



# NAZARBAYEV UNIVERSITY

**Department of Civil and Environmental Engineering**

**Capstone Project II**

**Design of multi-story hotel building in Los Angeles, California, USA**

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**Date of Submission: April 26, 2025**

## Declaration

We hereby declare that this report entitled “Design of high-rise hotel building in Riverside, California, USA” is the result of our 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.

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### **Acknowledgement**

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## **Ethical and Professional Considerations**

The design and construction of the 16-story luxury hotel at 1201 South Grand Avenue in Downtown Los Angeles with ethical considerations integrated at every level of the project. Because it is located in a high seismic zone, the structure is designed to meet the California Building Code (CBC) and ASCE 7, to be safe, resilient, and earthquake prepared, as well as NFPA codes for fire safety. Sustainable design practices, including energy-efficient systems, water-saving fixtures, and waste management, should target LEED certification, and an Environmental Impact Assessment guarantees minimal impact on the ecology.

The same holds for ethical standards in reporting and documentation, in addition to the design and construction processes. Reports are created with a high level of accuracy, transparency, and professionalism, making sure that all of the data, calculations, and findings are presented truthfully. It has proper acknowledgment of all team members and sources of external information — respecting intellectual property and collaboration. Avoids plagiarism by properly citing all external sources and providing references according to academic standards.

The project is also designed with the greater community in mind, engaging transparently, minimizing disruption, and honoring the local culture and architecture of Los Angeles. This incorporates all ethical and professional practices in ensuring safety, sustainability, inclusivity, and integrity in all sections of the project.

## Abstract

In this paper, we present the design and construction of a large-span 16-story luxury hotel with a basement located at 1201 South Grand Avenue, Los Angeles, California in a high seismic zone. The project utilizes a holistic architectural, structural, geotechnical, environmental, and construction engineering design approach to function, safety, and guest experience. The hotel will offer luxury accommodations and a wide range of amenities, including rooftop restaurants, a swimming pool, conference halls, a fitness center, and lounge areas, enhancing overall guest satisfaction.

This project features two parking areas which are a  $45 \times 56$  m<sup>2</sup> outdoor parking facility and an underground parking facility that offers extra space and structural support. This also offers savings from the associated construction stages by integrating the underground parking facility completely into the geotechnical and structural planning, thus creating high stability in the seismically active region. The design includes a storm sewer system to comply with the requirements for drainage and water management.

A 2020 GeoPentech investigation for the site — originally to be built with a 40-storey mixed-use building — is used for geotechnical analysis. The design adheres to IBC, ASCE, and LA Building Code standards, with special emphasis on seismic issues at the structural and foundation level. This hotel constitutes an innovative high-end hospitality experience when compared to the existing seismic conditions on site.

**Keywords:** luxury hotel, Los Angeles, seismic design, structural design, geotechnical analysis, parking design, storm sewer design, construction management.

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## **1. Introduction**

### **1.1. Project Description**

The presented poster shows the design of a 16-story hotel in downtown Los Angeles, California, USA. The poster contains information on site selection and analysis, design codes, architectural layout, geotechnical, environmental, and construction management details.

### **1.2. Problem Statement**

Los Angeles is one of the most popular destinations among tourists not only in the United States but also around the world. To maintain the high level of tourism, it was decided to build a hotel and a shopping center.

### **1.3. Project Objectives**

- To select the site location and conduct a site analysis to assess the constraints, opportunities, and conditions of the area
- To showcase the building's design concept and architectural plans
- To determine the soil profile and seismic zones
- To conduct an in-depth review of design standards and reports for each section
- To approximate the project costs, develop a project schedule, and define the project charter.

## **2. Architectural Design**

### **2.1. Overview of the Building**

The building is a 16-story hotel with 1-story underground parking. The first 5 floors are used for commercial, then it goes with the ten floors for accommodation, and the highest floor is used as a lounge bar with a terrace. The design idea was to present the beauty of Kazakh nature, particularly the Ustyurt Plateau, which is a vast and arid desert region that stretches across western Kazakhstan. Therefore, colors of the sand (beige shades) and a night open sky (midnight blue) were chosen for the building design. The Hotel will contain 214 rooms that have 4 different types according to their size (S/M/L/VIP). Following the International Building Code (IBC), occupancy of the first 5 floors and the last 16th floor is assumed for public usage, while floors from 6 to 15 are used for accommodating purposes. Below, a 3D View of the building is located.

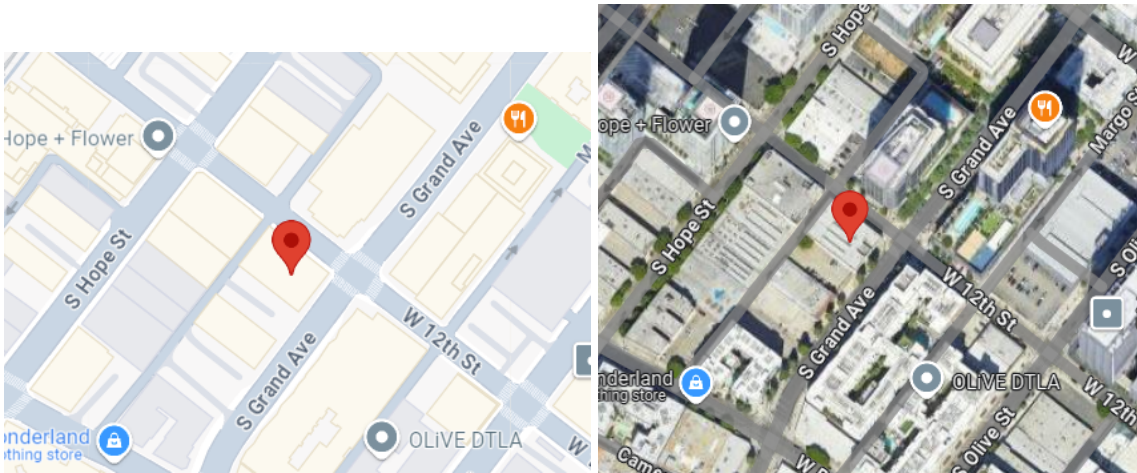


**Figure 2.1.** 3D View 1 - Rendered

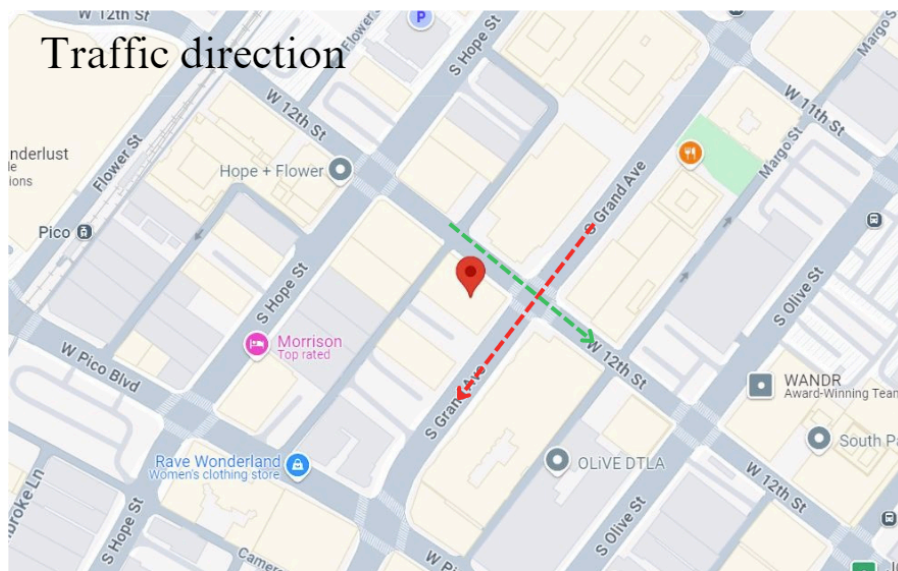
### **2.2 Site Analysis**

The available site with dimensions of 112x46 meters is located at 1201 S Grand Ave, Los Angeles, CA 90015, USA. Two adjacent buildings on Grand Avenue, starting from

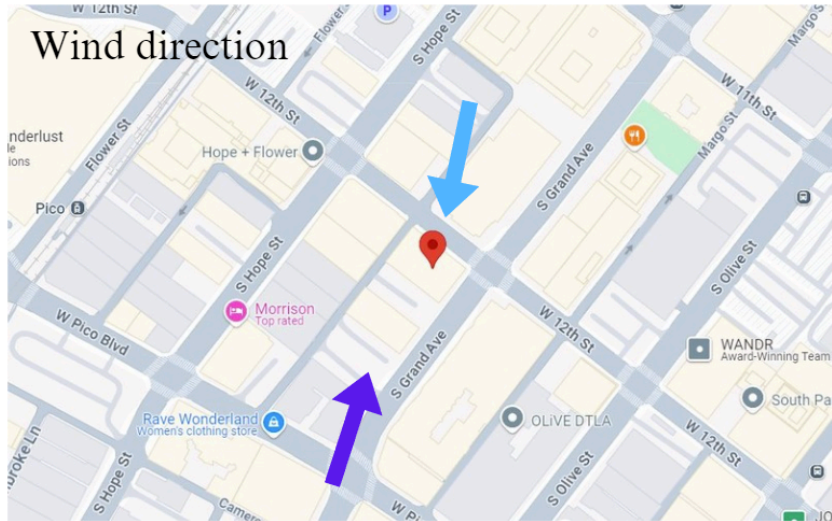
12th Street, are set to be demolished. Followingly, a parking area would be located at the corner of Grand Ave and 12th Streets and the building itself would be located behind the parking area, being oriented to Grand Ave. Historical background: The current building on the site was built in the first half of the 20th century for the Felix Chevrolet dealership. According to the US Department of Commerce, 2022, no snowfall is registered at site location. In the figures below, the precise location of the construction site, traffic, wind, and sun directions are provided.



**Figure 2.2.** Location of the building

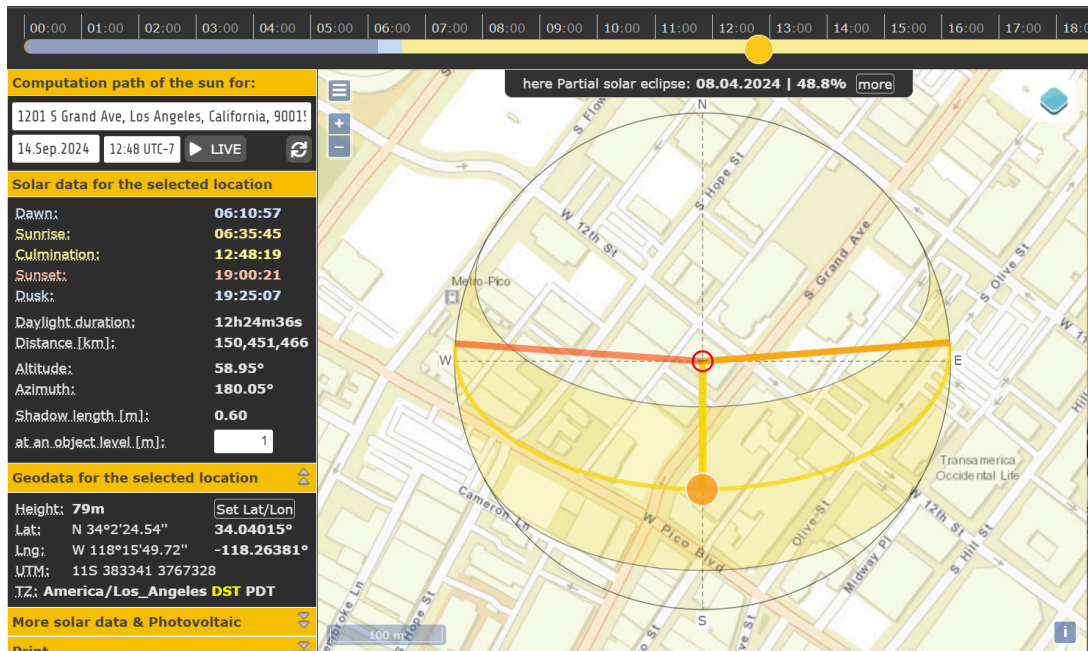


**Figure 2.3.** Traffic direction



**Figure 2.4. Wind Direction**

The first half of the day - is the southeast and south sides, while the second half is - the south and southwest sides. Daylight duration during summer time - from 12h to 12h 30 mins. Shadow length when the sun is at its zenith is only 0.6 m. Temperature: Highest in September - 39°C. Lowest in February - 2.7°C. (US Department of Commerce, 2022)



**Figure 2.5. Sun Direction**

To determine the seismicity and wind conditions on site, the ASCE Hazards tool was applied. Following ASCE-7-16, the following assumptions were made: Risk Category - 2,

Exposure Category B, and Soil Class - C (from the geotechnical report). The figure below shows the analysis of the tool.



**Address:**  
1201 S Grand Ave  
Los Angeles, California  
90015

## ASCE Hazards Report

**Standard:** ASCE/SEI 7-16      **Latitude:** 34.040174  
**Risk Category:** II              **Longitude:** -118.263792  
**Soil Class:** C - Very Dense Soil and Soft Rock      **Elevation:** 243.04505873216607 m (NAVD 88)

**Figure 2.6.** Input data for the ACSE Hazards tool

## Wind

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**Results:**

Wind Speed	42 m/s
10-year MRI	29 m/s
25-year MRI	32 m/s
50-year MRI	34 m/s
100-year MRI	36 m/s

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2  
 Date Accessed: Mon Nov 04 2024

**Figure 2.7.** Results for wind conditions

So, the design wind speed equals 42 m/s for exposure category B with a return period of 50 years. Maximum wind speed in history - 45 m/s.

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<b>Site Soil Class:</b>	C - Very Dense Soil and Soft Rock		
<b>Results:</b>			
S <sub>s</sub> :	1.937	S <sub>D1</sub> :	0.642
S <sub>1</sub> :	0.688	T <sub>L</sub> :	8
F <sub>a</sub> :	1.2	PGA :	0.828
F <sub>v</sub> :	1.4	PGA <sub>M</sub> :	0.993
S <sub>MS</sub> :	2.325	F <sub>PGA</sub> :	1.2
S <sub>M1</sub> :	0.964	I <sub>e</sub> :	1
S <sub>DS</sub> :	1.55	C <sub>v</sub> :	1.287

**Figure 2.8.** Results for seismic conditions

Summarizing wind and seismic conditions, it should be stated that the building has to be designed for high seismic and strong wind activities.

## 2.2. Material Selection

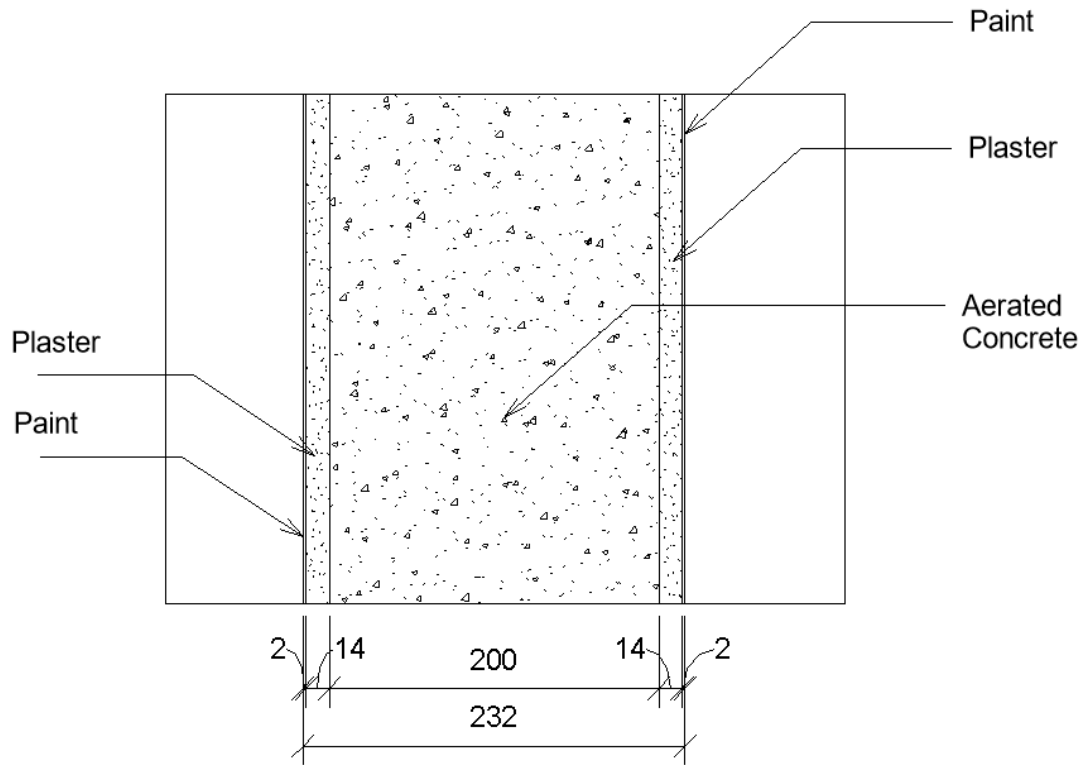
	<b>Materials</b>
<b>Hotel Structure</b>	<p><b>Reinforced concrete</b></p> <ul style="list-style-type: none"> <li>- Strength and Durability: High compressive strength and long-term stability.</li> <li>- Fire Resistance: Excellent fire-resistant properties.</li> <li>- Low Maintenance: Minimal upkeep required.</li> <li>- Thermal Mass: Moderates indoor temperatures through heat absorption and release.</li> </ul>
<b>Siding</b>	<p><b>Fiber-cement plates</b></p> <ul style="list-style-type: none"> <li>- Durability: Resistant to rot, pests, fire, and extreme weather conditions.</li> <li>- Low Maintenance: Requires minimal upkeep, retaining its appearance over time.</li> <li>- Versatility: Available in various styles, including wood-like finishes, while offering modern aesthetics.</li> <li>- Fire Resistance: Non-combustible, making it a safer choice for building exteriors.</li> <li>- Eco-Friendly: Made from sustainable materials like cement, sand, and cellulose fibers.</li> <li>- Long Lifespan: Typically lasts several decades with proper installation and care.</li> </ul>
<b>Interior</b>	<p><b>Ceramic Tiles</b> will be used on floors with public access:</p> <ul style="list-style-type: none"> <li>- Durability: Highly resistant to scratches, wear, and tear, making them ideal for high-traffic areas.</li> <li>- Water Resistance: Non-porous and resistant to water, making them suitable for kitchens and bathrooms.</li> <li>- Aesthetic Variety: Available in a wide range of colors, patterns, and textures to match various design preferences.</li> <li>- Ease of Maintenance: Simple to clean and maintain with just regular sweeping and mopping.</li> <li>- Hypoallergenic: Does not trap dust, allergens, or bacteria, promoting a healthier indoor environment.</li> <li>- Heat Resistance: Can withstand high temperatures, making them ideal for areas with radiant heating systems.</li> </ul> <p><b>Carpet Surface</b> will be used in accommodating floors:</p> <ul style="list-style-type: none"> <li>- Comfort: Soft and warm underfoot, adding a sense of coziness to spaces.</li> <li>- Sound Insulation: Absorbs sound, reducing noise levels and providing a</li> </ul>

	<p>quieter environment.</p> <ul style="list-style-type: none"> <li>- Safety: Reduces the risk of slipping and provides a softer surface to cushion falls.</li> <li>- Energy Efficiency: Acts as an insulating layer, retaining heat and reducing energy costs.</li> <li>- Affordability: Generally less expensive than other flooring options like hardwood or tiles.</li> </ul> <p><b>Sound-Resistant Gypsum Board with 4mm Plaster Wall Leveling</b> will be used at the accommodating spaces:</p> <ul style="list-style-type: none"> <li>- Ease of Installation: Lightweight and easy to cut, making it simple to install.</li> <li>- Cost-Effective: Affordable compared to other wall finishing materials.</li> <li>- Fire Resistance: Provides good fire resistance due to its non-combustible core.</li> <li>- Sound Insulation: Reduces noise transmission between rooms when used as interior walls.</li> <li>- Smooth Finish: Creates a uniform surface for painting or wallpapering.</li> <li>- Moisture Resistance: Moisture-resistant versions are available for bathrooms and kitchens.</li> </ul> <p><b>14 mm Plaster Wall Leveling</b> will be used in the corridor, hall, and lobby spaces</p> <ul style="list-style-type: none"> <li>- Durability: Creates a hard, long-lasting surface resistant to cracking.</li> <li>- Aesthetic Appeal: Allows for smooth, detailed finishes and decorative designs.</li> <li>- Fire Resistance: Provides excellent fire resistance due to its non-combustible nature.</li> <li>- Thermal Insulation: Helps regulate indoor temperatures by providing a thick insulating layer.</li> <li>- Versatility: Can be applied to a variety of surfaces, including brick, stone, and concrete</li> </ul>
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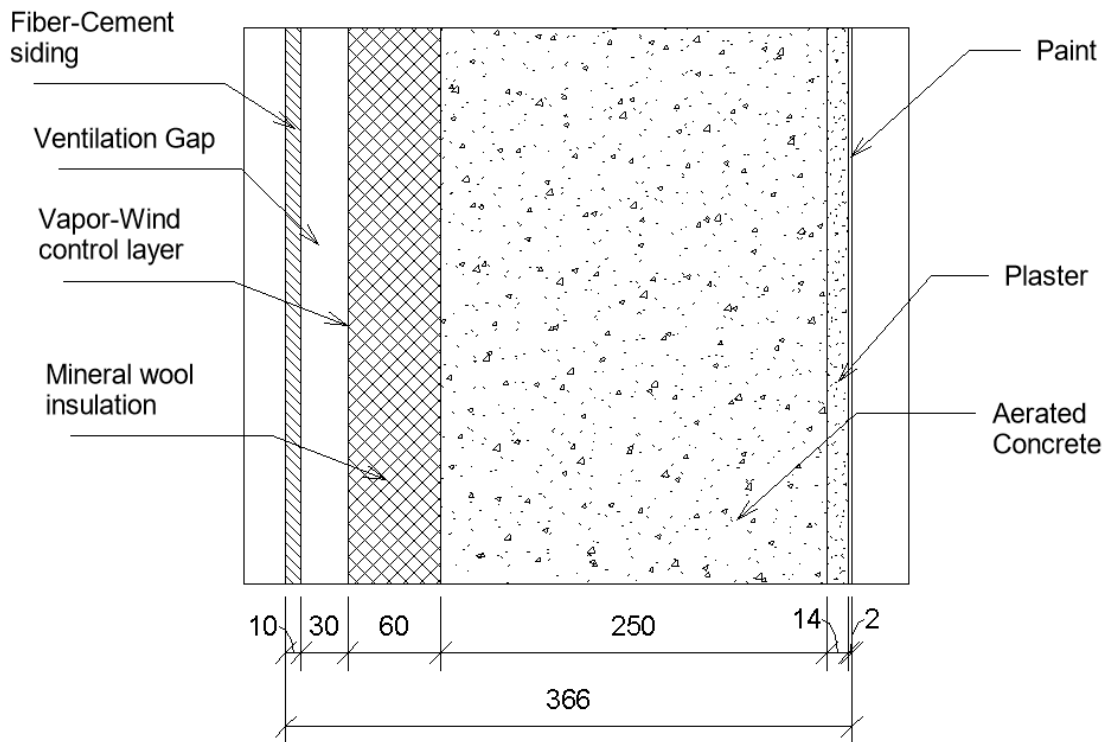
Table 2.1. Material selection

### 2.3. Sectional Views of Floor Slab, Ceilings, Walls

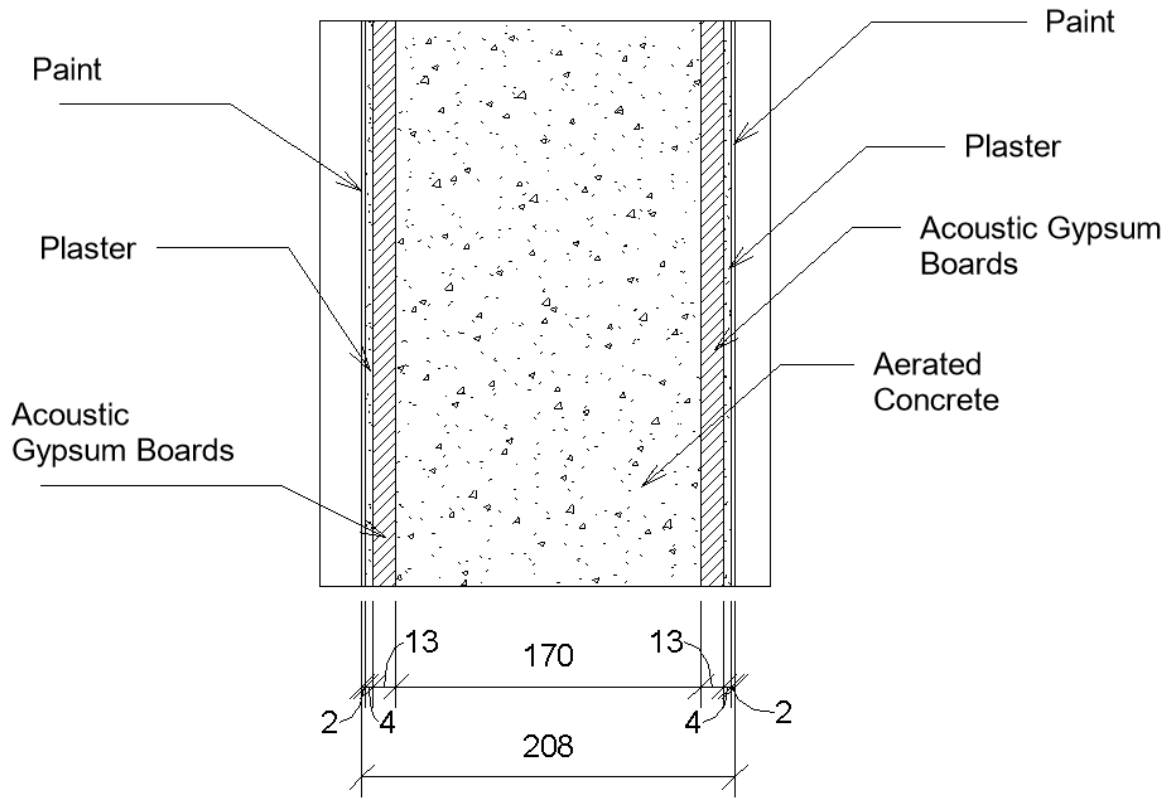
After determining the purposes of each room and materials of the project, all structural and non-structural components were drawn by layers, which are demonstrated on the figures below:



