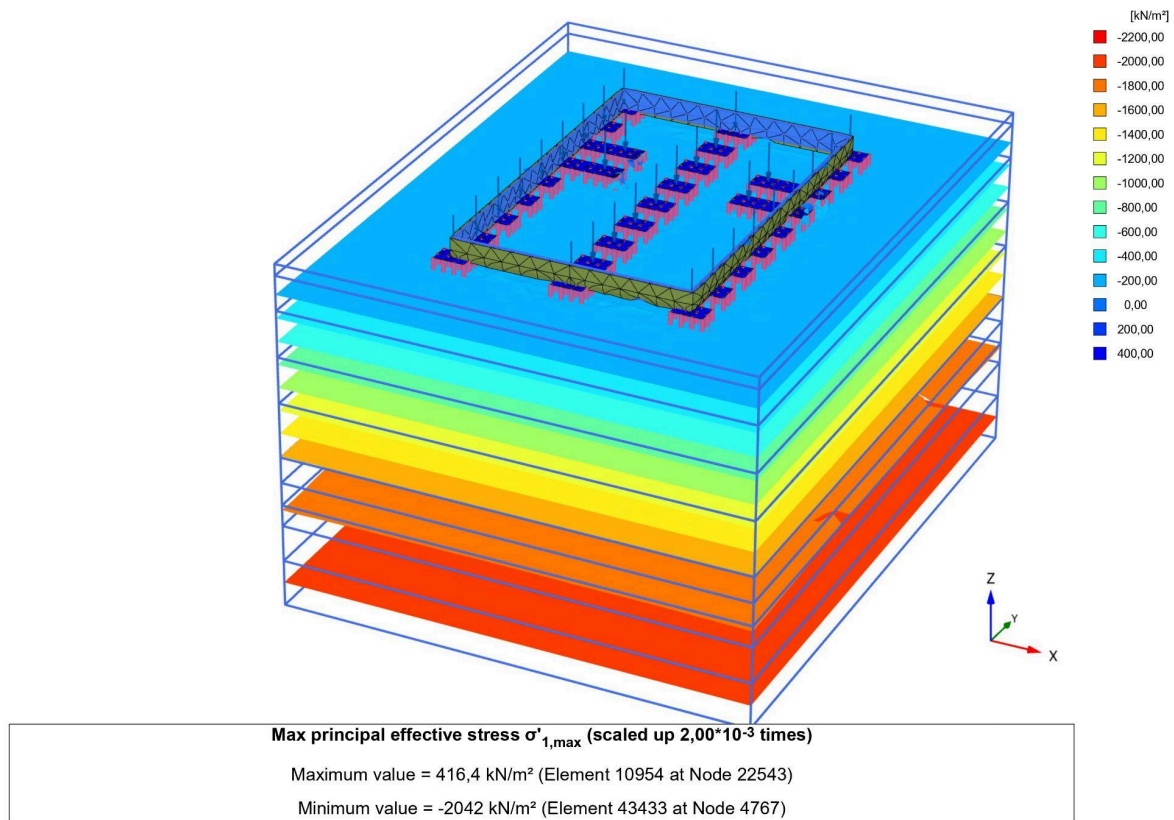


Figure 4.5.7.2.6 Maximum Principal Effective Stresses

The distribution of effective principal stresses through this plot gives an account of pore-water pressures. It has been observed that the maximum compressive effective stress is noted as -2042 kN/m² at the bottom of piles at the tips, proving successful loading transfer to the underlying deep bearing layers. Most of the positive effective stresses (to 416.4 kN/m²) occur in the upper part of the profile, along the retaining walls and the ground, where the effect of unloading is also notable. As can be seen by the effective stress analysis, the excess pore pressures will never reach critical levels, even at full design load, and the consolidation of the clay strata will occur without negative hydraulic gradients.

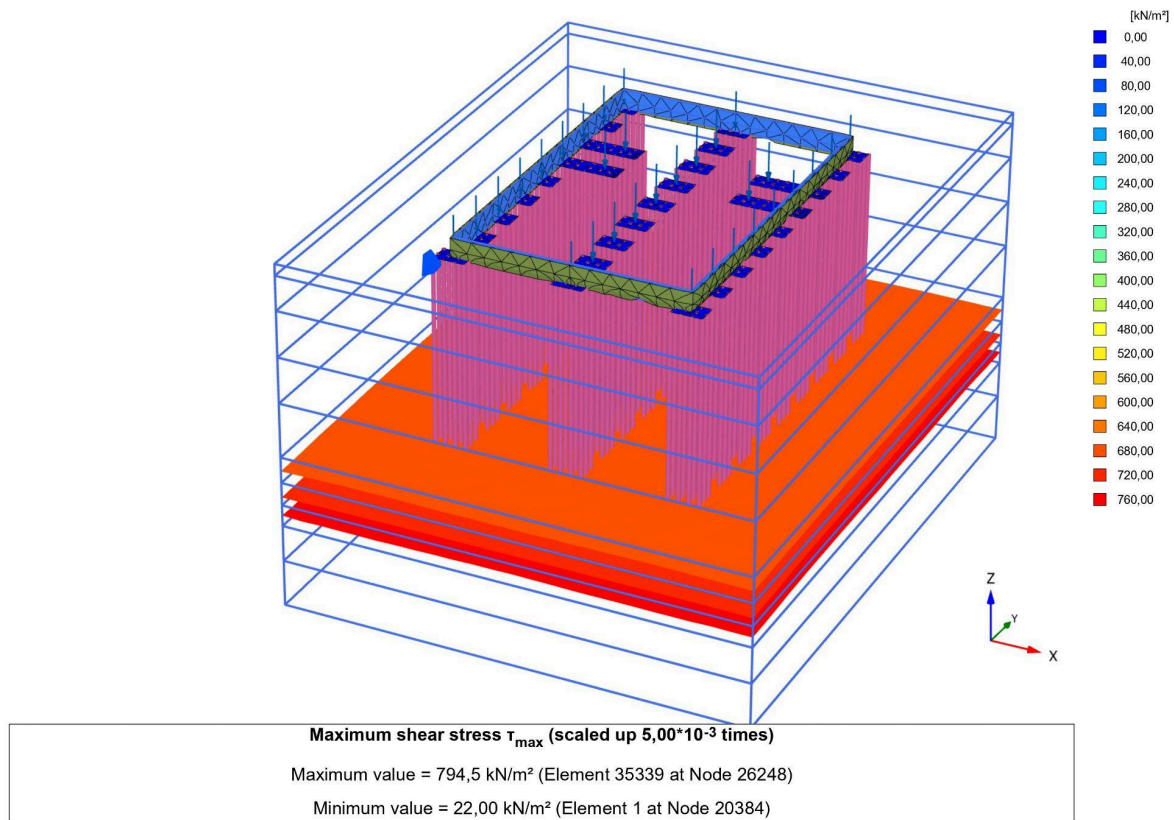


Figure 4.5.7.2.7 Maximum Shear Stresses

The shear stress distribution shows the areas which have the highest shearing forces exerted on the soil. The highest shear stress is seen to reach 794.5 kN/m² and occurs in the sand lenses and deep clay layers at the lower section of the pile shafts. This tendency coincides with the mobilization of the shaft friction and confirms the analysis of the presence of negative skin friction during the calculation of the analytical design. The lack of large areas with restraining shear stresses, verifies a sufficient level of safety regarding shaft resistance.

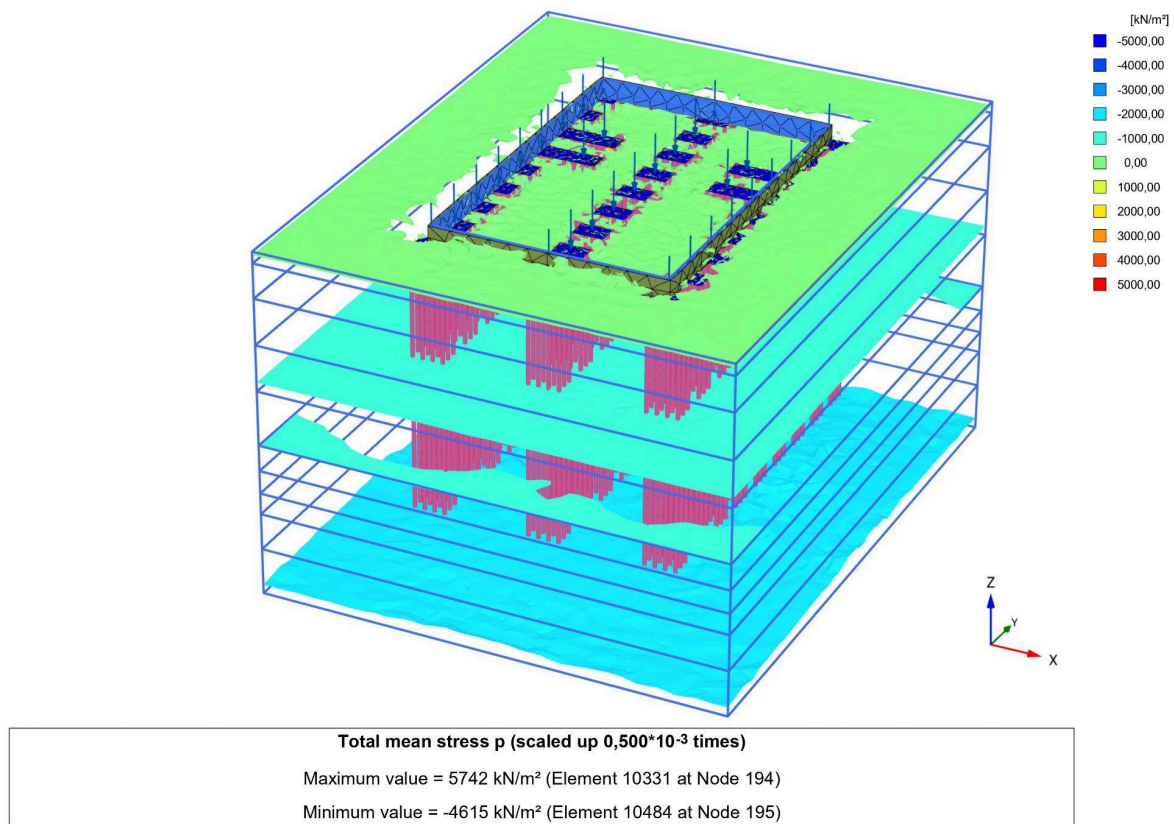


Figure 4.5.7.2.8 Total Mean Stresses

The overall mean stress gives information about the volumetric stress situation of the soil. It is the maximum of the average stress of 5742 kN/m² just under the pile cap and on the tip of the piles indicating the joint effect of the structural load and the overburden pressure of the soil. Stress with negative mean signs (-4615 kN/m²) are related to stress-relieving zones near the basement walls and excavation slopes.

4.5.7.3 Conclusion

The geotechnical design of the pile foundation system is well validated within the finite element analysis carried out in Plaxis 3D Ultimate. The calculated overall settlement of the entire foundation field on the design loads is 17 mm, and this is below the generally accepted serviceability measure of 20 mm of pile-supported structures. The value of this represents a cumulative impact of the compression of the piles, which is elastic, the instantaneous motion of

the bearing strata and the consolidation of the deep clay layers over the long run. The results of the stress distributions based on the model indicate an effective way of transferring the axial loads to lower layers of competent sand and stiff clay more than 45 m below depth, and mobilizing shear stresses on the pile shafts within permissible boundaries. Moreover, analysis of the overall foundation system, consisting of all 35 groups of piles along with the retaining walls and pile caps has shown that the effects of group-to-group interaction are small and they would not cause excessive differential settlements and overloading of individual piles. As a result, the foundation system, which includes 44 m-long, 0.5 m-diameter piles arranged in 4x3 groups at spaces of 3.5D with a cap at the top, is considered safe and serviceable. The data outcomes are closely approximated to hand-calculations and offer a strong validation of the design-assumptions and methods used in the prior sections.

4.6 Retaining Wall

4.6.1 Lateral Earth Pressure

4.6.1.1 Lateral earth pressure at rest

It is important to calculate the coefficient of earth pressure at rest at the beginning, and since the first two layers of soil are soft clay, it is reasonable to use Alpan's method for normally consolidated clays:

$$K_o = 0.19 + 0.233 \log PI \quad (63)$$

Then we find the Vertical effective stress σ_v by multiplying the unit weight of the soil by the soil thickness:

$$\sigma_v = \gamma * z \quad (64)$$

At $z = 2$ m (interface between layers 1 and 2):

$$\sigma_{v(2)} = \gamma_1 \times 2 = 18.5 \times 2 = 37.0 \text{ kPa}$$

At $z = 3.3$ m (base of the wall):

$$\sigma_{v(3.3)} = \gamma_1 \times 2 + \gamma_2 \times (3.3 - 2) = 27 + 18.8 \times 1.3 = 61.44 \text{ kPa}$$

And resultant at-rest force per unit length P_o is found by the following formula for each soil layer:

$$P = K_o * \sigma_v * H/2 \quad (65)$$

$$P_1 = \frac{1}{2} \sigma_v K_o * H = \frac{1}{2} (0.3869 * 37) \times 2 = 14.32 \text{ kN/m}$$

$$P_2 = \frac{1}{2} (\sigma_{v1} K_{o1} + \sigma_{v2} K_{o2}) \times H = \frac{1}{2} (37 * 0.3869 + 61.44 * 0.4) \times 1.3 = 25.28 \text{ kN/m}$$

$$P_o = P_1 + P_2 = 14.32 + 25.28 = 39.6 \text{ kN/m}$$

We found the location of the line of action of the resultant force:

$$\bar{z} = \frac{P_1(H/2) + P_2(H/3)}{P_o} = 1.31 \text{ m}$$

$$z_{\text{from base}} = H - \bar{z} = 3.3 - 1.31 \approx 1.99 \text{ m above the base}$$

Table 4.6.1.1.1 Lateral earth pressure at rest for the basement wall

Layer	Depth interval, m	$\gamma \text{ (kN/m}^3\text{)}$	PI	K_o	z, m	σ_v	$\sigma_h \text{ (kN/m}^2\text{)}$	P (kN/m)
1	0 – 2.0	18.5	7	0.3869	2	18.5	37.0	14.32
2	2.0 – 3.3	18.8	8	0.4	1.3	49.2	61.44	25.28

4.6.1.2 Rankine Active Earth Pressure

We start with the calculation of Rankine Active Earth Pressure, which is done by the formula below:

$$K_a = \tan^2(45 - \phi'/2) \quad (66)$$

Rankine active pressure σ_a' is calculated next

$$\sigma_a' = \sigma_0' K_a - 2c'K_a^{0.5} \quad (67)$$

The table below shows the calculation of active pressure on different depths below the ground surface level.

Table 4.6.1.2.1 Rankine active earth pressure for the basement wall

Depth, z (m)	σ_0' (kN/m ²)	Cohesion c (kPa)	K_a	σ_a'
0	0	18	0.5279	-26.15642177
2-	37	18	0.5279	-6.624121774
2+	37	20	0.4903	-9.867470117
3.3	61.44	20	0.4903	2.115461883

The pressure distribution diagram is graphed in the following figure.

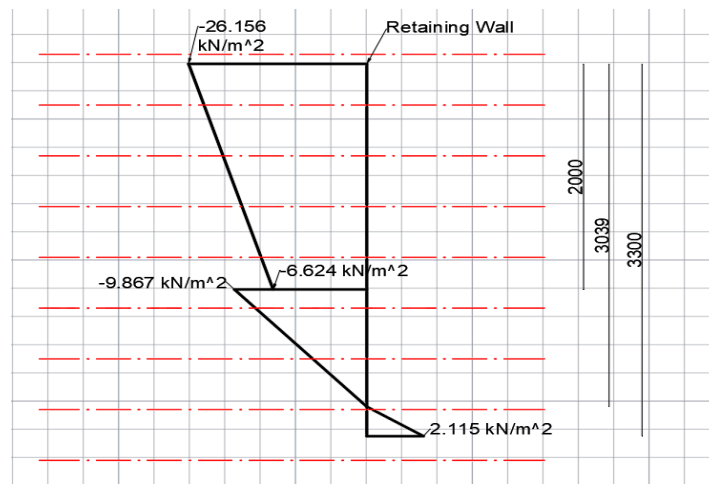


Figure 4.6.1.2.1 Active pressure distribution diagram

In this case, the total force per unit length is the sum of the areas of all the shapes in the graph:

$$P_a = \text{area 1} + \text{area 2} + \text{area 3} =$$

$$= 0.5 * (26.156 + 6.624) * 2 + 0.5 * 9.867 * (3.038 - 2) + 0.5 * 2.12 * (3.3 - 3.038) =$$

$$= 38.18 \text{ kN/m}$$

The distance to the location of the line of action of the resultant force:

$$\frac{-}{Z} = \frac{P_1 * z_1 + P_2 * z_2 + P_3 * z_3}{Pa} = 2.39 \text{ m}$$

4.6.1.3 Rankine Passive Earth Pressure

For this case, there is a separate coefficient of Rankine Passive Earth Pressure, which is calculated using the following formula:

$$K_p = \tan^2(45 + \varphi'/2) \quad (68)$$

Rankine passive pressure σ_p' is calculated as follows:

$$\sigma_p' = \sigma_0' K_p - 2c' K_p^{0.5} \quad (69)$$

The table below shows the calculation of passive pressure at different depths.

Table 4.6.1.3.1 Rankine passive earth pressure for the basement wall

Depth, z (m)	σ_0' (kN/m ²)	Cohesion c (kPa)	K_p	σ_p'
0	0	18	1.8944	49.549
2-	37	18	1.8944	70.093
2+	37	20	2.04	132.611
3.3	61.44	20	2.04	182.469

The pressure distribution diagram is graphed in the following figure.

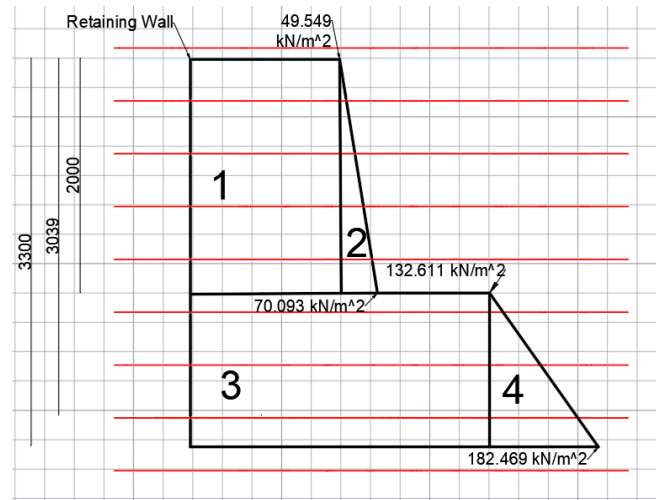


Figure 4.6.1.3.1 Passive pressure distribution diagram

In this case, the total force per unit length is the sum of the areas of all the shapes in the graph:

$$P_p = \text{area 1} + \text{area 2} + \text{area 3} + \text{area 4} =$$

$$= 2 * 49.549 + 0.5 * (70.093 - 49.549) * 2 + 1.3 * 132.6 + 0.5 * (182.5 - 132.6) * 1.3 =$$

$$= 324.44 \text{ kN/m}$$

The distance to the location of the line of action of the resultant force:

$$\bar{z} = \frac{P_1 * z_1 + P_2 * z_2 + P_3 * z_3 + P_4 * z_4}{P_p} = 1.22 \text{ m}$$

4.6.2 Proportioning Retaining Walls

Wall design begins with an assumption about the structure's dimensions. This process, called proportioning, can be useful for assessing the wall's stability. The figure below shows a cantilever wall with the assumed dimensions that will be used in the project.