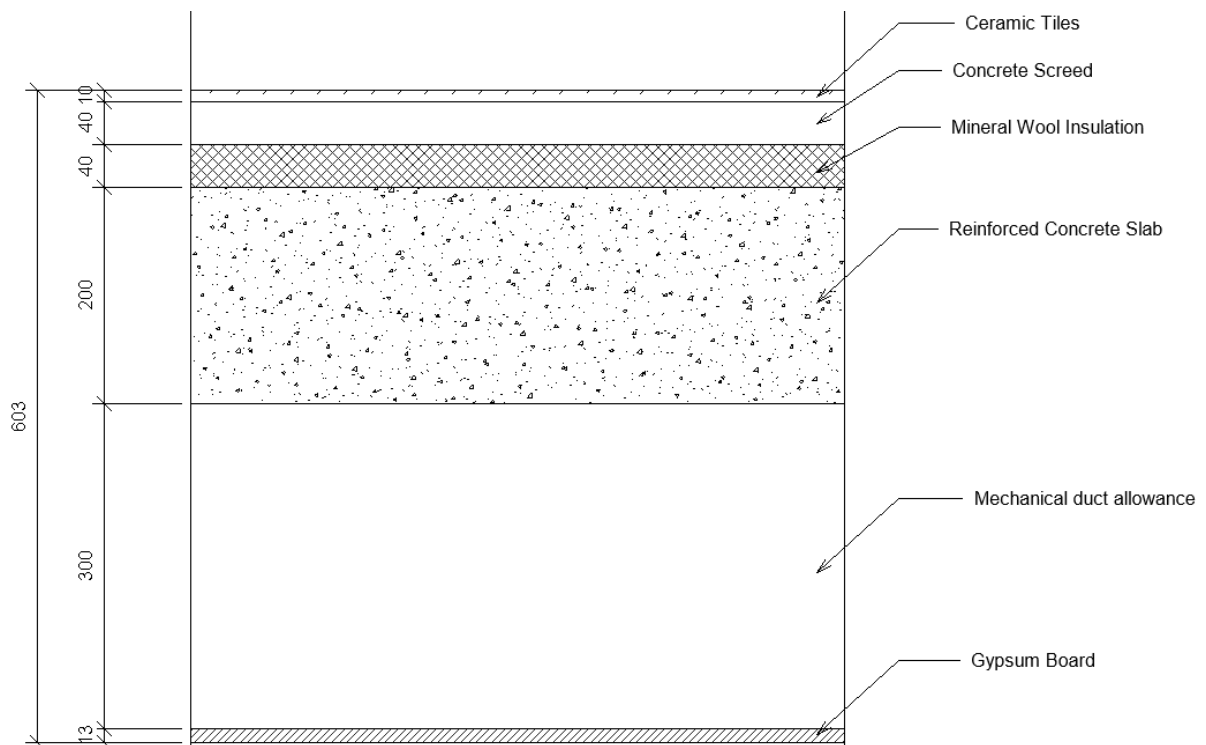
**Figure 2.9.** Corridor Wall's Sectional View



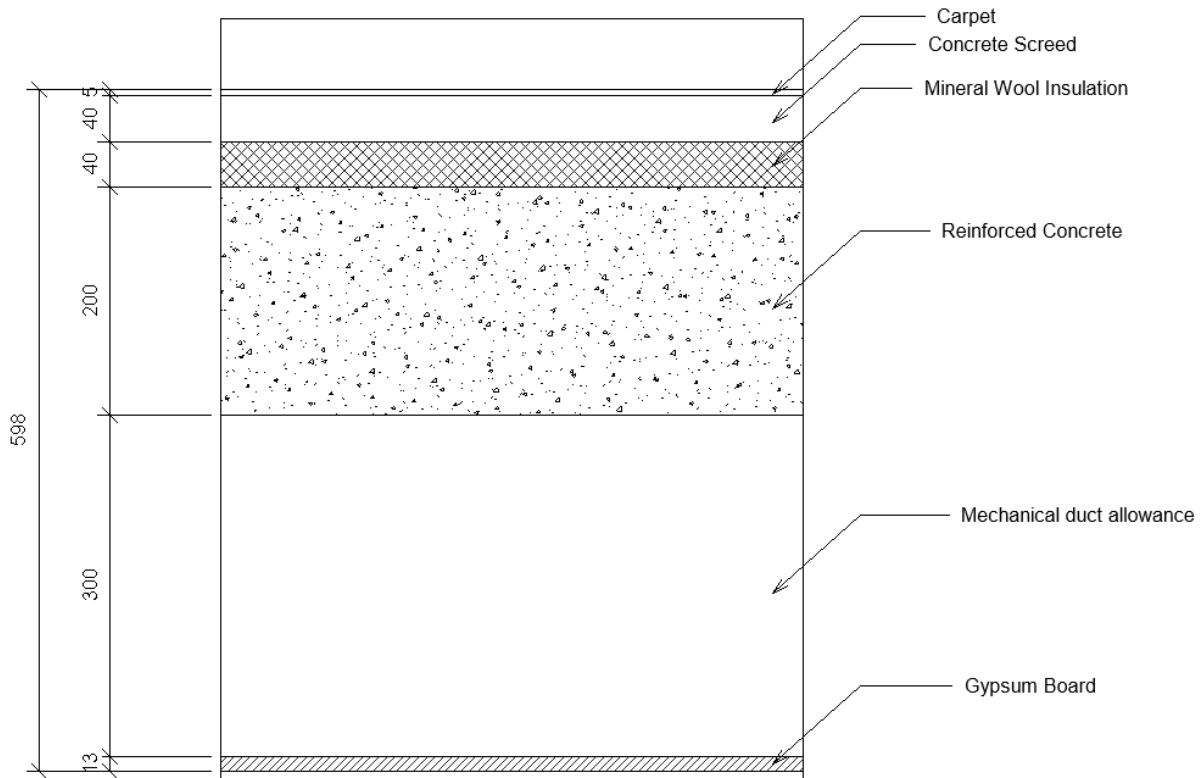
**Figure 2.10.** Exterior Wall's sectional view



**Figure 2.11.** Interior Wall's (Room separating) sectional view



**Figure 2.12.** Slab layers for Public Use - sectional view



**Figure 2.13.** Slab layers for accommodating use - sectional view

## 2.4. Site Layout and Geometry Design of the Building

As it was mentioned, the building is located in a high seismic and strong wind zone. The geometry shape of the building is squared to prevent a high torsional effect. Overall height from the ground surface equals 73.4 meters (76.9 including underground parking). Parking area was chosen to be located closer to a corner of Grand Ave and 12 th street in terms of easier traffic control. There will be 3 entrances and 4 exits, including ramp paths. The slope of the ramp is 12% which allows cars to climb up and down a vertical height of 3.5 meters in 30 meters. As the site is located in Downtown Los Angeles, landscape design is absent on a site plan due to high density of the buildings. Gross Area of the building is 2025 m<sup>2</sup>.

Function of each floors:

Floor 1 - Lobby, Reception

Floor 2 - Large Hall designed for events

Floor 3 - Four Conference Halls (Transferrable to permanent offices) for smaller events

Floor 4 - Kids' room, medical cabinet, gym, fresh bar, barbershop

Floor 5 - Restaurant

Floor 6-15 - Accommodation

Floor 16 - Lounge Bar

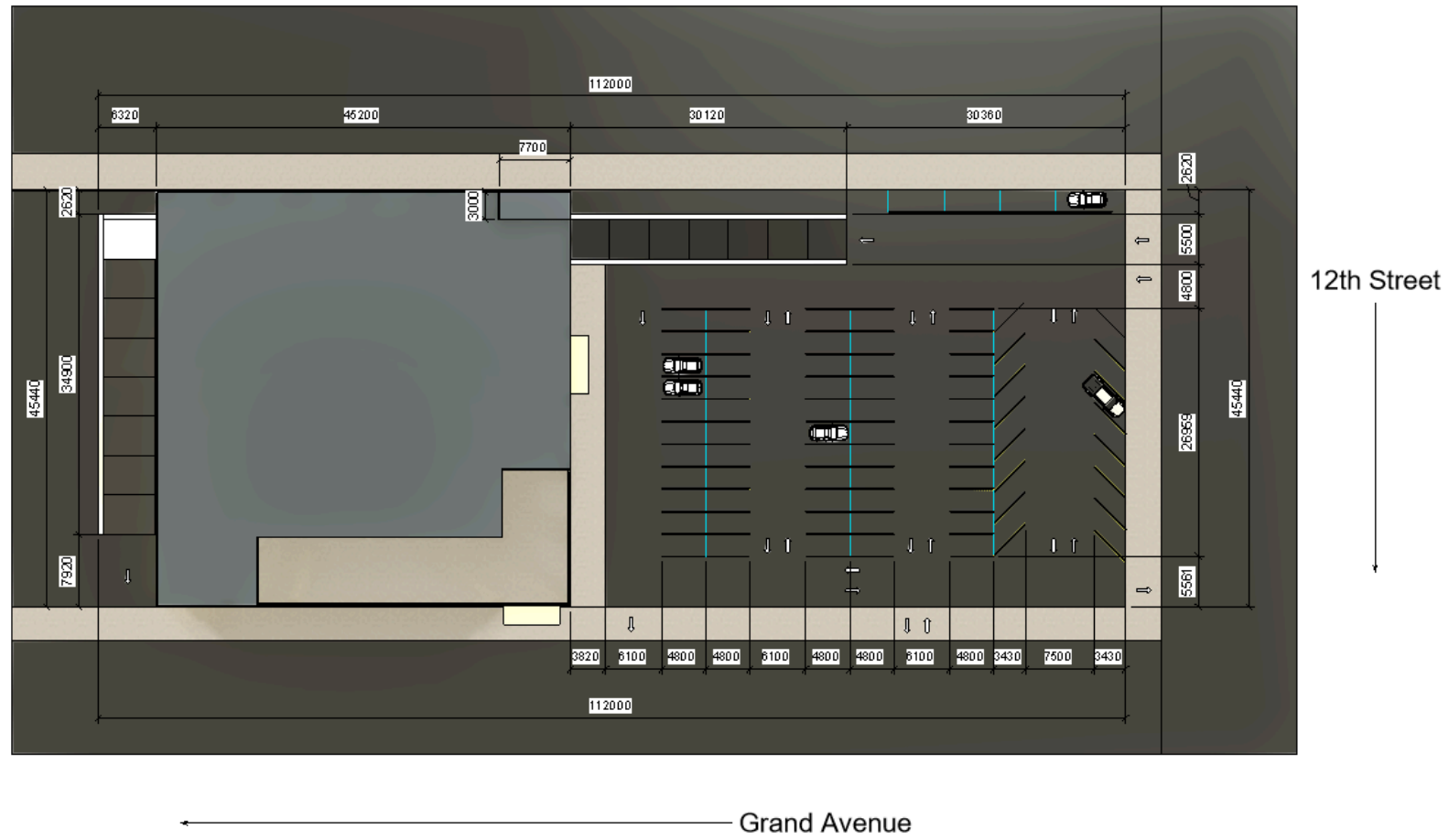
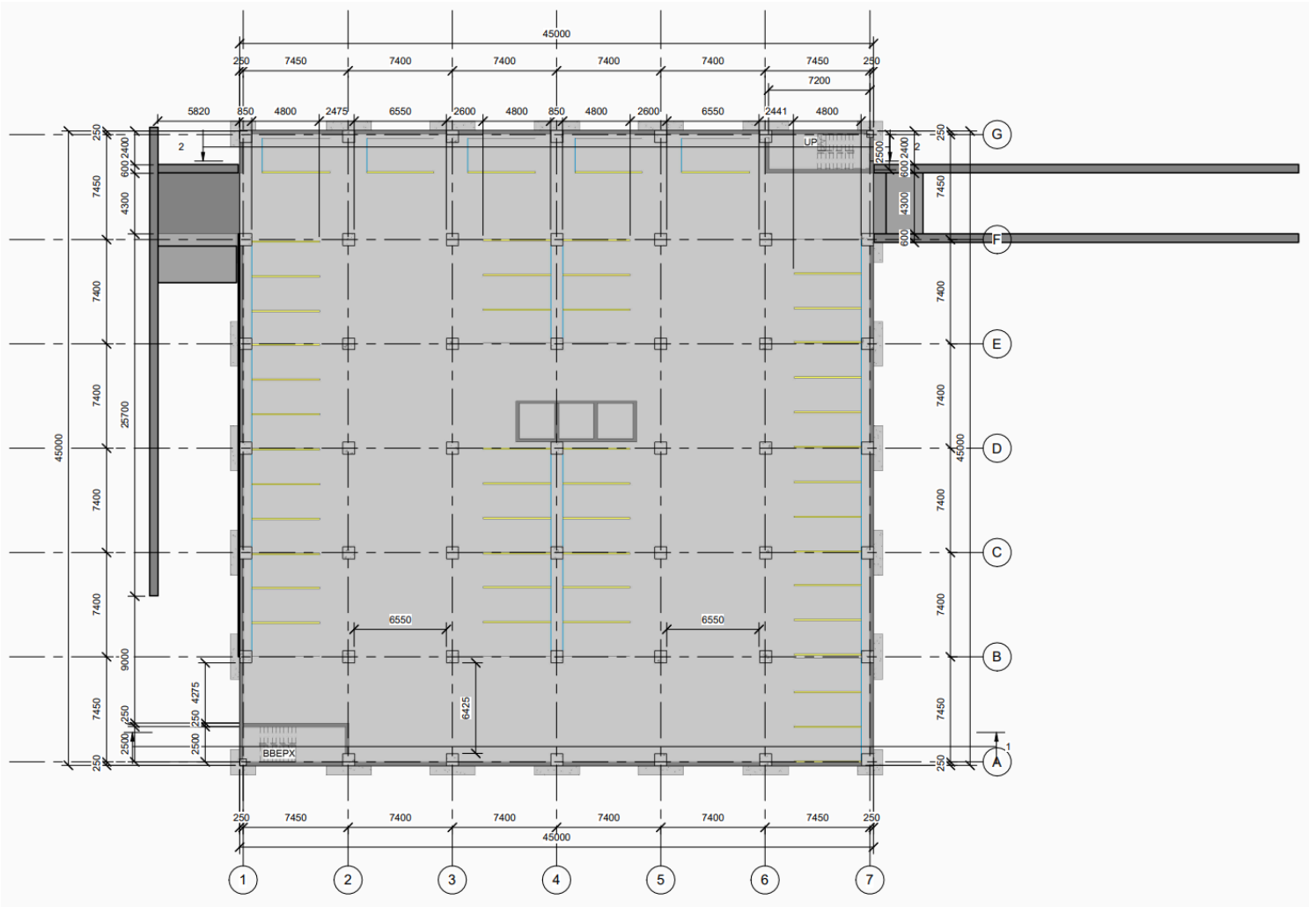
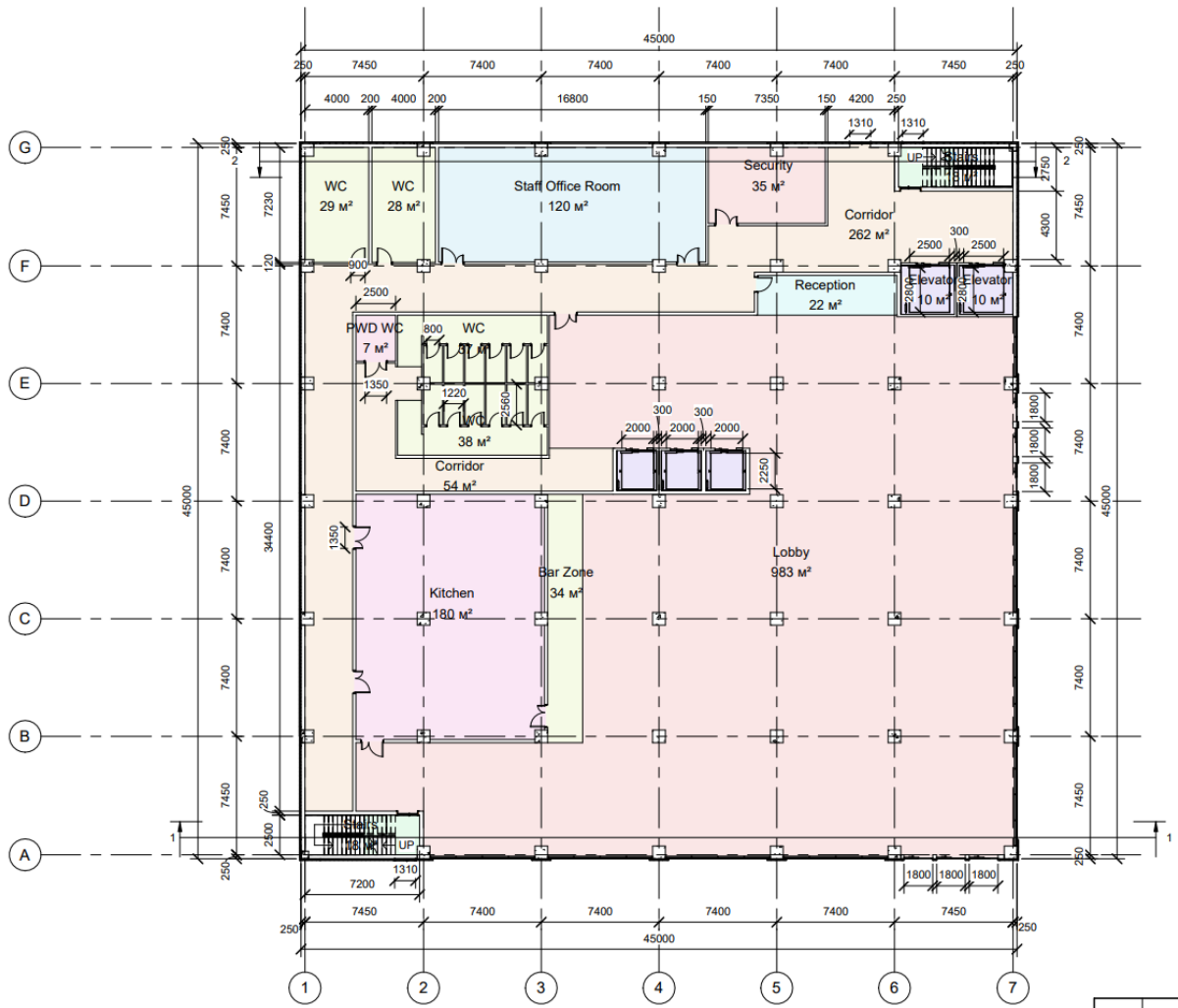


Figure 2.14. Site View



**Figure 2.15.** Layout of underground floor



**Figure 2.16.** Layout of floor 1

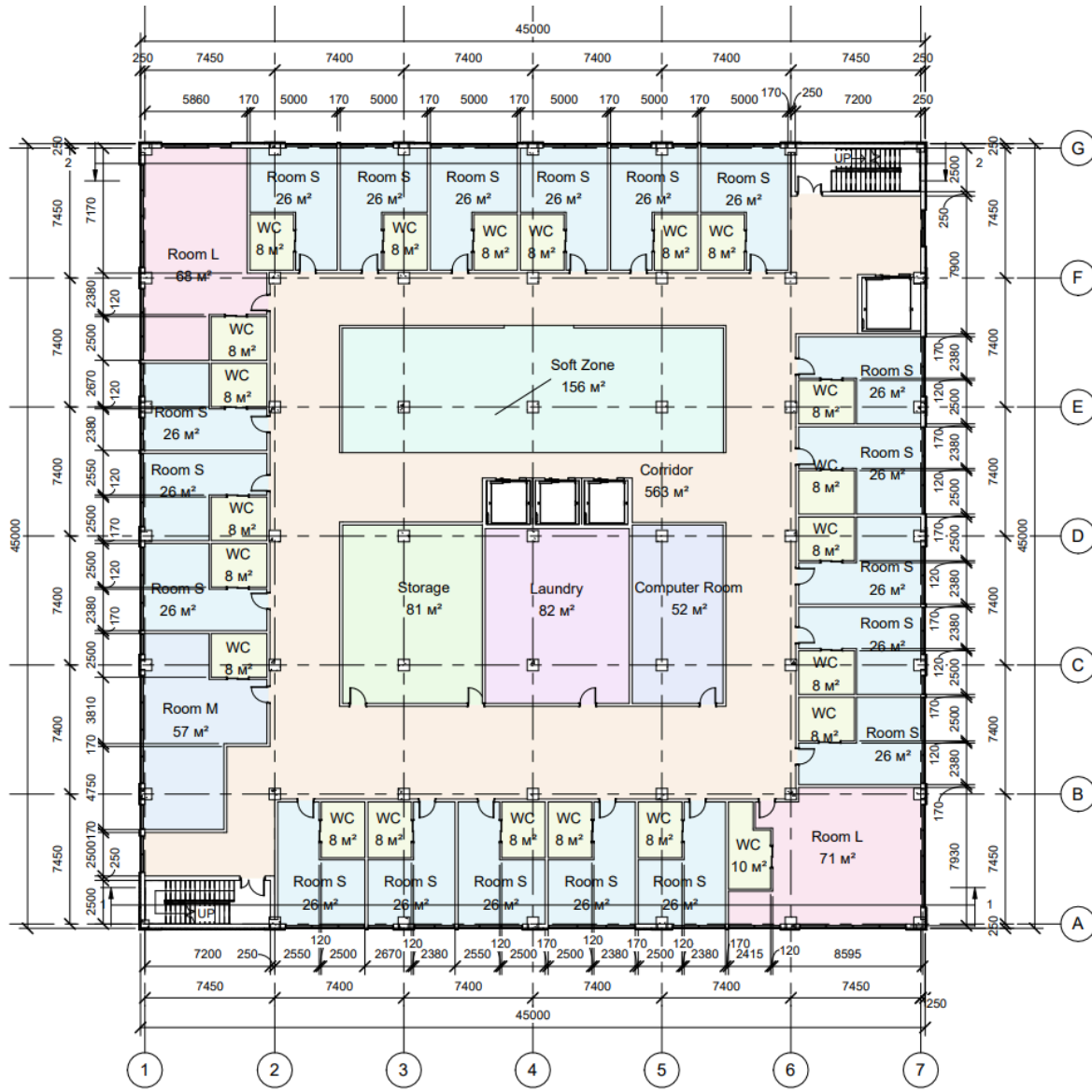
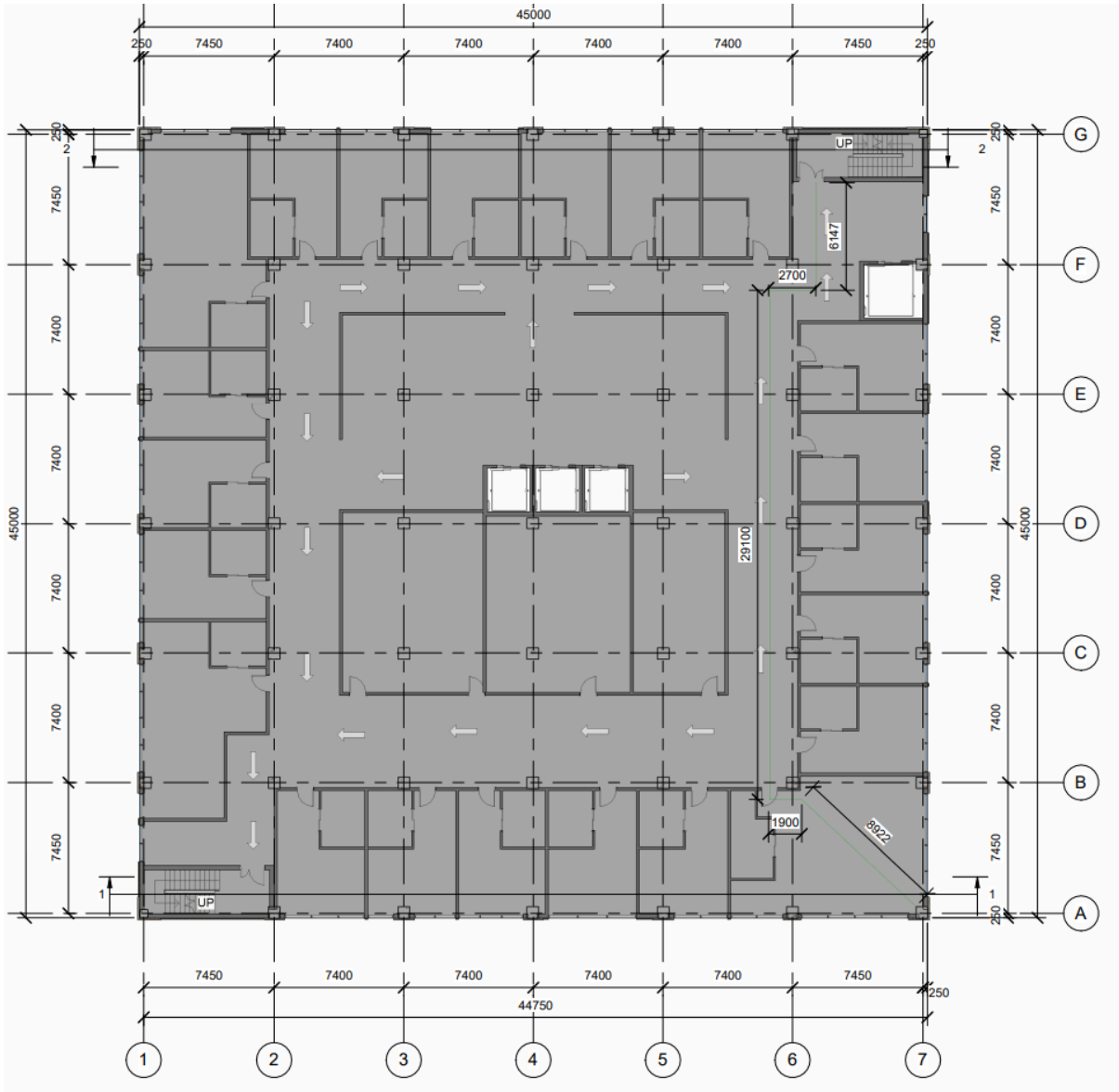


Figure 2.17. Layout of typical floor



**Figure 2.18.** Evacuation Plan

Square shape of the building is quite advantageable it terms of evacuation due to its symmetricity. The sum of distances in the plan above shows that the maximum distance to the evacuation exit is 48.8 meters.

## 2.5. IBC and LA Building Code Considerations

As the hotel is located in Los Angeles, all structural and non-structural elements' dimensions and quantity have to be designed in accordance with International Building Code and Los Angeles Building Code.

### 2.5.1. Parking Area

LA Code: Parking requirements in Los Angeles generally require 1 parking space per guest room. Minimum dimensions of one parking lot has to be

Per the LAMC Section 12.21 A.5(c):

Standard Parking Space: 2.6 x 5.5 m

Compact Parking Space: 2.3 x 4.6 m (if allowed)

Two-way traffic width, min: 7.2 m

Two-way traffic 45 degree parking, min: 5.5 m

One-way traffic 45/90 degree, min/min: 3.5/5.5 m

1 accommodation floor of the hotel = 22 rooms for floors 6-12, and 20 rooms for floors 13-15, which means 22 and 20 parking lots for one floor. The hotel would have 10 floors for accommodation, therefore,  $22*7+20*3 = 214$  parking lots minimum.

**Design:** Due to limitation of the space (Downtown Los Angeles), the site will not be able to provide 214 parking spaces. Optimized case is able to accommodate a maximum 123 parking spaces (50 for underground parking and 73 for outside). Both 90 degree and 45 degree parkings are used in the outside parking area. Two-way traffic width has taken to equals 6.1 m, while one-way traffic width is 4.8 meters.

### 2.5.2. Emergency Exit

**IBC:** At least two exits when the number of occupancies is larger than 49 for business occupancies etc, and larger than 29 for residential. Minimum Door size: 815 x 2032 mm. Maximum longest path is 76 meters. Minimum width of corridor: 915 mm. Minimum stair width: 1118 meters, riser height is 178 mm, and riser depth 280 mm.

**Design:** One accommodating floor serves 22 rooms with at least 2 guests, and therefore, 2 emergency stair shafts are designed. Door size: 1350 x 2100 mm. Maximum longest path is 48.8 meters. Minimum width of corridor: 2000 mm. Stair width: 1200 mm, max riser height is 176.5 mm, and riser depth 280 mm at least.

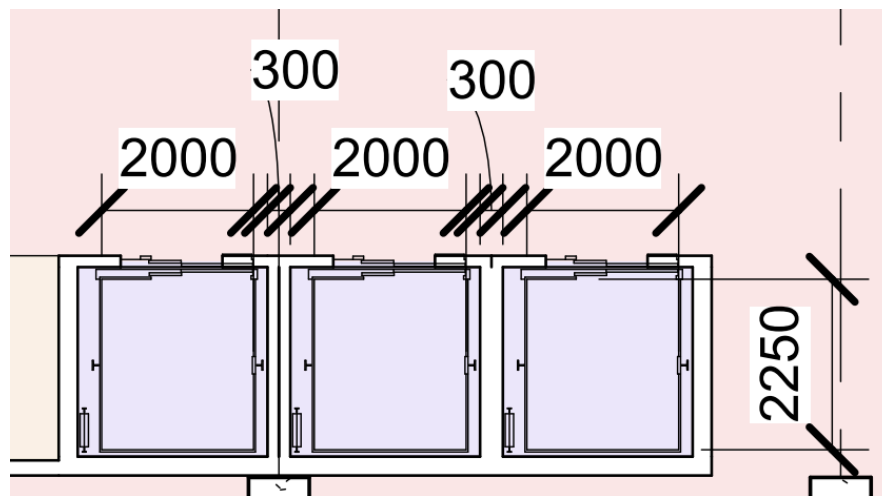
Размеры	
Требуемое количе...	34
Текущее количест...	34
Текущая высота п...	176.5
Текущая ширина п...	280.0

**Figure 2.19.** Minimum Riser Properties in the building from Revit (34 steps, 176.5 mm height, width 280 mm).

### 2.5.3. Elevators

**IBC:** Min Size: 2032 x 1372 mm.

**Design:** 2 staff and 3 passenger elevators are designed for fast service and accessibility of each floor of the building. Staff Size: 2800 x 2500, Passenger's: 2250 x 2000 mm.



**Figure 2.20.** Passenger Elevator Size

### 2.5.4. Hotel Rooms and Windows

**IBC:** Not specified in the IBC or LA codes, however the typical floor area of a standard room is 18-22 m<sup>2</sup>, ceiling height is 2.3 meters. Minimum window area = 8% of floor area of the room.

**Design:** Minimum Floor area of the hotel room in the whole building is 35 m<sup>2</sup>. Ceiling height equals (4 m - 0.2 m of slab - 0.3 m of mech.duct allowance - 0.013 m of gypsum layer) = 3.487 meters of clear height. All rooms have panorama windows.

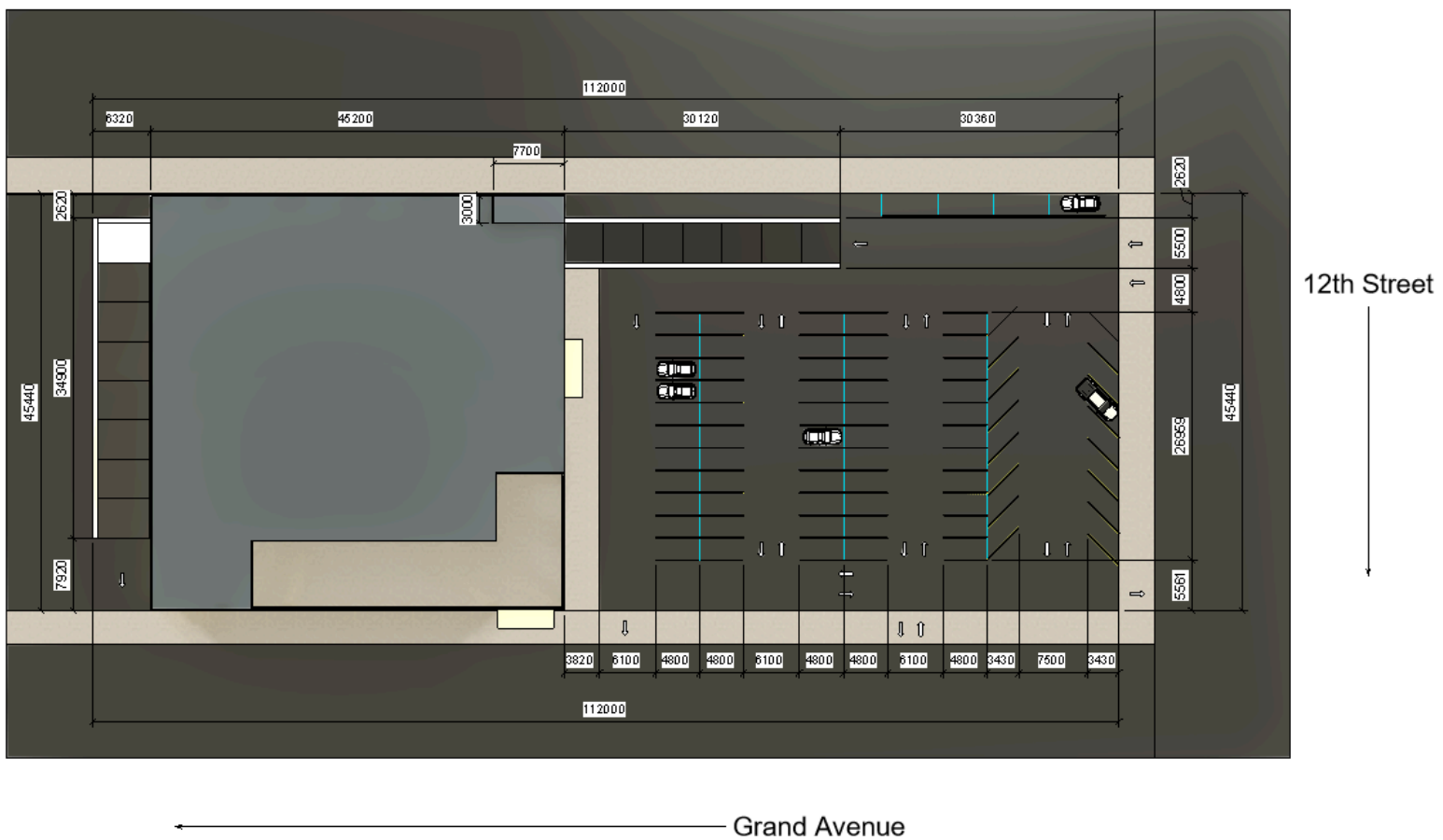
### 2.5.5. Allowable Building Height

According to the IBC, Construction Type I-A has unlimited height

### 2.5.6. Roof Slope and Ramp Design

As stated in Section 1507.10.1 of IBC, roof slope is 2% for drainage collection and ramp slope equals 12%.

### 2.6. Traffic Flow Design



**Figure 2.21.** Traffic Flow Through Parking Area

On the right side of the site layout the parking zone is located, where the flow direction of the car is indicated by white arrows. Followingly, the parking zone has two entries from 12th Street (one for underground parking, one for outside parking) and one exit. Regarding the Grand Avenue, there is one two-way entry and one exit. After that, the lane starting from 12th Street up until the corner and the lane after a turn on the corner along the hotel entrance are one-way directions. The lane along the hotel entrance is

assumed to be 6100 mm for comfortable dropoff of the hotel guests. Other roads in the parking area are suggested to be two-way directions.

### 2.7. Concrete mixture

A number of concrete mixes were created using Life365 software, accounting for heat conditions, earthquake resistance, corrosion resistance, and other elements identified in earlier research.

- Base case: w/c ratio 0.4, fly ash 15%, silica fume 0%, slag 30%
- Alternative 1: w/c ratio 0.4, fly ash 15%, silica fume 0%, slag 25%
- Alternative 2: w/c ratio 0.45, fly ash 10%, silica fume 0%, slag 30%
- Alternative 3: w/c ratio 0.4, fly ash 0%, silica fume 0%, slag 0%

### 2.8. Life cycle cost

Using the software, the total life cycle cost was calculated taking into account the following material costs:

- Concrete - \$76.46/cubic.yd
- Black steel - \$0.45/lb
- Membrane - \$3.07/sq.ft
- Sealer - \$0.65/sq.ft
- Inhibitor - \$5.68/gal
- Repair - \$37.16/sq.ft

When entering Los Angeles as the location, the software determined a chloride exposure level of 0.68% by weight of concrete and a temperature range of 55°F to 70°F.

The life cycle costs can be seen in Table 2.2.

Type of mixture	Construction cost, \$	Barrier cost, \$	Repair cost, \$	Life-cycle cost, \$
Base case	103256	0	108642	288898
Alternative 1	103256	0	93182	196437

Alternative 2	103256	0	281232	384488
Alternative 3	103256	0	380953	484209

Table 2.2 Total life cycle costs.

## 2.9. Service life

From LCC the service life of each mixture was calculated and can be seen in Table 2.3. It can be seen that the Alternative 1 has the most service life.

Type	Base case	Alternative 1	Alternative 2	Alternative 3
Service life	69.08	72.17	59.67	46.58

Table 2.3. Service life.

## 2.10. Corrosion prevention

Concrete mixtures are made using specific cement ingredients to reduce corrosion. While silica fume (5-10% by weight of cementitious materials) increases impermeability and chloride resistance, Portland cement binds the combination. Blast furnace slag (30-50%) increases sulphate and alkali-silica reaction, and fly ash (15-30%) reduces porosity and increases strength through pozzolanic processes.

## 2.11. Leadership in Energy and Environmental Design (LEED)

The USGBC created the Leadership in Energy and Environmental Design (LEED) standard, which is a standard for green building practices. To be rated, projects must first meet the basic minimum requirements of the program. Since 2004, California has required compliance with the certification system, which recognizes four performance levels: \*Certified (40-49 points), Silver (50-59), Gold (60-79), and Platinum (80+)\* (Government of California, n.d.).

The assessment is based on seven critical sustainability parameters:

**Sustainable Sites**

**Water Efficiency**

**Energy and Atmosphere**

**Material and Resources**

**Indoor Environmental Quality**

**Design Innovation**

**Regional Environmental Priorities**

The LEED assessment for this project, detailed in Figure 2.22, yielded a score of 79, earning it Gold certification and demonstrating its commitment to environmental stewardship and sustainable design excellence.

0	26	0	Location and Transportation	16
Y	16		Credit: LEED for Neighborhood Development Location	16
		N	Credit: Sensitive Land Protection	1
		N	Credit: High Priority Site and Equitable Development	2
Y	3		Credit: Surrounding Density and Diverse Uses	5
Y	5		Credit: Access to Quality Transit	5
		N	Credit: Bicycle Facilities	1
Y	1		Credit: Reduced Parking Footprint	1
Y	1		Credit: Electric Vehicles	1

0	7	0	Sustainable Sites	10
Y			Prereq: Construction Activity Pollution Prevention	Required
Y	1		Credit: Site Assessment	1
Y	2		Credit: Protect or Restore Habitat	2
Y	1		Credit: Open Space	1
Y	3		Credit: Rainwater Management	3
		N	Credit: Heat Island Reduction	2
		N	Credit: Light Pollution Reduction	1

0	5	0	Water Efficiency	11
Y			Prereq: Outdoor Water Use Reduction	Required
Y			Prereq: Indoor Water Use Reduction	Required
Y			Prereq: Building-Level Water Metering	Required
Y	2		Credit: Outdoor Water Use Reduction	2
		N	Credit: Indoor Water Use Reduction	6
Y	2		Credit: Optimize Process Water Use	2
Y	1		Credit: Water Metering	1

0	17	0	Energy and Atmosphere	33
Y			Prereq: Fundamental Commissioning and Verification	Required
Y			Prereq: Minimum Energy Performance	Required
Y			Prereq: Building-Level Energy Metering	Required
Y			Prereq: Fundamental Refrigerant Management	Required
Y	4		Credit: Enhanced Commissioning	6
Y	12		Credit: Optimize Energy Performance	18
Y	1		Credit: Advanced Energy Metering	1
		N	Credit: Grid Harmonization	2
		N	Credit: Renewable Energy	5
		N	Credit: Enhanced Refrigerant Management	1

0	10	0	Materials and Resources	13
Y			Prereq: Storage and Collection of Recyclables	Required
Y	3		Credit: Building Life-Cycle Impact Reduction	5
Y	2		Credit: Environmental Product Declarations	2
Y	1		Credit: Sourcing of Raw Materials	2
Y	2		Credit: Material Ingredients	2
Y	2		Credit: Construction and Demolition Waste Management	2

0	12	0	Indoor Environmental Quality	16
Y			Prereq: Minimum Indoor Air Quality Performance	Required
Y			Prereq: Environmental Tobacco Smoke Control	Required
Y	2		Credit: Enhanced Indoor Air Quality Strategies	2
		N	Credit: Low-Emitting Materials	3
Y	1		Credit: Construction Indoor Air Quality Management Plan	1
Y	2		Credit: Indoor Air Quality Assessment	2
Y	1		Credit: Thermal Comfort	1
Y	2		Credit: Interior Lighting	2
Y	3		Credit: Daylight	3
Y	1		Credit: Quality Views	1
		N	Credit: Acoustic Performance	1

0	0	0	Innovation	6
			Credit: Innovation	5
		N	Credit: LEED Accredited Professional	1

0	2	0	Regional Priority	4
Y	1		Credit: Regional Priority: Specific Credit	1
		N	Credit: Regional Priority: Specific Credit	1
Y	1		Credit: Regional Priority: Specific Credit	1
		N	Credit: Regional Priority: Specific Credit	1

0	79	0	TOTALS	Possible Points: 110
Certified: 40 to 49 points, Silver: 50 to 59 points, Gold: 60 to 79 points, Platinum: 80 to 110				

Figure 2.22. LEED scores.

### 2.11.1. Location and Transportation

The location was chosen for its proximity to the city's financial center. Since residents will work in the Downtown area, the property will be pedestrian-friendly and offer access to good public transportation. The availability of such environmentally friendly modes of transportation encourages local residents to walk five to ten minutes and reduces greenhouse gas emissions from private cars. Scores are shown in Figure 2.23.

0	26	0	Location and Transportation	16
Y	16		Credit: LEED for Neighborhood Development Location	16
		N	Credit: Sensitive Land Protection	1
		N	Credit: High Priority Site and Equitable Development	2
Y	3		Credit: Surrounding Density and Diverse Uses	5
Y	5		Credit: Access to Quality Transit	5
		N	Credit: Bicycle Facilities	1
Y	1		Credit: Reduced Parking Footprint	1
Y	1		Credit: Electric Vehicles	1

Figure 2.23. Location and transportation scoreboard.

### 2.11.2. Sustainable Sites

To ensure a harmonious interaction between the occupants, the building and its surroundings, a thorough pre-design assessment examined biodiversity, environmental conditions and adjacent natural regions. In addition to a rooftop drainage system that facilitates the management of urban rainfall, the design includes open spaces that connect the area with nature. Figure 2.24 shows the performance indicators for this category.

0	7	0	<b>Sustainable Sites</b>		<b>10</b>
Y			Prereq	Construction Activity Pollution Prevention	Required
Y	1		Credit	Site Assessment	1
Y	2		Credit	Protect or Restore Habitat	2
Y	1		Credit	Open Space	1
Y	3		Credit	Rainwater Management	3
		N	Credit	Heat Island Reduction	2
		N	Credit	Light Pollution Reduction	1

Figure 2.24. Sustainable sites scoreboard.

### 2.11.3. Water Efficiency

The project will utilize low-cost, non-potable water for outdoor use to meet LEED standards. Water shortages will be addressed and consumption will be reduced through a controlled water management system and conservation policies for residents. Graywater recycling technologies will also be implemented to reduce reliance on freshwater supplies. Figure 2.25 shows the LEED equivalent points for these water efficiency measures.

0	5	0	<b>Water Efficiency</b>		<b>11</b>
Y			Prereq	Outdoor Water Use Reduction	Required
Y			Prereq	Indoor Water Use Reduction	Required
Y			Prereq	Building-Level Water Metering	Required
Y	2		Credit	Outdoor Water Use Reduction	2
		N	Credit	Indoor Water Use Reduction	6
Y	2		Credit	Optimize Process Water Use	2
Y	1		Credit	Water Metering	1

Figure 2.25. Water efficiency scoreboard.

### 2.11.4. Energy and Atmosphere

This energy efficient design prioritizes:

Strategic solar harvesting by maximizing solar exposure

Intelligent zoning control and highly efficient HVAC systems

Real-time energy monitoring to maximize performance

Figure 2.26 shows the achieved energy efficiency ratings.

0	17	0	<b>Energy and Atmosphere</b>		<b>33</b>
Y			Prereq	Fundamental Commissioning and Verification	Required
Y			Prereq	Minimum Energy Performance	Required
Y			Prereq	Building-Level Energy Metering	Required
Y			Prereq	Fundamental Refrigerant Management	Required
Y	4		Credit	Enhanced Commissioning	6
Y	12		Credit	Optimize Energy Performance	18
Y	1		Credit	Advanced Energy Metering	1
		N	Credit	Grid Harmonization	2
		N	Credit	Renewable Energy	5
		N	Credit	Enhanced Refrigerant Management	1

Figure 2.26. Energy and atmosphere scoreboard.

### 2.11.5. Materials and Resources

By using additional cementitious materials (slag, fly ash and silica fume) to partially replace cement, the structural design uses environmentally friendly concrete compositions and reduces its carbon footprint. Before construction begins, a detailed waste management plan will be created, which will include material sorting and life cycle analysis of each building component. To ensure that construction waste is properly disposed of in landfills, strict waste tracking procedures will be monitored by on-site engineers. Figure 2.27 shows the environmental performance indicators for these sustainable strategies.

0	10	0	<b>Materials and Resources</b>		<b>13</b>
Y			Prereq	Storage and Collection of Recyclables	Required
Y	3		Credit	Building Life-Cycle Impact Reduction	5
Y	2		Credit	Environmental Product Declarations	2
Y	1		Credit	Sourcing of Raw Materials	2
Y	2		Credit	Material Ingredients	2
Y	2		Credit	Construction and Demolition Waste Management	2

Figure 2.27. Materials and resources scoreboard.

### 2.11.6. Indoor Environmental Quality

Through stringent smoke control procedures and improved HVAC performance, the project will maintain indoor air quality that exceeds minimum criteria. Other improvements will include:

Strategic use of low VOC materials can reduce indoor pollution.

Careful site placement to provide occupants with good views of the outdoors

Intelligent temperature control systems that use sunlight to measure and modify thermal comfort

Automated smart home technologies integrated with sophisticated lighting solutions

A building environmental management system will be used to continuously monitor these integrated systems. Figure 2.28 shows the score.

0	12	0	Indoor Environmental Quality		16
Y			Prereq	Minimum Indoor Air Quality Performance	Required
Y			Prereq	Environmental Tobacco Smoke Control	Required
Y	2		Credit	Enhanced Indoor Air Quality Strategies	2
		N	Credit	Low-Emitting Materials	3
Y	1		Credit	Construction Indoor Air Quality Management Plan	1
Y	2		Credit	Indoor Air Quality Assessment	2
Y	1		Credit	Thermal Comfort	1
Y	2		Credit	Interior Lighting	2
Y	3		Credit	Daylight	3
Y	1		Credit	Quality Views	1
		N	Credit	Acoustic Performance	1

Figure 2.28. Indoor environmental quality scoreboard.

## 3. Structural Design

### 3.1. Structural load calculations

#### 3.1.1. Calculation of Dead Loads

Structural and Non-structural dead loads for the floor and roof were determined in accordance with the guidelines provided in Chapter 3 of ASCE 7-16 (American Society of Civil Engineers, 2017). The floor dead loads were computed based on types of purposes of each floor. Detailed calculations for the dead loads and all layers considered on each floor, stair loads, and wall loads is shown in following tables.