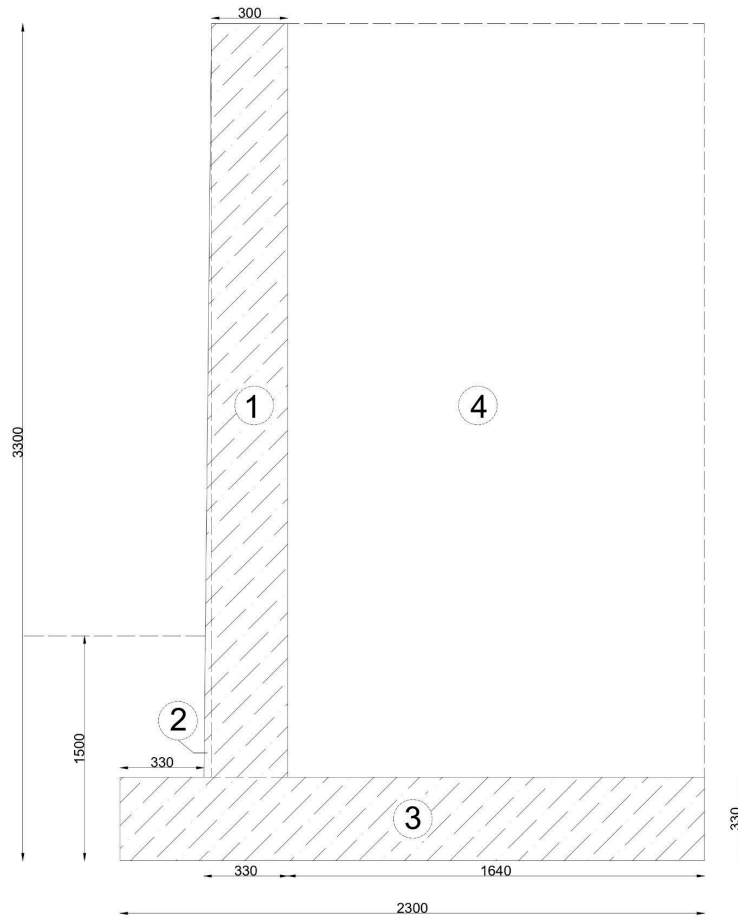


Figure 4.6.2.1 Cantilevered wall

Factor of Safety Against Overturning

Rankine's active force acts on the walls, and since we take its effect in the horizontal direction, the horizontal component of this force equals the initial force.

$$P_h = P_a = 38.18 \text{ kN/m}$$

Now, it is possible to determine the moment of resistance using the table below:

Table 4.6.2.1 Table of moments of resistance

Section	Area (m ²)	Weight length (kN/m)	Moment arm (m)	Moment (kN*m/m)
1	0.891	21.01	0.51	10.7151
2	0.04455	1.050489	0.35	0.36767115
3	0.759	17.89722	1.15	20.581803
4	4.871	90.587	1.48	134.06876

The sum of the moments is equal to:

$$\Sigma M_R = 10.72 + 0.37 + 20.58 + 134.07 = 165.73 \text{ kN} * \text{m/m}$$

The overturning moment is then calculated as follows:

$$M_0 = P_h \frac{H'}{3} = 38.18 * 3.3/3 = 42 \text{ kN} * \text{m/m}$$

Therefore factor of safety against overturning is

$$FS_{\text{overturning}} = \frac{\Sigma M_R}{M_0} = \frac{165.73}{42} = 3.95 > 2$$

Factor of Safety Against Sliding

The factor of safety against sliding can be estimated by the method below:

$$FS_{\text{sliding}} = \frac{(\Sigma V)\tan(k_1\phi_2') + Bk_2c'_2 + P_p}{P_a \cos\alpha} \quad (70)$$

It was assumed that $k_1 = k_2 = \frac{2}{3}$ and P_p is calculated for this case separately.

$$P_p = \frac{1}{2}K_p\gamma_2 D^2 + 2c'_2 K_p^{0.5} D = \frac{1}{2} * 2.04 * 18.8 * 1.5^2 + 2 * 20 * 2.04^{0.5} * 1.5 = 128.84 \text{ kN/m}$$

$$\text{Sum of the vertical force} = \Sigma V = 21.01 + 1.05 + 17.9 + 90.6 = 130.54 \text{ kN/m}$$

Hence,

$$FS_{sliding} = \frac{130.54 \cdot \tan(2/3 \cdot 20) + 2.3 \cdot 2/3 \cdot 20 + 128.84}{38.18 \cdot \cos 0} = 4.99 > 1.5$$

Factor of Safety Against Bearing Capacity Failure

The eccentricity of the resultant force was determined as follows:

$$e = \frac{B}{2} - \frac{\Sigma M_R - M_0}{\Sigma V} = \frac{2.3}{2} - \frac{165.73 - 42}{130.54} = 0.202 < \frac{B}{6} = 0.383 \quad (71)$$

Next, find the maximum and minimum pressures using the following formulas:

$$q_{heel} = \frac{\Sigma V}{B} \left(1 - \frac{6e}{B}\right) = \frac{130.54}{2.3} \left(1 - \frac{6 \cdot 0.202}{2.3}\right) = 26.83 \text{ kN/m}^2 \quad (72)$$

$$q_{toe} = \frac{\Sigma V}{B} \left(1 + \frac{6e}{B}\right) = \frac{130.54}{2.3} \left(1 + \frac{6 \cdot 0.202}{2.3}\right) = 86.69 \text{ kN/m}^2 \quad (73)$$

The ultimate bearing capacity of the soil is equal to:

$$q_u = c'_2 N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + 0.5 \gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i} = 407.36 \text{ kN/m}^2 \quad (74)$$

The factor of safety against bearing capacity is then equal to:

$$FS_{bearing} = \frac{q_u}{q_{toe}} = \frac{407.36}{86.69} = 4.7 > 3$$

4.6.3 Reinforcement of a Retaining Wall

Table 4.6.3.1 Initial design parameters for reinforced concrete structure

Parameter	Value
Concrete	C25/30 (f _{ck} = 25 MPa)
Reinforcement	A500C (f _{yk} = 500 MPa)
Footing thickness (h _f)	0.33 m

Wall thickness (h_w)	0.3 m
Active pressure (P_a)	38.18 kN/m

First, it is necessary to find the design moment at the base of the wall.

$$P_{a,Ed} = \gamma_G * P_a = 1.35 * 38.18 = 51.54 \text{ kN/m} \quad (75)$$

$$M_{Ed} = P_a * \frac{H'}{3} = 56.69 \text{ kN} * \text{m/m} \quad (76)$$

In this case, the shear force can be expressed as:

$$V_{Ed} = P_{a,Ed} = 51.54 \text{ kN/m}$$

Next is the effective cross-sectional depth, assuming a 50-mm protective layer and a rebar diameter of 16 mm.

$$d = h_w - c - \phi/2 = 300 - 50 - 8 = 242 \text{ mm} \quad (77)$$

Now it is possible to start selecting the longitudinal reinforcement.

$$\alpha_m = \frac{M_{Ed}}{bd^2 f_{cd}} = 0.0581 \quad (78)$$

The relative height of the compressed zone is then calculated using the formula below:

$$\xi = 1 - (1 - 2\alpha_m)^{0.5} = 0.0599 \quad (79)$$

In this case, the cross-sectional area of the reinforcement is calculated as:

$$A_s = \frac{M_{Ed}}{f_{yd} d (1 - 0.5\xi)} = 555 \text{ mm}^2 / \text{m} \quad (80)$$

Minimum reinforcement must also be verified.

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} bd = 327 \text{ mm}^2 / \text{m} \quad (81)$$

The calculated area of 555 mm²/m is greater than the minimum. Accepting Ø12 rods with a 200 mm spacing, area A_s in this case is taken as 565 mm²/m. Next, a shear check is performed; to do this, we first determine the ultimate load that the concrete can withstand.

$$V_{Rd,c} = [C_{Rd,c} k (100 p_l f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d \quad (82)$$

Where:

$$C_{Rd,c} = 0.18 / \gamma_c = 0.12;$$

$$k = 1 + (200/d) = 1.909 \leq 2;$$

$$p_l = A_s / (bd) = 0.002335$$

$$V_{Rd,c} = 0.12 * 1.909 * 1.80 * 1000 * 242 = 99864 \text{ N} \approx 99.9 \text{ kN/m}$$

$V_{Ed} = 51.54 \text{ kN/m} < V_{Rd,c}$, since this condition is met, transverse reinforcement is not required, and constructive stirrups are sufficient.

Next, the toe reinforcement must be performed. For this purpose, the toe pressure values from Section 4.6.1 will be used, where $q_{toe} = 86.69 \text{ kN/m}^2$ and $q_{heel} = 26.83 \text{ kN/m}^2$. Next, determine the design pressure values:

$$q_{toe,Ed} = 1.35 * 86.69 = 117 \text{ kN/m}^2$$

$$q_{heel,Ed} = 1.35 * 26.83 = 36.2 \text{ kN/m}^2$$

The pressure under the heel will vary linearly from q_{wall} to $q_{heel,Ed}$. First, $q_{wall,Ed}$ will be determined by interpolation.

$$q_{wall,Ed} = q_{toe,Ed} + (q_{heel,Ed} - q_{toe,Ed})x/B = 88.9 \text{ kN/m}^2 \quad (83)$$

In that case, the average pressure on the heel pad will be as follows:

$$q_{avg,heel} = (q_{wall,Ed} + q_{heel,Ed}) / 2 = 62.55 \text{ kN/m}^2 \quad (84)$$

In this case, the design moment in the cantilever is equal to:

$$M_{heel,Ed} = \frac{q_{avg,heel} L_{heel}^2}{3} = 45.04 \text{ kN} * \text{m/m} \quad (85)$$

The average pressure on the toe cantilever was calculated in the same way:

$$q_{avg,toe} = (q_{wall,Ed} + q_{toe,Ed}) / 2 = 102.95 \text{ kN/m}^2 \quad (86)$$

Moment at the root of the cantilever:

$$M_{toe,Ed} = \frac{q_{avg,toe} L_{toe}^2}{3} = 32.94 \text{ kN} * \text{m/m}$$

Since the base thickness is 0.33 m and the protective layer is 30 mm, the effective height for the reinforcement in the upper and lower zones will be:

$$d_{toe} = h_f - c_{top} - \phi/2 = 330 - 50 - 8 = 272 \text{ mm}$$

$$d_{heel} = h_f - c_{heel} - \phi/2 = 330 - 50 - 8 = 272 \text{ mm}$$

In order to calculate the top reinforcement for the heel, we use the same method as for the longitudinal reinforcement.

$$\alpha_m = \frac{M_{Ed}}{bd^2 f_{cd}} = 0.0365$$

$$\xi = 1 - (1 - 2\alpha_m)^{0.5} = 0.0372$$

$$A_s = \frac{M_{Ed}}{f_{yd} d (1 - 0.5\xi)} = 388 \text{ mm}^2 / m$$

Minimum reinforcement must also be verified.

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} bd = 368 \text{ mm}^2 / m$$

The calculated area of 388 mm²/m is greater than the minimum. Accepting Ø10 rods with a 200 mm spacing, the area A_s in this case is taken as 393 mm²/m. To calculate the bottom reinforcement for the toe, we use the same approach.

$$\alpha_m = \frac{M_{Ed}}{bd^2 f_{cd}} = 0.0267$$

$$\xi = 1 - (1 - 2\alpha_m)^{0.5} = 0.0271$$

$$A_s = \frac{M_{Ed}}{f_{yd} d (1 - 0.5\xi)} = 282 \text{ mm}^2 / m$$

Minimum reinforcement must also be verified.

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} bd = 368 \text{ mm}^2 / m$$

The calculated area of 282 mm²/m is lower than the minimum, the area is taken to be 368 mm²/m. Accepting Ø10 rods with a 200 mm spacing, area A_s in this case is taken as 393 mm²/m.

It is necessary to take into account important structural details. First, the anchoring of the wall reinforcement to the footing; according to EN 1992-1-1, the anchorage length for Ø12 should be $l_{bd} = 40\phi = 480 \text{ mm}$. Second, concrete with enhanced water resistance must be used, and the outer side of the retaining wall must be waterproofed with bitumen.

Table 4.6.3.2 Summary Table of Reinforcement of Retaining Wall

Element	Reinforcement diameter (mm)	Spacing (mm)	Position
Wall stem (vertical)	12	200	Inner face
Wall stem (horizontal)	10	200	Both faces
Toe slab (bottom)	10	200	Bottom of toe
Heel slab (top)	10	200	Top of heel

4.6.4 Sheet-Pile Walls

For the project under development, temporary sheet piles are required during excavation to prevent soil collapse. In our case, cantilever sheet piles will be used. Since our soil consists mainly of soft clay, calculations will be made using the cantilever sheet pile method in clay.

First, let's determine the theoretical depth of the sheet pile embedment using the following formula:

$$D^2[4c - (\gamma L_1 + \gamma' L_2)] - 2DP_1 - \frac{P_1(P_1 + 12cz'_1)}{(\gamma L_1 + \gamma' L_2) + 2c} \quad (87)$$

Hence, P_1 is the resultant of the active pressure above the bottom of the excavation, which has already been calculated in the section “Rankine Active Earth Pressure”, and it is equal to:

$$\begin{aligned}
 P_1 &= \text{area 1} + \text{area 2} + \text{area 3} = \\
 &= 0.5 * (26.156 + 6.624) * 2 + 0.5 * 9.867 * (3.038 - 2) + 0.5 * 2.12 * (3.3 - 3.038) = \\
 &= 38.18 \text{ kN/m}
 \end{aligned}$$

Now we need the arm of this resultant force relative to the bottom of the pit, which was also found in the previous sections:

$$\bar{z} = \frac{P_1 * z_1 + P_2 * z_2 + P_3 * z_3}{Pa} = 2.39 \text{ m}$$

Now we substitute the values for the depth formula:

$$D^2 [4 * 20 - (18.5 * 2 + 18.8 * 1.3)] - 2D * 38.18 - \frac{38.18(38.18 + 12 * 20 * 2.39)}{(18.5 * 2 + 18.8 * 1.3) + 2 * 20} = 18.56 * D^2 - 76.36 * D - 230.26$$

Solving this quadratic equation, we obtain $D = 6.136 \text{ m}$. Then, the net pressure at the bottom of the excavation is calculated and checked against the condition.

$$\sigma_6 = 4c - (\gamma L_1 + \gamma' L_2) = [4 * 20 - (18.5 * 2 + 18.8 * 1.3)] = 18.56 \text{ kN/m}^2 > 0$$

Knowing the theoretical value, we can now determine the practical embedding depth.

$$D_{actual} = D_{theor} * 1.5 = 9.2 \text{ m} \quad (88)$$

Next, we determine the maximum bending moment. To do this, we first use the formula below to find the depth to the point of zero shear force:

$$z' = \frac{P_1}{\sigma_6} = \frac{38.18}{18.56} = 2.06 \text{ m} \quad (89)$$

Maximum moment:

$$M_{max} = P_1(z' + z_1) - \frac{\sigma_6 * z'^2}{2} = 38.18 * (2.06 + 2.39) - \frac{18.56 * 2.06^2}{2} = 130.52 \text{ kN} * \text{m/m} \quad (90)$$

Now we find the minimum required cross-sectional modulus for profile selection. We assume that $\sigma_{all} = 172.5 \text{ MN/m}^2$.

$$S = \frac{M_{max}}{\sigma_{all}} = \frac{130.52}{172.5 * 10^3} = 7.57 * 10^{-4} \text{ m}^3/\text{m} \quad (91)$$

The PZ-22 model was selected as the sheet pile, as it fully meets the requirements.

4.6.5 Software Analysis of Retaining Wall and Sheet Piles

The stability and structural performance of the retaining system were verified using Geo5 geotechnical software. Two separate analyses were performed: one using the Retaining Wall module for the reinforced concrete cantilever wall and another using the Sheet Pile Wall module for the embedded sheet pile system. The objective of the numerical modelling was to verify stability against sliding and overturning, determine internal forces, and evaluate lateral displacement under short-term undrained soil conditions.

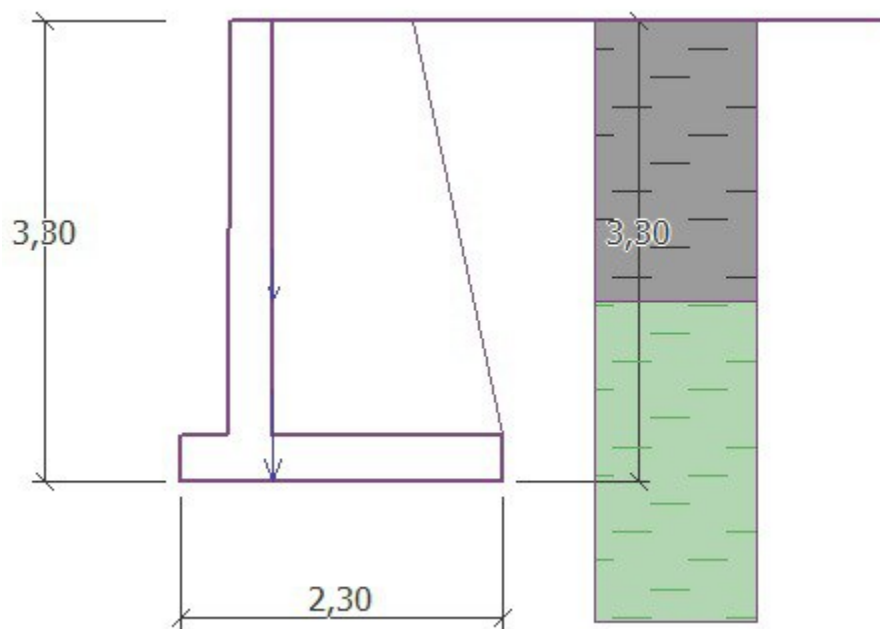


Figure 4.6.5.1 2D Model of the Retaining wall in Geo5 software

For the reinforced concrete retaining wall, the geometry was defined according to the structural design, with a wall height of 3.30 m and a base width of 2.30 m. The wall stem and base slab were modeled as rigid structural elements. The soil profile was based on the geotechnical investigation and included silty clay fill, very soft to soft clay, and underlying soft clay. For each layer, unit weight, undrained shear strength, friction angle, elastic modulus, and Poisson's ratio were specified. Short-term behavior was assumed, and undrained soil parameters were adopted.

The loading conditions included the self-weight of the wall, the weight of the retained soil wedge, and the active earth pressure acting on the back of the wall. Geo5 computed the earth pressure distribution using classical earth pressure theory. Resultant forces, application points, and moment arms were automatically determined and used in stability verification.

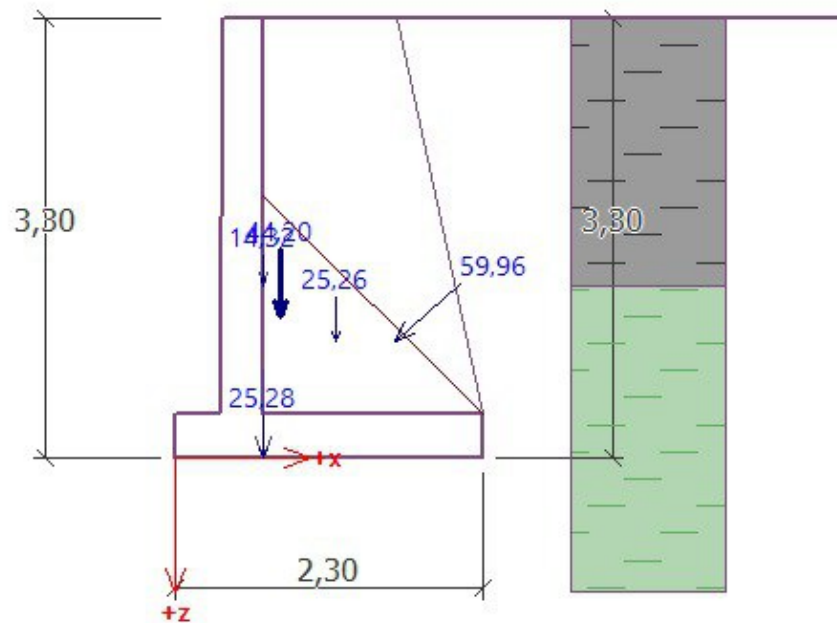


Figure 4.6.5.2 Internal Forces of the Retaining wall in Geo5 software

Sliding and overturning checks were performed within the program. The calculated factor of safety against sliding satisfied the required limit, and the utilization was 98.7%, indicating that the resisting forces were sufficient to counteract the horizontal driving forces. Overturning verification was also satisfactory, with stabilizing moments exceeding overturning moments and maintaining the resultant within the permissible zone of the base. The stress distribution beneath the footing remained acceptable, and no excessive eccentricity was observed. The pressure diagram confirmed a triangular active earth pressure distribution increasing with depth, consistent with theoretical expectations.

Verification : + - [1]						
No. Δ	Force	F_x [kN/m]	F_z [kN/m]	Applic. point x [m] z [m]		Coeff. [-]
1	Weight - wall	0,00	44,20	0,79	-1,06	1,000
2	Weight - earth wedge	0,00	52,47	1,21	-1,39	1,000
3	Active pressure	-30,32	44,94	1,80	-1,04	1,000
4	Force No. 1	0,00	14,32	0,66	-1,30	1,000
5	Force No. 2	0,00	25,28	0,66	0,00	1,000
6	Base anchorage	0,00	52,36	2,00	0,00	1,000

— Verification

OVERTURNING : SATISFACTORY (15,2%)

SLIP : SATISFACTORY (98,7%)

Figure 4.6.5.3 Verification of the Retaining Wall in Geo5 Software

The sheet pile wall was analyzed using the Geo5 Sheet Pile module as a cantilever embedded structure with a total length of 9.20 m. The soil layering corresponded to the site stratigraphy and included very soft to soft clays in the upper portion and soft clay below. Soil parameters were defined similarly to the retaining wall model, and short-term undrained behavior was considered. The wall was modeled as a flexible structural element interacting with the surrounding soil mass.

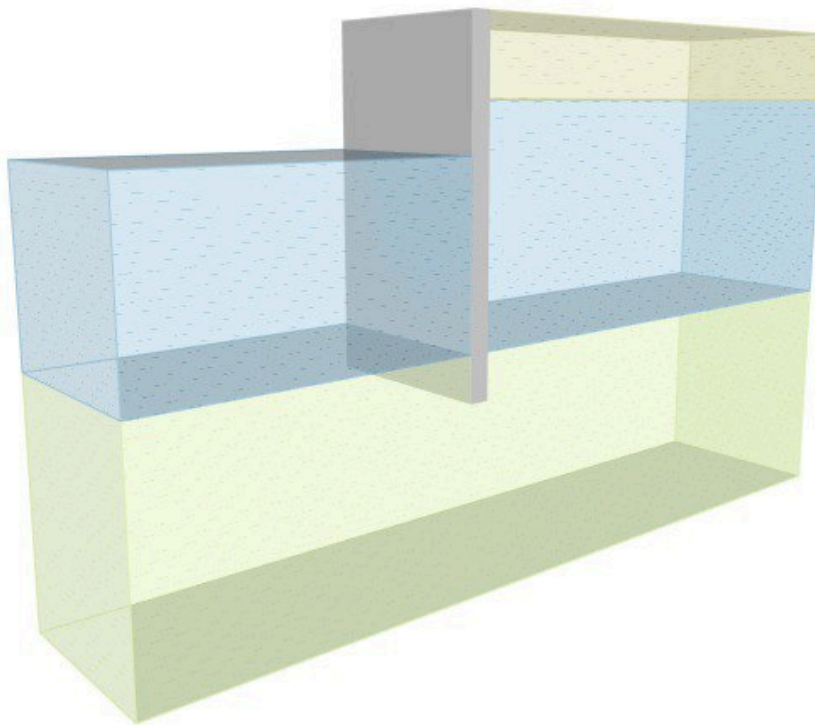


Figure 4.6.5.4 3D model of Sheet Pile Wall in Geo5 Software

The analysis included the calculation of earth pressures, internal shear forces, bending moments, and horizontal displacements along the wall depth. Geo5 generated shear force and bending moment diagrams based on beam-on-elastic-foundation theory, combined with classical earth-pressure assumptions. The maximum horizontal displacement obtained was 12.0 mm, occurring near the top of the sheet pile, which is characteristic of cantilever wall behavior. The maximum shear force reached 25.34 kN/m, and the maximum bending moment was 41.53 kNm/m. The maximum pressure acting on the structure was 21.38 kPa. The deformation pattern demonstrated expected cantilever action, with rotation occurring near the excavation level and passive resistance mobilized in the embedded zone.

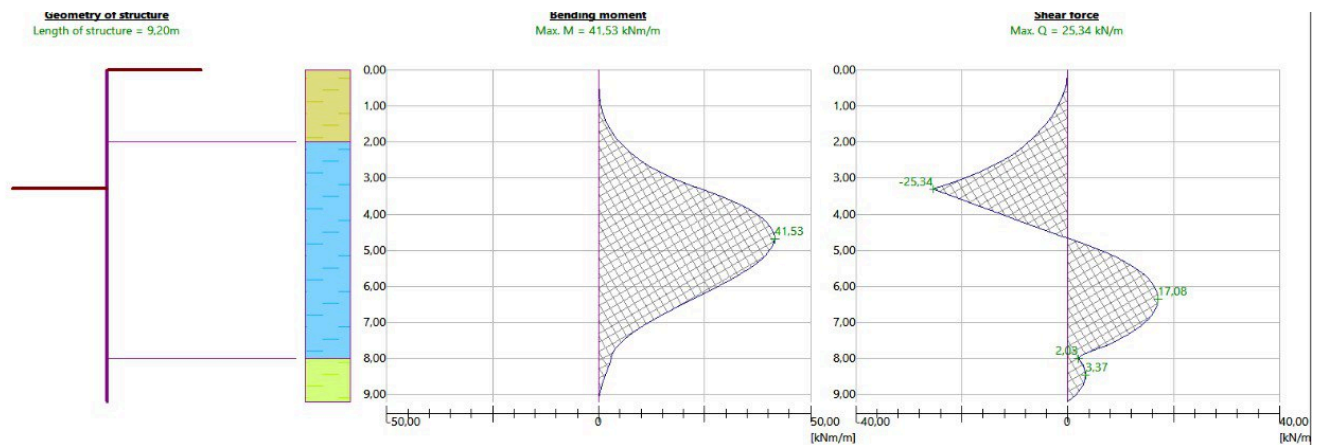


Figure 4.6.5.5 Sheet Pile Moment and Shear Force in Geo5 Software

The magnitude of displacement remained within acceptable serviceability limits for temporary excavation support systems. The internal force values were consistent with theoretical predictions, thereby verifying the selected sheet pile section.



Figure 4.6.5.6 Displacement and Pressure Action on Sheet Pile in Geo5 Software

Some differences between Geo5 outputs and hand calculations are expected and should not be treated as an error by default. The hand method idealizes earth pressure distributions, assumes simplified soil behavior (often fully active on the retained side and a predefined passive distribution on the embedded side), and typically applies conservative construction-stage assumptions. Geo5, in contrast, applies its own internal modeling choices for pressure redistribution, resultant locations, and stability verification, and it is sensitive to how the user defines soil layers, wall–soil interface parameters, surcharge placement, and passive resistance reduction. In addition, as noted in the semester deliverables, the limited availability of certain Geo5 functions and model options can prevent outputs from matching the hand-calculated values closely, even when both approaches are reasonable for preliminary design. Therefore, the role of the software in this section is to verify feasibility and identify governing checks, not to replace the design basis.

Results

Analysis properly completed.

Maximum values of internal forces acting on the structure

Maximum shear force = 25,34 kN/m

Maximum moment = 41,53 kNm/m

Maximum displacement = 12,0 mm

Figure 4.6.5.7 Overall Results on Sheet Pile in Geo5 Software

Overall, the Geo5 numerical analysis confirmed that both the reinforced concrete retaining wall and the sheet pile wall satisfy stability and serviceability requirements. The selected cantilever retaining wall geometry satisfies global stability requirements, with sliding being the critical check. The cantilever sheet pile system with 9.2 m embedment and the selected steel section are serviceable for temporary excavation support based on the predicted displacement response.

4.7 Construction Procedure

The foundation of the hotel is a combination of the temporary retaining structure, the twelve bored cast-in-place piles of the 4×3 pile groups, the pile caps, the basement slab and the permanent perimeter retaining wall designed in Section 4.6. The sequence of installation is presented top-down in the sequence of placement. The first task is to clear the site of vegetation and any remaining fill, to level and compact the working platform to provide a minimum undrained bearing capacity of 150 kPa to support the drilling rig and crawler crane, and to set out the building grid from protected external benchmarks with a total station to ± 25 mm in plan. The position of each of the twelve piles in the groups is marked on the platform and protected with a steel stake and a run-off trench is excavated around the perimeter to collect surface water and feed it into a sump outside the working area.

The first structure to be placed is the temporary support of steel sheet piles around the basement plan. The sheet piles provide a cofferdam to retain the soil during excavation, and to protect the adjacent sites and the permanent retaining wall during its curing. AZ-type or equivalent U-section interlocking sheets of some 6 m length are driven vertically around the perimeter using a vibratory hammer that is preferred to impact driving due to noise and vibration restrictions in the developed Humble area. The sheet piles need to extend at least 2.5 m below the 3.30 m basement formation level to develop the passive earth-pressure resistance in the clay to hold the wall in place during excavation. The interlock joints are checked and the plumbness of the lead pile is adjusted during the first metre of driving because a misaligned pile will affect the whole wall. After the basement perimeter is complete, the basement is excavated in 1.0 to 1.5 m lifts using a tracked excavator, disposing of the spoil direct to trucks for disposal. The surface

clays (Layers 1 to 3) are clean-cut with a toothless bucket, without ripping. Below a depth of about 1.5 m, one tier of internal steel struts and whalers is installed around the perimeter to provide support for the sheet-pile wall against the active earth pressure that was calculated in Section 4.6. The sheet-pile wall is monitored between each lift and if the inward deflection on any panel is greater than about 11 mm, which corresponds to about $L/300$ for a 3.30 m retained wall, then the preload on the struts is increased. The last 150-200 mm down to the final design formation level of 3.30 m is dug out by hand, to avoid any disturbance to the clay that provides bearing for the basement slab. It is checked and signed off by the engineer prior to concrete placement.

Once the basement is excavated and the working platform established, the piles are installed. Each of the twelve bored piles in every group is installed using the temporary-casing method, which was preferred over the stabilisation by a slurry of bentonite because the top 33.5 m of the stratification is predominantly soft-to-medium clays in which the stabilising pressure of the slurry column can be lost at inter-beds and cause necking of the shaft, while a steel casing provides positive radial support and also allows visual inspection of the shaft bottom before concreting. A hydraulic rotary drilling rig, such as the Bauer BG series class with a minimum of 120 kNm torque to reach the design depth of 44 m in cohesive soils, is set up so that its centre-line coincides with the pile mark. A Ø560 mm steel casing (oversize by 60 mm with respect to the nominal pile diameter to allow for the passage of drilling tools) is placed upright and checked with a two-axis inclinometer. The casing is then driven or vibrated through the upper two layers of clay to a depth of 8 to 10 m, which is sufficient to provide a water-tight seal to prevent groundwater seepage from perched water tables and collapse of the upper soil into the open borehole. The material inside the casing is removed with a bucket auger through the clay layers, and with a rock auger through the dense sand and silty-sand band between 33.5 and 45.7 m depths. The tool is regularly retrieved and the spoil dumped into a side-dumping skip and removed from the site. As the borehole progresses beyond the tip of the casing, the clay walls of the upper layers are temporarily stable for the short time required to insert the next string of tool, but the bore is never left open for more than one day. When the bucket tip reaches the design tip elevation of 49.30 m (medium-stiff clay of Layer 9), the base is cleaned with a cleaning bucket, and, if needed, an air-lift, and then examined for intact bearing soil and absence of any sloughed

soil or debris. The most frequent cause of excessive settlements of a bored pile in clay is loose debris in the pile tip, which should be removed.

The pre-fabricated cage of each pile, comprising the eight longitudinal Ø35 mm bars confined by the Ø16 mm spiral at 70 mm pitch with 75 mm cover as described in Section 4.5.5 is lifted by the crawler crane and dropped into the cased hole. The cage is fixed with concrete spacer wheels at 3 m gaps to ensure the cover, and is suspended from a spreader bar at the casing head to prevent it from sitting on the base of the borehole, thereby damaging the cleaned bearing layer. The cage verticality is verified before the concreting. Self-compacting concrete of class C30/37 with a slump of 180 to 210 mm is placed via a tremie pipe which is positioned at 0.3 m above the borehole base. Concreting is continuous. The tremie must always be kept at least 2 m below the top of fresh concrete so that the concrete isn't contaminated with water and spoil, and segregation of the concrete does not occur. Concrete is poured at a rate that keeps the rise in the borehole at a rate of 3-4 m per hour and is over-cast by 0.5-1.0 m above the design cut-off level of 5.30 m because the top layer of concrete is usually contaminated with laitance and is removed later during head trimming. The temporary casing is gradually removed during the concreting, while the fresh concrete level inside casing is maintained above the groundwater and soil pressure outside to prevent water or soil from leaking into the fresh concrete. The twelve piles in each group are drilled in a skip sequence in the 4×3 grid, i.e. first the alternate piles are installed and then the remaining piles in between are drilled after the adjacent concrete has developed a minimum of 24 hours of strength. This is a critical sequence for the case of $3.5D = 1.75$ m centre-to-centre spacing in the group so that a fresh concrete pile is not affected by the vibration and the lateral soil displacement that would be caused by the installation of the next pile.

A minimum of seven days after installation, the 0.5 to 1.0 m of over-cast concrete at the top of each pile is chipped away with air chisels to the design cut-off level of 5.30 m below ground surface. The exposed longitudinal bars are cleaned and bent out to provide the embedment into the pile cap, and the development length for the Ø35 mm bars into the cap is 30 times the bar diameter or 1,050 mm, as per ACI 318-19. A 75 mm blinding concrete of class C15 is then placed over the cut-off tops of each group to provide a flat surface above the dusty pile tops on which to assemble the form of the pile cap. The pile cap of plan dimensions 6.35×4.60 m and depth 1.65 m as used in the Plaxis 3D model of Section 4.5.7 is formed on each group

using side shutters, the bottom reinforcement mat is placed on 50 mm thick concrete spacers to maintain the cover to the bottom face, and the cage shown in Section 4.5.4 is tied above the exposed pile dowels so that all the dowels are fully lapped into the cap cage. The pile cap is cast in one go to eliminate the possibility of a cold joint in an element that must transfer the whole column load (21,920 kN) to the 12 piles. Given the 1.65 m depth of the cap, special attention is given to heat of hydration, with low-heat cement used, and controlled curing with wet hessian or curing blankets for 72 hours to minimise the temperature differential. The cap is cured for at least 14 days before the column load is applied. Once all the pile caps are cast, the 0.35 m basement slab is cast on top of the caps with a two-layer high-density polyethylene (HDPE) waterproofing membrane installed between the caps and the basement slab. All joints in the membrane are heat-welded and the membrane is folded up around the perimeter to overlap with the waterproofing system of the retaining wall to create a tank which will seal the basement against water leak. While the site's groundwater table is at about 50.3 m (and thus well below the slab), the clays above may contain perched water in the event of a long-duration rainfall event and the waterproofing system is designed for that condition rather than for a continuous hydrostatic load. The slab is reinforced with a bi-directional top and bottom mat and is wet poured with control joints formed at about 6 m intervals to control shrinkage cracking and later filled with hydrophilic waterstop strips.

The permanent reinforced-concrete cantilever retaining wall designed in Section 4.6, with the 3.30 m retained height and the 0.33 m thick footing, is now built around the outside of the basement on the top of the slab. The wall is constructed in 3 to 4 m lifts to control the form pressures, with a construction joint and a shear key at the top of each lift, and the longitudinal and vertical reinforcement being continuous through the joints with lapped or coupled bars. A waterstop is installed at each construction joint to ensure continuity of the waterproof membrane. The sheet-pile cofferdam remains in place to retain the earth pressure so that the wall is not loaded until it has reached the required strength. After the wall has gained the required 28-day design strength, the annular space between the wall and the sheet-pile cofferdam is filled with 300 mm (compacted) lifts of well-graded granular fill with a 40 mm maximum particle size, to a minimum of 95 per cent of the Standard Proctor maximum dry density, as confirmed by nuclear density testing on the site. Compaction near the wall is done with a walk-behind plate compactor, rather than a vibratory roller, to prevent overloading the wall with excessive lateral forces as the

wall is still curing. Finally, the temporary sheet piles are extracted in the inverse sequence of their installation, using a vibratory extractor. The small void left by the web and the two interlocks is immediately filled with a fluid sand-cement grout through a tremie pipe as each sheet pile is extracted, to ensure no soil and surface settlement occurs behind the permanent wall. The extraction is monitored by survey targets on the neighbouring buildings and the road surface, and a trigger value for settlement of 10 mm is set, at which point extraction is temporarily stopped and the grouting rate is increased until the movements stabilise. At this stage the temporary works are complete, the permanent retaining wall is in place to carry the in situ soil pressure and the substructure is ready to take the superstructure loads.

5. Environmental Engineering

5.1 Local Regulations

5.1.1 State regulatory framework

In Texas, the Texas Commission on Environmental Quality (TECQ) regulates solid-waste management in Title 30 of the Texas Administrative Code (TAC) with the assistance of the Texas Health and Safety Code and the Texas Food Establishment Rules (TFER).

The Chapter 330 of the 30 TAC forms the fundamental technical requirements to the storage and collection of the municipal solid waste (MSW) used by the commercial generators, such as the hotels.

The Chapter 328 of the TAC defines the recyclable work and establishes handling regulations of the facilities opting to introduce the recycles program, in which recyclables should be separated by MSW and should be unaffected by contamination, odor and vectors (TCEQ, 2023b).

Chapter 332 of TAC is the regulation of the management of organic wastes, such as food scraps, and it specifies the storage and pathogen-reduction requirements of the facilities that organize organics (TCEQ, 2023c).

These regulations are outlined in the statutory Texas Health and Safety Code Chapter 361 which gives local jurisdiction to implement the commercial generator container placement and

storage standards and collection requirements through local jurisdiction of the City of Humble and Harris County and the City of Houston.

In the case of the restaurant and kitchen of the hotel, the Texas Food Establishment Rules (25 TAC Chapter 228) apply further since grease and oils and other wastes are supposed to be stored in closed and labeled containers and removed by specific service providers (TDSHS, 2023).

Collectively, the state structure demands that the Humble hotel:

- store all MSW and food-service trash in covered, airtight cans;
- prevent overflows, odors and infestation by pests by sufficient capacity and collection;
- use licensed haulers, authorized facilities on all disposal and organics management.

5.1.2 Local ordinances and franchise system in Humble

The City of Humble does not have an indexed municipal sanitation ordinance that specifically regulates commercial waste handling. Consequently, TCEQ regulations, especially with regard to commercial waste-storage, and Harris County public-health authority are the major enforcers of commercial wastes-store requirements in Humble. In its Solid Waste Program, Harris County Public Health mandates that trash be kept in rodent-proofed, lidded containers and be picked up commonly enough to avoid smell, spillage and infestation of vermin (Harris County Public Health, 2024).

The Humble area also has operational practices that are regional based on the requirements set by the City of Houston Solid Waste Management Department. These standards specify the dumpster to be leak-free, closed when not used, located on paved pads, and in a sanitary condition (City of Houston SWMD, 2024).

Haulers that cater to Humble (Waste Management and Republic Services) work on these regional conditions and offer homogenous collection services. Their service policies focus on avoiding contamination of the recycling material and they provide collection services once a week up to several times a week depending on the commercial needs.

5.1.3 Implications for bin sizing, colours and siting at the Humble hotel

With the common densities of commercial-sector mixed MSW and food waste, this corresponds to at least 2.3 m^3 of residual (general) trash per day and 0.2 m^3 of separated organics. An average 4-yard front-load dumpster has approximately 3.06 m^3 capacity and an 8-yard has an approximate of 6.12 m^3 . In this regard, a single 4-yard general trash dumpster would adequately suffice, in pure volumetric terms.

The container structure that has been proposed to be used by Humble hotel is based on the normal business arrangement of major haulers that are based in the Houston area, among them Waste Management, Republic Services and FCC Environmental Services.

Organics are placed in one 8-yard green dumpster, as green containers are the industry-standard color of food-waste and compostable materials, and 8-yard front-load containers are widely accessible by both haulers to high-volume putrescible waste (Waste Management, 2024; Republic Services, 2024). It is proposed to collect three times a week as the food waste in the Houston climate produces odors and attracts pests rapidly, and pickup services report that restaurants and hotels often have to have several pickups every week to keep the environment hygienic (FCC Environmental Services, 2024).

General trash (residual MSW) will have two 4-yard grey or black dumpsters, which can be attributed to the typical color scheme of refuse containers, as well as the volume capacity of this type of trash given the mass of waste that the hotel produces daily. Houston haulers usually provide 4-yard containers to hotels and commercial buildings and have two to three pickups a week, which is also in line with the industry practice of mixed commercial trash (Waste Management, 2024; Republic Services, 2024).

The recycles will be picked up with 1 4-yard blue single-stream recycling dumpster, which is the same color required by both WM and Republic Services to use with commingled recyclables and is suitable to the amount of recyclable waste that the hotel is likely to produce. It should be collected twice a week since commercial recycling collections are often collected fewer than trash collections made and could be provided by the haulers with one or two weekly pickups of paper, cardboard and mixed recycling (Republic Services, 2024).

Containers are all located in the loading-dock enclosure, which is in line with the regional requirements according to which the dumpsters should be placed in a paved area, be inaccessible to the general public, and closed when not in use, as well as accessible to the collection vehicles (City of Houston Solid Waste Management Department, 2024). All in all, the chosen sizes, colors and frequencies are those typical in the availability and normal operations of commercial generators in the Houston met.

The corresponding bin configuration is summarised in Table 5.1.3.1.

Table 5.1.3.1 Waste containers for a 78-room hotel in Humble, Texas

Waste stream	Size	Quantity	Colour (industry practice)	Location	Typical collection frequency
Organics (food waste)	8 yd	1	Green	Loading-dock enclosure	3 times per week
General trash (residual MSW)	4 yd	2	Grey/black	Loading-dock enclosure	2-3 times per week
Single-stream recycling	4 yd	1	Blue	Loading-dock enclosure	2 times per week



Figure 5.1.3.1 Three types of bins

Source: Hanh Phát Packaging & Plastic. (2025).

<https://hanpak.com.vn/faq-why-is-it-ok-to-put-plastic-bags-in-food-waste-but-not-in-the-green-recycling-bin/>

5.2 Waste generation rate

5.2.1 Guest related waste

Hotel waste generation rate is dependent on the type of hotel, level of service, and occupancy. Pirani and Arafat (2014) observe that the quantity of solid waste produced by a hotel per guest per day ranges between 0.83 kg and 1.22 kg, depending on the type and size of the hotel both in the United States and internationally. Equally, the California Integrated Waste Management Board (2006) indicates that the average planning worth of U.S. hotels is around 2.2 lb. per guest per night which is equal to 1.0 kg/guest/day. According to these two sources, the average guest generation rate of 1.0 kg/guest/day is used as a design standard of average hotel visitors in Texas, which is consistent with average national indicators in the US hotel industry.

The hotel occupancy rate in the U.S. is defined as 63.5 percent, as reported by the American Hotel & Lodging Association (2023), which is a sufficient starting point when calculating the daily waste-volume in this paper. Since the hotel operations literature and data on feasibility studies always indicate that mid-range and full-service hotels normally move between one and two guests per occupied room, an engineering average of 1.5 guests per occupied room was taken. The given value is typically used in the research on the industry as a typical ratio of

business and tourist hotels (International Tourism Partnership, 2017). On this premise, it is assumed that there will be approximately 75 in-house guests per day when the hotel operates normally:

$$N_{guests, avg} = 78 \times 1.5 \times 0.635 \approx 75 \text{ guests}$$

The expected guest-related solid waste generation is therefore:

$$W_{guests} = N_{guests, avg} \times 1.0 \text{ kg/guest/day} \approx 75 \times 1.0 = 75 \text{ kg/day}$$

5.2.2 Restaurant and kitchen related waste

Food service represents one of the largest waste-generating activities in hotel operations. The restaurant dining area is 460.5 m^2 . Full-service restaurant design guidelines recommend allocating 12-15 sq ft per guest ($\approx 1.11\text{-}1.39 \text{ m}^2/\text{seat}$) (TheFork Manager, 2025).

Using the mid-range average of 1.25 m^2 per seat, the seating capacity is:

$$N_{seats} = \frac{460.5}{1.25} = 368 \text{ seats}$$

Nevertheless, daytime full seating occupancy is not at full service level in the full-service restaurants. The statistics recorded in terms of industry F&B capture rate show that the percentage of hotel restaurant capacity that is most likely to be used daily is approximately 60 (National Restaurant Association, 2021). A 90 diners/day increment is made to cover the additional demand of the non-residents visiting the restaurant. Thus, daily diners number is to be expected as:

$$N_{diners} = 368 \times 0.6 + 90 = 311 \text{ diners/day}$$

The waste generated by the restaurant and kitchen can be estimated by taking the sum of the waste generation rates that can be derived empirically by the international hospitality waste audits. According to BusinessWaste (2024), the average quantity of waste generated by hotels and restaurants per plated meal amounted to 0.46 kg of waste, including front-of-house waste, e.g., disposable napkins, packaging, and plate leftovers. In the case of back-of-house operations, the literature analyzing the food service industry revealed that waste in the kitchen preparation is

usually between 0.3 and 0.6 kg/meal served depending on the type of menu and kitchen efficiency (Charlebois et al., 2015). A design value of 0.35 kg per meal is taken to estimate the level of kitchen waste which is supposed to be conservative when ensuring a low estimate of mid-scale hotels.