<b>Interior Wall Load</b>				
<b>Location</b>	<b>Material</b>	<b>Thickness, mm</b>	<b>Load, kPa</b>	<b>Wall load, kPa</b>
Corridors 1st floor, separation between staff and guest areas t=232	Finishing	2	0.04	<b>1.9</b>
	Plaster	14	0.24	
	Aerated Concrete	200	1.2	
	Plaster	14	0.24	
	Finishing	2	0.04	
Room Separation, common walls t=208	Finishing	6	0.04	<b>2.07</b>
	Gypsum Board	13	0.1	
	Aerated Concrete	170	1.02	
	Gypsum Board	13	0.1	
	Finishing	6	0.04	
Bathroom interior t=112	Cement Tiles	10	0.77	<b>2.64</b>
	Masonry - Clay Brick	102	1.87	
Bathroom exterior t=131	Cement tiles	10	0.77	<b>2.78</b>
	Masonry - Clay Brick	102	1.87	
	Gypsum Board	13	0.1	
	Finishing	6	0.04	
<b>Exterior wall Load</b>				
<b>Location</b>	<b>Material</b>	<b>Thickness, mm</b>	<b>Load, kPa</b>	<b>Wall load, kPa</b>
Edge walls t=366	Fiber cement siding (steel studs)	10	0.19	<b>2.632</b>
	Air gap	30	0	
	Vapor & Wind control layer	0	0.03	
	Mineral wool insulation	60	0.132	
	Aerated Concrete 250 mm	250	2	
	Plaster	14	0.24	
	Finishing	2	0.04	

**Table 3.1.1** Calculation of the load of each type of wall per meter squared.

Wall Load per Slab					
	Type of Wall	Wall Load per m2	Area (from Revit)	Total Load, kN	Uniform Load, kPa
Slab 1	Corridor	1.9	673	1278.7	0.631
	Room Separation	2.07	446.7	924.669	0.457
	t=120 toilet	2.78	321	892.38	0.441
	t=100 toilet	2.64	271	715.44	0.353
	Exterior Wall	2.632	885	2329.32	1.150
<b>Total:</b>					<b>3.032</b>
Slab 2	Corridor	1.9	345	655.5	0.324
	Room Separation	2.07	370.2	2.07	0.001
	t=120 toilet	2.78	281	781.18	0.386
	t=100 toilet	2.64	294	776.16	0.383
	Exterior Wall	2.632	732	1926.624	0.951
<b>Total:</b>					<b>2.045</b>
Slab 3	Corridor	1.9	177	336.3	0.166
	Room Separation	2.07	701.2	1451.484	0.717
	t=120 toilet	2.78	173	480.94	0.238
	t=100 toilet	2.64	279	736.56	0.364
	Exterior Wall	2.632	876	2305.632	1.139
<b>Total:</b>					<b>2.623</b>
Slab 4	Corridor	1.9	88	167.2	0.083
	Room Separation	2.07	891.2	1844.784	0.911
	t=120 toilet	2.78	158	439.24	0.217
	t=100 toilet	2.64	210	554.4	0.274
	Exterior Wall	2.632	732	1926.624	0.951
<b>Total:</b>					<b>2.436</b>
Slab 5	Corridor	1.9	410	779	0.385
	Room Separation	2.07	315.2	652.464	0.322
	t=120 toilet	2.78	280	778.4	0.384
	t=100 toilet	2.64	294	776.16	0.383
	Exterior Wall	2.632	732	1926.624	0.951
<b>Total:</b>					<b>2.426</b>
Slab 6-15	Room Separation	2.07	1616.7	3346.569	1.653
	Exterior Wall	2.632	579	1523.928	0.753

<b>Total:</b>					<b>2.405</b>
<b>Slab 16</b>	Corridor	1.9	517	982.3	0.485
	Room Separation	2.07	315.2	652.464	0.322
	t=120 toilet	2.78	280	778.4	0.384
	t=100 toilet	2.64	294	776.16	0.383
	Exterior Wall	2.632	526	1384.432	0.684
<b>Total:</b>					<b>2.259</b>
<b>Slab 17 (Roof)</b>					
	Exterior Walls	2.632	11.88	31.26816	0.015
<b>Total:</b>					<b>0.015</b>
<b>Slab -1 (Parking)</b>					
	Exterior Walls	2.47	485	1197.95	0.592
<b>Total:</b>					<b>0.592</b>

**Table 3.1.2.** Detailed calculation of Wall Load to the Slab

<b>Wall Load to Slab</b>	
Location	Total Wall Load, kPa
Slab 1	3.032
Slab 2	2.045
Slab 3	2.623
Slab 4	2.436
Slab 5	2.426
Slab 6 - 15	2.405
Slab 16	2.259
Slab 17	0.015
Slab -1	0.592

**Table 3.1.3.** Summarized calculation of Wall Load per Slab

It was assumed that walls would have a uniformly distributed load, so the calculated load (kN/m<sup>2</sup>) for the partition walls is:

$$\text{Floor area} = 2025 \text{ m}^2$$

Uniformly distributed load of partition walls =

$$\sum_{i=1}^n (\text{wall load}_i \times \text{total wall area on slab}_i) / 2025$$

H, m	h rise, mm	d, mm	n/2	Length with Landings, mm	Landing depth, mm	Width of stairs	Volume per floor, m <sup>3</sup>	Density, kg/m <sup>3</sup>	Axial Load, kN	Load, kPa	# of stair shafts in building	Load per Floor Slab, kPa
6	177	280	17	7160	1220	1200	3.6	2407.3 2	84.38	0.042	2	0.083
5	157	280	16	6880	1360	1200	3.1		72.40	0.036		0.072
4	167	280	12	5760	1920	1200	2.8		66.64	0.033		0.066
3.5	103	280	17	7160	1220	1200	2.1		49.10	0.024		0.048

**Table 3.1.4.** Stair Load Calculation

Ceiling Load, kPa	
Mechanical Duct Allowance	0.19
Gypsum Layer	0.1
	<b>0.29</b>

**Table 3.1.5.** Ceiling Load Calculation

Summarizing, Dead Load is the sum of slab, partition wall, stairs, and ceiling loads.

### 3.1.2. Calculation of Live Loads

For the Hotel, accommodation floors are assumed as residential building and floors designed for public use are assumed as floors with public access, therefore, the minimum uniformly distributed loads based on ASCE 7-16 were identified and shown below (American Society of Civil Engineers, 2017):

Live load reduction should be applied for all the uniformly distributed load except public rooms (20% reduction permitted if the floor supports two or more floors above) according to ASCE 7-16. The reduction for floor live load should be applied using the following formula:

$$L = L_0 \left( 0.25 + \frac{4.57}{\sqrt{K_{LL} A_r}} \right) \quad (3.1.)$$

Live load element factor,  $K_{LL}$ , for the structural members will be:

- Interior columns:  $K_{LL} = 4$
- Exterior columns without cantilever slabs:  $K_{LL} = 4$

- Corner columns:  $K_{LL} = 4$
- Edge beams without cantilever slabs:  $K_{LL} = 2$
- Interior beams:  $K_{LL} = 2$

Live Load, kPa							
	Type of floor	Beam interior	Beam Exterior	Column Exterior	Column Corner	Column Interior	Notes
Slab 1-5	Public rooms support $\geq 2$ floors	3.83	3.83	3.83	3.83	3.83	20% reduction allowed as an exception
Slab 6-15	Hotel Rooms	1.67	1.92	1.32	1.92	1.07	
Slab 16	Public rooms support 1 floor	4.79	4.79	4.79	4.79	4.79	No reduction allowed ASCE-7

**Table 3.1.6.** Live Load Calculation

	Beam interior	Beam Exterior	Column Exterior	Column Corner	Column Interior
<b>K(LL)</b>	2	2	4	1	4
<b>At, m<sup>2</sup></b>	27.38	13.69	27.38	13.69	54.76
<b>K(LL) x At</b>	54.76	27.38	109.52	13.69	219.04

**Table 3.1.7.** Live Load assumptions

Whereas, for roof live load, the following reduction should be applied:

$$L_r = L_0 R_1 R_2 \quad \text{where } 0.58 \leq L_r \leq 0.96 \quad (3.2)$$

The reduction factor  $R_1$  can be determined as follows:

$$R_1 = \begin{cases} 1 & \text{for } A_T \leq 18.58 \text{ m}^2 \\ 1.2 - 0.011A_T & \text{for } 18.58 \text{ m}^2 < A_T < 55.74 \text{ m}^2 \\ 0.6 & \text{for } A_T \geq 55.74 \text{ m}^2 \end{cases} \quad (3.3)$$

The reduction factor  $R_2$  for ordinary flat roofs is 1.

The roof live load calculations for each structural member can be seen in Table 3.1.8, 3.1.9.

R1		R2		
At	R1	Slope, %	F	R2
54.76	0.59764	2	0.24	1

**Table 3.1.8.** Calculation of reduction factors

Roof Live Load			
Lo, kPa	R1	R2	Lr, kPa
0.96	0.59764	1	0.58

**Table 3.1.9.** Calculation of Roof Live Load

### 3.1.3. Calculation of Snow Loads

According to ASCE 7-16, Los Angeles City is not considered to have any snow loads, therefore, snow load was not calculated for the design project (American Society of Civil Engineers, 2017).

### 3.1.4. Calculation of Wind Loads

Wind load calculations were performed according to the ASCE 7-16. Basic wind speed in Los Angeles, California was found as  $V = 42 \text{ m/s}$  based on Figure 26.5-1A of ASCE 7-16 (American Society of Civil Engineers, 2017).

- Exposure category B (city)
- Topographical Effect:  $K_{zt} = 1$  (flat land)
- Direction Effect:  $K_d = 0.85$  (main wind load force resisting system (LFRS))
- Height Effect

The velocity pressure exposure coefficient can be calculated by formula:

$$K_z = 2.01 \left( \frac{z}{z_g} \right)^{2/\alpha}, \text{ for } z < 15 \text{ ft} \quad (3)$$

$$K_z = 2.01 \left( \frac{z}{z_g} \right)^{2/\alpha}, \text{ for } 15 \text{ ft} \leq z \leq z_g \quad (3.)$$

- Gust effect

The Gust effect was calculated considering 2 possible wind directions.

For case 1:  $B = 45 \text{ m}$ ,  $L = 45 \text{ m}$ ,  $m$  (number of frames) = 7

For case 2:  $B = 45 \text{ m}$ ,  $L = 45 \text{ m}$ ,  $m$  (number of frames) = 7

Therefore, the Gust effect is calculated for one case with consideration of same gust.

Case 1 (Y-axis)

Since  $H = 73.4 \text{ m} > 18 \text{ m}$ , the building is not a “low-rise” building. Because the building is comprised of reinforced concrete moment frames, estimate Gust factor:

Gust effect							
na	0.3220706396	<1 hz	H	71	m	232.951	ft
	flexible building		c	0.3			
			l	97.54	m		
			epsilon	0.3333333333			
			alpha dash	0.25			
z dash	42.6	>9.14	b dash	0.45			
Iz	0.2356240708		L	45			
Lz	158.1196888		B	45			
n1=na	0.322		beta	0.02			
Vz	27.15277394						
N1	1.875525109						
Rn	0.09260728383						
Rh	0.2248325397		nh	3.873941982			
Rb	0.3249523766		nb	2.455315341			
Rl	0.1142549992		nl	8.219968749			
R	0.4443670088						
gr	3.910115393						
Q	0.8115589212						
<b>Gf</b>	<b>0.903</b>						

**Table 3.1.10.** Gust Effect

Case 1 (X-axis)

The same calculations were performed for this case, but only L and B were changed.

$$G_f = 0.925 \left( \frac{1 + 1.7I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7g_v I_z} \right) = 0.903$$

$$L/B=1$$

External pressure coefficient  $C_p$ :

Windward = 0.8 and Leeward = -0.5

Velocity pressures, wind pressure, and wind forces acting on each floor were calculated using formulas below for each case.

Velocity pressure calculation

$$q_z = 0.613 K_z K_{zt} K_d V^2 \tag{3.33}$$

Design wind pressure calculation

$$P = q_z GC_{p,windward} - q_h GC_{p,leeward} \quad (3.34)$$

Force at top floor

$$F_{21} = (B/\text{number of frames}) \cdot h_{21}/2 \cdot P_{21} \quad (3.35)$$

Force at  $i_{th}$  floor

$$F_i = (B/\text{number of frames}) \cdot (P_{i+1} \cdot h_{i+1}/2 + P_i \cdot h_i/2) \quad (3.36)$$

Case 1													
number of frames	7												
B (m)	45												
L (m)	45												
Windward	Story	h (m)	z (m)	Kz	V (m/s)	Kd	Kzt	q (Pa)	Gf	Pw, Pl (Pa)	P (Pa)	Fi (kN)	F per frame (kN)
	17	2.4	73.4	1.270	42	0.85	1	1167.58	0.903	843.61	1370.87	74.03	37.01
	16	5	71	1.258	42	0.85	1	1156.55	0.903	835.64	1362.90	227.35	32.48
	15	4	66	1.232	42	0.85	1	1132.67	0.903	818.38	1345.64	274.43	39.20
	14	4	62	1.211	42	0.85	1	1112.61	0.903	803.90	1331.15	240.91	34.42
	13	4	58	1.188	42	0.85	1	1091.61	0.903	788.72	1315.98	238.24	34.03
	12	4	54	1.164	42	0.85	1	1069.55	0.903	772.78	1300.04	235.44	33.63
	11	4	50	1.138	42	0.85	1	1046.29	0.903	755.98	1283.23	232.49	33.21
<b>Cp</b>	<b>10</b>	4	46	1.112	42	0.85	1	1021.66	0.903	738.18	1265.44	229.38	32.77
<b>0.8</b>	<b>9</b>	4	42	1.083	42	0.85	1	995.45	0.903	719.24	1246.50	226.07	32.30
	<b>8</b>	4	38	1.052	42	0.85	1	967.38	0.903	698.96	1226.22	222.54	31.79
	<b>7</b>	4	34	1.020	42	0.85	1	937.13	0.903	677.10	1204.36	218.75	31.25
	<b>6</b>	4	30	0.984	42	0.85	1	904.21	0.903	653.31	1180.57	214.64	30.66
	<b>5</b>	5	26	0.944	42	0.85	1	867.98	0.903	627.14	1154.40	236.12	33.73
	<b>4</b>	5	21	0.888	42	0.85	1	816.60	0.903	590.02	1117.28	255.56	36.51
	<b>3</b>	5	16	0.822	42	0.85	1	755.56	0.903	545.91	1073.17	246.43	35.20
	<b>2</b>	5	11	0.739	42	0.85	1	678.85	0.903	490.49	1017.75	235.23	33.60
	<b>1</b>	6	6	0.621	42	0.85	1	570.90	0.903	412.49	939.75	241.36	34.48
<b>Leeward</b>	<b>17</b>		73.4	1.270	42	0.85	1	1167.58	0.903	-527.25 85617			
<b>Cp</b>													
<b>-0.5</b>													

Table 3.1.11. Case 1

<b>Case 2</b>				
<b>ex</b>	6.75			
<b>Story</b>	<b>Pw, PI (Pa)</b>	<b>MT (kN-m)</b>	<b>0.75Fi (kN)</b>	<b>Fdirect (kN)</b>
17	843.61	53.44	55.52	27.76
16	835.64	310.49	170.51	24.36
15	818.38	306.55	205.83	29.40
14	803.90	303.25	180.68	25.81
13	788.72	299.80	178.68	25.53
12	772.78	296.17	176.58	25.23
11	755.98	292.34	174.37	24.91
10	738.18	288.28	172.04	24.58
9	719.24	283.97	169.56	24.22
8	698.96	279.35	166.91	23.84
7	677.10	274.37	164.06	23.44
6	653.31	268.95	160.98	23.00
5	627.14	262.99	177.09	25.30
4	590.02	254.53	191.67	27.38
3	545.91	244.48	184.82	26.40
2	490.49	231.86	176.42	25.20
1	412.49	214.09	181.02	25.86
	-527.2585617			

**Table 3.1.12. Case 2**

<b>Case 3</b>				
<b>Story</b>	<b>0.75F1 (kN)</b>	<b>F1 per frame (kN)</b>	<b>0.75F2 (kN)</b>	<b>F2 per frame (kN)</b>
17	55.52	27.76	55.52	27.76
16	170.51	24.36	170.51	24.36
15	205.83	29.40	205.83	29.40
14	180.68	25.81	180.68	25.81
13	178.68	25.53	178.68	25.53
12	176.58	25.23	176.58	25.23
11	174.37	24.91	174.37	24.91
10	172.04	24.58	172.04	24.58
9	169.56	24.22	169.56	24.22
8	166.91	23.84	166.91	23.84
7	164.06	23.44	164.06	23.44
6	160.98	23.00	160.98	23.00
5	177.09	25.30	177.09	25.30
4	191.67	27.38	191.67	27.38
3	184.82	26.40	184.82	26.40
2	176.42	25.20	176.42	25.20
1	181.02	25.86	181.02	25.86

**Table 3.1.13. Case 3**

Case 4							
ex	6.75						
ey	6.75						
Story	Pw, Pl (Pa) - x axis	Pw, Pl (Pa) - y axis	MT (kN-m)	0.563F1 (kN)	0.563F2 (kN)	F1 direct (kN)	F2 direct (kN)
17	843.61	843.61	274.55	41.68	41.68	20.84	20.84
16	835.64	835.64	466.14	128.00	128.00	18.29	18.29
15	818.38	818.38	460.24	154.51	154.51	22.07	22.07
14	803.90	803.90	455.28	135.63	135.63	19.38	19.38
13	788.72	788.72	450.10	134.13	134.13	19.16	19.16
12	772.78	772.78	444.64	132.55	132.55	18.94	18.94
11	755.98	755.98	438.89	130.89	130.89	18.70	18.70
10	738.18	738.18	432.81	129.14	129.14	18.45	18.45
9	719.24	719.24	426.33	127.28	127.28	18.18	18.18
8	698.96	698.96	419.40	125.29	125.29	17.90	17.90
7	677.10	677.10	411.92	123.16	123.16	17.59	17.59
6	653.31	653.31	403.78	120.84	120.84	17.26	17.26
5	627.14	627.14	394.83	132.94	132.94	18.99	18.99
4	590.02	590.02	382.13	143.88	143.88	20.55	20.55
3	545.91	545.91	367.05	138.74	138.74	19.82	19.82
2	490.49	490.49	348.09	132.43	132.43	18.92	18.92
1	412.49	412.49	321.42	135.89	135.89	19.41	19.41

**Table 3.1.14. Case 4**

### **Torsional Effect, Wind**

Given the building's susceptibility to torsional effects due to variations in stiffness and mass centers, the torsional effect must be accounted for, and the seismic forces on each frame need to be calculated.

The mass center was determined as the first step. To calculate the mass center, the slabs, floor finishing, partition walls, and exterior walls were considered. For this building, the areas of the elevator and stair shafts were excluded from the calculations, as they lack slabs, while all other components were assumed to be uniformly distributed over the floor area. The building dimensions are 45 m in the longitudinal direction and 45 m in the transverse direction. After establishing the origin for the mass center determination, the necessary calculations were carried out as follows:

$x = 22.31 \text{ m}$  (due to symmetrical nature of the building)

$$y = \frac{A_{\text{floor}} y_{\text{floor}} - A_{\text{elevator shaft}} y_{\text{elevator shaft}} - A_{\text{stairs shaft, 1}} y_{\text{stairs shaft, 1}} - A_{\text{stairs shaft, 2}} y_{\text{stairs shaft, 2}}}{A_{\text{floor}} - A_{\text{shafts}}}$$

$$y = 22.34 \text{ m}$$

So, mass center:  $x = 22.31 \text{ m}$ ,  $y = 22.34 \text{ m}$ .

Afterwards, the stiffness center was determined by calculating the stiffness of the moment frame. Firstly, the cracking moment of inertia of beams and columns were determined using the following equations:

$$I_{b,cr} = \frac{1}{12} b h^3 \cdot 0.35$$

$$I_{c,cr} = \frac{1}{12} b h^3 \cdot 0.7$$

Next, the stiffness of frame can be calculated using the following formula:

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

Force needed to generate an inter-story rotation:

$$C_F = h \cdot D \cdot \# \text{ of columns per frame} \cdot \# \text{ of frames}$$

Consequently, the stiffness center can be calculated.

Accordingly, stiffness center:  $x' = 26.16 \text{ m}$ ,  $y' = 23.15 \text{ m}$  (uniform spacing between columns and major beams)

<b>Mass center</b>	<b>m</b>
x	22.31
y	22.34
<b>Stiffness center</b>	
x'	26.15719654
y'	23.15250431
<b>Eccentricity</b>	
ex'	-3.85
ey'	-0.81
e_ax	2.25
e_ay	2.25
ex	-1.60
ey	1.44

**Table 3.1.15.** Eccentricity

Accordingly, torsion moment, direct force, and torsional force will be calculated as:

$$T = eF$$

$$F_{direct} = \frac{F_x}{\# \text{ of frames}}$$

$$F_{torsion} = \frac{T * C_f * (x_i \text{ or } y_i)}{\sum C_f * (x_i^2 \text{ or } y_i^2)}$$

$$F_{total} = F_{direct} \pm F_{torsion}$$

Case 2 Both Sides				
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	A	-1.5499	25.86	24.31
	B	-1.0332	25.86	24.83
	C	-0.5166	25.86	25.34
	D	0.0000	25.86	25.86
	E	0.5166	25.86	26.38
	F	1.0332	25.86	26.89
	G	1.5499	25.86	27.41
Case 2 Second Side				
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
2	A	-1.6785	25.20	23.52
	B	-1.1190	25.20	24.08
	C	-0.5595	25.20	24.64
	D	0.0000	25.20	25.20
	E	0.5595	25.20	25.76
	F	1.1190	25.20	26.32
	G	1.6785	25.20	26.88
Case 2 Second Side				
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
3	A	-1.7699	26.40	24.63
	B	-1.1799	26.40	25.22
	C	-0.5900	26.40	25.81
	D	0.0000	26.40	26.40
	E	0.5900	26.40	26.99
	F	1.1799	26.40	27.58

	G	1.7699	26.40	28.17
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
4	A	-1.8426	27.38	25.54
	B	-1.2284	27.38	26.15
	C	-0.6142	27.38	26.77
	D	0.0000	27.38	27.38
	E	0.6142	27.38	28.00
	F	1.2284	27.38	28.61
	G	1.8426	27.38	29.22
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
5	A	-1.9039	25.30	23.39
	B	-1.2692	25.30	24.03
	C	-0.6346	25.30	24.66
	D	0.0000	25.30	25.30
	E	0.6346	25.30	25.93
	F	1.2692	25.30	26.57
	G	1.9039	25.30	27.20
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
6	A	-1.9470	23.00	21.05
	B	-1.2980	23.00	21.70
	C	-0.6490	23.00	22.35
	D	0.0000	23.00	23.00
	E	0.6490	23.00	23.65
	F	1.2980	23.00	24.30
	G	1.9470	23.00	24.94
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
7	A	-1.9863	23.44	21.45
	B	-1.3242	23.44	22.11
	C	-0.6621	23.44	22.78
	D	0.0000	23.44	23.44

	E	0.6621	23.44	24.10
	F	1.3242	23.44	24.76
	G	1.9863	23.44	25.42
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
8	A	-2.0223	23.84	21.82
	B	-1.3482	23.84	22.50
	C	-0.6741	23.84	23.17
	D	0.0000	23.84	23.84
	E	0.6741	23.84	24.52
	F	1.3482	23.84	25.19
	G	2.0223	23.84	25.87
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
9	A	-2.0558	24.22	22.17
	B	-1.3705	24.22	22.85
	C	-0.6853	24.22	23.54
	D	0.0000	24.22	24.22
	E	0.6853	24.22	24.91
	F	1.3705	24.22	25.59
	G	2.0558	24.22	26.28
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
10	A	-2.0870	24.58	22.49
	B	-1.3913	24.58	23.19
	C	-0.6957	24.58	23.88
	D	0.0000	24.58	24.58
	E	0.6957	24.58	25.27
	F	1.3913	24.58	25.97
	G	2.0870	24.58	26.66
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
11	A	-2.1163	24.91	22.79
	B	-1.4109	24.91	23.50

	C	-0.7054	24.91	24.20
	D	0.0000	24.91	24.91
	E	0.7054	24.91	25.62
	F	1.4109	24.91	26.32
	G	2.1163	24.91	27.03
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
12	A	-2.1441	25.23	23.08
	B	-1.4294	25.23	23.80
	C	-0.7147	25.23	24.51
	D	0.0000	25.23	25.23
	E	0.7147	25.23	25.94
	F	1.4294	25.23	26.66
	G	2.1441	25.23	27.37
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
13	A	-2.1703	25.53	23.36
	B	-1.4469	25.53	24.08
	C	-0.7234	25.53	24.80
	D	0.0000	25.53	25.53
	E	0.7234	25.53	26.25
	F	1.4469	25.53	26.97
	G	2.1703	25.53	27.70
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
14	A	-2.1954	25.81	23.62
	B	-1.4636	25.81	24.35
	C	-0.7318	25.81	25.08
	D	0.0000	25.81	25.81
	E	0.7318	25.81	26.54
	F	1.4636	25.81	27.28
	G	2.1954	25.81	28.01
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)

15	A	-2.2193	29.40	27.18
	B	-1.4795	29.40	27.92
	C	-0.7398	29.40	28.66
	D	0.0000	29.40	29.40
	E	0.7398	29.40	30.14
	F	1.4795	29.40	30.88
	G	2.2193	29.40	31.62
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
16	A	-2.2477	24.36	22.11
	B	-1.4985	24.36	22.86
	C	-0.7492	24.36	23.61
	D	0.0000	24.36	24.36
	E	0.7492	24.36	25.11
	F	1.4985	24.36	25.86
	G	2.2477	24.36	26.61
	Case 2 Second Side			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
17				
	G	1.2036	27.76	28.96

**Table 3.1.16. Torsional Effect from Wind Case 2**

	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	1	-2.3269	19.41	17.09
	2	-1.5512	19.41	17.86
	3	-0.7756	19.41	18.64
	4	0.0000	19.41	19.41
	5	0.7756	19.41	20.19
	6	1.5512	19.41	20.96
	7	2.3269	19.41	21.74
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
2	1	-2.5200	18.92	16.40
	2	-1.6800	18.92	17.24
	3	-0.8400	18.92	18.08
	4	0.0000	18.92	18.92

	5	0.8400	18.92	19.76
	6	1.6800	18.92	20.60
	7	2.5200	18.92	21.44
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
3	1	-2.6572	19.82	17.16
	2	-1.7715	19.82	18.05
	3	-0.8857	19.82	18.93
	4	0.0000	19.82	19.82
	5	0.8857	19.82	20.71
	6	1.7715	19.82	21.59
	7	2.6572	19.82	22.48
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
4	1	-2.7664	20.55	17.79
	2	-1.8443	20.55	18.71
	3	-0.9221	20.55	19.63
	4	0.0000	20.55	20.55
	5	0.9221	20.55	21.48
	6	1.8443	20.55	22.40
	7	2.7664	20.55	23.32
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
5	1	-2.8583	18.99	16.13
	2	-1.9056	18.99	17.09
	3	-0.9528	18.99	18.04
	4	0.0000	18.99	18.99
	5	0.9528	18.99	19.94
	6	1.9056	18.99	20.90
	7	2.8583	18.99	21.85
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
6	1	-2.9231	17.26	14.34
	2	-1.9488	17.26	15.31
	3	-0.9744	17.26	16.29
	4	0.0000	17.26	17.26
	5	0.9744	17.26	18.24
	6	1.9488	17.26	19.21