The restaurant and kitchen waste generation rates are therefore calculated as:

$$W_{restaurant} = N_{diners} \times 0.46 = 311 \times 0.46 \approx 143 \text{ kg/day (front-of house waste)}$$

$$W_{kitchen} = N_{diners} \times 0.35 = 311 \times 0.35 \approx 109 \text{ kg/day (back-of house waste)}$$

$$W_{F\&B} = 143 + 109 \approx 252 \text{ kg/day}$$

Table 5.2.2.1 Waste generation rates for each stream

Source	Daily Waste (kg/day)	Annual Waste (kg/year)	% of Total
Guest Rooms	75	27,375	22.9 %
Restaurant (Front-of-House)	143	52,195	43.8 %
Kitchen (Back-of-House)	109	39,785	33.3 %
Total (Hotel)	327	119,355	100 %

5.3 Solid waste composition

5.3.1 Guest-Related Solid Waste Composition

Hotel related waste refers to the waste that is generated by the guests in the rooms, corridors, lounges, and other common areas. This waste is significantly different from kitchen or restaurant waste since it mainly has paper, plastics and packaging waste but not food remnants. This is why organics constitute a lesser percentage of the overall than the back-of-house operations.

Surveys conducted within the United States and other developed countries always give evidence that organic wastes of guest related areas do not surpass 25% of total wastes. CalRecycle (2014) indicates that the hotel industry in Massachusetts has food and vegetative waste that makes up about 25% of the entire waste stream. This is in line with the results of the U.S. Environmental Protection Agency (2022), who found that commercial premises not involved with food production like hotel rooms, offices, and retail spaces usually produced less than a quarter of organics, and the rest consisted of paper and packaging materials. Equally, the European Commission (2013) and Juvan et al. (2023) concluded that the organics of the guest floors in the hotels without kitchens are within the range of 15% to 25%, which corroborates the appropriateness of the range in terms of design use in hospitality facilities.

The greatest waste percentage in guest areas is always caught by paper and cardboard. U.S. EPA (2022) indicated that 23.1% of municipal solid waste stream in the United States is paper and paperboard, whereas CalRecycle (2014) also revealed that the commercial sector (hotels) generates up to 30% of waste volumes on paper (mainly brochures, tissues, and packaging). Consequently, the mean value of 28% is used in this category.

Another important element of guest-related waste is plastics, and they are produced mainly through single-use toiletries, water containers, and wrappers. In the U.S. EPA (2022) plastics were estimated at 12.9% of total MSW and CalRecycle (2014) at 10-15% of hotel and lodging facilities. In this regard, 12% is taken on board.

There is the use of glass and metals that are in the form of beverage containers and cans. Both the EPA (2022) and Juvan et al. (2023) mention that these materials combined do not often surpass 10% of the overall waste stream in guest related locations. In this way, glass and metals are estimated at 5% each.

Lastly, the remaining part of the waste stream includes residual or miscellaneous waste consisting of textile waste, sanitary waste, bathroom waste, and minor non-recyclable plastics. According to the European Commission (2013), these elements constitute 20-22% of the total waste in guests.

Table 5.3.1.1 Estimated Composition of Guest-Related Solid Waste in Hotels

Waste Category	Share (%)	Typical Constituents
Organics	25	Food scraps from room service, used napkins, paper towels, compostable items, biodegradable toiletries
Paper & Cardboard	28	Brochures, tissues, newspapers, packaging, toilet paper, office paper
Plastics	12	Water bottles, toiletry containers, wrappers, plastic liners, disposable packaging
Glass	5	Beverage bottles, jars, cosmetic containers
Metals	5	Aluminum cans, foil, small metal lids and packaging
Residual / Miscellaneous	25	Textiles (linens, towels), sanitary waste, mixed non-recyclables, bathroom refuse

5.3.2 Kitchen & Restaurant Solid Waste Composition

The kitchen and restaurant operation produces one of the bigger figures in the overall waste production of a hotel facility. Preparation of food, plate waste, wrapping of incoming food, and drink containers are all parts of the waste that add to the bulk in this sector of waste.

A recent segmentation study of hotels and restaurants by Saeteros-Hernandez et al. (2025) shows that in food service/restaurant operations, the bulk of waste stream usually comprises 40-60% organic material. More indicators are the findings of Tatàno et al. (2017), who concluded that food-service facilities produce an organics-dominated waste stream, and the food waste makes a significant portion of the total mass.

Similarly, according to Juvan et al. (2023), paper/cardboard tend to constitute between 6 to 40% of the hospitality industry waste stream, with plastics constituting between 5 to 15%. The proportions that were adopted in terms of glass (7%), metals (4%), and residual waste (4%) are

within the empirically recorded ranges within hospitality setups. Tatàno et al. (2017) observed a typical glass content in restaurant waste of 6-10% and Pirani and Arafat (2014) reported a metal content of 3-5% of all hotel and restaurant waste. Equally, Juvan et al. (2023) provided residual/miscellaneous fractions ranging 2-8%, and this indicates that the allocation assumed in this paper is realistic and consistent with the literature.

According to the given above, the estimate of the waste composition in the restaurant and kitchen area of the proposed 78-room hotel in Humble, Texas is the following:

Table 5.3.2.1 Estimated Composition of Kitchen and Restaurant Solid Waste in Hotels

Category	Share (%)	
Organics	52	Food scraps (pre-consumer & plate waste)
Paper & Cardboard	22	Packaging, disposable paper goods, delivery cartons
Plastics	11	Food service film, plastic packaging, containers
Glass	7	Bottles, jars from beverages/condiments
Metals	4	Cans, foil, kitchen metal scrap
Residual/Other	4	Ceramics, contaminated items, mixed waste

5.4 Solid waste quantity

In order to convert the compositions in percentages to the real masses, the sum of the daily wastes estimated to be produced by each subsystem was multiplied by each material category share. In the case of the waste associated with guests, the overall weight is 75 kg/day. In the case of the food-and-beverage (F&B) subsystem (restaurant + kitchen), it is 36.45 kg/day. Masses were then transformed into weekly and annual. An example is provided of one of the tables; the rest of the rows are also computed similarly.

5.4.1 For guest related waste

Guest-related waste (rooms, corridors, lobby) was estimated at 75 kg/day. Using the composition profile developed in Section 5.3.2 (25 % organics, 28 % paper & cardboard, 12% plastics, 5% glass, 5% metals, 25% residual), each category mass is:

$$m_{category, day} = 75 \text{ kg/day} \times share(\%)$$

For example, organics:

$$\text{Daily mass: } m_{organics, day} = 75 \text{ kg/day} \times 0.25 = 18.75 \text{ kg/day}$$

$$\text{Weekly mass: } m_{organics, week} = m_{organics, day} \times 7$$

$$\text{Annual mass: } m_{organics, year} = m_{organics, day} \times 365$$

Table 5.4.1.1 Guest-related solid waste quantities by category

Waste category	Share (%)	Daily mass (kg/day)	Weekly mass (kg/week)	Annual mass (kg/year)	Ton/year (t/year)
Organics	25	18.75	131	6,844	7
Paper & cardboard	28	21	147	7,665	8
Plastics	12	9	63	3,285	3
Glass	5	3.75	26	1,369	1
Metals	5	3.75	26	1,369	1
Residual / misc.	25	18.75	131	6,844	7
Total	100	75	525	27,375	27

**All annual and tonnage values have been rounded to the nearest whole number for clarity*

5.4.2 For restaurant related waste

Using the kitchen/restaurant composition from Section 5.3.1 (52% organics, 22% paper & cardboard, 11% plastics, 7% glass, 4% metals, 4% residual), each category mass is:

$$m_{category, day} = 36.45 \text{ kg/day} \times share(\%)$$

Daily, weekly and annual values are obtained in the same way as for guest waste.

Table 5.4.2.1 Kitchen and restaurant solid waste quantities by category

Waste category	Share (%)	Daily mass (kg/day)	Weekly mass (kg/week)	Annual mass (kg/year)	Ton/year (t/year)
Organics	52	131	917	47,818	48
Paper & cardboard	22	55	388	20,221	20
Plastics	11	28	194	10,113	10
Glass	7	18	123	6,439	6
Metals	4	10	71	3,679	4
Residual / misc.	4	10	71	3,679	4
Total	100	252	1,764	91,462	92

Overall, the hotel generates solid waste of 118,837 kg/year, which is 118 t/year.

5.5 Market values of recyclables

The rate of recycling performance has a high variation in terms of the categories of material used and it is dependent on the infrastructure in the state and the state markets. As Texas is a voluntary system of recycling, the realistic rates recovery rates were calculated based on the verified data sources on the U.S. territory, including the U.S. Environmental Protection Agency (EPA), CalRecycle, and the Texas Commission on Environmental Quality (TCEQ), etc.

According to EPA (2022), paper and cardboard rank among the most commonly recovered resources in the U.S., and about 68% of these types of materials are recycled, which is caused by great demand in the market and well-developed systems of collecting these wastes.

Plastic waste was classified as PET, HDPE and mixed plastics by using valid U.S. national resin-distribution data. The assumed near-zero practical recyclability of mixed plastics, comprising LDPE, PP and PS, and film, was in agreement with the results of the same report and EPA national evaluations. The recent EPA-based estimates as published by Resource-Recycling (2025) show that PET bottles are being recycled at an average of 23.2% and HDPE bottles are being recycled at 22.7% in the U.S. It is also reported by the U.S. Environmental Protection Agency (EPA, 2020) that HDPE constitutes about 35% of all amounts of plastic waste produced in the United States, whereas PET comprises about 18%. Using these matching datasets, the other 47% of plastic waste was placed into mixed plastics (LDPE, film, PP, PS and other low-value polymers), which have minimal or no chance of recovery in commercial environments.

Aluminum recycling performance in the United States is very different based on the type of product. The number of beverage cans returned as a result of the interaction of hotels with aluminum is the most suitable indicator as the hotel industry produces almost all the aluminum in the form of cans. Can Manufacturers Institute stated that 77% of the aluminum beverage cans were recycled (Can Manufacturers Institute, 2024). In the case of other metals, such as steel packaging and non-aluminum mixed metal, the EPA national recycling rate of 27.8% is the best and most accurate estimate of the non-aluminum metals recovery in the United States (U.S. Environmental Protection Agency, 2025). Approximately 19% of the metal fraction is aluminum, and the rest of the $\approx 81\%$ is composed of steel and other metals (U.S. Environmental Protection Agency, 2025).

Table 5.5.1.1 Calculated recycled mass results

Category	Hotel + F&B Total (t/year)	Recycling Rate (%)	Recyclable (t/year)
Paper & cardboard	28	68%	19

PET	2	23.20%	1
HDPE	5	22.70%	1
Aluminum	1	77%	1
Other metals	4	27.80%	1
Total recyclable mass	23 t/year		

Sources: EPA (2022); EPA (2020); EPA (2025); Resource-Recycling (2025); Can Manufacturers Institute (2024).

Therefore, the recycling potential can be calculated using the following formula:

$$\text{Recycling potential} = \frac{\text{Recyclable waste}}{\text{Total waste}} \times 100\%$$

$$\frac{23 \text{ t/year}}{119 \text{ t/year}} \times 100\% \approx 19.3\%$$

In the case of plastic recyclables, realistic scrap values of post-consumer plastic gathered during hotel business were defined by trade listing Texas. An authorized listing on TradeIndia indicates that HDPE bottle regrind scrap supplied out of Houston, Texas sells at \$400 per metric ton, which is a typical mid-value of baled and sorted post-consumer HDPE packaging (TradeIndia, 2025a). Likewise, PET bottle scrap with the U.S. origin is quoted in TradeIndia at \$350 per metric ton, which indicates situations of the recent market of recovered PET beverage containers in mixed commercial recycling streams (TradeIndia, 2025b). As they are directly linked with real material offers related to the suppliers registered in Texas and scrap produced in the U.S., they offer the locally relevant reference values to approximate that the plastic fraction produced by the hotel may potentially lead to the recycling revenue.

The recovered fibre prices were directly based on proven data in the U.S. market. Waste360 (2024) wrote that the US national average price of old corrugated containers (OCC) in 2024 was \$72.81 per ton, and residential mixed paper was \$40.25 per ton in 2024 (Waste360, 2024). The paper stream of the hotel is a mixture of cardboard packaging (OCC) and lower grade paper products like tissues, brochures and office paper is a mixture of paper and cardboard material stream to be practically recycled. As such, \$73 per ton which is the value of the checked

OCC as per the check-in value by the Waste360 (2024) is suitable as a representative recyclable market price that the hotel would charge in their paper and cardboard stream in their operations projections.

The values of recyclable metals in markets were calculated based on the publicly available scrap-market values of the U.S. verified commercial recycling facilities. In the case of aluminium beverage cans, ScrapMonster cites a national average of about \$0.52 per pound that translates to about \$1,040 per ton (ScrapMonster, 2025). In the United States, mixed and ferrous scrap prices are wide with regard to grade and the level of contamination in non-aluminium metals. According to a validated commercial price list by J.R. on the Advanced Recyclers, the typical mixed ferrous scrap types have a wide range of between \$80 and \$210 per ton (J.R. Advanced Recyclers, 2025). In order to be conservative and not overestimate the potential value of the mixed ferrous recyclables, a value of mid-range of \$150 per ton was used in this research.

Then we multiply recyclable mass $\times$ price/ton = annual market value.

Table 5.5.1.2 Estimated Market Value of Recyclable Materials

Category	Recyclable mass (t/year)	Unit price (USD/t)	Annual market value (USD/yea
Paper & Cardboard	19	73	1,387
PET	1	350	350
HDPE	1	400	400
Aluminum	1	1,040	1,040
Other metals	1	150	150
TOTAL	23	—	≈ \$3,327/year

Sources: TradeIndia (2025a); TradeIndia (2025b); Waste360 (2024); ScrapMonster (2025); J.R.'s Advanced Recyclers (2025).

The recycling program of the hotel has an estimated annual recovery value worth \$3327 and can provide the hotel with reasonable financial returns and help to meet the Texas waste-management expectations.

5.6 Waste management options

Through the generated municipal solid waste measurement of the hotel, three alternatives to waste management, namely recycling, composting, and energy recovery incineration, have been considered. These alternatives have been evaluated based on the calculation of material flows, recovery efficiencies provided in the literature as well as market/energy performance measures and the portion of the waste quantities, which cannot be recycled by these alternatives, is assumed to be discarded in landfill plants.

5.6.1 Recycling

Recycling is the key waste management strategy that can be used to manage dry and recoverable wastes that will be produced by the hotel, such as paper and cardboard, plastic packaging, and metal fractions. The potential of recycling was calculated as the summation of the total annual tons of each of the material streams of the recycling material, and the material-based recycling rates according to the regulations authority and industrial sources. Those rates are based on actual recovery efficiencies instead of theoretical maximums and hence represent a realistic measure of actual practical results of recycling under normal operating conditions (EPA, 2020; EPA, 2022; Resource-Recycling, 2025).

The economic dimension of recycling was evaluated in Section 5.5, where the recoverable mass of each dry recyclable fraction was combined with current Texas market prices to estimate an annual revenue of approximately \$3,327/year (Table 5.5.1.2).

5.6.2 Composting

Composting was chosen as one of the main waste management alternatives because of a high percentage of organic waste produced by the hotel that is mostly related to food preparation, restaurants, and guest dining. The mass of organic waste was calculated per year to be:

$$M_{organics} = 55 \text{ t/year}$$

Practically, reported composting diversion rates of between 60 and 70 percent in the traditional hospitality and commercial food-service operations are caused by contamination, unsteady separation of sources, and operational conditions. However, in this study a higher composting efficiency factor was adopted, defined as

$$f_{compost} = 0.85$$

relevant to the controlled operational environment of the hotel, waste management by the staff, specific containers of organic waste, and regular collection routines.

The annual mass of waste diverted to composting was therefore calculated as:

$$M_{compost} = M_{organics} \times f_{compost} = 55 \times 0.85 = 46.75 \approx 47 \text{ t/year}$$

To evaluate the contribution of composting within the overall waste management system, the composting potential was calculated relative to the total annual solid waste generation of the hotel, estimated as:

$$P_{compost} = \frac{M_{compost}}{M_{total}} \times 100\% = \frac{47}{119} \times 100\% = 39.3\%$$

5.6.3 Incineration

Incineration was used as a theoretical waste management option to establish the best potential energy recovery of the total waste stream (municipal solid waste) generated in the hotel. Unlike the recycling and composting methods which are applied to the separated waste streams, incineration was applied to the total waste stream, which is a mixture of organics, paper and cardboard, plastics, residual waste, glass and metals. This was a worst-case scenario of diversion where all the waste streams are assumed to be disposed of in a waste-to-energy plant.

The energy recovery of the waste was calculated with the help of the upper heating values (UHV) which differ from the lower ones. These are used since they are the total of gross calorific value of materials including the latent heat of vaporization of water content (for wet materials) and are normally used in theoretical studies on energy potential to give the maximum energy recoverable (Tchobanoglous et al., 1993; ICA, 2023).

The total annual energy potential was calculated using the following expression:

$$E_{inc} = M \times HV$$

where

M is the annual waste mass (kg/year) and

HV is the upper heating value (kJ/kg).

The resulting energy values were converted from megajoules per year to kilowatt-hours per year using:

$$E_{kWh/year} = \frac{E_{MJ/year}}{3.6}$$

The results of this calculation are shown in Table 5.6.3.1 and that means that the total waste stream has the potential of the average energy of around 40.6 kWh on a theoretical basis. The biggest share in this potential is due to plastics and paper due to their high calorific values, while glass and metals have negligible contribution as they are not combustible.

Table 5.6.3.1 The incineration calculated

Waste type	Annual mass (t/year)	Heating value (kJ/kg)	Energy (MJ/year)	Average power (kWh)
Organics	55	4,652	255,86	8.11
Paper & cardboard	28	16,747	468,916	14.87
Plastics	13	32,564	423,332	13.42
Residual / misc.	11	11,630	127,930	04.06
Glass	7	140	980	0.03
Metals	5	689	3,445	0.11
TOTAL	119	—	1,280,463	40.6

It is significant to note that this analysis is the best case scenario without taking into account the actual losses in the form of water content, combustion effectiveness, flue gas treatment and other energy conversion efficiency. Additionally, due to the absence of waste-to-energy (WTE) facilities in the vicinity of the project and the need to prioritise material and organic waste diversion, it is not recommended that incineration should be used as a potential waste management method in this facility. The remaining waste fractions that are not recoverable by recycling and composting are disposed of in landfill at WM - Atascocita Landfill (Harris County, Texas) which is a licensed Type I landfill operating under TCEQ 30 TAC Chapter 330 (Waste Management, 2024).

5.7 Waste Management Plan

According to the analysis of waste management options outlined in Section 5.6, incineration is not an option for this facility given that a waste-to-energy (WTE) facility does not exist within the Houston metropolitan service area, and the presence of more preferred recovery options for major waste fractions (TCEQ, 2023a). The chosen waste management plan is thus based on two active diversion strategies (dry recyclables recovery and composting of organic waste) and residual landfill disposal for fractions whose technical recovery is not feasible for the

conditions of this facility. The plan aims to divert at least 55% of the total annual waste generation from landfill, based on the commercial diversion targets promoted under the TCEQ Title 30 TAC Chapters 328 and 332 (TCEQ, 2023a; EPA, 2023)..

5.7.1 Waste accumulation: quantities and streams

Waste is generated in two subsystems at the hotel: the guest floors (hotel rooms, corridors, lobbies, gym, spa, conference rooms and commercial units on Floors 1-3) and the food and beverage (restaurant front-of-house and kitchen back-of-house on Floor 1). According to the generation rates and composition determined in this analysis, the total daily waste generation is 328 kg/day (119 t/year). This total is broken down in Table 5.7.1.1 according to the waste components, including the accumulation rates (in m³) required in the sizing and frequency of collection of the waste containers.

Table 5.7.1.1 Daily and weekly waste mass and volume accumulation

Waste component	Daily mass (kg/day)	Weekly mass (kg/week)	Bulk density (kg/m ³)	Daily volume (m ³ /day)	Weekly volume (m ³ /week)
Organics	150	1,050	305	0.492	4
Paper & cardboard	76	532	85	0.894	7
Plastics	37	259	85	0.435	3
Metals	14	98	490	0.029	0.2
Glass	22	154	320	0.069	0.5
Residual misc.	29	203	270	0.107	1
TOTAL	328	2,296	—	2	16

The total volume of waste accumulated in a week (about 16 m³) is spread over three streams. Paper, cardboard, plastics, metals and glass (collectively referred to as recyclables) represent the largest volume stream (7 m³/week). Organics represent approximately 4 m³/week and are the largest stream by weight, due to the large amounts of daily food waste resulting from the restaurant and kitchen activities. The remaining 1 m³/week is non-recyclables. This defines the minimum size of containers at the external collection zone (rear of the building) and the minimum collection frequency to avoid spilling, odour and insect problems (TCEQ, 2023b; Harris County Public Health, 2024).

5.7.2 Waste accumulation locations and infrastructure requirements

Internal and external spatial scales of waste accumulation are located at two points in the hotel: distributed internal generation points on eight floors, and a centralised external collection zone at the back of the hotel.

Internally, waste is generated at all times in guest rooms, bathrooms, corridors, lobbies, in the restaurant, kitchen, supermarket, gym, spa and conference rooms. These generation points are spread across the entire building and an internal distributed bin system is needed to ensure that waste is collected at the point of generation, and later transported to the external zone. An adequate quantity and size of bins need to be provided at each location to store the daily generation until the next collection. Details of bin type, size and numbers for each location are provided in Table 5.7.2.1.

Table 5.7.2.1 Indoor waste bin types and required capacities

Bin Type	Description	Floors 1-3 (Public/Service)	Floors 4-8 (Guest Rooms)	Total	Size
Sorting Set	Three compartments for source separation (Recyclables, Non-recyclables, Organics)	8	10	18	3×50 L = 150 L
Small Bin (S)	For WC stalls and guest room bathrooms (unsorted)	12	156	168	20-30 L
Medium Bin (M)	For guest rooms, offices, and lounges (unsorted)	23	0	23	40-50 L
Large Bin (L)	For restaurant and high-traffic service areas	1	0	1	60 L

Bin sizes were chosen from the standard product sizes commonly used in hotels and other commercial facilities (Uline, 2024; Busch Systems, 2024), as opposed to calculated based on volume, to ensure that bins are practical and readily available. Guest room and bathroom bins are unsorted, as source separation by guests in non-public areas is impractical (Pirani & Arafat, 2014); housekeeping staff will sort waste generated from these bins into the main streams while cleaning guest rooms. Bins in public and service areas can be sorted at the point of disposal for areas accessible only to staff and guests.

The hotel's waste streams are collected externally at a collection zone at the back of the hotel property. This must feature a concrete pad with drainage, chain-link fencing and vehicle access to permit licensed haulers to access the containers without having to enter the guest-accessible areas of the hotel. This arrangement is mandated by City of Houston Solid Waste Management Department (SWMD) standards for placement of commercial dumpsters

(City of Houston SWMD, 2024) and aligns with TCEQ standards for container siting in 30 TAC Chapter 330 (TCEQ, 2023b). This zone must include three distinct external containers, each for a different waste stream, of sizes to suit the corresponding total weekly volumes to be collected for each waste stream.

5.7.3. Responsible personnel.

Collection and disposal of waste is undertaken by the hotel's current housekeeping staff who collect waste from internal bins when cleaning rooms and common spaces, then transfer the consolidated waste to the external waste zone via service corridors and emergency exits. This practice minimises the need for a dedicated waste-collection team, as is typical in the hospitality industry (Knox et al., 2014).

Waste handling compliance is managed by a waste supervisor (one per shift) who oversees source segregation, internal waste transfers to the external zone (completed on time) and the external containers, which are to be picked up by licensed waste contractors on schedule. The supervisor also acts as the hotel's liaison with waste service contractors. Justification for personnel and staffing estimates are provided in Section 5.8.

Each waste stream is transported and finally disposed of by licensed contractors that are selected based on the stream and the final disposition location. The contractors, frequency of collection and final disposition facility are provided in Section 5.9.

5.8 Waste Collection Plan

The strategy, bin types and responsibilities were detailed in Section 5.6. This section provides the staffing calculations, collection frequencies, bin emptying routes and floor plans to implement the strategy.

The productivity of housekeeping in hotels is usually determined using a room per room attendant per shift (Knox et al., 2014). The empirical references in the operations domain of hotels report that the average room attendant services 14-16 rooms in 8 hours (Knox et al., 2014).

5.8.1 Personnel and staffing

In this case, for the hotel planning purposes, it is assumed that the 78 guest rooms need to be serviced every day (occupied rooms, stayovers, and due-outs), as is the case in the majority of full-service hotels:

$$N_{rooms} = 78$$

$$N_{\text{housekeepers/shift}} = \frac{78}{14} \approx 6$$

So, based strictly on guestrooms, the hotel needs:

- 6 housekeepers per shift
- 12 housekeepers total across 2 shifts.

The first 3 floors consist of lobby/reception hall + restaurant + supermarket + billiards + conference rooms + gym + spa and introduce a significant public area cleaning load (which is not accounted in the room attendant productivity measure (due to the explicit room attendant 14-16 rooms/shift metric) (Knox et al., 2014).

Thus, 2 extra housekeepers are needed per shift to clean the public and commercial areas of Floors 1-3:

- 8 housekeepers per shift
- 16 housekeepers in total (2 shifts)

A Waste supervisor is a designated employee of the operation department who needs to coordinate and monitor the implementation and compliance of the designated waste management system of the hotel, instead of performing housekeeping operations on a daily basis. The role of this position is to responsibly sort waste (recycling, organic and residual), on time for profit, and liaise with housekeeping and external waste service providers. For the mid-sized hotel (as described above) it is sufficient to employ only one waste supervisor per shift as the waste is generated in the context of operational cycles (for example, the end of housekeeping rounds and kitchen shifts) and the waste supervision task is mainly assigned to the housekeeping staff. Therefore, the supervisor task is managerial and supervisory, which includes quality control, co-ordination and corrective action and not the continuous physical work with waste handling.

Table 5.8.1.1 Waste collection frequency by stream.

Waste stream	Container type	Volume (m ³ /week)	Collection frequency	Hauls per year
Recyclables	4-yd dumpster	~7	2/per week	104
Organics	240-L carts (×2)	~4	2/per week	104
Non-recyclables	4-yd dumpster	~1	3/per week	156

5.8.2. Internal Waste Collection Layout and Movement Strategy.

To ensure that the hotel handles waste fast and efficiently, different layouts for waste collection were designed for each floor using the principles of functional zoning, circulation and architectural design. The layouts consider the placement of the bins, their accessibility and internal waste transportation. This ensures that areas used by guests are not disrupted too much and efficiency standards are reached.

Bin placement takes into account key architectural principles such as separating service flows from guest flows, locating them near areas of high traffic and ensuring that escape routes and vertical circulation areas are always accessible (Neufert, 2012). At the main circulation areas (elevator and transition areas) where waste is concentrated, there are sorting stations with three streams of waste: recyclables, non-recyclables and organics.

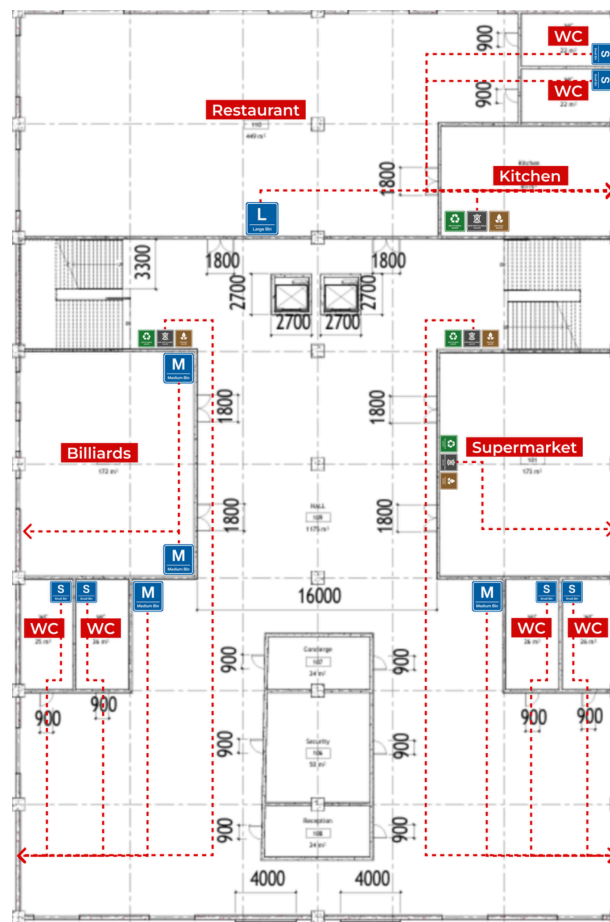


Figure 5.8.2.1 First floor waste collection layout.

The lobby, restaurant, supermarket and business are located on the first floor. The ground floor produces the most waste stream and it has the most diverse waste streams; it has a combination of sorting bins and medium- to big-size bins. Sorting bins are placed around

staircases and lift cores for a better collection of waste. Larger bins are located in places with more service, such as the restaurant's kitchen. Waste is collected in the internal routes (marked with red dotted lines) by housekeeping staff to go to the emergency exits. This makes sure the movement of waste is away from guest routes.

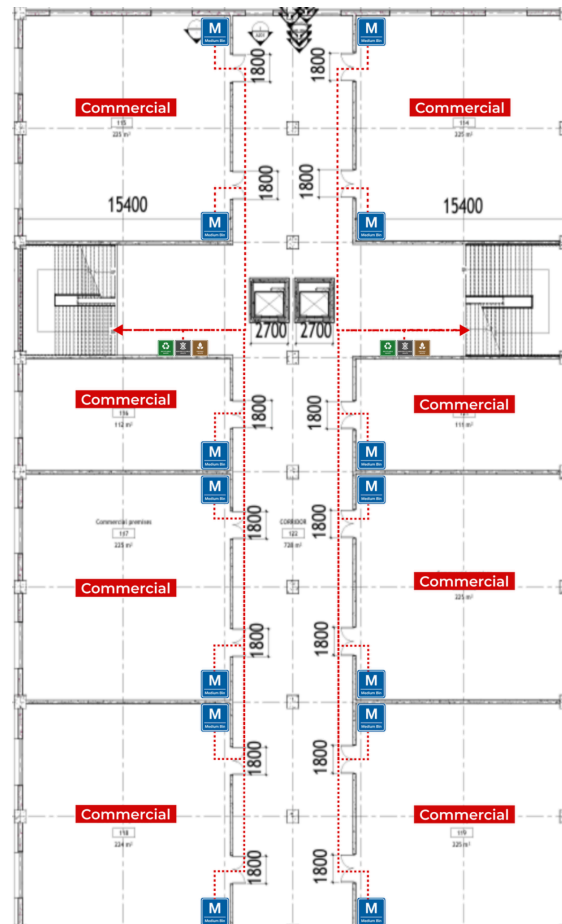


Figure 5.8.2.2 Second floor waste collection layout.

The second floor, which is mainly occupied by offices, adopts a similar approach for centralising bins. There are small-to-medium-sized bins along the corridors at regular intervals to make them accessible without blocking movement. There are also sorting stations near vertical circulation to allow people to dispose of their waste before moving up or down. Trash collection routes still coincide with service corridors and emergency exits, and are not crossed by public routes.

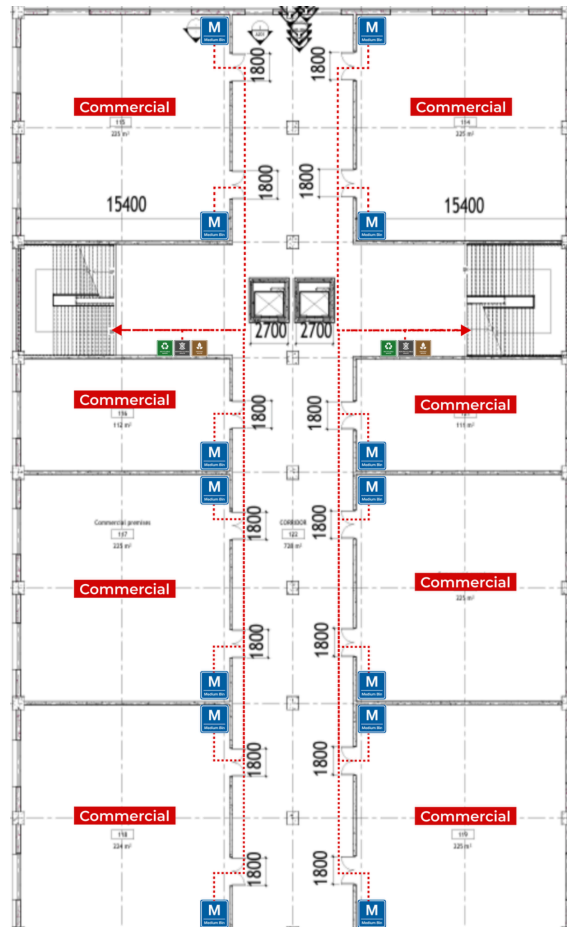


Figure 5.8.2.3 Third floor waste collection layout

The third floor is home to conference rooms, a gym, spa and children's playground. The bins are located in different areas based on the activities and the volume of waste produced. Small bins are placed in the restrooms and changing rooms, and medium bins in the corridors and at the entrance to functional areas. The sorting stations are still in the middle. This ensures that there are high-frequency and localised waste streams, and also keeps the interaction with the waste system consistent for users.

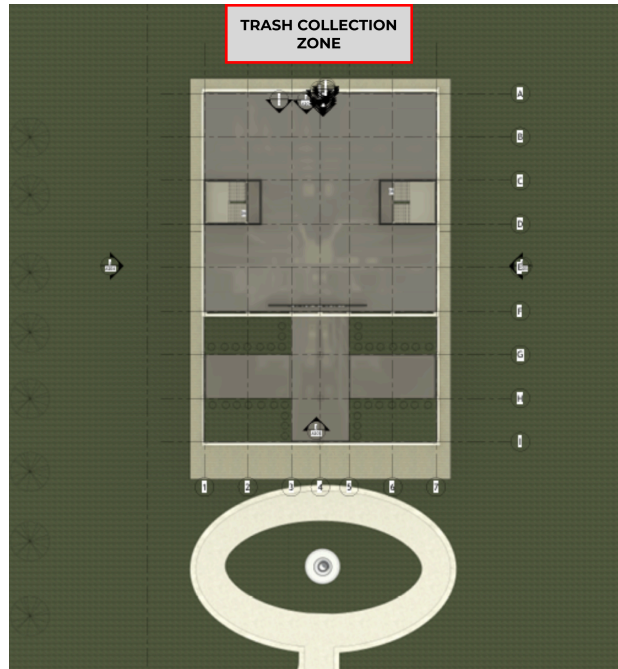


Figure 5.8.2.4 Overall site plan and external collection zone

At the building scale, all the waste streams from the hotel go to a specific external waste collection site at the rear of the hotel. This area has centralised recycling and organic and non-recycling bins. This allows waste collection vehicles to easily access it, but it's removed from the main guests' entry to the site. Each floor has internal transport routes to emergency exits. These are secure waste transfer points from the inside to the outside.

5.9 Integrated solid waste management plan

Goal statement

The aim of this waste management plan is to optimize landfill diversion by reaching a minimum total diversion rate of 55% of annual municipal solid waste (MSW) generation through the simultaneous execution of source-separated dry recyclables recovery and composting of organic waste. This goal is in line with the commercial waste diversion rates endorsed under TCEQ Title 30 TAC Chapters 328 and 332, and is in line with the levels of performance identified as feasible for full-service hotels with managed source separation in the United States (EPA, 2023; TCEQ, 2023a). To reach this goal, the following actions were taken: (1) material-specific recycling rates were applied to the hotel's dry waste stream to estimate how much recyclable mass could be recovered; (2) the 85% composting efficiency factor (which is higher than the 60-70% factor generally used for less controlled commercial environments) was justified by the hotel's exclusive sort centre and supervised collection service; and (3) a three-stream source-separation plan with clearly assigned container types, employee and contractor responsibilities was developed to support these recovery targets on site. The plan's

expected result is the diversion of 70 t/year waste from landfill, or a total diversion rate of 58.8%, surpassing the minimum target.

Overview of the integrated plan

The integrated solid waste management (ISWM) plan synthesises the research in Sections 5.5, 5.6 and 5.7 to develop a connected, site-specific plan for the proposed 78-room hotel in Humble, Texas. This plan is based on the waste management hierarchy outlined in Texas Health and Safety Code Chapter 361, which favours material recovery and organic diversion over residual disposal (TCEQ, 2023a). Its logic of operation - from generation to collection, storage outside the hotel, transportation, and disposal - is shown in Figure 5.8.1.

Point of generation and source separation

Solid waste is produced on all eight floors of the hotel in guest rooms, bathrooms, lobbies and corridors, restaurant, kitchen, supermarket, conference rooms, gym and spa, amounting to 328 kg/day (119 t/year) as calculated in Section 5.4. The main waste streams are recyclables (paper/cardboard, plastics, metals and glass), organics (food and kitchen waste) and non-recyclables (residual and mixed waste).

Source separation is provided at 18 three-compartment sorting stations across all floors, each with three 50-litre bins for recyclables, organics and non-recyclables. Stations are located at all elevator lobbies, corridor transitions and service zones on every floor, according to circulation principles that separate service areas from guest flows (Neufert, 2012). Guest rooms and bathrooms are equipped with unsorted bins due to the impossibility to reliably separate waste streams at source in private areas of this type of hotel; waste is collected and allocated to the proper stream by housekeeping staff in daily rounds (Pirani & Arafat, 2014).

Internal collection and transfer

Internal waste collection is done by 16 housekeeping staff in two eight-hour shifts. Waste is collected and consolidated from all stations, and then transferred to the external collection area via service corridors and emergency exits, maintaining a total separation of waste transfer from guests and occupants (Knox et al., 2014). Each shift has a waste supervisor who monitors sorting compliance, schedules transfers, and communicates with external collection contractors. This management approach is appropriate for a mid-sized hotel like this one, where waste streams are tied to operational cycles such as housekeeping and kitchen operations (see Section 5.7).

All internal streams are brought to the external waste zone at the back of the hotel property. This area is configured to meet City of Houston Solid Waste Management Department guidelines for placement of dumpsters on a paved, fenced, and accessible pad (City of Houston SWMD, 2024). Three external containers are maintained: one 4-yard front-load dumpster

(recyclables), one 4-yard front-load dumpster (non-recyclables) and two 240-litre sealed wheeled carts (organics) colour-coded to match the conventions of Houston area licensed collectors (Waste Management, 2024; Republic Services, 2024).

Off-site collection and final disposition - recyclables

Recyclables are generated at an estimated rate of 7 m³/week, and collected twice weekly by a licensed hauler. Sorted dry recyclables - paper and cardboard, PET, HDPE, aluminium, and other metals - are taken to the North Main Neighborhood Depository/Recycling Center operated by the City of Houston where single-stream co-mingled recyclables are sorted, baled, and shipped to secondary markets (City of Houston SWMD, 2024). Using material-specific recycling rates obtained from EPA (2020), EPA (2022) and Resource-Recycling (2025), 23 t/year of recyclables are recovered from the hotel's dry waste stream with an estimated market value of \$3,327/year (see Table 5.5.1). This represents a dry recyclables diversion rate of 19.3% of the annual waste stream.

Off-site collection and final disposition - organics

Organic waste from the restaurant, kitchen, and guest-floor sorting stations is collected in approximately 4 m³/week and is picked up twice per week by Houston-based Federal Recycling & Waste Solutions, a local contractor for commercial food-waste collection and diversion to local composting facilities (Federal Recycling & Waste Solutions, 2024). The total organics waste stream of 55 t/year was adjusted with a composting efficiency factor of 85%, justifiable given the hotel's separate source-separation infrastructure, monitored daily collection, and sealed organics containers, which help minimise contamination compared to the 60-70% efficiency reported for less-controlled commercial food-service operations (TCEQ 2023c; Saeteros-Hernandez et al. 2025). The composted material of 47 t/year is a diversion of 39.3% of the total annual waste stream generated, and represents the greatest contribution to the plan's landfill diversion goal.

Off-site collection and final disposition - non-recyclables

The estimated residual non-recyclable waste stream, or 1 m³/week, is collected three times per week (the most frequent of the three streams) by Waste Management Inc. and disposed at the WM - Atascocita Landfill, Harris County, Texas, which is a licensed Type I municipal solid waste landfill governed by TCEQ 30 TAC Chapter 330 (Waste Management, 2024). Residual waste is estimated to be approximately 49 t/year of waste that cannot technically be recycled (single-stream recycling) or composted given the operating conditions of this facility.

Combined diversion performance and goal attainment

The combined diversion of the recycling and composting streams is 70 t/year from landfill, or a total diversion of 58.8% of the annual waste stream (23 t recycled + 47 t composted, out of 119 t total). This achievement exceeds the minimum diversion goal of 55%, and validates the effectiveness of the two-stream diversion strategy for a hotel of such size and activity type. Only the remaining 49 t/year, which cannot be recovered locally, goes to landfill in accordance with the TCEQ (2023a) waste hierarchy principle and the EPA (2023) commercial waste minimisation guidelines.

Monitoring and continuous improvement

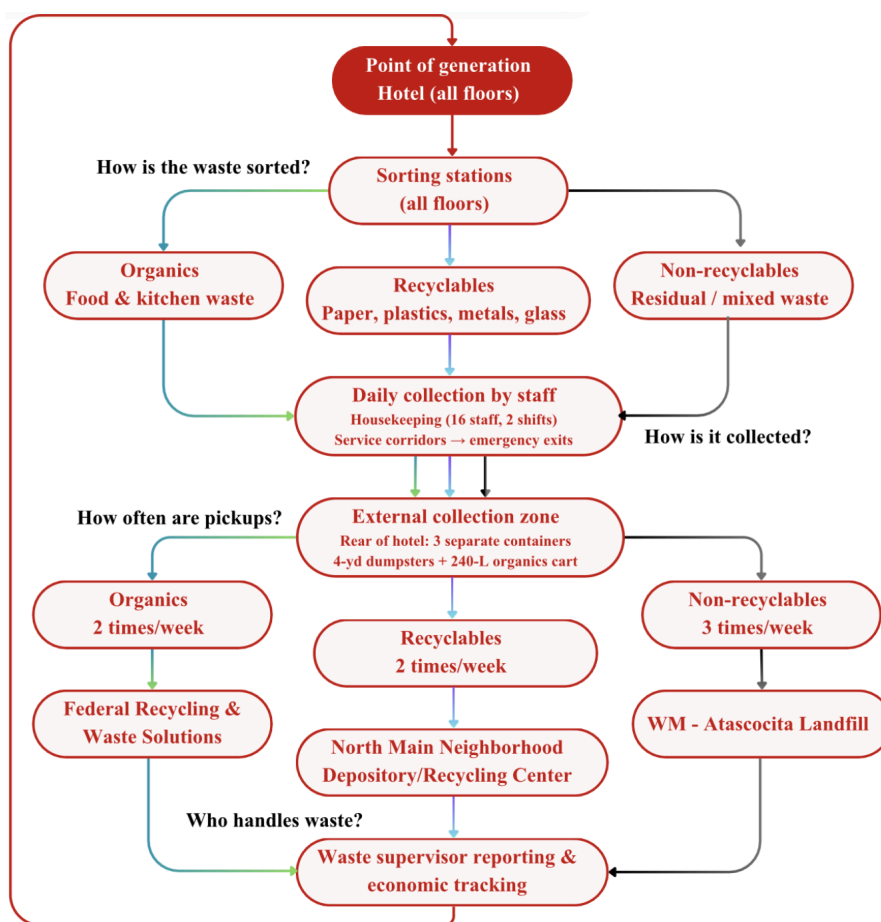


Figure 5.9.1.1 Integrated solid waste management plan flowchart.

The monitoring of plan performance is the role of the waste supervisor, who records container fill, haul frequency and contamination rates of sorting. This data is analysed monthly and reported to the facility manager to detect any inconsistencies with the diversion goals, and to initiate corrective measures in sorting or collection frequency. This monitoring and feedback process, depicted in Figure 5.9.1.1, allows the plan to adapt to changing waste generation patterns and to achieve the 55% diversion target over time (EPA, 2023).