	7	2.9231	17.26	20.19
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
7	1	-2.9820	17.59	14.61
	2	-1.9880	17.59	15.61
	3	-0.9940	17.59	16.60
	4	0.0000	17.59	17.59
	5	0.9940	17.59	18.59
	6	1.9880	17.59	19.58
	7	2.9820	17.59	20.58
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
8	1	-3.0362	17.90	14.86
	2	-2.0241	17.90	15.87
	3	-1.0121	17.90	16.89
	4	0.0000	17.90	17.90
	5	1.0121	17.90	18.91
	6	2.0241	17.90	19.92
	7	3.0362	17.90	20.94
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
9	1	-3.0864	18.18	15.10
	2	-2.0576	18.18	16.13
	3	-1.0288	18.18	17.15
	4	0.0000	18.18	18.18
	5	1.0288	18.18	19.21
	6	2.0576	18.18	20.24
	7	3.0864	18.18	21.27
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
10	1	-3.1333	18.45	15.32
	2	-2.0888	18.45	16.36
	3	-1.0444	18.45	17.40
	4	0.0000	18.45	18.45
	5	1.0444	18.45	19.49
	6	2.0888	18.45	20.54
	7	3.1333	18.45	21.58
	Case 4 both sides			

Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
11	1	-3.1773	18.70	15.52
	2	-2.1182	18.70	16.58
	3	-1.0591	18.70	17.64
	4	0.0000	18.70	18.70
	5	1.0591	18.70	19.76
	6	2.1182	18.70	20.82
	7	3.1773	18.70	21.88
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
12	1	-3.2189	18.94	15.72
	2	-2.1460	18.94	16.79
	3	-1.0730	18.94	17.86
	4	0.0000	18.94	18.94
	5	1.0730	18.94	20.01
	6	2.1460	18.94	21.08
	7	3.2189	18.94	22.16
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
13	1	-3.2584	19.16	15.90
	2	-2.1723	19.16	16.99
	3	-1.0861	19.16	18.08
	4	0.0000	19.16	19.16
	5	1.0861	19.16	20.25
	6	2.1723	19.16	21.33
	7	3.2584	19.16	22.42
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
14	1	-3.2960	19.38	16.08
	2	-2.1973	19.38	17.18
	3	-1.0987	19.38	18.28
	4	0.0000	19.38	19.38
	5	1.0987	19.38	20.47
	6	2.1973	19.38	21.57
	7	3.2960	19.38	22.67
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
15	1	-3.3319	22.07	18.74

	2	-2.2212	22.07	19.85
	3	-1.1106	22.07	20.96
	4	0.0000	22.07	22.07
	5	1.1106	22.07	23.18
	6	2.2212	22.07	24.29
	7	3.3319	22.07	25.40
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
16	1	-3.3746	18.29	14.91
	2	-2.2497	18.29	16.04
	3	-1.1249	18.29	17.16
	4	0.0000	18.29	18.29
	5	1.1249	18.29	19.41
	6	2.2497	18.29	20.54
	7	3.3746	18.29	21.66
	Case 4 both sides			
Floors	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
17	1	-1.9876	20.84	18.85
	2	-1.3250	20.84	19.51
	3	-0.6625	20.84	20.18
	4	0.0000	20.84	20.84
	5	0.6625	20.84	21.50
	6	1.3250	20.84	22.16
	7	1.9876	20.84	22.83

**Table 3.1.17. Torsional Effect from Wind Case 4**

### 3.1.5. Calculation of Seismic Loads

To determine the seismic loads acting on the building, the Equivalent Lateral Force (ELF) procedure will be utilized (American Society of Civil Engineers, 2017). According to the site investigation report, the soil was classified as Class C, which will be used to evaluate the seismic design parameters for the site (GeoPentech, 2020). The seismic parameters for the location were determined using the ASCE7 Hazard Tool online map. Since the structure is a residential complex, it was assigned a Risk Category II. Based on the tool's findings, the mapped acceleration parameters were identified as follows:

$$S_s = 1.937 g, S_1 = 0.688 g$$

Accordingly, based on the site class, the soil condition correction coefficients were used to find the adjusted parameters:

$$F_a = 1.2, F_v = 1.4$$

$$S_{MS} = F_a S_s$$

$$S_{M1} = F_v S_1$$

$$S_{MS} = 1.2 \cdot 1.937 = 2.324 \text{ g}$$

$$S_{M1} = 1.4 \cdot 0.688 = 0.963 \text{ g}$$

Next, the maximum considered earthquake parameters were adjusted for design acceleration parameters:

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

$$S_{DS} = \frac{2}{3} \cdot 2.324 = 1.55 \text{ g}$$

$$S_{D1} = \frac{2}{3} \cdot 0.963 = 0.642 \text{ g}$$

Based on the calculations of seismic design parameters, the seismic design category was determined as D (American Society of Civil Engineers, 2017). Using the identified seismic design category, the Lateral Force Resisting System was determined as Special Reinforced Concrete Moment Frame with the following parameters:

Response modification coefficient,  $R = 8$

Overstrength factor,  $\Omega = 3$

Design amplification factor,  $C_d = 5.5$

Importance factor,  $I_e = 1$

Furthermore, the periods of design spectrum were calculated:

$$T_L = 8 \text{ s}$$

$$T_0 = 0.2 \frac{S_{D1}}{S_{DS}}$$

$$T_0 = 0.2 \frac{0.642}{1.55} = 0.083 \text{ s}$$

$$T_s = \frac{S_{D1}}{S_{DS}}$$

$$T_s = \frac{0.642}{1.55} = 0.414 \text{ s}$$

Approximate fundamental period was calculated using the following equation:

$$T_a = C^t h_n^x$$

Where,

$T_a$  - approximate fundamental period

$C^t$  and  $x$  - approximate period parameters defined in Table 12.8-2 (ASCE 7-16, 2016)

$h_n$  - building height above the ground to its highest point (in our building,

$$h_n = 73.4 \text{ m})$$

From Table 12.8-2 in ASCE 7-16, for concrete moment-resisting frames,

$C^t = 0.0466$ ,  $x = 0.9$  (American Society of Civil Engineers, 2017). Accordingly, the approximate fundamental period can be calculated as:

$$T_a = 0.0466 \cdot 73.4^{0.9} = 2.321 \text{ s}$$

To calculate the fundamental period of the structure, the following formula was used:

$$T = C_u T_a$$

Where,

$C_u$  - coefficient for upper limit on calculated period defined in Table 12.8-1

(American Society of Civil Engineers, 2017). For our building, this coefficient is 1.4.

$$T = 1.4 \cdot 2.321 = 3.25 \text{ s}$$

Since  $T^*$  analytical value is not available,  $T_a$  will be taken for further calculation.

For the ELF procedure, the following equation should be used to determine the seismic base shear:

$$V = C_s W$$

Where,

$C_s$  - seismic response coefficient

$W$  - effective seismic weight of the building

Firstly, to calculate the seismic response coefficient, the following equation was used:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$$C_s = \frac{1.55}{\left(\frac{8}{1}\right)} = 0.19375$$

However, there are limitations for the coefficient values: (1) it should not exceed the following:

$$C_s = \frac{S_{D1}}{T(\frac{R}{I_e})} \quad \text{for } T \leq T_L$$

$$C_s = \frac{0.642}{3.25(\frac{8}{1})} = 0.025 < 0.19375 \rightarrow C_s = 0.025$$

And (2) it should not be less than:

$$C_s = 0.5S_1/(R/I_e) \quad \text{for } S_1 \geq 0.6 \text{ g}$$

$$C_s = 0.5 \cdot 0.688/(8/1) = 0.043 > 0.025 \rightarrow C_s = 0.043 \text{ (controlled)}$$

The effective seismic weight was then calculated by summing the weights of the columns, beams, slab, floor finishing, stairs, exterior walls, and partition walls. For the calculations, the weights of the slab, floor finishing, and exterior and partition walls, provided in kPa, were multiplied by the floor area of 2025 m<sup>2</sup>.

Since effective seismic weight and the seismic response coefficients are known, the seismic base shear can be calculated:

Slab level	Column Size, m		R.C. Unit Weight, kN/m <sup>3</sup>	Story clean height, m	n	Column Weight, kN	Beam Size, m		Length	n	Beam weight, kN	Dead + Partition Loads, kPa	Floor weight, kN
	a	b					h	b					
18												6.326	146.131
17	0.3	0.3	25	2.2	2	9.900	0.6	0.3	7.1	1	21.300	6.340	10810.034
16	0.3	0.3	25	4.8	41	442.800	0.6	0.3	7.1	72	1533.600	8.498	19184.563
15	0.35	0.35	25	3.8	47	546.963	0.6	0.3	7.05	84	1776.600	8.417	19368.469
14	0.4	0.4	25	3.8	47	714.400	0.6	0.3	7	84	1764.000	8.417	19523.306
13	0.45	0.45	25	3.8	47	904.163	0.6	0.3	6.95	84	1751.400	8.417	19700.469
12	0.5	0.5	25	3.8	47	1116.250	0.6	0.3	6.9	84	1738.800	8.417	19899.956
11	0.5	0.5	25	3.8	47	1116.250	0.6	0.3	6.9	84	1738.800	8.417	19899.956
10	0.55	0.55	25	3.8	47	1350.663	0.6	0.3	6.85	84	1726.200	8.417	20121.769
9	0.6	0.6	25	3.8	47	1607.400	0.6	0.3	6.8	84	1713.600	8.417	20365.906
8	0.6	0.6	25	3.8	47	1607.400	0.6	0.3	6.8	84	1713.600	8.417	20365.906
7	0.65	0.65	25	3.8	47	1886.463	0.6	0.3	6.75	84	1701.000	8.417	20632.369
6	0.65	0.65	25	3.8	47	1886.463	0.6	0.3	6.75	84	1701.000	8.417	20632.369
5	0.7	0.7	25	4.8	47	2763.600	0.6	0.3	6.7	84	1688.400	9.229	23140.183
4	0.75	0.75	25	4.8	47	3172.500	0.6	0.3	6.65	84	1675.800	8.771	22609.463
3	0.75	0.75	25	4.8	47	3172.500	0.6	0.3	6.65	84	1675.800	8.671	22407.103
2	0.8	0.8	25	4.8	47	3609.600	0.6	0.3	6.6	84	1663.200	8.146	21768.637
1	0.85	0.85	25	5.8	47	4923.838	0.6	0.3	6.55	84	1650.600	9.827	26474.058
-1	0.85	0.85	25	3.3	47	2801.494	0.6	0.3	6.55	84	1650.600	6.697	18013.671
												<b>Total weight, kN</b>	<b>365064.319</b>

**Table 3.1.18. Weight of the building**

$$V = C_s W = 0.043 \cdot 365064.319 = 15697.766 \text{ kN}$$

Next, the vertical distribution of seismic forces should be calculated using the following equation:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$$

Where,

$C_{vx}$  - vertical distribution factor

$w_i$  and  $w_x$  - portion of total effective seismic weight at each level  $i$  or  $x$

$h_i$  and  $h_x$  - height (m) from the ground to level i or x

$k$  - an exponent related to the structure period as following:

- If  $T \leq 0.5$  s  $\rightarrow k = 1$
- If  $T \geq 2.5$  s  $\rightarrow k = 2$
- $T$  between 0.5 s and 2.5 s should be interpolated as in our case:

$$k = \frac{2-1}{2.5-0.5} \cdot 2.321 + 1 = 1.91$$

After a series of calculations for each story, the lateral seismic force at each floor can be seen in Table 3.17, 3.18. It should be noted that the weight on the roof was added to the 18th floor's weight.

Story Force						
Slab level	Floor height	hx	Floor weight, kN	wxhx <sup>k</sup>	Cvx	V, kN
18	2.4	76.9	10956.165	43925915.90	0.08130792	15697.766
17						
16	5	74.5	19184.563	72394650.41	0.13400423	15697.766
15	4	69.5	19368.469	64004020.60	0.11847297	15697.766
14	4	65.5	19523.306	57607943.29	0.10663368	15697.766
13	4	61.5	19700.469	51537395.64	0.09539695	15697.766
12	4	57.5	19899.956	45782292.83	0.08474412	15697.766
11	4	53.5	19899.956	39890733.50	0.07383870	15697.766
10	4	49.5	20121.769	34770375.92	0.06436080	15697.766
9	4	45.5	20365.906	29959496.72	0.05545574	15697.766
8	4	41.5	20365.906	25129545.21	0.04651539	15697.766
7	4	37.5	20632.369	20976635.32	0.03882825	15697.766
6	4	33.5	20632.369	16910139.68	0.03130107	15697.766
5	5	29.5	23140.183	14875163.74	0.02753428	15697.766
4	5	24.5	22609.463	10192766.05	0.01886705	15697.766
3	5	19.5	22407.103	6531258.00	0.01208951	15697.766
2	5	14.5	21768.637	3602672.71	0.00666863	15697.766
1	6	9.5	26474.058	1953266.36	0.00361554	15697.766
-1	3.5	3.5	18013.671	197262.36	0.00036514	15697.766
			<b>total w<sub>i</sub>h<sub>i</sub><sup>k</sup></b>	540241534.2		<b>V, kN check</b>

**Table 3.1.19.** Cvx calculation

Story Force	Transverse (NS)			Longitudinal (NS)		
Slab level	Fx, kN	Fdirect (kN)	T (kN- m)	Fx (kN)	Fdirect (kN)	T (kN-m)
18 (stair roof)	1276.35	182.34	1835.86	1276.35	182.34	-2036.73
17 (roof)						
16	2103.57	300.51	3025.69	2103.57	300.51	-3356.75
15	1859.76	265.68	2675.01	1859.76	265.68	-2967.70
14	1673.91	239.13	2407.69	1673.91	239.13	-2671.13
13	1497.52	213.93	2153.97	1497.52	213.93	-2389.66
12	1330.29	190.04	1913.44	1330.29	190.04	-2122.81
11	1159.10	165.59	1667.21	1159.10	165.59	-1849.63
10	1010.32	144.33	1453.21	1010.32	144.33	-1612.21
9	870.53	124.36	1252.14	870.53	124.36	-1389.14
8	730.19	104.31	1050.27	730.19	104.31	-1165.19
7	609.52	87.07	876.71	609.52	87.07	-972.63
6	491.36	70.19	706.75	491.36	70.19	-784.08
5	432.23	61.75	621.70	432.23	61.75	-689.72
4	296.17	42.31	426.00	296.17	42.31	-472.61
3	189.78	27.11	272.97	189.78	27.11	-302.84
2	104.68	14.95	150.57	104.68	14.95	-167.05
1	56.76	8.11	81.64	56.76	8.11	-90.57
-1	5.73	0.82	8.24	5.73	0.82	-9.15

**Table 3.1.20.** Seismic Load Calculation

### Torsional Effect, Seismic

Given the building's susceptibility to torsional effects due to variations in stiffness and mass centers, the torsional effect must be accounted for, and the seismic forces on each frame need to be calculated.

The mass center was determined as the first step. To calculate the mass center, the slabs, floor finishing, partition walls, and exterior walls were considered. For this building, the areas of the elevator and stair shafts were excluded from the calculations, as they lack slabs, while all other components were assumed to be uniformly distributed over the floor area. The building dimensions are 45 m in the longitudinal direction and 45 m in the transverse direction. After establishing the origin for the mass center determination, the necessary calculations were carried out as follows:

$$x = 22.31 \text{ m (due to symmetrical nature of the building)}$$

$$y = \frac{A_{\text{floor}} y_{\text{floor}} - A_{\text{elevator shaft}} y_{\text{elevator shaft}} - A_{\text{stairs shaft, 1}} y_{\text{stairs shaft, 1}} - A_{\text{stairs shaft, 2}} y_{\text{stairs shaft, 2}}}{A_{\text{floor}} - A_{\text{shafts}}}$$

$$y = 22.34 \text{ m}$$

So, mass center:  $x = 22.31 \text{ m}$ ,  $y = 22.34 \text{ m}$ .

Afterwards, the stiffness center was determined by calculating the stiffness of the moment frame. Firstly, the cracking moment of inertia of beams and columns were determined using the following equations:

$$I_{b,cr} = \frac{1}{12}bh^3 \cdot 0.35$$

$$I_{c,cr} = \frac{1}{12}bh^3 \cdot 0.7$$

Next, the stiffness of frame can be calculated using the following formula:

$$D = \frac{12E}{h^2} \left( \frac{I_{c,cr} \cdot I_{b,cr}}{L \cdot I_{c,cr} + h \cdot I_{b,cr}} \right)$$

Force needed to generate an inter-story rotation:

$$C_F = h \cdot D \cdot \# \text{ of columns per frame} \cdot \# \text{ of frames}$$

Consequently, the stiffness center can be calculated.

Accordingly, stiffness center:  $x' = 26.16 \text{ m}$ ,  $y' = 23.15 \text{ m}$  (uniform spacing between columns and major beams)

<b>Mass center</b>	<b>m</b>
x	22.31
y	22.34
<b>Stiffness center</b>	
x'	26.15719654
y'	23.15250431
<b>Eccentricity</b>	
ex'	-3.85
ey'	-0.81
e_ax	2.25
e_ay	2.25
ex	-1.60
ey	1.44

**Table 3.1.21.** Eccentricity

Accordingly, torsion moment, direct force, and torsional force will be calculated as:

$$T = eF$$

$$F_{direct} = \frac{F_x}{\# \text{ of frames}}$$

$$F_{torsion} = \frac{T * C_f * (x_i \text{ or } y_i)}{\sum C_f * (x_i^2 \text{ or } y_i^2)}$$

$$F_{total} = F_{direct} \pm F_{torsion}$$

Floors	Transverse				Longitudinal			
	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
-1	A	-0.0597	0.82	0.76	1	0.0662	0.82	0.89
	B	-0.0398	0.82	0.78	2	0.0441	0.82	0.86
	C	-0.0199	0.82	0.80	3	0.0221	0.82	0.84
	D	0.0000	0.82	0.82	4	0.0000	0.82	0.82
	E	0.0199	0.82	0.84	5	-0.0221	0.82	0.80
	F	0.0398	0.82	0.86	6	-0.0441	0.82	0.77
	G	0.0597	0.82	0.88	7	-0.0662	0.82	0.75
	Transverse				Longitudinal			
1	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-0.5910	8.1080	7.5170	2	0.6558	8.1080	8.7637
	B	-0.3940	8.1080	7.7140	2	0.4372	8.1080	8.5452
	C	-0.1970	8.1080	7.9110	3	0.2186	8.1080	8.3266
	D	0.0000	8.1080	8.1080	4	0.0000	8.1080	8.1080
	E	0.1970	8.1080	8.3050	5	-0.2186	8.1080	7.8894
	F	0.3940	8.1080	8.5020	6	-0.4372	8.1080	7.6708
	G	0.5910	8.1080	8.6990	7	-0.6558	8.1080	7.4522
	Transverse				Longitudinal			
2	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN

	A	-1.0900	14.9568	13.8668	2	1.2095	14.9568	16.1663
	B	-0.7267	14.9568	14.2302	2	0.8063	14.9568	15.7632
	C	-0.3633	14.9568	14.5935	3	0.4032	14.9568	15.3600
	D	0.0000	14.9568	14.9568	4	0.0000	14.9568	14.9568
	E	0.3633	14.9568	15.3202	5	-0.4032	14.9568	14.5537
	F	0.7267	14.9568	15.6835	6	-0.8063	14.9568	14.1505
	G	1.0900	14.9568	16.0469	7	-1.2095	14.9568	13.7474
	<b>Transverse</b>				<b>Longitudinal</b>			
3	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-2.6352	27.1152	24.4799	2	2.9236	27.1152	30.0387
	B	-1.7568	27.1152	25.3583	2	1.9490	27.1152	29.0642
	C	-0.8784	27.1152	26.2367	3	0.9745	27.1152	28.0897
	D	0.0000	27.1152	27.1152	4	0.0000	27.1152	27.1152
	E	0.8784	27.1152	27.9936	5	-0.9745	27.1152	26.1406
	F	1.7568	27.1152	28.8720	6	-1.9490	27.1152	25.1661
	G	2.6352	27.1152	29.7504	7	-2.9236	27.1152	24.1916
	<b>Transverse</b>				<b>Longitudinal</b>			
4	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-4.1126	42.3163	38.2037	2	4.5626	42.3163	46.8788
	B	-2.7417	42.3163	39.5745	2	3.0417	42.3163	45.3580
	C	-1.3709	42.3163	40.9454	3	1.5209	42.3163	43.8371
	D	0.0000	42.3163	42.3163	4	0.0000	42.3163	42.3163
	E	1.3709	42.3163	43.6871	5	-1.5209	42.3163	40.7954
	F	2.7417	42.3163	45.0580	6	-3.0417	42.3163	39.2746
	G	4.1126	42.3163	46.4288	7	-4.5626	42.3163	37.7537
	<b>Transverse</b>				<b>Longitudinal</b>			
5	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-6.0018	61.7557	55.7539	1	5.0463	70.20	75.25
	B	-4.0012	61.7557	57.7545	2	5.0463	70.20	75.25

	C	-2.0006	61.7557	59.7551	3	2.5231	70.20	72.73
	D	0.0000	61.7557	61.7557	4	0.0000	70.20	70.20
	E	2.0006	61.7557	63.7563	5	-2.5231	70.20	67.68
	F	4.0012	61.7557	65.7569	6	-5.0463	70.20	65.16
	G	6.0018	61.7557	67.7575	7	-7.5694	70.20	62.63
	<b>Transverse</b>				<b>Longitudinal</b>			
6	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-6.8229	70.2041	63.3812	1	7.5694	70.2041	77.7735
	B	-4.5486	70.2041	65.6555	2	5.0463	70.2041	75.2504
	C	-2.2743	70.2041	67.9298	3	2.5231	70.2041	72.7272
	D	0.0000	70.2041	70.2041	4	0.0000	70.2041	70.2041
	E	2.2743	70.2041	72.4784	5	-2.5231	70.2041	67.6810
	F	4.5486	70.2041	74.7527	6	-5.0463	70.2041	65.1578
	G	6.8229	70.2041	77.0270	7	-7.5694	70.2041	62.6347
	<b>Transverse</b>				<b>Longitudinal</b>			
7	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-8.4636	87.0866	78.6229	1	9.3897	87.0866	96.4763
	B	-5.6424	87.0866	81.4441	2	6.2598	87.0866	93.3464
	C	-2.8212	87.0866	84.2653	3	3.1299	87.0866	90.2165
	D	0.0000	87.0866	87.0866	4	0.0000	87.0866	87.0866
	E	2.8212	87.0866	89.9078	5	-3.1299	87.0866	83.9566
	F	5.6424	87.0866	92.7290	6	-6.2598	87.0866	80.8267
	G	8.4636	87.0866	95.5502	7	-9.3897	87.0866	77.6968
	<b>Transverse</b>				<b>Longitudinal</b>			
8	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-10.1393	104.3278	94.1885	1	11.2487	104.3278	115.5764
	B	-6.7595	104.3278	97.5683	2	7.4991	104.3278	111.8269
	C	-3.3798	104.3278	100.9480	3	3.7496	104.3278	108.0773
	D	0.0000	104.3278	104.3278	4	0.0000	104.3278	104.3278

	E	3.3798	104.3278	107.7075	5	-3.7496	104.3278	100.5782
	F	6.7595	104.3278	111.0873	6	-7.4991	104.3278	96.8287
	G	10.1393	104.3278	114.4670	7	-11.2487	104.3278	93.0791
	<b>Transverse</b>				<b>Longitudinal</b>			
9	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-12.0881	124.3798	112.2917	1	13.4107	124.3798	137.7905
	B	-8.0587	124.3798	116.3211	2	8.9405	124.3798	133.3202
	C	-4.0294	124.3798	120.3504	3	4.4702	124.3798	128.8500
	D	0.0000	124.3798	124.3798	4	0.0000	124.3798	124.3798
	E	4.0294	124.3798	128.4091	5	-4.4702	124.3798	119.9096
	F	8.0587	124.3798	132.4385	6	-8.9405	124.3798	115.4393
	G	12.0881	124.3798	136.4678	7	-13.4107	124.3798	110.9691
	<b>Transverse</b>				<b>Longitudinal</b>			
10	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-14.0291	144.3526	130.3235	1	15.5642	144.3526	159.9168
	B	-9.3528	144.3526	134.9999	2	9.8862	144.3526	154.2388
	C	-4.6764	144.3526	139.6762	3	4.9431	144.3526	149.2957
	D	0.0000	144.3526	144.3526	4	0.0000	144.3526	144.3526
	E	4.6764	144.3526	149.0290	5	-4.9431	144.3526	139.4095
	F	9.3528	144.3526	153.7054	6	-9.8862	144.3526	134.4665
	G	14.0291	144.3526	158.3818	7	-14.8292	144.3526	129.5234
	<b>Transverse</b>				<b>Longitudinal</b>			
11	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-16.0951	165.6103	149.5152	1	17.8562	165.6103	183.4665
	B	-10.7301	165.6103	154.8802	2	11.9041	165.6103	177.5144
	C	-5.3650	165.6103	160.2453	3	5.9521	165.6103	171.5623
	D	0.0000	165.6103	165.6103	4	0.0000	165.6103	165.6103
	E	5.3650	165.6103	170.9753	5	-5.9521	165.6103	159.6582