5.10 Finances of Waste Management Plan

5.10.1. Capital Expenditure (CapEx)

Capital expenditure is the up-front, one-time cost needed to set up the waste management system at the hotel. This involves the purchase of waste bins, external collection bins, building the waste collection zone and installation, as well as other infrastructure components such as signs and staff training. These investments are made before the system is put into use and they represent the physical infrastructure needed for all subsequent waste management operations.

The capital cost of the proposed system is \$17,233 as shown in Table 5.10.1.1 The infrastructure and setup costs dominate this total, especially the cost of the outdoor waste collection area, which includes a concrete pad, drainage and fencing to securely and properly store the waste. The cost of equipment, such as indoor sorting units and different types of bins, is lower, as it is based on the choice of commercial, off-the-shelf products, rather than highly specialized or automated equipment.

The estimated cost uses price data and supplier costs for Texas. Bin and container unit costs were sourced from commercial vendors and manufacturers (e.g., Uline, Rubbermaid, Busch Systems) commonly used in commercial and hospitality settings. Dumpster pricing was estimated based on local waste management companies in Texas (e.g., Waste Connections TX). Costs for construction and installation of systems were based on industry cost databases and labor rates (e.g., RSMeans, BLS Texas labor rates), and costs for safety equipment and training were sourced from regulatory and occupational safety resources (e.g., OSHA).

These references were chosen to ensure that all the calculations represent achievable costs for a mid-sized hotel in Texas, rather than average or global costs. In cases where specific information was not available, mid-point estimates were applied to avoid underestimating costs and thus ensure the integrity of the financial model.

In general, the CapEx can be considered reasonable and adequate for the proposed project, given that the system is based on bin collection (no compactors nor other advanced processing technologies are used). This is consistent with the low-volume of waste generation (119 t/year) and allows for the implementation of a cost-effective and efficient system.

Table 5.10.1.1 Capital Expenditure (CapEx).

Item	Qty	Unit Cost	Total	Justification & Source
Equipment				
Indoor Sorting Sets(3×50 L per station recyclables, organics, non-recyclables)	18	\$120	\$2,160	Busch Systems Trilogy 3-stream station, \$115-\$125/unit (BuschSystems.com , 2024)

Item	Qty	Unit Cost	Total	Justification & Source
Small Bins (20–30 L) WC stalls & guest room bathrooms unsorted	168	\$18	\$3,024	Rubbermaid Deskside FG295600BLA, \$16-\$20/unit (Zoro.com , \$18.49/unit)
Medium Bins (40-50 L) Guest rooms, offices, lounges unsorted	23	\$30	\$690	Rubbermaid BRUTE 10-gal FG261000, \$28-\$32/unit (Uline.com , 2024)
Large Bin (60 L) Restaurant back-of-house, high-traffic service area	1	\$45	\$45	Rubbermaid BRUTE 14-gal FG264360, \$40-\$50/unit (Uline.com , 2024)
4-Yard External Dumpster (Recyclables) Rear-of-hotel collection zone	1	\$900	\$900	Steel 4-yd front-load dumpster, \$800-\$1,100 (Waste Connections TX , 2024)
4-Yard External Dumpster (Non-Recyclables) Rear-of-hotel collection zone	1	\$900	\$900	Steel 4-yd front-load dumpster, \$800-\$1,100 (Waste Connections TX , 2024)
240-L Wheeled Organics Carts Rear-of-hotel, sealed compostables	2	\$180	\$360	Rehrig Pacific 96-gal wheeled cart, \$165-\$195/unit (RehrigPacific.com , 2023)
Subtotal: Equipment			\$8,079	
Infrastructure & Setup				
Waste collection area constructionConcrete pad, chain-link fencing, drainage, rear of hotel	1	\$4,500	\$4,500	RS Means commercial concrete pad + fencing, Texas unit cost (RSMeans , 2024)
Multilingual sorting signage (laminated) All floors - 30 signs total	30	\$15	\$450	Commercial laminated signs, \$10-\$20/sign (4imprint.com , 2024)
Staff PPE & initial training materialsGloves, hi-vis vests for 18 personnel (16 housekeeping + 2 supervisors)	1	\$500	\$500	OSHA PPE & training bundle, ~\$28/employee initial cost (OSHA.gov , 2023)
Installation labor: bin placement, signage installation, contractor setup	1	\$1,500	\$1,500	2 workers × 12 hrs × \$62/hr (BLS Texas install labor , 2024)
Contingency (15%)			\$2,204	Unforeseen costs, setup challenges
TOTAL CapEx			\$17,233	

5.10.2. Net Operational Expenditure

Personnel Costs

Labor costs are the largest share of the operating costs, as waste handling in hospitality operations is very labor intensive. The annual cost of labor for waste management is estimated to be \$176,651, as shown in Table 5.10.2. This comprises two full-time equivalent (FTE) waste supervisors for coordination and management, and partial allocation of 16 housekeeping staff who are involved in daily waste collection and handling activities as part of their responsibilities.

Labor hourly rates were determined using Texas-specific labor market data from the U.S. Bureau of Labor Statistics (BLS), in which supervisory positions typically fall in the range of \$20-\$24/hour, and housekeeping staff in the range of \$14-\$18/hour (BLS, 2024). For this study, the following mid-points were chosen: \$22/hour for supervisors and \$16/hour for housekeeping staff.

To estimate the annual salary, we used the following assumptions. Supervisors are assumed to be full-time employees working normal working days throughout the year and housekeeping staff only work a fraction of their time on waste-related activities. This is calculated as:

$$\text{Annual salary} = (2 \text{ supervisors} \times 8 \text{ h/day} \times 365 \text{ days}) + (16 \text{ housekeepers} \times 0.25 \text{ h/day} \times 365 \text{ days})$$

This is because supervisors work an 8-hour shift every day while housekeepers devote 15 minutes (0.25 hours) daily to waste handling activities including collecting and transporting waste from inside bins to outside collection points. This is in line with operational guidelines for hospitality waste management, in which waste-handling is part of the cleaning process (Knox et al., 2014).

The analysis not only considers the base salaries but also the mandatory employer contributions to payroll taxes and benefits as mandated by U.S. and Texas laws. FICA (Federal Insurance Contributions Act) is at 7.65% (employer contributions to Social Security and Medicare). FUTA (Federal Unemployment Tax Act) is applied at 0.6% for the first \$7,000 of every employee's pay, to provide federal unemployment benefits. The state level includes Texas State Unemployment Insurance (TX SUI) at a 2.7% rate, as determined by the Texas Workforce Commission (TWC, 2024).

Further costs include health insurance, estimated at \$7,000 per employee per year based on employer data from across the country (Kaiser Family Foundation, 2024), and workers' compensation insurance, estimated at 2% of payroll for janitorial-related occupations (NCCI Texas, 2024). An employer contribution of 4% of salary to a 401(k) retirement plan is also factored in as per industry standards in the U.S.

This approach allows personnel costs to be calculated as a fully loaded cost of labor, including wages, taxes and benefits. The calculated number is in line with industry benchmarks, in which personnel costs are often the largest component of operational costs in hotel waste management systems.

Table 5.10.2.1 Personnel: Fully Loaded Annual Salary.

Cost Component	Waste Supervisor (2 FTEs)	Housekeeping - waste handling (16 FTEs)	Total	Justification
Hourly rate	\$22.00/hr	\$16.00/hr	—	BLS SOC 37-1011 TX supervisor \$20-\$24/hr; SOC 37-2012 housekeeper \$14-\$18/hr (BLS, 2024)
Annual salary (2 supervisors × 8h × 365d; 16 housekeepers × 0.25h/day × 365d)	\$64,240	\$23,360	\$87,600	Supervisors: full shift × 2 FTEs. Housekeeping: 15 min/day waste allocation per Knox et al. (2014)
FICA (7.65%)	\$4,914	\$1,787	\$6,701	Mandatory federal payroll tax, employer share
FUTA (0.6%)	\$385	\$140	\$525	On first \$7,000 per employee
TX SUI (2.7% new employer rate)	\$1,734	\$630	\$2,364	Texas Workforce Commission new employer SUI rate (TWC, 2024)
Health insurance	\$14,000	\$56,000	\$70,000	TX avg employer cost ~\$7,000/FTE/yr (KFF Employer Health Benefits Survey, 2024)
Workers' Comp	\$1,285	\$4,672	\$5,957	TX class 9015 (janitorial) ~2% of payroll (NCCI TX, 2024)
401(k) Match (4%)	\$2,570	\$934	\$3,504	Common employer benefit 4% of base salary match
Total fully loaded	\$89,128	\$87,523	\$176,651	

Maintenance, Supplies, and Recurring Operational Costs

Beyond personnel costs, the waste management system also entails many ongoing operational costs, including waste collection, consumables, maintenance, and regulatory fees. These inputs are estimated to cost \$24,170 per year, as shown in Table 5.10.2.2

The major component of these costs is the cost of waste collection and disposal services, which is dependent on the chosen collection frequency and container sizes. The annual cost of hauling was determined using:

$$\text{Annual cost} = \text{number of hauls per year} \times \text{cost per haul}$$

Non-recyclable waste is assumed to be collected three times per week (156 hauls/year) at an average cost of \$75 per haul (commercial costs in Texas) (Waste Management Inc. TX, 2024). Recycling is collected twice a week (104 hauls/year) with an average cost of \$50 per haul, based on industry practices that typically charge lower rates for recycling collection than landfill (Republic Services TX, 2024). Organic waste collection is also assumed to be twice per week, at a cost of \$40 per haul (City of Austin Resource Recovery, 2024).

Besides the hauling, other consumables such as liners are needed. Liners were estimated to cost \$600 a year, based on an estimated 3,500 bags used per year for a cost of \$0.08 per bag, based on commercial supplies (Uline, 2024).

The cost for maintenance of equipment was estimated at 2-3% of the total equipment value annually, based on industry average maintenance cost for cleaning & facility equipment (ISSA Cleaning Industry Benchmark, 2022). This includes bin replacement, maintenance and repair of carts.

Personal protective equipment (PPE) also needs to be replaced to ensure safe waste handling. This includes gloves, reflective vests and masks for employees and is estimated to cost \$60 per year per employee (OSHA, 2023).

Compliance with regulatory requirements is also factored in by including the costs of Texas Commission on Environmental Quality (TCEQ) registration, which is estimated at \$300 to \$700 per year for commercial solid waste operators (TCEQ, 2024). We have chosen to use \$500 for this study.

Lastly, the costs of education and awareness are included to promote proper source segregation and minimise contamination. These include costs to reprint signs and materials, estimated at \$750 annually (using typical commercial printing costs).

In summary, the ongoing operational costs were estimated using Texas-specific service rates, city fee schedules and industry standards, to ensure that the financial model accurately represents the true costs for a commercial hospitality facility. These costs are relatively small compared to personnel costs, but critical to ensure the performance, safety and regulatory compliance of the waste management system.

Table 5.10.2.2 Maintenance, Supplies & Recurring Operational Costs.

Item	Annual Cost	Justification
Waste collection service non-recyclables(4-yd dumpster, 3× per week)	\$11,700	156 hauls/yr × \$75/haul; TX commercial pickup avg. \$60-\$90/haul (Waste Management Inc. TX, 2024)
Waste collection service recyclables(4-yd dumpster, 2× per week)	\$5,200	104 hauls/yr × \$50/haul; recycling hauls ~40% lower than residual (Republic Services TX, 2024)
Organics / compostables pickup(240-L carts, 2× per week)	\$4,160	104 hauls/yr × \$40/haul; TX commercial organics fee schedule (City of Austin Resource Recovery, 2024)
Bin liners / waste bags(~3,500 bags/yr across all bins)	\$280	Commercial 60-gal liners ~\$0.08/bag × 3,500 bags/yr (Uline.com, 2024)
Equipment maintenance(bin replacements, cart servicing)	\$500	~2-3% of bin asset value annually (ISSA Cleaning Industry Benchmark, 2022)
PPE replenishment(gloves, hi-vis vests, masks for 18 staff)	\$1,080	~\$60/employee/yr replacement cycle (OSHA PPE replacement benchmark, 2023)
TCEQ annual compliance registration(TX solid-waste commercial facility)	\$500	TX Commission on Environmental Quality commercial solid waste annual fee \$300-\$700 (TCEQ, 2024)
Ongoing education & awareness(Annual reprinting of sorting guides, signage refreshes)	\$750	Estimated annual reprint and campaign cost 30 sign sets at ~\$25 reprint
Total	\$24,170	

The overall annual operational costs (OpEx) of the proposed waste management system is estimated to be \$223,604, as shown in Table X. This includes personnel, waste collection and disposal, maintenance and supplies, as well as annual depreciation of capital costs. Labor costs are the largest component of OpEx, in line with the high-cost, labor-intensive nature of waste management in the hospitality industry.

Capital depreciation was estimated using a straight-line depreciation rate over 10 years, with an annual cost of \$1,723, enabling initial investments to be spread over the system's lifetime.

The annual revenue from the recovery of recyclable materials is calculated at \$3,327, based on the price and quantity of recoverable material. With this revenue, the net annual cost for the system is \$220,277, revealing that while recycling revenue partially offsets the cost of the system, the waste management system is overall a cost center. This outcome is common in hotels where improvements in environmental performance are made primarily through regulatory compliance, sustainability gains and long term cost reductions, rather than profitability.

5.10.3. Net Annual Cost Summary

The net annual cost of the waste management system is \$220,277, after deducting revenues from recycling. While the recycling revenue generates a small profit, it can only offset a small part of the total cost of the system, which is mostly driven by labour and service costs. This shows that the waste management plan is a cost centre, where the primary benefits are environmental performance, compliance and operational efficiency rather than profit.

Table 5.10.3.1 The Net Annual Cost Summary.

Item	Amount (USD)
Personnel salaries	\$176,651
Hauling & disposal	\$21,060
Maintenance & supplies	\$24,170
Depreciation (CapEx / 10 yr)	\$1,723
Total Annual OpEx (Gross)	\$223,604

Less: Recycling revenue	-\$3,327
Net Annual Cost	\$220,277

The waste management system operates as a net cost centre but the achieved diversion rate of 58.8% confirms that the main environmental objective of diverting at least 55% of the total annual waste stream from landfill is achieved within this financial framework, and the cost is in line with the standard operational cost for mid-sized commercial hospitality operations in Texas (EPA, 2023; TCEQ, 2023a).

6. Construction Management

An overview of the construction project is provided in the Project Charter. It includes all of the project's key dates and features. It also includes general information about the projected cost and the reason for the building.

6.1. Project Charter

Table 6.1.1 Project Charter

General Information	
Project Title	Design of an 8-storey Hotel with One Basement in Humble, Texas, USA
Project	This project proposes the development of an 8-story hotel with one basement level on 6819 Manor Way, Humble, Texas. The location

description	is underlain by deep soft clay soils and is situated in a strong wind zone (basic wind speed > 40 m/s). The design will follow local architectural traditions while integrating modern structural, environmental, and construction-management practices. The interim stage focuses on defining geotechnical, structural, architectural, and environmental measures, which will be further refined during detailed design.
Project Start Date	September 2025 (Design Phase)
Project End Date	July 2028 (Estimated Completion)
Project Manager	Arsen Bekzhan
Project Sponsor	Nazarbayev University
Purpose	Goal: ● To construct a safe, sustainable, and culturally distinctive hotel facility that complies with Houston building codes and accounts for wind and geotechnical conditions. Benefits: ● Provides high-quality hospitality services. ● Introduces environmentally friendly building practices. ● Contributes to the local economy and tourism growth.
Project Scope	● Creation of an integrated design covering geotechnical, structural, architectural, environmental, and management

	parts. • Supervision and control of the construction process. • Soil improvement, deep foundations, excavation support, and drainage strategies. • Sustainability measures: waste management, flood mitigation, energy, and water efficiency. • Operational and maintenance planning for long-term use.
Project Deliverables	• 8-story hotel building with a basement. • Guest rooms, lobby, restaurants, and conference areas. • Basement level for MEP and service functions. • Parking areas and landscape development. • Documentation: geotechnical, structural, environmental, and CM studies.
Risks and Issues	• Potential cost overruns from the foundation and excavation. • Environmental risks: flooding, waste management, and noise. • Geotechnical uncertainties: settlement, stability, groundwater. • Safety hazards during deep excavation and structural erection. • Weather delays due to hurricanes and heavy rainfall.

Assumptions Dependencies	• The project will be delivered within the planned timeline. • All stakeholders complete tasks with quality outcomes. • The structural system and foundations are stable and safe. • Flood and wind design meet ASCE 7 and FEMA standards. • The hotel will achieve sustainable operation with green strategies. • Traffic and parking are managed safely on-site.		
Estimated budget	\$30,183,500		
Scheduling	Phase	Deadline	Status
	Project Management	14.11.2025	In progress
	Pre-Construction	20.03.2025	Not started
	Construction	02.09.2027	Not started
	Finishing	13.02.26	Not started
Team members	Aisha Nurymgali Alikhan Kurmangaliyev Arsen Bekzhan Assylbek Bakzhan		

	Azamat Yerassyluly Sanzhar Urmanov Yeladil Yelaman
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6.2 Feasibility study

The feasibility study was done at the initial stage of the project to establish the viability and long-term sustainability of building the proposed hotel in Humble, Texas. The analysis is conducted by examining the location, the environment, the economic potential of the project, and the risks posed by the project to ensure it completes and functions successfully.

6.2.1 Site Analysis

The chosen location at 6819 Manor Way, Humble, TX 77396 is 3 acres in size and offers a lot of space to build, stage, and store. Its suburban nature, being accessible to Houston metropolitan facilities, provides access to business hubs, airports, shopping, and entertainment centers. The building footprint enables inclusion of amenities of the hotel, e.g., spa, gym, and retail facilities on the first three levels, whereas the remaining five levels will include 78 guest rooms. The services and parking have been planned with adequate provisions at the basement level to ensure the movement of traffic and the convenience of guests.

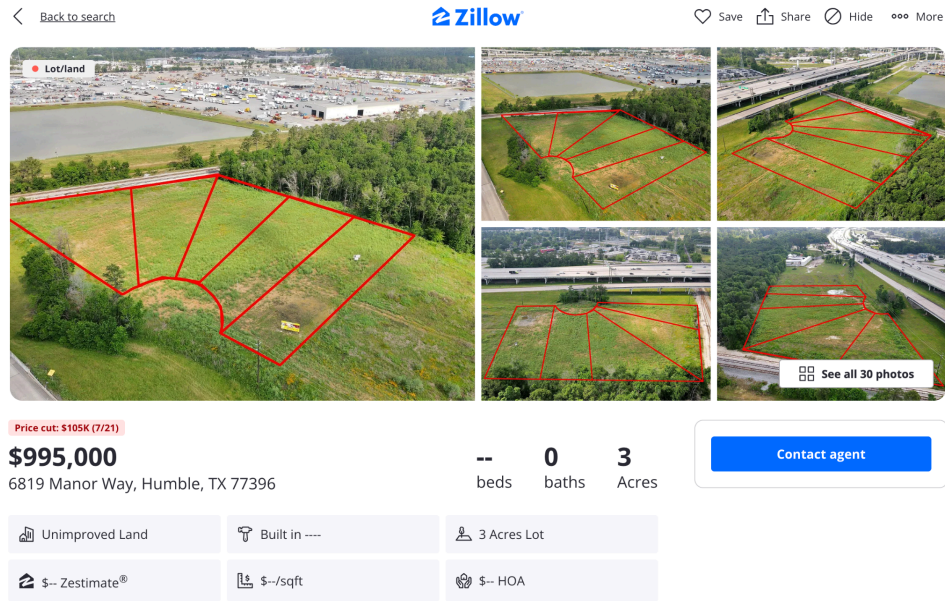


Figure 6.2.1. Site Sale Announcement for 6819 Manor Way, Humble, TX



Figure 6.2.2 3D view of the construction site

6.2.2 Environmental Analysis

The projected occurrence and velocity of hurricane, tornado, or severe storm winds have an extreme risk of hurricane in Humble (TX hurricane map and climate risk report). Hurricanes pose the greatest threat to it. There are higher average maximum wind speeds in Humble than 30

years ago, and 92 percent of houses in Humble have some risk (TX hurricane map and climate risk report). Humble lies within a high wind zone, whereby the winds are basic exceeding 40 m/s and the underlying soil of the site is characterized by deep soft clay soils. These situations demand very strong design criteria, such as deep pile bases, soil enhancement plans, and drainage systems to handle the risk of groundwater and settlement. Also, FEMA-compliant drainage and stormwater management systems will be used to mitigate the risk of floods. The project will be designed according to the ASCE 7 standards of wind and seismic design and will also meet the requirements of the Texas environmental permitting and sustainability requirements.

6.2.3 Economic Analysis

The hotel construction and its operation should have an important economic impact on Humble and the Houston region in general. In the construction process, the project will have direct jobs for engineers, contractors, and laborers, and indirect jobs for suppliers and logistics. When the hotel is completed, the hotel will provide permanent hospitality, management, maintenance, and retail employment.

Its estimated cost of construction is 30.18 million. The hotel has 78 rooms, with a projected average daily rate (ADR) of approximately \$205-220, and an expected occupancy rate of 70-75%. This means the hotel will generate about \$4,720,000 in room revenue yearly. The hotel will also. Extra income of \$2,175,000 will be guaranteed by leasing commercial space on the second and third floors for different purposes: shopping stores, spa and gym services, conference and event rent, and food and beverage outlets. The payback period under conservative assumptions is estimated as 6-7 years; thus, the project is economically viable.

6.2.4 Risk Assessment

Geotechnical uncertainties (settlement and the stability of excavations), harsh weather delays (hurricanes, rainy conditions), and the construction cost being exceeded by the fluctuation in the price of materials are just some of the possible risks. These risks will be addressed by conducting proper soil investigations, robust foundation design, segmented procurement plans, and buffer allowance in the schedule. Diversification of revenue streams and adoption of a flexible pricing strategy will mitigate operational risks like occupancy changes because of

economic recessions. The risks associated with compliance in terms of updates to the code will be dealt with by engaging the local authorities early.