	F	10.7301	165.6103	176.3404	6	-11.9041	165.6103	153.7062
	G	16.0951	165.6103	181.7054	7	-17.8562	165.6103	147.7541
	<b>Transverse</b>				<b>Longitudinal</b>			
12	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-18.4722	190.0697	171.5974	1	20.4934	190.0697	210.5631
	B	-12.3148	190.0697	177.7549	2	13.6623	190.0697	203.7319
	C	-6.1574	190.0697	183.9123	3	6.8311	190.0697	196.9008
	D	0.0000	190.0697	190.0697	4	0.0000	190.0697	190.0697
	E	6.1574	190.0697	196.2271	5	-6.8311	190.0697	183.2385
	F	12.3148	190.0697	202.3845	6	-13.6623	190.0697	176.4074
	G	18.4722	190.0697	208.5419	7	-20.4934	190.0697	169.5763
	<b>Transverse</b>				<b>Longitudinal</b>			
13	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-20.7943	213.9625	193.1682	2	23.0695	213.9625	237.0321
	B	-13.8629	213.9625	200.0997	2	15.3797	213.9625	229.3422
	C	-6.9314	213.9625	207.0311	3	7.6898	213.9625	221.6524
	D	0.0000	213.9625	213.9625	4	0.0000	213.9625	213.9625
	E	6.9314	213.9625	220.8940	5	-7.6898	213.9625	206.2727
	F	13.8629	213.9625	227.8254	6	-15.3797	213.9625	198.5828
	G	20.7943	213.9625	234.7568	7	-23.0695	213.9625	190.8930
	<b>Transverse</b>				<b>Longitudinal</b>			
14	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-23.2436	239.1650	215.9214	1	25.7869	239.1650	264.9519
	B	-15.4958	239.1650	223.6693	2	17.1913	239.1650	256.3563
	C	-7.7479	239.1650	231.4171	3	8.5956	239.1650	247.7606
	D	0.0000	239.1650	239.1650	4	0.0000	239.1650	239.1650
	E	7.7479	239.1650	246.9129	5	-8.5956	239.1650	230.5694
	F	15.4958	239.1650	254.6608	6	-17.1913	239.1650	221.9738

	G	23.2436	239.1650	262.4087	7	-25.7869	239.1650	213.3781
	<b>Transverse</b>				<b>Longitudinal</b>			
15	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-25.8243	265.7190	239.8946	1	28.6500	265.7190	294.3689
	B	-17.2162	265.7190	248.5027	2	19.1000	265.7190	284.8189
	C	-8.6081	265.7190	257.1108	3	9.5500	265.7190	275.2689
	D	0.0000	265.7190	265.7190	4	0.0000	265.7190	265.7190
	E	8.6081	265.7190	274.3271	5	-9.5500	265.7190	256.1690
	F	17.2162	265.7190	282.9352	6	-19.1000	265.7190	246.6190
	G	25.8243	265.7190	291.5433	7	-28.6500	265.7190	237.0690
	<b>Transverse</b>				<b>Longitudinal</b>			
16	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	-29.2098	300.5535	271.3437	2	32.4058	300.5535	332.9593
	B	-19.4732	300.5535	281.0803	2	21.6039	300.5535	322.1574
	C	-9.7366	300.5535	290.8169	3	10.8019	300.5535	311.3554
	D	0.0000	300.5535	300.5535	4	0.0000	300.5535	300.5535
	E	9.7366	300.5535	310.2901	5	-10.8019	300.5535	289.7515
	F	19.4732	300.5535	320.0267	6	-21.6039	300.5535	278.9496
	G	29.2098	300.5535	329.7633	7	-32.4058	300.5535	268.1477
	<b>Transverse</b>				<b>Longitudinal</b>			
17	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
					6	-24.9537	181.8434	156.8898
	G	33.7389	181.84	215.58	7	-37.4305	181.8434	144.4129

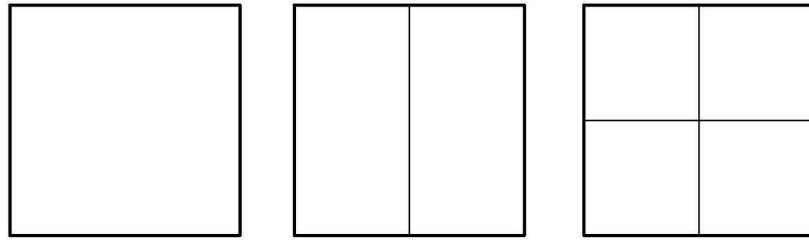
**Table 3.1.22.** Torsional Effect

### 3.2. Estimation of Member Sizes

#### 3.2.1. Determination of Structural Layout

The spacing between columns was taken as 7.4 m to provide bigger space for the building occupancies.

Three potential GLRS layouts were evaluated, and the most suitable layout was selected based on material costs, determined by calculating the volumes of structural components. The first layout includes major beams and a two-way slab, while the second features a one-way slab supported by major beams and one minor beam. The third option consists of a two-way slab with major beams and two minor beams, as illustrated in Figure 3.1.



**Figure 3.2.1** Potential GLRS layouts

The preliminary member size estimation was done for the further analysis based on the followings statements from ACI 318-19 Chapters 7-10 and heuristic approach (American Concrete Institute, 2022):

- The thickness of major beams under a two-way slab without any interior beams is equal to 8% of its span.
- The width of major beams under a two-way slab without any interior beams is equal  $b = \frac{h}{2}$ .
- The thickness of simply supported minor beams is equal to  $h = \frac{l}{16}$ .
- The thickness of simply supported one-way slab is equal to  $h = \frac{l}{20}$ .

#### Option 1: Two-way slab

For the conventional design, the lengths of both the slab and beam were taken as 7.4 m. As previously mentioned, the thickness of the major beam (h) was set at 8% of its length, while its width (b) was defined as  $b=h/2$ .

$$h_{major} = 8\%L = 0.08 * 7.4 = 0.592 \text{ m} = 0.6 \text{ m}$$

$$b_{major} = \frac{h}{2} = 0.3 \text{ m}$$

To determine the slab thickness, it was necessary to calculate the moment of inertia and the elastic modulus of both the beam and slab. C40 grade concrete was selected for both components, as it is commonly used for structural members designed to bear heavier loads (Base Concrete, 2023). The calculations performed are as follows:

$$f_{cb} = f_{cs} = 40 \text{ MPa}$$

The modulus of elasticity can be calculated using the following equation:

$$E_b = E_s = 4700\sqrt{f_c} \quad (3.3)$$

$$E_b = E_s = 4700\sqrt{f_c} = 29725.41 \text{ MPa}$$

$$I_{b,cr} = \frac{1}{12} \cdot b \cdot h^3 \cdot 0.35$$

$$I_{s,cr} = \frac{1}{12} \cdot L \cdot h^3 \cdot 0.25$$

After calculating the members' moments of inertia the stiffness ratio should be calculated:

$$\alpha_{fm} = \frac{E_{cb} I_{b,cr}}{E_{cs} I_{s,cr}} \quad (3.4)$$

### Option 2: Two-way slab with minor beams

The similar procedure was repeated for this option. Thickness of major beam was assumed to be 8% of its length, whereas thickness of simply-supported minor beam is equal to  $\frac{l}{16}$ . Thickness of slab was assumed to be 0.2 m.

$$h_{major} = 8\% \cdot L = 0.08 \cdot 7.4 = 0.6 \text{ m}$$

$$b_{major} = 0.6/2 = 0.3 \text{ m}$$

$$h_{minor} = L/16 = 7.4/16 = 0.4625 \text{ m}$$

$$b_{major} = 0.4625/2 = 0.23125 \text{ m}$$

Then, the moment of inertia and stiffness ratio was calculated.

$$\alpha_{fm} = \frac{\alpha_{f,major} + \alpha_{f,minor}}{2}$$

### Option 3: One-way slab with minor beam

For this last case, the dimensions of major and minor beams were found the same way as in the previous option.

$$h_{major} = 0.6 \text{ m}; b_{major} = 0.3 \text{ m}$$

$$h_{minor} = 0.463 \text{ m}; b_{major} = 0.231$$

The thickness of simply supported one-way slab is equal to  $\frac{l}{20}$ .

$$h_{slab} = \frac{L}{20} = \frac{3.7}{20} = 0.185 \text{ m}$$

Beams and Slab															
Two way															
	L (m)	h (m)	b (m)	I (m)	fc (MPa)	Ec (MPa)	alpha f1	alpha f2	alpha fm	beta	fy (MPa)	ln (m)	h (m)	V (m <sup>3</sup> )	
Slab	7.400	0.174	7.400	0.000301	40	29725.410	6.277	6.277	6.277	1.000	420	7.100	0.090	9.504	
Beam	7.400	0.600	0.300	0.001890	40	29725.410								3.787	
<b>Total:</b>														<b>13.291</b>	
Two way with minors															
	L (m)	h (m)	b (m)	I (m)	fc (MPa)	Ec (MPa)	alpha f1	alpha f2	alpha fm	beta	fy (MPa)	ln (m)	h (m)	V (m <sup>3</sup> )	n
Slab	3.700	0.168	3.700	0.000151	40	29725.410	12.554	4.432	8.493	1.000	420	3.434	0.090	2.300	4
Major Beam	7.400	0.600	0.300	0.001890	40	29725.410								0.959	4
Minor Beam	7.400	0.463	0.231	0.000667	40	29725.410								0.276	2
<b>Total:</b>														<b>13.587</b>	
One way with minors															
	L (m)	h (m)	b (m)	I (m)	fc (MPa)	Ec (MPa)	V (m <sup>3</sup> )								
Slab	7.400	0.185	3.700	0.000301	40	29725.410	10.131								
Major Beam	7.400	0.600	0.300	0.001890	40	29725.410	5.328								
Minor Beam	7.400	0.463	0.231	0.000667	40	29725.410	0.791								
<b>Total:</b>							<b>16.250</b>								

**Table 3.2.1** GLRS Layout calculation

Based on the calculations of dimensions and total volumes of structural members, the layout that results in the lowest volume, and accordingly, the lowest cost was chosen as the layout system for the whole building.

The dimensions of slab, major and minor beams for all three cases were summarized in Table 3.10. In summary, a two-way slab was chosen for design.

### 3.2.2. Estimation of Size Dimensions of Columns

Load combination was applied to estimate the dimensions of columns:

$$w_u = 1.2D + 1.6L + 0.5L_r \quad (3.6)$$

$$P_u = w_u \cdot A_T + n_{\text{columns above}}$$

$$\phi P_n = 0.8\phi(0.85f'_c A_c + f_y A_s) \quad (3.7)$$

Reinforcement ratio was assumed as 1%, hence,  $A_s = 0.01A_c$ ,  $\phi = 0.65$  (based on ACI 318-19),

$$f'_c = 40 \text{ MPa}, f_y = 420 \text{ MPa}.$$

In sum, it is needed to obtain a strength value larger than the demand value. Detailed Calculation is available on the next tables:

Dead Load, kPa	Live Load, kPa	Roof Live Load, kPa	Tributary Area of Column, m <sup>2</sup>	Total Load, kPa	Factored Load, kN/m <sup>2</sup>	Area of Slab, m <sup>2</sup>	Axial Force Interior, kN	Axial Force Exterior, kN	Axial Force Corner, kN
9.924	3.83	0	54.76	13.754	18.03642305	2025	987.675	493.837	246.919
8.205	3.83	0		12.035	15.97365406		874.717	437.359	218.679
8.782	3.83	0		12.612	16.66662117		912.664	456.332	228.166
8.882	3.83	0		12.712	16.78622532		919.214	459.607	229.803
9.306	3.83	0		13.136	17.29461051		947.053	473.526	236.763
8.559	1.07	0		9.629	11.9827954		656.178	328.089	164.044
8.558	4.79	0		13.348	17.93378562		982.054	491.027	245.514
6.341	0	0.58		6.921	7.89972928		1700	432.589	216.295
6.326	0	0.58	11.55	6.906	7.8812	23.1	91.028		
6.697	3.83	0	54.76	10.527	14.16449007	2025	775.647	387.824	193.912

**Table 3.2.2** Axial Force per column calculation

Level	Axial Load Pu (kN)	Axial Force Exterior, kN	Axial Force Corner, kN
18	0	0	0
17	91.028	91.028	91.028
16	523.617	307.322	199.175
15	1505.671	798.349	444.689
14	2161.849	1126.438	608.733
13	2818.027	1454.527	772.778
12	3474.205	1782.616	936.822
11	4130.383	2110.705	1100.867
10	4786.561	2438.794	1264.911
9	5442.738	2766.883	1428.955
8	6098.916	3094.972	1593.000
7	6755.094	3423.061	1757.044
6	7411.272	3751.150	1921.089
5	8067.450	4079.239	2085.133
4	9014.503	4552.765	2321.897
3	9933.716	5012.372	2551.700
2	10846.381	5468.704	2779.866
1	11721.098	5906.063	2998.545
-1	12708.772	6399.900	3245.464
Foundati on	13484.420	6787.724	3439.376

**Table 3.2.3** Axial Force on each floor

Level	Iteration	b, m	Column Area, m <sup>2</sup>	Concrete Class, kPa	Reinforcement Grade, kPa	Bar Area, m <sup>2</sup>	Demand Pu, kN	Strength Pn, kN
Slab -1	1	0.7	0.49	40000	420000	0.0049	13484.42	9733.36
	2	0.8	0.64	40000	420000	0.0064		12712.96
	3	0.85	0.7225	40000	420000	0.007225		14351.74
Slab 1,2	1	0.8	0.64	40000	420000	0.0064	12708.77	12712.96
	2	0.75	0.5625	40000	420000	0.005625		11173.5
	3	0.7	0.49	40000	420000	0.0049		9733.36
Slab 3-4	1	0.65	0.4225	40000	420000	0.004225	10846.38	8392.54
	2	0.7	0.49	40000	420000	0.0049		9733.36
	3	0.75	0.5625	40000	420000	0.005625		11173.5
Slab 5	1	0.65	0.4225	40000	420000	0.004225	9014.50	8392.54
	2	0.7	0.49	40000	420000	0.0049		9733.36
	3	0.75	0.5625	40000	420000	0.005625		11173.5
Slab 6-7	1	0.65	0.4225	40000	420000	0.004225	8067.45	8392.54
	2	0.7	0.49	40000	420000	0.0049		9733.36
	3	0.75	0.5625	40000	420000	0.005625		11173.5
Slab 8-9	1	0.65	0.4225	40000	420000	0.004225	6755.09	8392.54
	2	0.6	0.36	40000	420000	0.0036		7151.04
	3	0.55	0.3025	40000	420000	0.003025		6008.86
Slab 10	1	0.6	0.36	40000	420000	0.0036	5442.74	7151.04
	2	0.55	0.3025	40000	420000	0.003025		6008.86
	3	0.5	0.25	40000	420000	0.0025		4966
Slab 11-12	1	0.55	0.3025	40000	420000	0.003025	4786.56	6008.86
	2	0.5	0.25	40000	420000	0.0025		4966
	3	0.45	0.2025	40000	420000	0.002025		4022.46
Slab 13	1	0.5	0.25	40000	420000	0.0025	3474.20	4966
	2	0.45	0.2025	40000	420000	0.002025		4022.46
	3	0.4	0.16	40000	420000	0.0016		3178.24
Slab 14	1	0.45	0.2025	40000	420000	0.002025	2818.03	4022.46
	2	0.4	0.16	40000	420000	0.0016		3178.24
	3	0.35	0.1225	40000	420000	0.001225		2433.34
Slab 15	1	0.4	0.16	40000	420000	0.0016	2161.85	3178.24
	2	0.35	0.1225	40000	420000	0.001225		2433.34
	3	0.3	0.09	40000	420000	0.0009		1787.76
Slab 16	1	0.35	0.1225	40000	420000	0.001225	1505.67	2433.34
	2	0.3	0.09	40000	420000	0.0009		1787.76
	3	0.25	0.0625	40000	420000	0.000625		1241.5

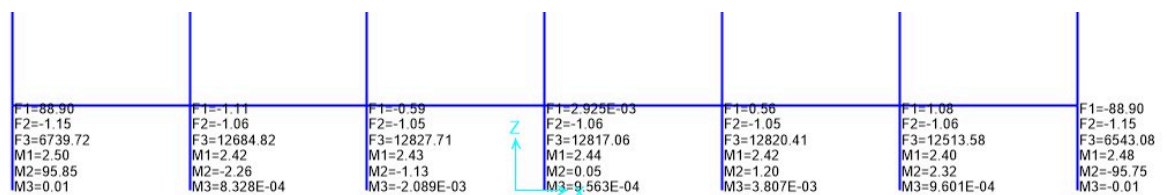
Table 3.2.4 Minimum allowable column sizing

Square Column Size			
Level	Length/ width, m	Demand	Strength
18	0	91.028	1787.76
17	0	523.617	1787.76
16	0.3	1505.671	1787.76
15	0.35	2161.849	2433.34
14	0.4	2818.027	3178.24
13	0.45	3474.205	4022.46
12	0.5	4130.383	4966
11	0.5	4786.561	4966
10	0.55	5442.738	6008.86
9	0.6	6098.916	7151.04
8	0.6	6755.094	7151.04
7	0.65	7411.272	8392.54
6	0.65	8067.450	8392.54
5	0.7	9014.503	9733.36
4	0.75	9933.716	11173.5
3	0.75	10846.381	11173.5
2	0.8	11721.098	12712.96
1	0.8	12708.772	12712.96
-1	0.85	13484.420	14351.74

**Table 3.2.5.** Summarized Loads and Column Size

### Reaction Forces using SAP2000 for the frame

The load combination was also applied to the 3D frame in SAP2000 to determine the reaction forces. The final reaction forces at the bottom of the columns on the basement floor for the selected frame no. 3 are shown in Figure 3.2.2.



**Figure 3.2.2.** The reaction forces generated under the columns for the 3rd frame using 3D frame in SAP2000

The critical reaction forces under the interior and exterior columns are:

$$\text{Interior column: } F_z = 12684.82 \text{ kN}$$

$$\text{Exterior column: } F_z = 6739.72 \text{ kN}$$

### Comparison of Reaction Forces for the 3rd frame from different calculation methods

The comparison of reaction forces for the 4rd frame under exterior and interior columns can be seen in Table 3.2.4.

	$P_u, kN$	$.P_u, kN$
Column	Hand calculations	SAP2000 values
Exterior	13356.99	13280.58
Interior	6724.01	6739.72
Corner	3407.52	3507.78

**Table 3.2.4. Comparison of reaction forces under columns for the 3rd frame.**

Since the method relies on the average value of the inflection points for both fixed and pinned supports, rough manual calculations could not give the correct answer. In contrast, the 3D model automatically calculated the self-weight of the elements by entering their weight directly into SAP2000.

### 3.3. Structural Analysis

#### 3.3.1. Analytical models development in software

This section describes the creation of an analytical model using SAP2000 software. Important structural elements such as section details, boundary conditions, building geometry and material qualities are included in the model. To ensure accurate modeling of the structural behavior, each component has been carefully described. These modeling parameters and how they are used in finite element analysis are explained in detail in the following subsections.

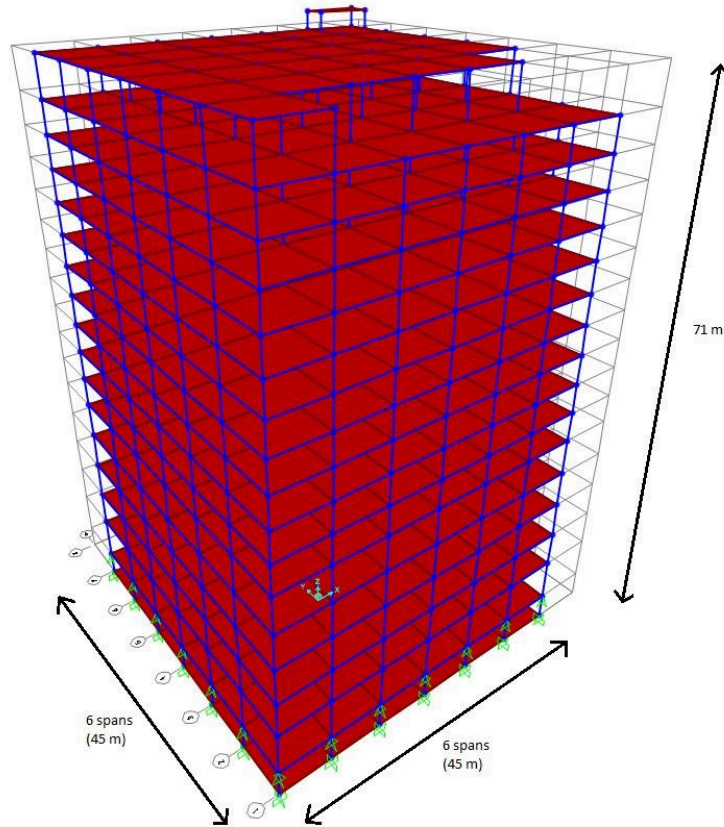
#### Model geometry

Using SAP2000, we developed a comprehensive structural model in which:

Beams and columns were assigned as frame elements

Slabs were modeled as shell elements

The complete architectural geometry is presented in Figure 3.3.1.



**Figure 3.3.1 The building geometry.**

## Materials

The residential building's structural components utilize reinforced concrete as the primary construction material.

*Concrete:*

The material properties of the concrete can be seen in Table 3.3.1.

**Table 3.3.1. Concrete properties.**

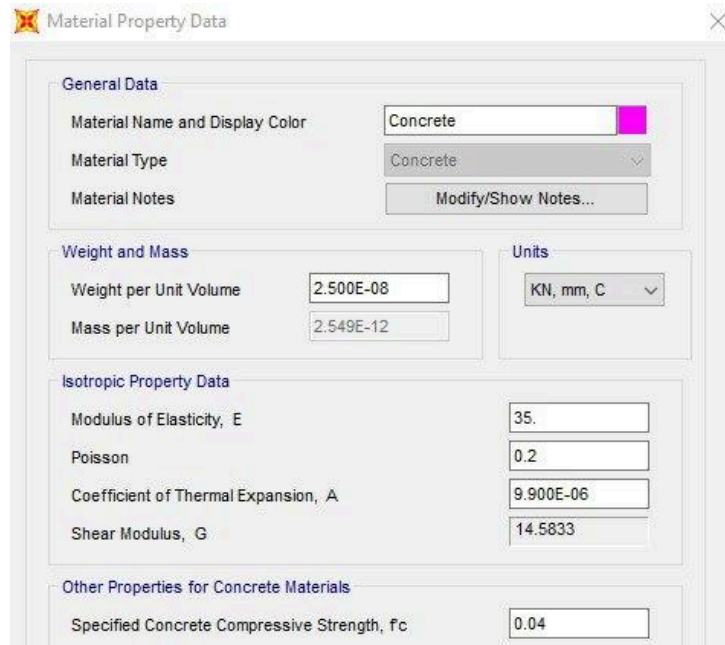
Component	$f'_c$ , MPa	$\rho$ , kN/m <sup>3</sup>	$E_c$ , MPa	$\nu$	$G_c$ , MPa
Beams, columns and slabs	40	23.6	34942	0.2	14583

C40 grade concrete was specified for all load-bearing elements, as this high-strength mix is widely employed in heavy construction projects (Base Concrete, 2023). The modulus of elasticity  $E_c$  was calculated using the following formula:

$$E_c = 4700\sqrt{f'_c} \quad (3.1)$$

For the C40 grade concrete, a Poisson's ratio of 0.2 was adopted, falling within the typical range of 0.15 to 0.25 for concrete materials. The shear modulus  $G_c$  was subsequently derived using the standard mechanical relationship:

$$G_c = \frac{E_c}{2(1+\nu)} \quad (3.2)$$



**Figure 3.3.2.** Material property in SAP2000: concrete.

*Reinforcing steel:*

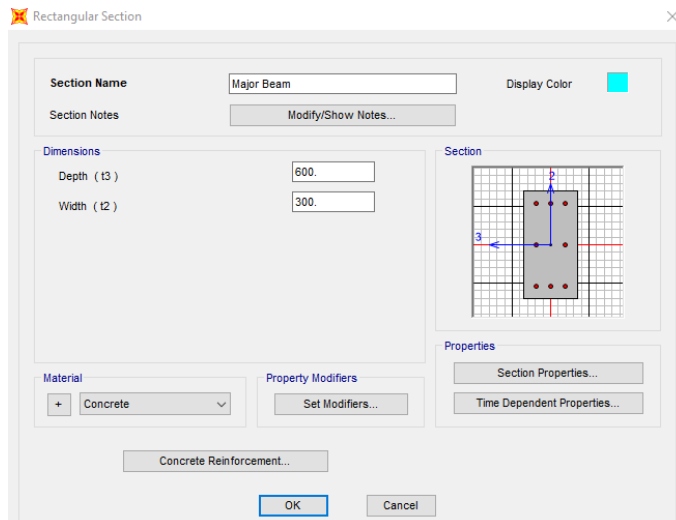
The properties of the reinforcement can be seen in Table 3.3.2.