Conclusion

This feasibility study proves that the 8-story hotel with one basement located at 6819 Manor Way, Humble, TX 77396 is feasible and sustainable. This project is ideally positioned to achieve long-term success and community goodwill because of its strategic location, good cultural identity, seamless modern sustainable practices, and financial projections.

6.3 Cost estimation

Using the RSMeansOnline platform, cost estimates for our proposed hotel project were generated. The figure and table below present the outcomes of these calculations. While the overall construction cost appears relatively high, it should be noted that this reflects not only the scale and features of the facility but also the elevated local construction prices in the Houston metropolitan area.

Table 6.3.1. Detailed cost estimation of the project

Costs	Description	Price
Land cost	3-acres land, 130,680 sq.ft land	\$995,000
Preparation of site	Clearing the land, grading and leveling the ground	\$150,000
Pre-Construction	Grubbing, soil testing & analysis.	\$450,000
Foundation	Foundation construction: excavation, footing & basement.	\$2,740,000
Concrete material	Construction of slabs, beams, columns.	\$3,820,000

cost & Steel Reinforcement	Installation of steel reinforcement to meet structural requirements.	
Exterior & Interior Walls	Several-layered walls for exterior and interior walls.	\$8,165,000
Elevators	Installation of 2 elevators.	\$730,000
Cost of Labor and Insurance	Labor cost of workers.	\$4,025,000
Equipment	Purchasing and renting of all required equipment.	\$834,500
Total taxes	Local and Governmental taxes.	\$932,000
Technical equipment and IT systems	Software tools for Hotel Management, Networking, WI-FI maintenance, Systems of Safety.	\$625,000
Interior elements	Furniture, fixtures, essential equipment of guest rooms, commercial spaces, restaurants and other hotel spaces.	\$2,780,000
Emergency savings (15%)	A fund for reserve to meet cost overruns and unexpected spendings.	3,937,000
Total cost	Sum of all spendings	30,183,500

6.4 Work breakdown structure

In this project, the Work Breakdown Structure (WBS) was developed and presented in Figure 6.4.1. Every stage has its subsections that are the key steps needed to accomplish the hotel project in a sequential and logical order. It consists of six major phases:

1. Project Management
2. Pre-Construction
3. Construction
4. Finishing and Installation
5. Quality Assurance and Control
6. Completion and Closeout.

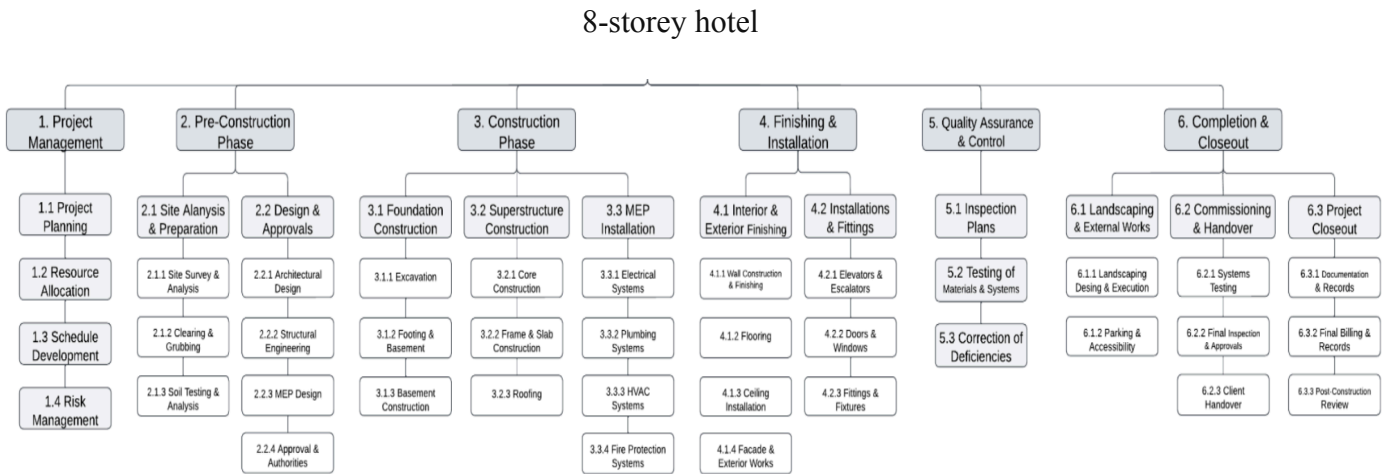


Figure 6.4.1. Work Breakdown Structure

6.5 Scheduling

The application of scheduling is important in the construction management of the 8-story hotel project, in developing the overall scope of the project into a plan based on time. In this type of schedule, each task is allocated a start and finish date, considering the dependencies between the tasks, the resources allocated, and the external constraints (Kawai & Tasaki, 2024). The efficient scheduling in the construction industry helps in making sure that every phase in the project has a logical arrangement, thus ensuring that the construction project is completed within its budget and within the allocated time. The most commonly used scheduling tools include Gantt charts, which are easy to use and follow the progress of the tasks and their completion. The visual representation of these gives the practitioners the ability to track the progress of individual tasks, as well as to identify possible scheduling conflicts.

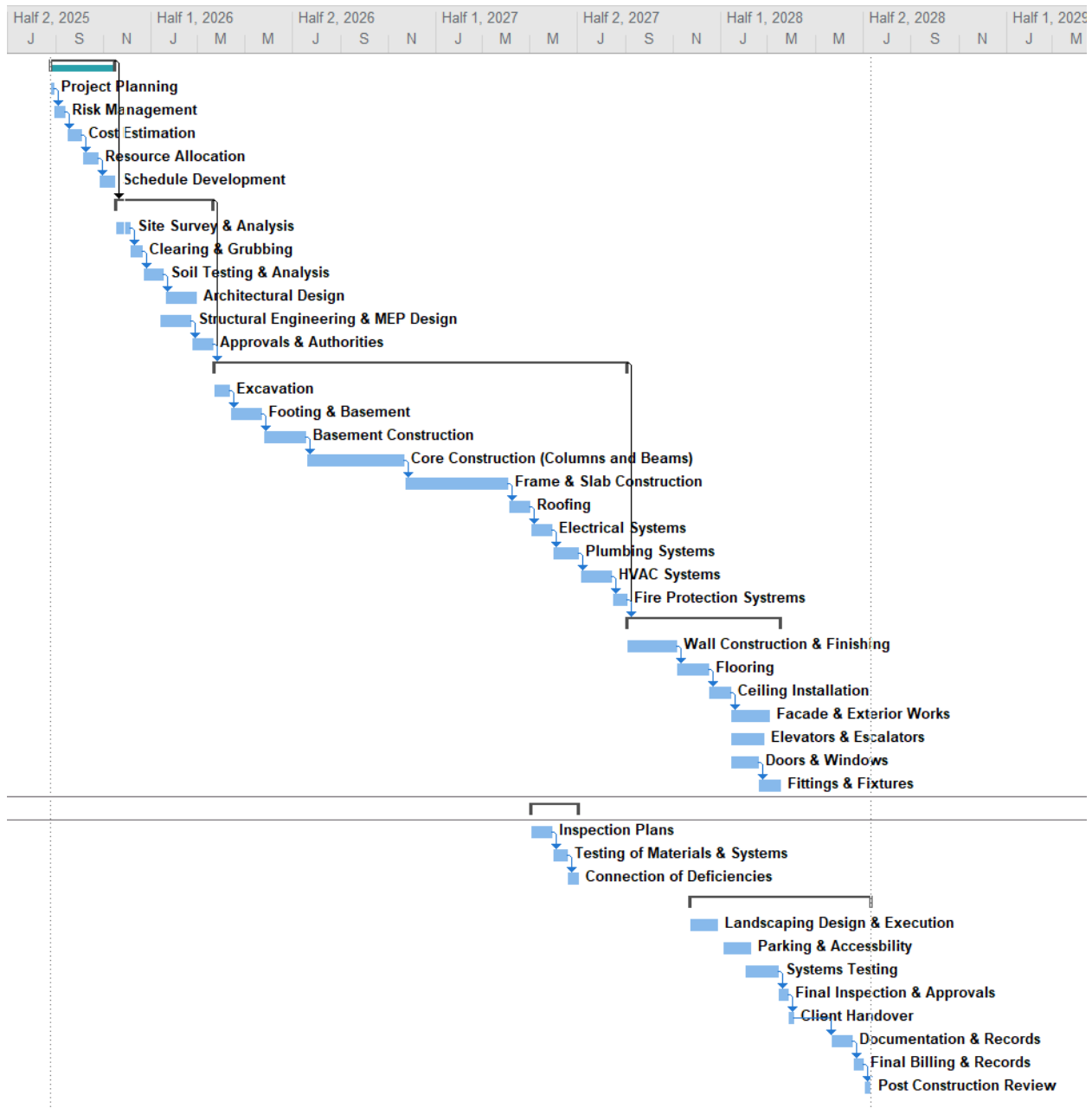


Figure 6.5.1. Gantt Chart

		1	Project Management	60 days	Mon 8/25/25	Fri 11/14/25
		1.1	Project Planning	4 days	Mon 8/25/25	Thu 8/28/25
		1.2	Risk Management	11 days	Fri 8/29/25	Fri 9/12/25
		1.3	Cost Estimation	15 days	Mon 9/15/25	Fri 10/3/25
		1.4	Resource Allocation	15 days	Mon 10/6/25	Fri 10/24/25
		1.5	Schedule Development	15 days	Mon 10/27/25	Fri 11/14/25
		2	Pre-Construction Phase	90 days	Mon 11/17/25	Fri 3/20/26
		2.1	Site Survey & Analysis	14 days	Mon 11/17/25	Thu 12/4/25
		2.2	Clearing & Grubbing	11 days	Fri 12/5/25	Fri 12/19/25
		2.3	Soil Testing & Analysis	20 days	Mon 12/22/25	Fri 1/16/26
		2.4	Architectural Design	30 days	Mon 1/19/26	Fri 2/27/26
		2.5	Structural Engineering & MEP Design	30 days	Mon 1/12/26	Fri 2/20/26
		2.6	Approvals & Authorities	20 days	Mon 2/23/26	Fri 3/20/26
		3	Construction Phase	379 days	Mon 3/23/26	Thu 9/2/27
		3.1	Excavation	15 days	Mon 3/23/26	Fri 4/10/26
		3.2	Footing & Basement	30 days	Mon 4/13/26	Fri 5/22/26
		3.3	Basement Construction	40 days	Mon 5/25/26	Fri 7/17/26
		3.4	Core Construction (Columns and Beams)	90 days	Mon 7/20/26	Fri 11/20/26
		3.5	Frame & Slab Construction	95 days	Mon 11/23/26	Fri 4/2/27
		3.6	Roofing	20 days	Mon 4/5/27	Fri 4/30/27
		3.7	Electrical Systems	20 days	Mon 5/3/27	Fri 5/28/27
		3.8	Plumbing Systems	25 days	Mon 5/31/27	Fri 7/2/27
		3.9	HVAC Systems	30 days	Mon 7/5/27	Fri 8/13/27
		3.10	Fire Protection Systems	14 days	Mon 8/16/27	Thu 9/2/27
		4	Finishing & Installation	140 days	Fri 9/3/27	Thu 3/16/28
		4.1	Wall Construction & Finishing	45 days	Fri 9/3/27	Thu 11/4/27
		4.2	Flooring	30 days	Fri 11/5/27	Thu 12/16/27
		4.3	Ceiling Installation	20 days	Fri 12/17/27	Thu 1/13/28
		4.4	Facade & Exterior Works	35 days	Fri 1/14/28	Thu 3/2/28
		4.5	Elevators & Escalators	30 days	Fri 1/14/28	Thu 2/24/28
		4.6	Doors & Windows	25 days	Fri 1/14/28	Thu 2/17/28
		4.7	Fittings & Fixtures	20 days	Fri 2/18/28	Thu 3/16/28
		5	Quality Assurance & Control	44 days	Mon 5/3/27	<u>Thu 7/1/27</u>
		5.1	Inspection Plans	20 days	Mon 5/3/27	Fri 5/28/27
		5.2	Testing of Materials & Systems	14 days	Mon 5/31/27	Thu 6/17/27
		5.3	Connection of Deficiencies	10 days	Fri 6/18/27	Thu 7/1/27
		6	Completion & Closeout	166 days	Tue 11/23/27	Tue 7/11/28
		6.1	Landscaping Design & Execution	25 days	Tue 11/23/27	Mon 12/27/27
		6.1	Parking & Accessibility	25 days	Tue 1/4/28	Mon 2/7/28
		6.3	Systems Testing	30 days	Tue 2/1/28	Mon 3/13/28
		6.4	Final Inspection & Approvals	10 days	Tue 3/14/28	Mon 3/27/28
		6.5	Client Handover	5 days	Tue 3/28/28	Mon 4/3/28
		6.6	Documentation & Records	20 days	Mon 5/22/28	Fri 6/16/28
		6.7	Final Billing & Records	10 days	Mon 6/19/28	Fri 6/30/28
		6.8	Post Construction Review	7 days	Mon 7/3/28	Tue 7/11/28

Figure 6.5.2. Scheduling details in Microsoft Project

The Gantt chart in the attachment was generated in Microsoft Project, which is an effective software in project management and has ensured the correct scheduling and allocation of tasks. The realistic working calendar used in this project has included holidays, Saturdays, and Sundays as non-working days in order to have a balanced workload and sustain efficiency within

an assumed 8 work hours. The dependency logic provided in the schedule is a vivid representation of the project flow, and certain tasks can be extended over time to make sure that the crucial milestones are completed on time. The chart will include a comprehensive timeline between August 2025 and July 2028, the full project life cycle through its six major stages, namely, Project Management, Pre-Construction, Construction, Finishing and Installation, Quality Assurance and Control, and Completion and Closeout.

The schedule starts with Project Management, where parallel actions such as Risk management, cost estimation, resource allocation, and schedule development will form the governance and baseline of the whole project. It is then succeeded by the Pre-Construction Phase, which has a duration of 90 days (until the beginning of March 2026). This stage includes Site Survey and Analysis, Soil Testing, and an extensive Architectural, Structural Engineering, and MEP Design, which culminates in the all-important Approving and Authorities phase, which in turn is a significant precursor to the beginning of physical construction.

The longest part of the project is the Construction Phase, during which the project is 379 days (March 2026 to March 2027). This stage starts with Excavation, after which Footing and basement Construction take place. Core Construction (Columns and Beams) is an important requirement, which takes 90 days. The order is Frame & Slab Construction and Roofing, then the installation of major MEP Systems (Electrical, Plumbing, HVAC, and Fire Protection). The time of these activities is the complexity of the work of a multi-storey hotel that requires careful coordination and accuracy in implementation. Overlapping with this is the Quality Assurance and Control stage, which requires 44 days (beginning May 2027). Activities here start with Inspection Plans and Testing of Materials and Systems, ensuring the hotel meets all the functional and safety standards upon handover.

It will take 140 days (since September 2027) in the Finishing & Installation phase. This phase converts the structure into a functional hotel by activities such as Wall Construction and Finishing, Flooring, ceiling installation, and facade and exterior works. The last phase of Completion and Closeout will require 83 days (since the beginning of March 2028). The other important activities are landscaping design and implementation, parking and accessibility, and

critical systems testing (taking 30 days) to fully commission all hotel systems. The stage ends with final inspection and approvals, client handover, administrative final tasks of documentation and records, final billing and records, and a post-construction review (scheduled until July 2028).

Microsoft Project is used to ensure that the project is closely controlled using working hours and days off of the real world, with the ability to use overtime on critical pathways to achieve a high quality fully functioning hotel.

6.6 Annual Cash Inflow

Table 6.6.1. Net profit in a Year

Name	Amount
Revenue of rooms	\$4,720,000
Revenue of restaurant	\$1,320,000
Revenue of commercial renting	\$2,175,000
Operational expenses	\$3,700,000
Net Profit	\$4,515,000

The Revenue from rooms is projected at \$4,720,000. This figure is derived from the hotel's unique room mix and expected occupancy rates, leveraging its strategic location near Bush Intercontinental Airport (IAH) and the Generation Park business hub in Humble, Texas.

The hotel features a total of 78 rooms across three categories:

- 18 Standard Rooms (37.5 sq. m.)
- 40 Standard + Rooms (56.25 sq. m.)
- 20 Luxury Suites (67.5 sq. m.)

Assuming an average daily rate (ADR) of approximately \$205-220 and a calculated average occupancy of about 70-75% across all room types for a 365-day year, this revenue stream is achievable. The \$4,720,000 demonstrates the strong potential for guest accommodation income within the Houston-area market.

Revenue from commercial renting is another substantial element in the cash inflow of the annual cash as it constitutes 2,175,000. This revenue source is produced within several commercial blocks that are devoted: The ground floor area is assigned to a billiards room and a supermarket, each 166 sq.m. The second floor (2,400 sq. m. area) will be devoted to commercial use, six large (231 sq. m.) and two smaller (115.5 sq. m.) commercial spaces will be located there. The proposed strategy will take advantage of the fact that there are no nearby malls or clothing shops and provide local residents and hotel guests with an easy place to shop. The third floor has rentable areas comprising the large conference halls (600 sq. m.), used to hold meetings, company presentations, and even events.

Such a diversified business plan offers a constant, non-lodging source of revenue, which can be important in mitigating the overall project risk.

The total of the operating revenues of the room, the restaurant and the commercial rentals amount to \$8,215,000 (\$4,720,000 +\$1,320,000 +\$2,175,000). With Operational expenses deducted and recorded as 3,700,000, resulting Net Profit stands at 4,515,000, which shows a strong economic feasibility of operation in this strategy that consists of hospitality, retail, and recreational services. The good financial figures are an indication of a good future to the payback period of 6-7 years of the hotel in terms of its long run success in the competitive Texas market.

6.7 Risk Management

Risk management is essential in achieving the successful delivery of projects on time, within budget, and satisfying the expectations of stakeholders. In order to accomplish this, it is important to have clear methods of risk identification, assessment, and constant monitoring. In the absence of such strategies, any form of interference, such as a labor strike that causes shortages in materials or a supply chain caused by bad weather, would increase the costs and halt the progress. Research recommends the use of a risk matrix to effectively rank threats and their probability and effects. The results of this assessment immediately guide the most appropriate targeted response, which is avoiding, transferring, or reducing the risk (Gad et al, 2019).

Through qualitative and quantitative analyses in a risk framework of a project, groups of people may come up with a comprehensive risk register. This practice encourages the industry's best standards and allows dynamic, flexible planning. The objective is to reduce the losses in case of time or funds constraint deadlines, and the project's durability and ability to face unforeseen changes with minimal losses.

The 3x3 Matrix Applied to the Hotel Project. A 3x3 risk matrix is a tool for constructing hazard management in the Hotel Construction Project. The scores of the risks in this matrix are determined by a composite measure of Severity Level and Probability Level. Rather than developing the entire risk plan, this matrix offers a simple three-tiered system of analyzing the impact of risks and risk probability: Strong, Moderate, and Low risk.

The visual system enables the project team to simply achieve a quick visual assessment of which risks demand the early resources and mitigation efforts. It is an effective tool because of its simplicity and effectiveness. It is also able to give a clear guideline on how risks are categorized, and both the management and on-site teams are able to interpret and understand the risks and properly control the risks. Such a systematic arrangement enables the project team and make relevant decisions, risks priorities, and eventually enhance the project's resilience and completion on time.

Table 6.7.1. 3x3 Matrix of Dangerous Event

Dangerous event's severity				
Dangerous event's probability		Minor	Moderate	Serious
	Rare	Low risk [1]	Low risk [2]	Moderate risk [4]
	Possible	Low risk [2]	Moderate risk [4]	High risk [6]
	Certain	Moderate risk [3]	High risk [6]	High risk [9]

The risk score is calculated through the multiplication of the probability of the risk by its severity:

Table 6.7.2. Table of risk scores and risk reduction management

Risk		Category of risk	Probability	Severity	Risk score	Risk reduction management
Project Management	PM 1	Workforce inefficiency	1	2	2	Use time monitoring tools to evaluate performance and offer bonuses for accomplishing certain tasks.
	PM 2	Weak corporate culture	2	2	4	Establish strong project leadership, open communication lines,

						and schedule corporate events to foster team cohesion.
	PM 3	Project scope modifications	2	2	4	Prepare a contingency budget and timeline to address unforeseen revisions from.
	PM 4	Project delays	2	2	4	Execute precise project planning and tracking.
	PM 5	Employee strikes	3	2	4	Improve scheduling efficiency and ensure workforce versatility to minimize disruption from labor action
	PM 6	Lack of qualified labour	3	3	9	Contract specialized work packages to established local service providers.
Design	D 1	Changes in design	2	1	2	Streamline the change request authorization procedure to accelerate project timelines.

	D 2	Flawed design specifications	2	2	4	Periodic checks can detect and rectify design discrepancies.
	D 3	Non-compliance with construction standards	1	3	3	Track evolving requirements and execute appropriate project revisions..
Construction	C 1	System failures	1	3	3	Conduct regular upkeep checks and arrange for short-term equipment leasing to cover unexpected breakdowns.
	C 2	Potential dangers	3	2	6	Enforce strict safety measures protocol, provide compulsory daily safety orientation and ensure requisite protective clothing is always used.
	C 3	Increased project expenditure	1	2	2	Have sustained cost control, establish rigid controls on expenditure, and seek to reduce the prices of the vendors.

Environmental	E1	Inaccuracies in the environmental impact study (EIS)	1	2	2	Conduct a thorough review of environmental studies.
	E2	Assessment of environmental impact	2	2	4	Reduce adverse environmental consequences, utilize eco-friendly materials, and pursue LEED certification standards.
	E3	Impact of climate conditions	3	2	6	Monitor weather conditions closely and come up with strong emergency plans.

External	R 1	Adjustments in statutory requirements	1	2	2	Hire a compliance professional to take care of the arising regulatory issues.
	R 2	Political challenges	1	2	2	Disperse the supply base in different geographic locations so as to avoid dependence in a given geographic location.

	R 3	Substandard material quality	2	3	6	Conduct the required inspection of incoming materials and undertake constant quality checks during construction.
	R 4	Inflation issue	3	3	9	Negotiate fixed prices of the basic materials and keep a special contingency fund in case of any unforeseen expense.
Geotechnical	G1	Unforeseen soil issues	2	3	6	Perform comprehensive geotechnical studies during the project's preliminary phase.
	G2	High windy area	3	3	9	Utilize contemporary wind-resistant construction techniques and specialized materials.

Table 6.7.3. 3x3 Matrix of Dangerous event's probability

Dangerous event's severity				
Dangerous event's		Minor	Moderate	Serious
	Rare		PM1, C1, E1, R1, R2	D3

probability	Possible	D1	PM2, PM3, PM4, D2, E2	R3, G1
	Certain		PM5, C2, E3	PM6, C3, R4, G2

The building project's risk assessment includes a thorough table and risk matrix, identifying possible risks based on probability and severity. High likelihood (serious&certain) risks, such as employee strikes, inflation issues, and high winds, are highlighted in red and require quick action with contingency plans and structural safeguards. Regular monitoring is necessary for moderate risks like changes in design and project delays, while basic protections handle low-risk concerns like workforce inefficiency and equipment mistakes. This structure focuses on addressing the primary challenges to project stability and efficiency, ensuring their presence.

6.8 Quality Management

One of four major factors which define the success of a project is quality management, as well as cost, time, and safety (Ansah et al., 2019). It contributes significantly to the long-term reputation of a construction company as customer satisfaction is a necessary goal. That is why managers and superintendents should make sure that materials employed and the work done should be of the necessary quality. Contractors are also involved in covering financial implications of work not in line with the original plans. The high quality is beneficial to all members of the team since it decreases the number of reworks and enhances the overall performance of the project. The superintendent in this project is charged with the responsibility of conducting quality checks besides the other roles. A proactive method of quality control planning will assist in the successful implementation of the project as it will eliminate defects and would eliminate unnecessary repairs. Hence, the project manager and superintendent should succeed in planning and controlling the construction operations to make sure that the materials, labor, and subcontractors meet or even surpass the contract requirements.

6.8.1 Quality Control Measures

Quality control measures have a few major steps that are followed. First of all, there is a close consideration in terms of quality and competent subcontractors choice. Secondly, quality is ensured through proper orderly submission and approval process. The deliveries of the approved materials to the site are subjected to a respecification. When any of the materials fail to meet the requirements, they are discarded, and cannot be offloaded in the truck. Other than this there are frequent checks on materials that have been kept in the jobsite to confirm that it has not been damaged due to the weather conditions and wrong storage facilities.

The subcontractors would be required to carry out mock-ups prior to undertaking any kind of construction work either in foundation works or wall construction such that they are able to demonstrate the standard of their workmanship. These mock-ups are deliberated and accepted to suit the needs of the project. At the same time, the superintendent verifies the availability of all the necessary supplies, equipment, and tools and prepares them in advance before the work can be approved.

The QS inspector or the superintendent at the site controls the work after the construction work has started to ensure that the workmanship is up to the required quality standard that has been approved. The checks are undertaken on a daily basis where one gets to know that the work is being carried out as indicated by the contract documents and the technical specifications. Daily basis reports are developed to reflect the levels of progress going on at the site and bring forth the weaknesses or failures. Any issues awaiting completion are recorded in a log of deficiency which is always maintained until corrective actions have been undertaken and the issues sorted out to the latter.

The project is also pre-final inspected towards the end of the project, such that to view all the work done as well as to see whether the construction has been carried out as per the set standards of quality. Based on the defects that may persist, a punch list takes note of the same and enough time is provided to make the necessary repair and work. Finally, through the superintendent, the owner representative of the company and a representative of the company,

the final check is done and the owner is assured that all the activities are satisfactorily done before the project is handed over.

Quality metrics in construction project

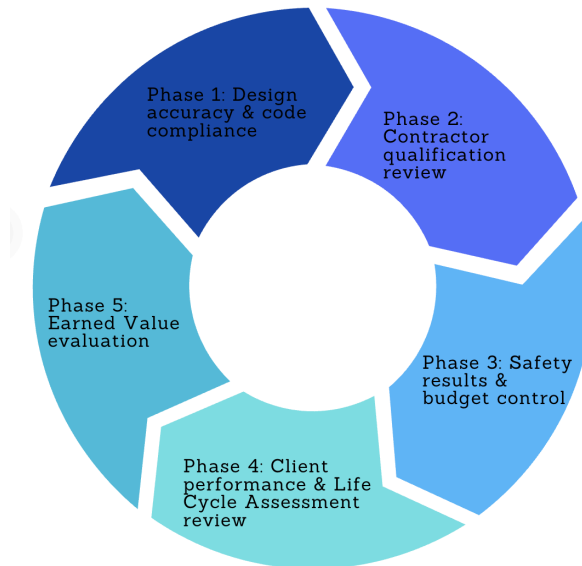


Figure 6.8.1. Quality metrics

Table 6.8.1. Description of the quality metrics

Metric category	Description	Target
Phase 1: Design accuracy& code compliance	Verifies architectural and engineering drawings are in accordance with building codes and specifications	100% code compliance
Phase 2: Contractor qualification review	Assesses subcontractor qualifications, mock-up and access to necessary equipment and tools before work commences	All subs pre-qualified before start
Phase 3: Safety results& budget control	Inspects site daily to ensure work complies with drawings and tech specs; log of deficiencies until resolved	Zero open deficiencies; $\leq 10\%$ costvariance
Phase 4: Client performance& Life Cycle Assessment	Pre-final inspection of all work; punch list prepared for remaining issues and corrected before turnover	100% punch list items resolved
Phase 5: Earned Value evaluation	Superintendent and owner's rep final walk-through verifies all work satisfactorily completed	Owner sign-off before project handover

6.8.2. Quality Control Checklist

To ensure the structural integrity and long-term serviceability of the Asyl Nomad hotel, a specific Building Construction Quality Checklist was created as the key quality control (QC) metric. This framework assures that all stages of physical execution precisely correspond to the design specifications, Texas building regulations, and ACI 318-19 standards set during the structural design process.

As shown in Figure 6.8.2, the checklist is carefully divided into four separate phases, each representing a crucial milestone in the construction process.

Building Construction Quality Checklist

Project Information:

- **Project Name:** “Asyl Nomad” 8-storey hotel
- **Location:** 6819 Manor Way, Humble, Houston, USA
- **Project Manager:** Arsen Bekzhan
- **Date of Inspection:** 07.04.2026
- **Inspection Team Members:**
 - Azamat Yerassyluy
 - Yeladil Yelaman
 - Alikhan Kurmangaliyev

Phase 1: Pre-Construction & Site Preparation

- ☐ Validate site clearance of all waste, rubble, and unwanted greenery.
- ☐ Audit site topography to ensure ground elevation and flatness align with technical blueprints.
- ☐ Certify the safe extraction of any toxic or hazardous substances found on-site.
- ☐ Evaluate soil stability protocols and the effectiveness of runoff prevention systems.
- ☐ Confirm the positioning and accessibility of all primary utility hookups.

Phase 2: Foundation & Substructure Inspection

- ☐ Validate that excavation matches authorized blueprints regarding specific depths and dimensions.
- ☐ Audit soil density to ensure ground compaction meets the required load-bearing standards.
- ☐ Authenticate rebar placement, ensuring a 3.0-inch (75 mm) clear cover per ACI 318-19.
- ☐ Certify that the concrete composition strictly adheres to the 6000 psi design mix.
- ☐ Confirm the integration of drainage systems and waterproofing to prevent hydrostatic pressure.

Phase 3: Superstructure & Vertical Circulation

- ☐ Verify vertical and horizontal alignment of column cores for the 8-story tower.
- ☐ Inspect floor slab framing and reinforcement for guest room levels (Floors 2-8).
- ☐ Audit elevator cores and staircase framing in compliance with IBC fire safety and egress.
- ☐ Validate Mechanical, Electrical, and Plumbing (MEP) rough-ins within the structural frame.
- ☐ Confirm fire-stopping and soundproofing installations between guest suites.

Phase 4: Finishing & Sustainability Integration

- ☐ Inspect exterior cladding and thermal barriers for energy efficiency.
- ☐ Verify the installation of solar panels and green roofing systems on the roof level.
- ☐ Conduct a final audit of waste management systems for occupancy type R1.
- ☐ Complete the final LEED minimum requirement checklist for sustainable design.
- ☐ Obtain all necessary municipal permits and certifications for building handover.

Figure 6.8.2. Quality Control Checklist

6.9 Procurement Planning

A key aspect of project implementation is stakeholder management which entails the ordered organization of the different people and institutions that a construction project would affect. The effective project delivery process in the construction industry is considerably dependent on how well one manages to negotiate the dynamic and fragmented relationships within the multiplicity of participants. This is a very intricate network of stakeholders interactions that is normally witnessed in a construction project (Chinyio and Olomolaiye, 2010).

The inherent guidelines of efficient stakeholder relationship management comprise:

Stakeholder Identification. This is the first step that considers any internal and external entities involved in the project. Ordinary parties include the client and design consultants, regulatory agencies and the end-users.

Expectation and Needs Analysis. It is important to evaluate specific goals of every group in order to harmonize the project goals. An example of these would include the neighbors who would be preoccupied with ensuring that the project does not disrupt their traffic and their dust load, and the investors and financial sponsors who would be mostly concerned with the returns on the investment (ROI) and the schedule to completion.

Strategic Communication. Having strong information exchange channels will make sure that all the parties are updated on the milestones of the project, changes in the design and risks involved. In addition, offensive communication systems play a crucial role in reducing the conflict as well as solving the imminent disagreements that occur during the construction period.

Active Engagement. The inclusion of stakeholders in the process by ensuring active involvement in routine site meetings, consultations with the people and feedback loops would ensure a collaborative atmosphere. Such openness helps not only to establish mutual trust but also a sense of responsibility in the project team.

A full-fledged stakeholder management approach will reduce risks in the project, make the project execution smooth, and will create a base of trust that may lead to subsequent business relationships and favorable corporate image.

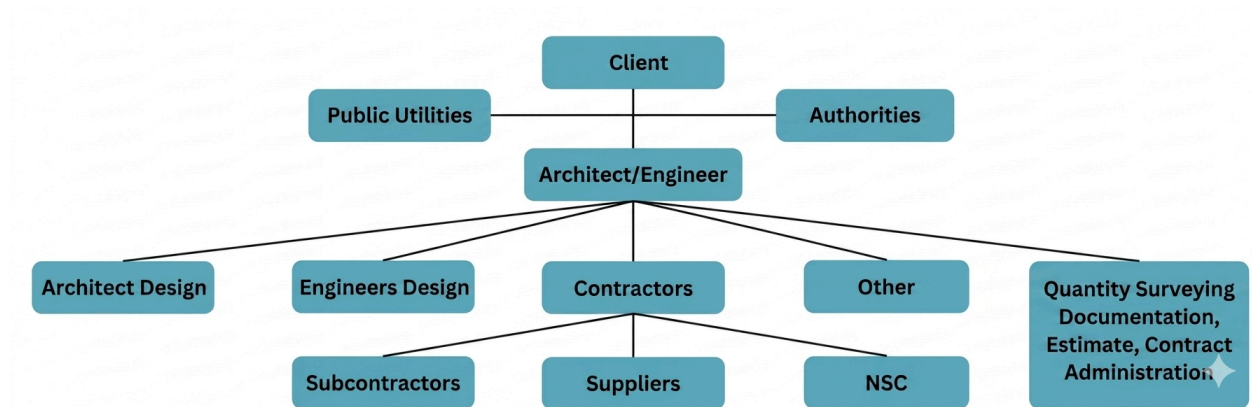


Figure 6.9.1. Stakeholders hierarchy in Construction Procurement

The Client/Property Owner is the top stakeholder in any project. They are integral to the project's success, being the source of funding, strategic decision making and controlling the pace of progress. The second level of stakeholders is the Architect and Engineering team. They play a management role in their relationship with external stakeholders such as Public Utilities and Authorities and in the coordination of the specialist stakeholders below. They are responsible for the technical management of the work and allocation of work packages to the design teams, general contractors and surveyors.

A Stakeholder Analysis Matrix is used to classify each party based on this project structure and their role, interest and influence.

Interest: This criteria measures the stakeholder's interest in the successful outcome of the project.

Influence: This measure determines the stakeholder's ability to influence the project's direction and outcomes.

In this system, the amount of responsibility allocated to a stakeholder is directly correlated to his or her ranking in these two categories; as interest and influence increase the responsibility of the stakeholder in the project lifecycle increases.

Table 6.9.1. Stakeholder Matrix

Stakeholder's Position	Role	Influence	Interest
Owner	Initiation and ownership	High	High
Civil Engineer	Structural design and technical oversight of construction	Moderate	Moderate
Architect	Preparation of architectural design	Moderate	High
Construction Contractor	Overall coordination and management	High	High
Subcontractors	Carrying out specific construction activities and material supply	Moderate	Moderate
Government Authorities	Issuing permits and conducting regulatory inspections	High	Moderate
Utility Service Providers	Supplying essential services (water, electricity, etc.)	Moderate	Low
Financial Institutions	Providing financial support and funding	High	Moderate
Legal Advisors	Managing contractual and legal matters	Low	Moderate
Environmental Consultants	Ensuring compliance with environmental standards and regulations	Low	Moderate

6.10 Construction Safety

Safety management in construction is vital to the well being of workers and free running of the activities hence the main concern of all stakeholders in a project. Safety standards such as OSHA standards have to be followed strictly in the construction of a residential building. To begin with, a special team should develop an elaborate safety scheme, whereby workers should be aware of all the possible hazards, risks, and preventive measures. This safety plan should also be kept on a regular basis as the construction phases go on. Moreover, the workers should be provided with personal protective equipment and trained about hazard prevention and management. In addition to the workforce, every visitor to the site will be required to comply with the set safety measures. Lastly, the construction work should never jeopardize the security of the people. This

encompasses dealing with external risks that are connected with the material transportation, stockpiling, and excessive noise (Hughes and Ferrett, 2015).

Table 6.10.1. Safety Risk Register

Hazard Description	Probability (1–5)	Impact Level (1–5)	Risk Score
Working at elevated levels	4	4	16
Slipping, tripping, or stumbling	3	3	9
Mobile equipment and moving machinery	4	3	12
Fire or explosion incidents	2	4	8
Building or structural failure	2	5	10
Insufficient protective equipment	3	3	9
Exposure to extreme temperatures	3	3	9
Electrical shock and power hazards	3	4	12
Excessive noise exposure	2	2	4
Caught-in or caught-between incidents	3	4	12
Hazardous or toxic material exposure	3	3	9
Hand–arm vibration effects	3	3	9

6.11 Construction site planning

The site plan for the 8-storey hotel project at 6819 Manor Way is designed to support material logistics and safe vertical construction in a windy area.

Access & Circulation: Two entrances provide access to the site. Vehicles can circulate unidirectionally around the central Building zone from the gate to the Excavated ground and storage.

Crane Operations: A Crane zone is located on the west of the building footprint. Due to high wind loads ($> 40\text{m/s}$) and weak clay soil conditions, the crane is positioned to reach the entire construction footprint inside a 50m radius, while being stable.

Material Logistics: Dedicated zones (stockpile) are set aside for:

- Material stockpile: On the west side for miscellaneous materials.
- Reinforcement stockpile: Adjacent to the crane for immediate use.
- Waste material: Kept in the north-east corner for safety and cleanliness.

On-Site Facilities: The Office Camp, Equipment & Park, and worker facilities are located on the east and south sides of the site. This keeps offices and rest areas away from the dangerous Construction Zone, in the centre of the site.

This arrangement allows the basement excavation and 8-storey construction to proceed without site traffic issues.

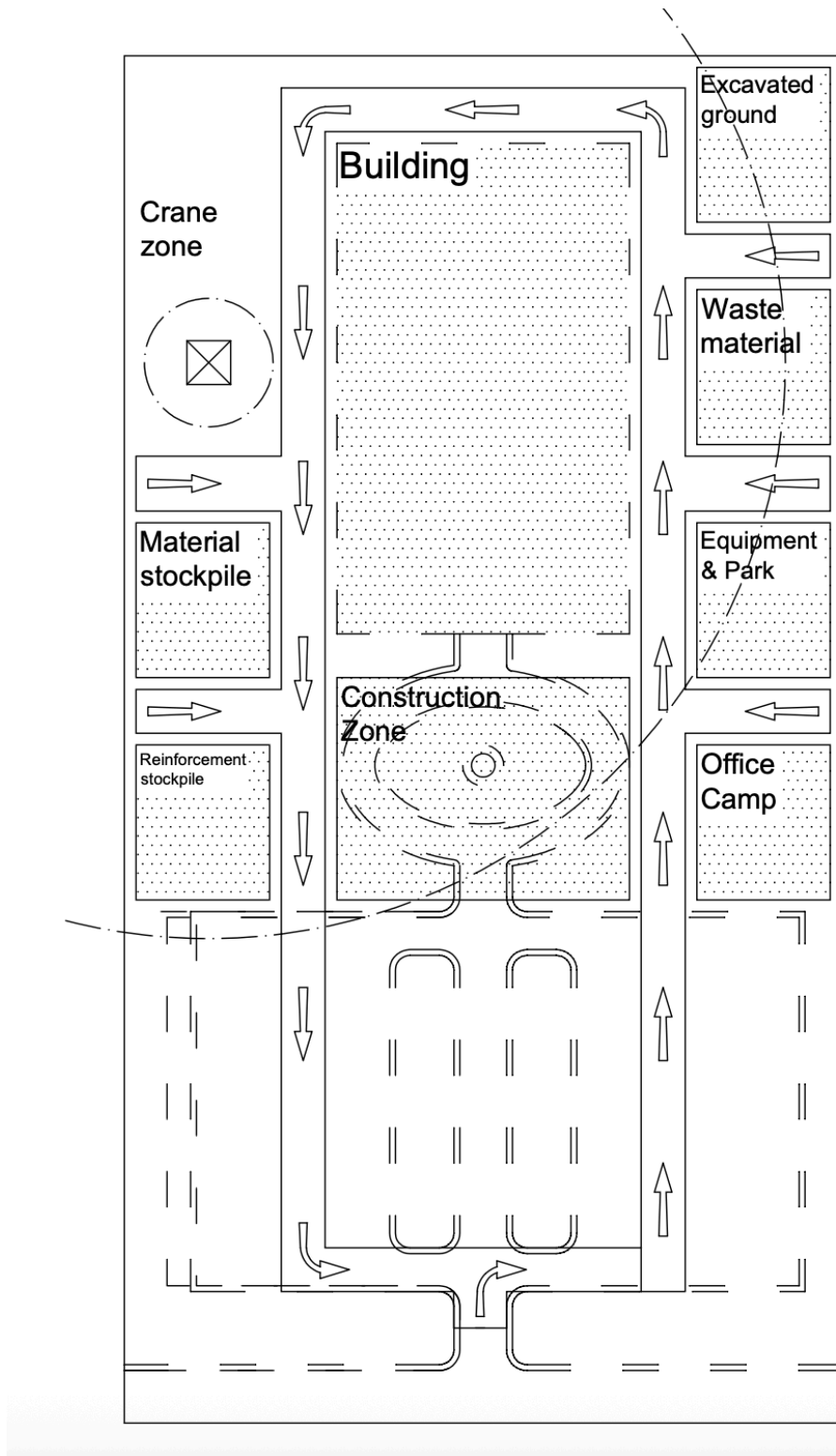


Figure 6.11.1. Construction Site Layout

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Appendices

Table. Beam reinforcement design for the beam of Floor 8 between Frames 2 and 3

	End	Mid	End
Top reinforcement bars	2#8	2#8	4#8
Area, in²	1.57	1.57	3.14
Bottom reinforcement bars	2#8	4#8	4#8
Area, in²	3.14	3.14	3.14
Reinforcement ratio, %	1.69	1.69	2.25

Shear reinforcement	#3@275	#3@275	#3@275
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Table. Beam reinforcement design for the beam of Floor 8 between Frames 3 and 4

	End	Mid	End
Top reinforcement bars	2#8	2#8	6#9
Area, in²	1.57	1.57	6.00
Bottom reinforcement bars	6#8	2#8	4#8
Area, in²	4.71	1.57	3.14
Reinforcement ratio, %	2.25	1.13	3.28
Shear reinforcement	#3@120	#3@275	#3@120

Table. Moment Capacity for Flexure

	Point A	Point B	Point C	Point D	Point E	Point F
MD	-3356.080	1711.44	-3425.17	-3425.16	1711.44	-3356.080
ML	-387.870	197.05	-394.45	-394.45	197.05	-387.870
Mhyp	-1761.09	3492.7435	8746.577	8746.577	3492.7435	-1761.09
Mu	-6408.978	5861.7515	4005.253	4005.265	5861.7515	-6408.978

Basic Parameter Analysis	Point A (T-T)	Point B (T-C)	Point C (T-T)	Point D (T-T)	Point E (T-C)	Point F (T-T)
Stop	614043426.99	0	614043426.99	614043426.99	0	614043426.99
Sbot	0	189546451.63	0	0	189546451.63	0
P (kN)	3463.32	3463.32	3463.32	3463.32	3463.32	3463.32

P/A	-1.519	-1.519	-1.519	-1.519	-1.519	-1.519
fr + (P/A), MPa	5.53896 9683	5.53896 9683	5.538969 683	5.53896968 3	5.538969683	5.538969683
Mcr, kN-m	3401.16 7926	1049.89 2049	3401.167 926	3401.16792 6	1049.892049	3401.167926
1.2Mcr	4081.40 1511	1259.87 0459	4081.401 511	4081.40151 1	1259.870459	4081.401511
phi Mn	4829.10 498	4829.10 498	4829.104 98	4829.10498	4829.10498	4829.10498
Status	OK	OK	OK	OK	OK	OK