**Table 3.3.2. Reinforcing steel properties.**

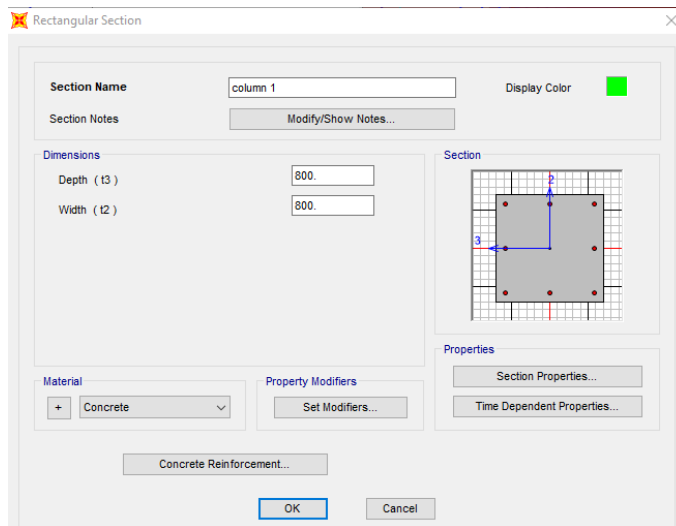
$E$ , MPa	$\rho$ , kN/m <sup>3</sup>	$\nu$	$f_y$ , MPa
200000	76.97	0.3	413.69

**Sections**

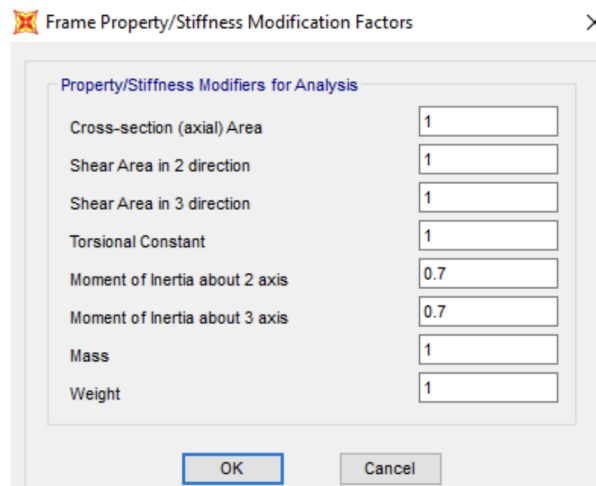
Next, the frame sections were defined. All column sizes for each floor were determined and created. The main beam with its size was also created. The frame section properties and property modifiers, which include the crack moment of inertia factor, can be seen in Figures 3.3.3 to 3.36.



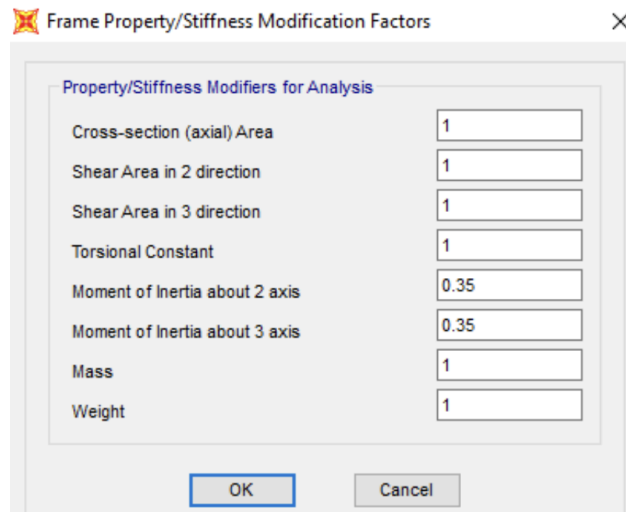
**Figure 3.3.3.** Section properties: major beams.



**Figure 3.3.4.** Section properties: column.



**Figure 3.3.5.** Property modification factors: column.



**Figure 3.3.6.** Property modification factors beams.

The cracking moments of inertia were used for the structural members as can be seen in Table 3.3.3.

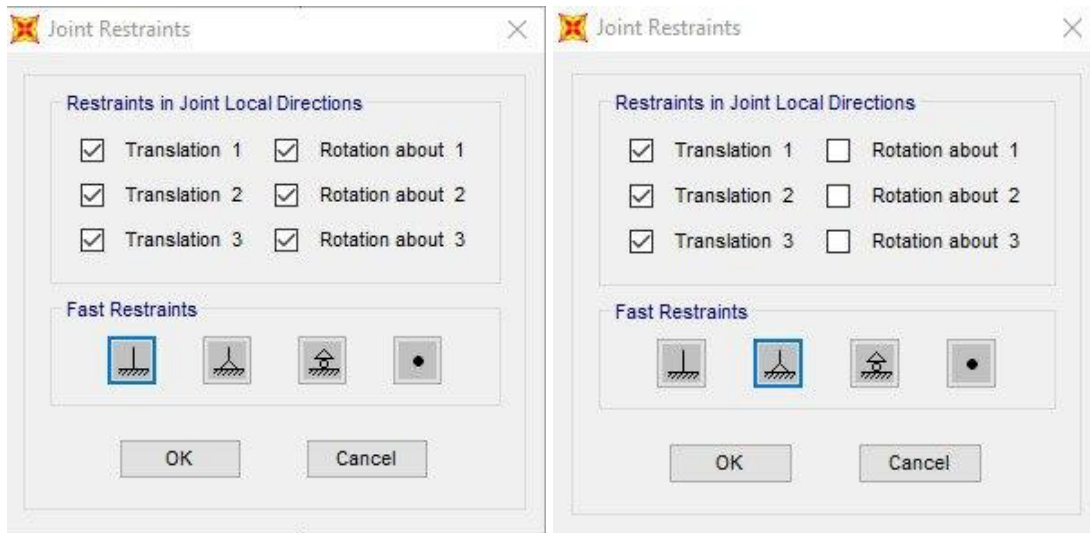
**Table 3.3.3. Cracking moment of inertia factors of structural members.**

<b>Component</b>	<b>Cracking moment of inertia factors</b>
Beams	0.35
Slabs	0.7
Columns	0.25
Stairs	0.25

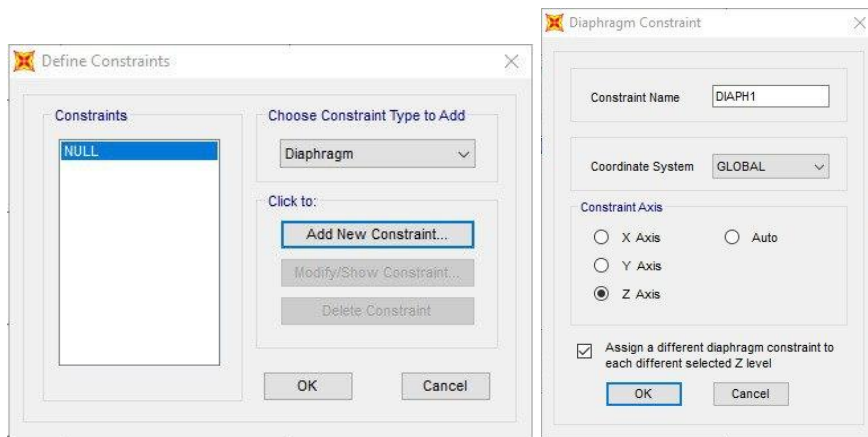
### **Elements and Boundary connections**

In addition, joint restraints and diaphragm restraints were applied. Below the basement, a fixed support was applied to prevent any rotation and ensure the stability of the structure. While at the joints between the underground parking and the basement floors, where contact with the ground occurs, a pinned support was designed to allow rotation to compensate for potential ground settlement.

Next, it is important to apply diaphragm restraints to resist lateral forces such as wind and earthquake. The restraints can be seen in Figures 3.3.7 and 3.3.8.



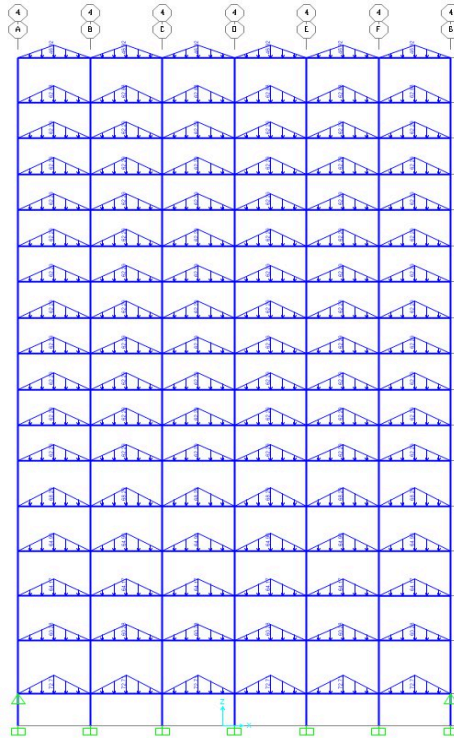
**Figure 3.3.7.** Joint restraints: a) fixed support, and b) pinned support respectively.



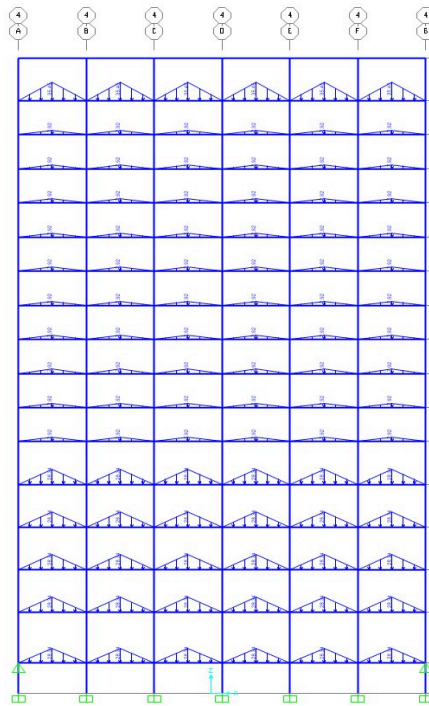
**Figure 3.3.8.** Diaphragm constraints in SAP2000

### Load assignments

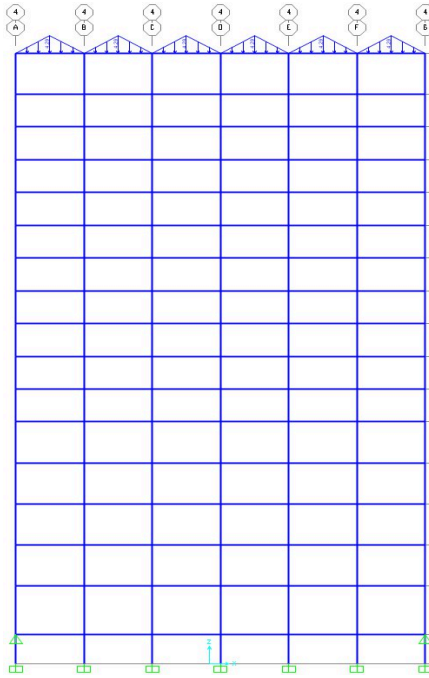
Based on the parameter definitions, a 2D frame-building model was constructed. Dead, Live, and roof live loads were assigned.



**Figure 3.3.9.** Assigned loads on 2D frames in SAP2000: dead load.

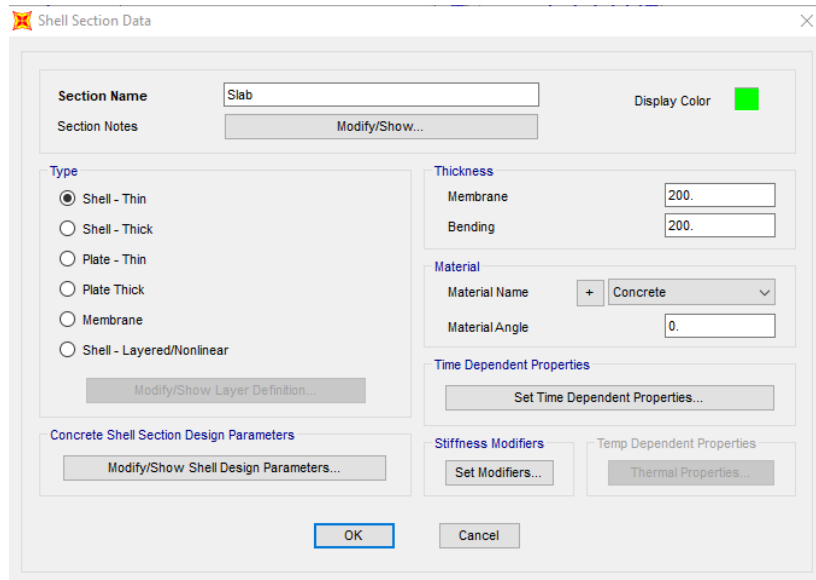


**Figure 3.3.10.** Assigned loads on 2D frames in SAP2000: live load.



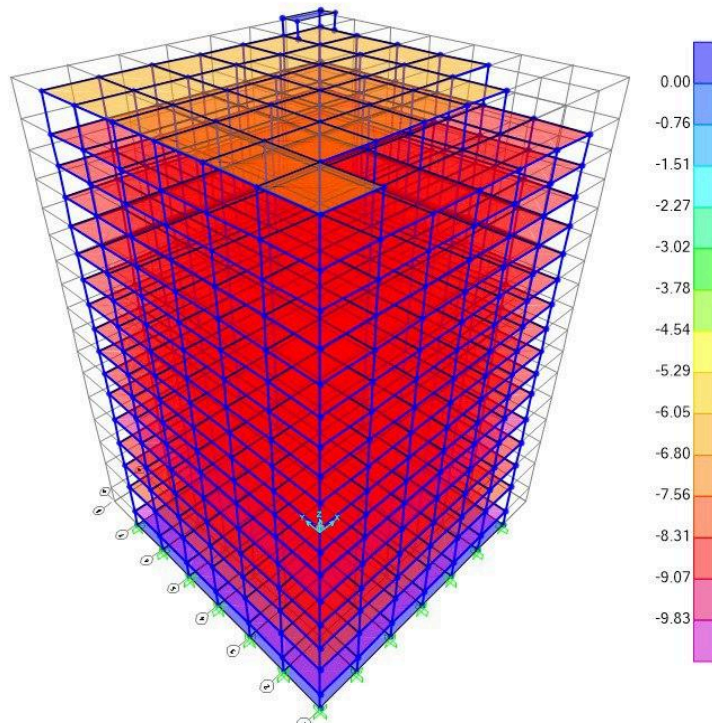
**Figure 3.3.11.** Assigned loads on 2D frames in SAP2000: roof live load respectively.

The building structure was then designed in 3D in SAP2000. The slab properties were also defined as shown in Figure 3.3.12.

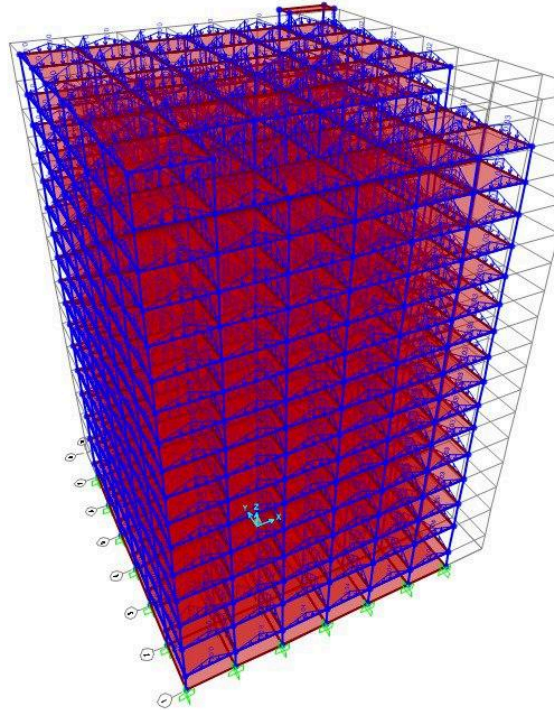


**Figure 3.3.12.** Slab section properties.

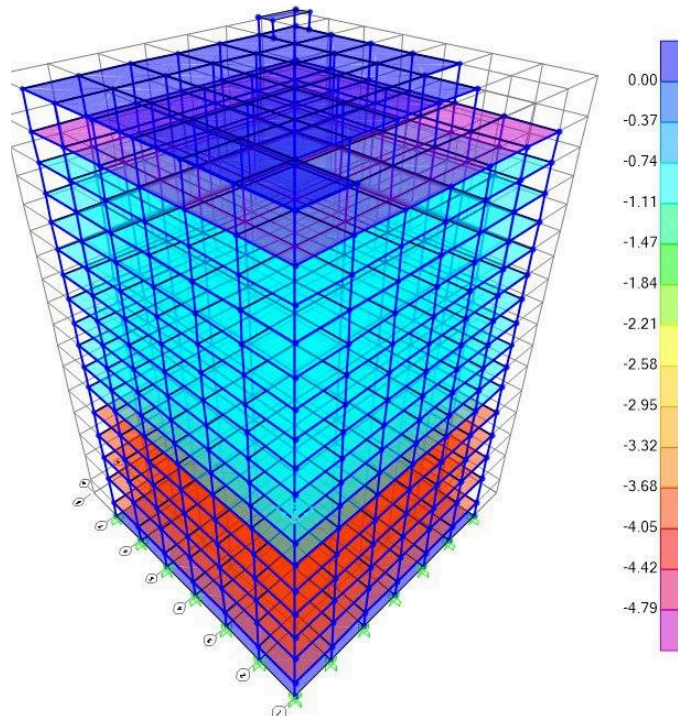
The loads were then allocated to the slab, and because the elevator shafts and stairs do not have slabs, the weights that the elevators and stairs apply were allocated to the nearby beams per the load transfer in the two-way slab. Figures 3.3.13 through 3.3.17 show the applied dead loads, live loads, and live roof loads on the building's 3D model.



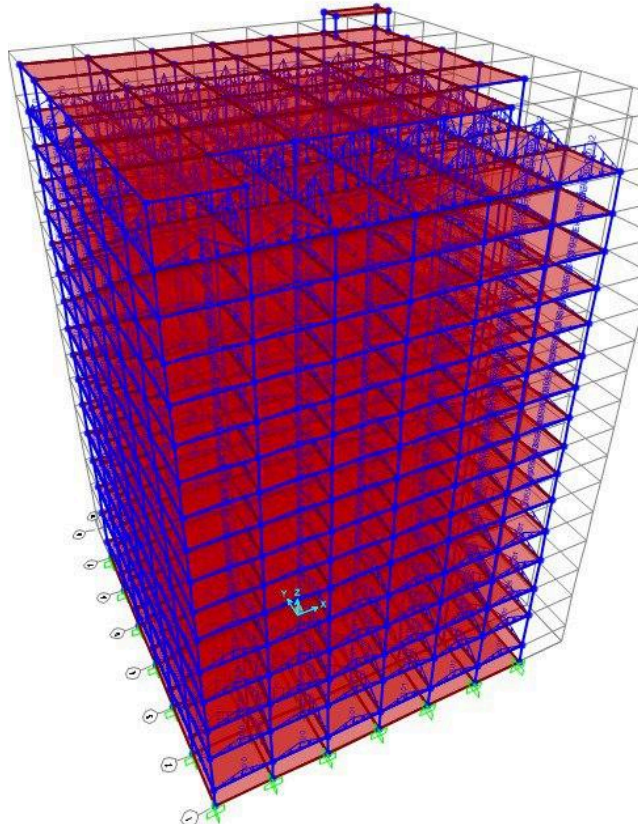
**Figure 3.3.13.** Applied dead area loads on a 3D model in SAP2000



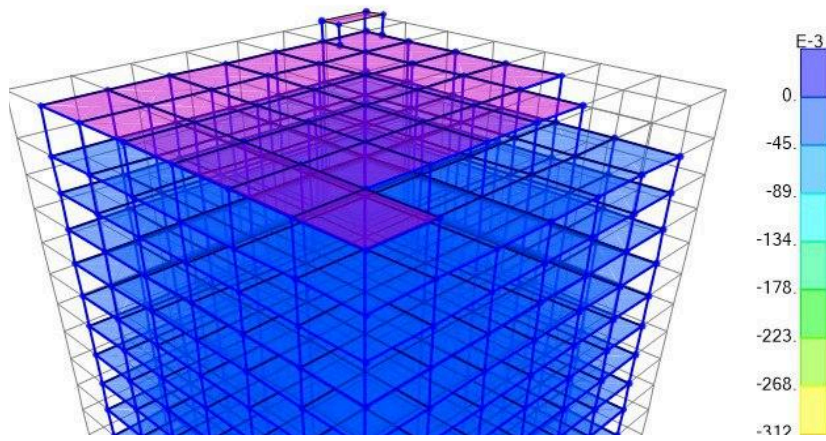
**Figure 3.3.14.** Applied dead frame loads on the 3D model in SAP2000



**Figure 3.3.15.** Applied live area loads on a 3D model in SAP2000



**Figure 3.3.16.** Applied live frame loads on the 3D model in SAP2000



**Figure 3.3.17.** Applied roof live area loads on a 3D model in SAP2000

## Load combinations

The load combinations were chosen according to the Chapter 2 of ASCE 7-16. The load combinations include the dead, live, roof live, seismic and wind loads. The load combinations created in SAP2000 can be seen in Figure 3.3.18. During the analysis the critical load combinations were identified forming the maximum moments and forces:  $1.2D + 1L \pm 1Ey$  and  $0.9D + 1Ey$ .

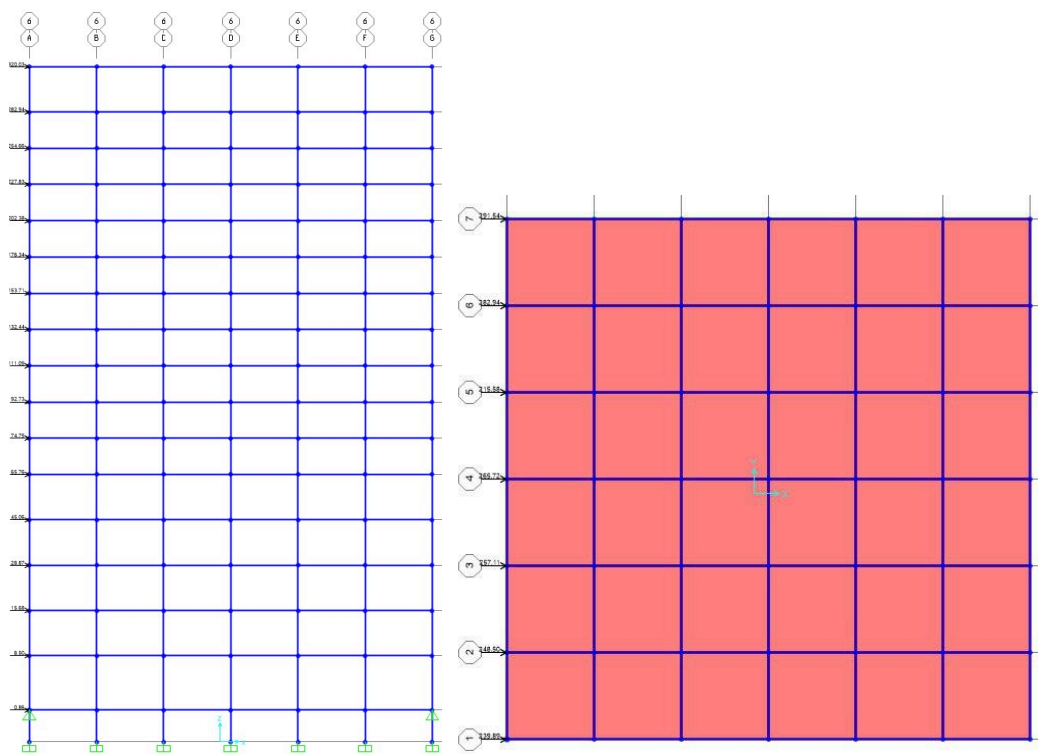
1.4D	1.2D+1L-1W(8)	0.9D+1W(5)	
1.2D+1.6L+0.5Lr	1.2D+1L+1W(9)	0.9D-1W(5)	
1.2D+1L+1W	1.2D+1L-1W(9)	0.9D+1W(6)	
1.2D+1L-1W	1.2D+1L+1W(10)	0.9D-1W(6)	
1.2D+1L+1W(2)	1.2D+1L-1W(10)	0.9D+1W(7)	
1.2D+1L-1W(2)	1.2D+1L+1W(11)	0.9D-1W(7)	
1.2D+1L+1W(3)	1.2D+1L-1W(11)	0.9D+1W(8)	
1.2D+1L-1W(3)	1.2D+1L+1W(12)	0.9D-1W(8)	1.2D+1L-1Ex
1.2D+1L+1W(4)	1.2D+1L-1W(12)	0.9D+1W(9)	1.2D+1L+1Ey
1.2D+1L-1W(4)	0.9D+1W	0.9D-1W(9)	1.2D+1L-1Ey
1.2D+1L+1W(5)	0.9D-1W	0.9D+1W(10)	0.9D+1Ex
1.2D+1L-1W(5)	0.9D+1W(2)	0.9D-1W(10)	0.9D+1Ey
1.2D+1L+1W(6)	0.9D-1W(2)	0.9D+1W(11)	0.9D-1Ey
1.2D+1L-1W(6)	0.9D+1W(3)	0.9D-1W(11)	0.9D-1Ex
1.2D+1L+1W(7)	0.9D-1W(3)	0.9D+1W(12)	1.2D+1L
1.2D+1L-1W(7)	0.9D+1W(4)	0.9D-1W(12)	1.2D
1.2D+1L+1W(8)	0.9D-1W(4)	1.2D+1L+1Ex	1.2D+1.6L

**Figure 3.3.18. Load combinations used in SAP2000.**

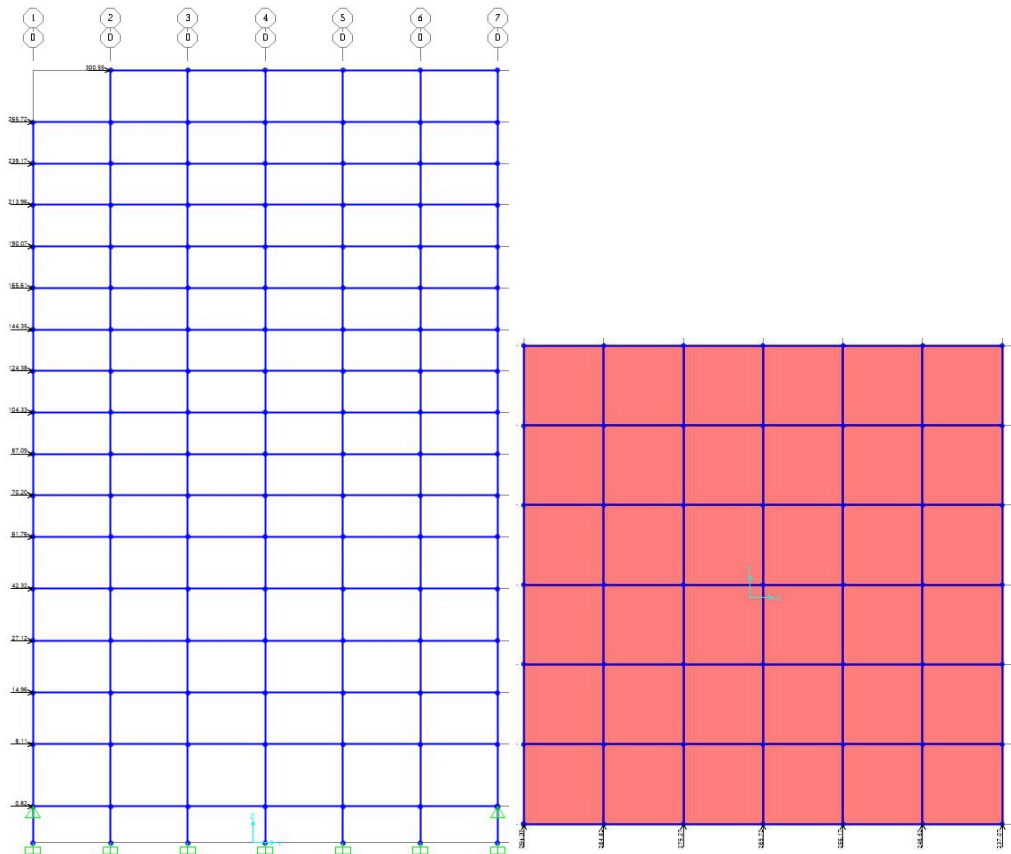
### 3.3.2. Lateral drift analysis and check under wind and seismic loads

#### 2D Frame analysis lateral drift under Wind and Seismic Loads

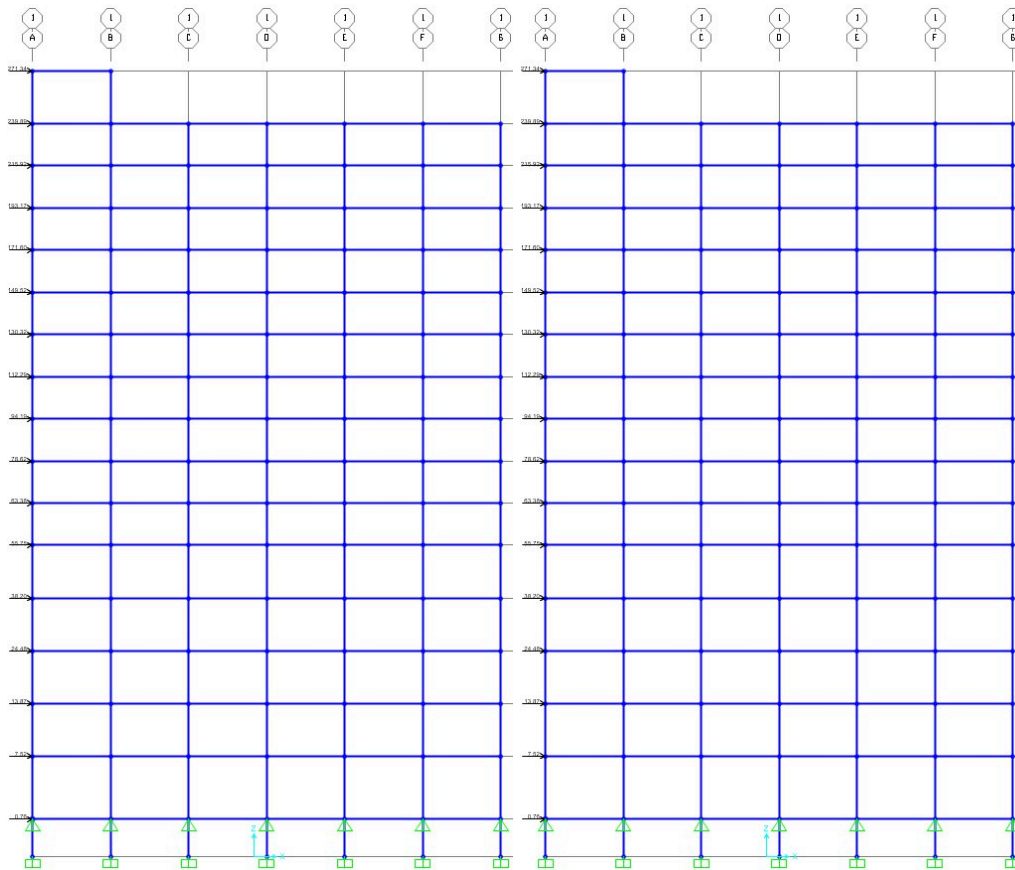
To assign the lateral forces, a procedure similar to that described in Section 3.1.1 was followed by defining the material properties, frame sections, and restraints. The wind load on the selected frame was assigned using  $F_{total}$  for each joint. The load was assigned based on Cases 2 and 4. Similarly, the seismic load was also applied using  $F_{direct}$  calculations for each connection. For the seismic forces, the torsional effect was included. The assigned forces can be seen in Figures 3.3.19 to 3.3.21.



**Figure 3.3.19.** Assigned lateral forces on 2D frame in SAP2000: Seismic loads (Transverse)



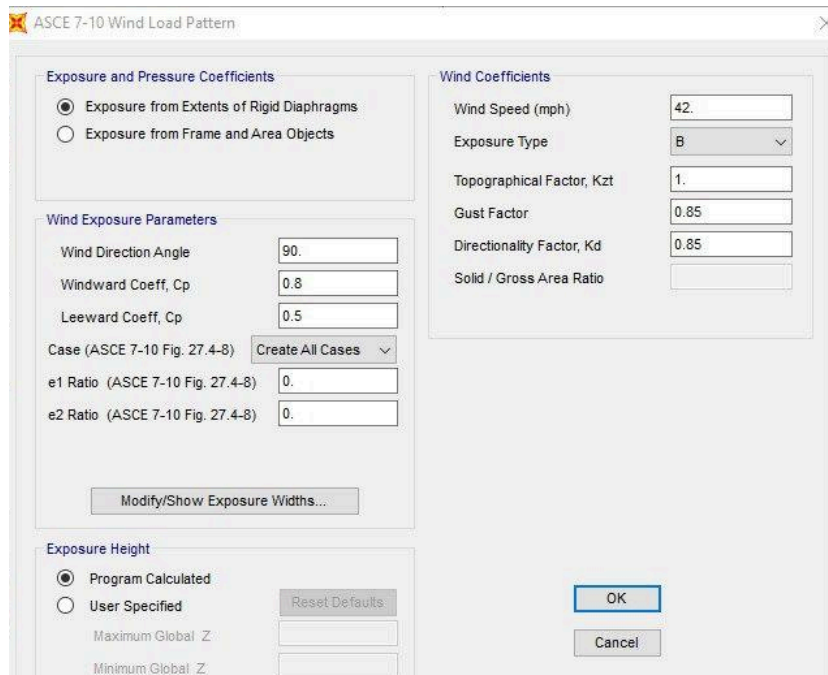
**Figure 3.3.20.** Assigned lateral forces on 2D frame in SAP2000: Seismic loads (Longitudinal)



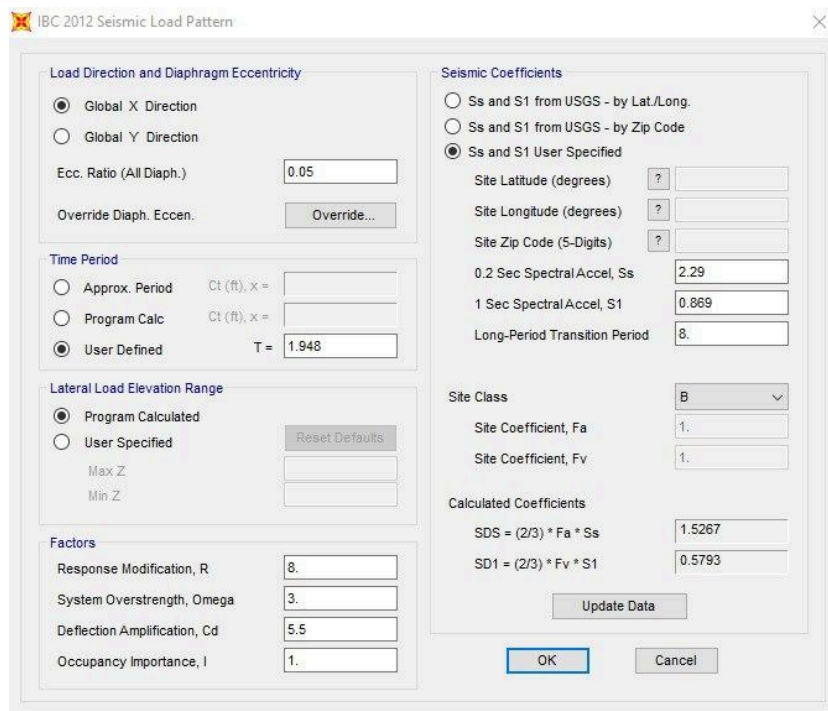
**Figure 3.3.21.** Assigned lateral forces on 2D frame in SAP2000: Wind load in a) Case II and b) Case IV respectively.

### 3D Frame analysis lateral drift under Wind and Seismic Loads

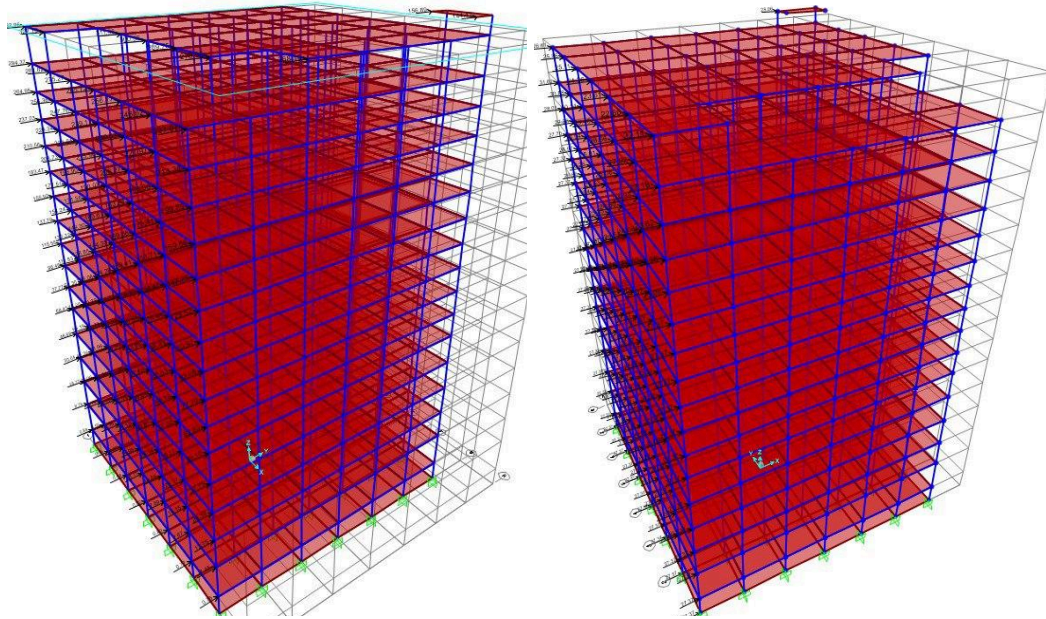
It is not possible to apply lateral loads to the joints to assign them to the 3D frame. Instead, SAP2000 uses internal procedures to perform the calculations, requiring the user to provide important parameters for earthquakes such as acceleration periods, fundamental period, response modification, etc., and wind such as wind speed, action type,  $K_{zt}$ , gust factor, etc. Figures 3.3.22 and 3.3.24 show the parameters assigned to seismic and wind loads, respectively. It should be noted that seismic loads were applied in both the x and y directions, and wind loads were applied to each example.



**Figure 3.3.22.** Wind load pattern specification in SAP2000



**Figure 3.3.23.** Seismic load pattern in SAP2000



**Figure 3.3.24.** Applied lateral forces in 3d view

### Hand calculation for lateral drift analysis under Wind Load

The results of the shear and flexural drift analysis is summarized in the Tables 3.3.3 to 3.3.5.

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm <sup>4</sup> )	Ib,cr (mm <sup>4</sup> )
16	5000	5000	32.48	32.48	5.41	5.41	4.73E+08	1.89E+09
15	4000	4500	39.20	71.68	11.95	8.68	8.75E+08	1.89E+09
14	4000	4000	34.42	106.10	17.68	14.82	1.49E+09	1.89E+09
13	4000	4000	34.03	140.13	23.36	20.52	2.39E+09	1.89E+09
12	4000	4000	33.63	173.76	28.96	26.16	3.65E+09	1.89E+09
11	4000	4000	33.21	206.97	34.50	31.73	3.65E+09	1.89E+09
10	4000	4000	32.77	239.74	39.96	37.23	5.34E+09	1.89E+09
9	4000	4000	32.30	272.04	45.34	42.65	7.56E+09	1.89E+09
8	4000	4000	31.79	303.83	50.64	47.99	7.56E+09	1.89E+09
7	4000	4000	31.25	335.08	55.85	53.24	1.04E+10	1.89E+09
6	4000	4000	30.66	365.74	60.96	58.40	1.04E+10	1.89E+09
5	5000	4500	33.73	399.47	66.58	63.77	1.40E+10	1.89E+09
4	5000	5000	36.51	435.98	72.66	69.62	1.85E+10	1.89E+09
3	5000	5000	35.20	471.18	78.53	75.60	1.85E+10	1.89E+09
2	5000	5000	33.60	504.78	84.13	81.33	2.39E+10	1.89E+09
1	6000	5500	34.48	539.26	89.88	87.00	2.39E+10	1.89E+09
0		3000				44.94		

**Table 3.3.3. Shear Drift under Wind Load**

<b>Ic-avg (mm<sup>4</sup>)</b>	<b>db (mm)</b>	<b>dc (mm)</b>	<b>dt (mm)</b>	<b>Interstory (mm)</b>	<b>Abs (mm)</b>
2.36E+08	0.37	4.01	4.39	6.30	202.16
6.74E+08	1.52	2.31	3.84	4.34	195.85
1.18E+09	2.60	2.24	4.85	5.17	191.51
1.94E+09	3.60	1.90	5.50	5.82	186.34
3.02E+09	4.59	1.55	6.15	6.64	180.52
3.65E+09	5.57	1.56	7.13	7.58	173.88
4.49E+09	6.54	1.49	8.02	8.35	166.30
6.45E+09	7.49	1.19	8.68	9.12	157.95
7.56E+09	8.43	1.14	9.57	9.99	148.82
8.99E+09	9.35	1.06	10.41	10.84	138.83
1.04E+10	10.26	1.01	11.26	15.30	128.00
1.22E+10	17.50	1.83	19.33	19.97	112.70
1.62E+10	19.10	1.50	20.61	21.39	92.73
1.85E+10	20.74	1.44	22.18	22.92	71.34
2.12E+10	22.32	1.35	23.66	30.12	48.42
2.39E+10	34.38	2.20	36.58	18.29	18.29
		0.00	0.00		0.00

**Table 3.3.4. Shear Drift under Wind Load (Continuation)**

Story	M (N-mm)	a=b (m)	A (mm <sup>2</sup> )	f <sub>i</sub>	Dq <sub>i</sub> (rad)	q <sub>i</sub> (rad)	Interstory (mm)	Absolute (mm)
16	1.62E+08	0.3	90000	6.16E-11	3.08E-07	4.09E-05	0.20	2.02
15	4.49E+08	0.35	122500	1.25E-10	5.01E-07	4.06E-05	0.16	1.82
14	8.74E+08	0.4	160000	1.86E-10	7.45E-07	4.01E-05	0.16	1.66
13	1.43E+09	0.45	202500	2.42E-10	9.67E-07	3.93E-05	0.16	1.50
12	2.13E+09	0.5	250000	2.91E-10	1.16E-06	3.84E-05	0.15	1.34
11	2.96E+09	0.5	250000	4.04E-10	1.61E-06	3.72E-05	0.15	1.18
10	3.92E+09	0.55	302500	4.42E-10	1.77E-06	3.56E-05	0.14	1.04
9	5.00E+09	0.6	360000	4.74E-10	1.90E-06	3.38E-05	0.14	0.89
8	6.22E+09	0.6	360000	5.90E-10	2.36E-06	3.19E-05	0.13	0.76
7	7.56E+09	0.65	422500	6.11E-10	2.44E-06	2.96E-05	0.12	0.63
6	9.02E+09	0.65	422500	7.29E-10	2.92E-06	2.71E-05	0.11	0.51
5	1.10E+10	0.7	490000	7.68E-10	3.84E-06	2.42E-05	0.12	0.40
4	1.32E+10	0.75	562500	8.01E-10	4.00E-06	2.04E-05	0.10	0.28
3	1.56E+10	0.75	562500	9.44E-10	4.72E-06	1.64E-05	0.08	0.18
2	1.81E+10	0.8	640000	9.64E-10	4.82E-06	1.16E-05	0.06	0.10
1	2.13E+10	0.8	640000	1.14E-09	6.82E-06	6.82E-06	0.04	0.04
0		0.8	0.64				0.00E+00	

**Table 3.3.5. Flexural Drift under Wind Load**

### **Hand calculations for lateral drift analysis under Seismic Load**

The same procedure was performed for the drift analysis under Seismic Loading. The results of the calculations are shown in Tables 3.3.6-3.3.9.

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm <sup>4</sup> )	Ib,cr (mm <sup>4</sup> )
16	5000	5000	300.08	300.08	50.01	50.01	4.73E+08	1.89E+09
15	4000	4500	265.30	565.38	94.23	72.12	8.75E+08	1.89E+09
14	4000	4000	238.79	804.17	134.03	114.13	1.49E+09	1.89E+09
13	4000	4000	213.63	1017.79	169.63	151.83	2.39E+09	1.89E+09
12	4000	4000	189.77	1207.56	201.26	185.45	3.65E+09	1.89E+09
11	4000	4000	165.35	1372.91	228.82	215.04	3.65E+09	1.89E+09
10	4000	4000	144.13	1517.04	252.84	240.83	5.34E+09	1.89E+09
9	4000	4000	124.18	1641.22	273.54	263.19	7.56E+09	1.89E+09
8	4000	4000	104.16	1745.38	290.90	282.22	7.56E+09	1.89E+09
7	4000	4000	86.95	1832.33	305.39	298.14	1.04E+10	1.89E+09
6	4000	4000	70.09	1902.43	317.07	311.23	1.04E+10	1.89E+09
5	5000	4500	61.66	1964.09	327.35	322.21	1.40E+10	1.89E+09
4	5000	5000	42.25	2006.33	334.39	330.87	1.85E+10	1.89E+09
3	5000	5000	27.07	2033.41	338.90	336.65	1.85E+10	1.89E+09
2	5000	5000	14.93	2048.34	341.39	340.15	2.39E+10	1.89E+09
1	6000	5500	7.93	2056.27	342.71	342.05	2.39E+10	1.89E+09
0		3000				171.36		

**Table 3.3.6. Shear Drift under seismic load**

Ic-avg (mm <sup>4</sup> )	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	Abs (mm)
4.73E+08	2.82	15.23	18.05	31.13	854.84
6.74E+08	10.40	15.77	26.17	28.41	823.71
1.18E+09	16.46	14.20	30.66	32.03	795.29
1.94E+09	21.90	11.51	33.41	34.60	763.26
3.02E+09	26.74	9.05	35.79	37.75	728.66
3.65E+09	31.01	8.69	39.70	41.17	690.91
4.49E+09	34.73	7.90	42.63	43.30	649.75
6.45E+09	37.95	6.01	43.97	45.08	606.45
7.56E+09	40.70	5.50	46.20	47.04	561.37
8.99E+09	43.00	4.89	47.88	48.59	514.32
1.04E+10	44.88	4.40	49.29	64.74	465.74
1.22E+10	72.60	7.59	80.20	80.31	401.00
1.62E+10	74.55	5.87	80.42	80.76	320.69
1.85E+10	75.86	5.25	81.10	81.19	239.93
2.12E+10	76.65	4.62	81.27	99.69	158.74
2.39E+10	110.99	7.12	118.11	59.05	59.05

**Table 3.3.7. Shear Drift under seismic load (Continuation)**

Story	M (N-mm)	a=b (m)	A (mm <sup>2</sup> )	f <sub>i</sub>	D <sub>qi</sub> (rad)
16	1.50E+09	0.3	90000	4.67E-10	2.34E-06
15	3.76E+09	0.35	122500	8.61E-10	3.44E-06
14	6.98E+09	0.4	160000	1.22E-09	4.89E-06
13	1.10E+10	0.45	202500	1.53E-09	6.12E-06
12	1.59E+10	0.5	250000	1.78E-09	7.12E-06
11	2.14E+10	0.5	250000	2.40E-09	9.58E-06
10	2.74E+10	0.55	302500	2.54E-09	1.02E-05
9	3.40E+10	0.6	360000	2.65E-09	1.06E-05
8	4.10E+10	0.6	360000	3.19E-09	1.28E-05
7	4.83E+10	0.65	422500	3.20E-09	1.28E-05
6	5.59E+10	0.65	422500	3.71E-09	1.48E-05
5	6.57E+10	0.7	490000	3.76E-09	1.88E-05
4	7.58E+10	0.75	562500	3.78E-09	1.89E-05
3	8.59E+10	0.75	562500	4.28E-09	2.14E-05
2	9.62E+10	0.8	640000	4.21E-09	2.11E-05
1	1.09E+11	0.8	640000	4.75E-09	2.85E-05
0			0		

**Table 3.3.8. Flexural Drift under seismic load**

qi (rad)	Interstory (mm)	Absolute (mm)
2.03E-04	1.02	9.57
2.01E-04	0.80	8.55
1.98E-04	0.79	7.74
1.93E-04	0.77	6.95
1.87E-04	0.75	6.18
1.79E-04	0.72	5.44
1.70E-04	0.68	4.72
1.60E-04	0.64	4.04
1.49E-04	0.60	3.40
1.36E-04	0.55	2.81
1.24E-04	0.49	2.26
1.09E-04	0.54	1.77
8.99E-05	0.45	1.22
7.10E-05	0.35	0.77
4.96E-05	0.25	0.42
2.85E-05	0.17	0.17
	0.00E+00	

**Table 3.3.9. Flexural Drift under seismic load (Continuation)**

### Comparison of lateral drifts for hand, 2D and 3D SAP calculations

In addition to hand calculations, the SAP2000 2D and 3D models of Frames A and 3 were built to analyze and compare the lateral drifts induced by wind and seismic loads. Furthermore, for the seismic resistance, each story should act as different resisting frames (ASCE, 2016). Therefore, the inter-story seismic deflections were amplified to account for the maximum inelastic displacement ( $\delta_x$ ) by using the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (3.3)$$

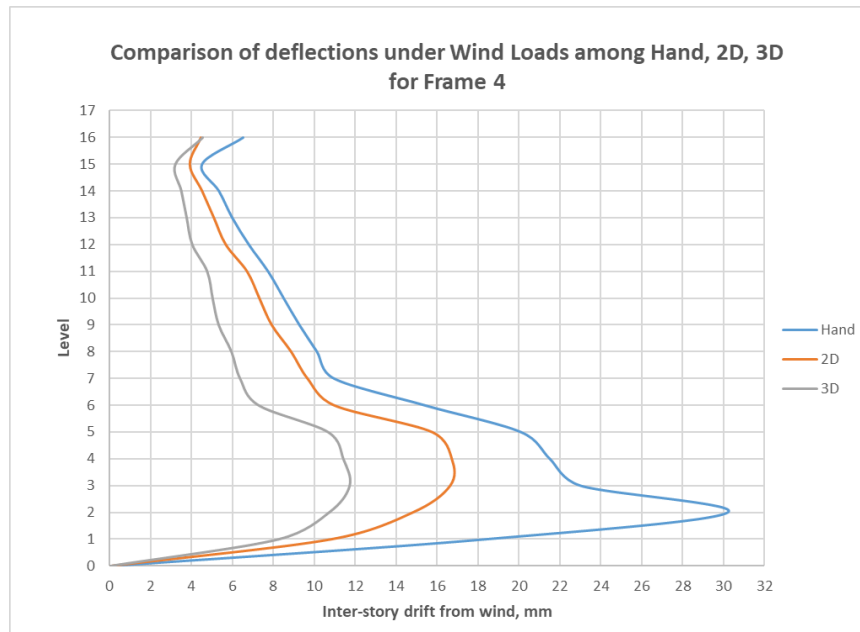
Where  $\delta_{xe}$  is an interstory elastic deflection and  $C_d$  is the deflection amplification factor (e.g.,  $C_d = 5.5$ , as we have special reinforced concrete frames). Subsequently, the allowable story drift limit should also be identified as:

$$\Delta_a = 0.020h_{sx} \quad (3.4)$$

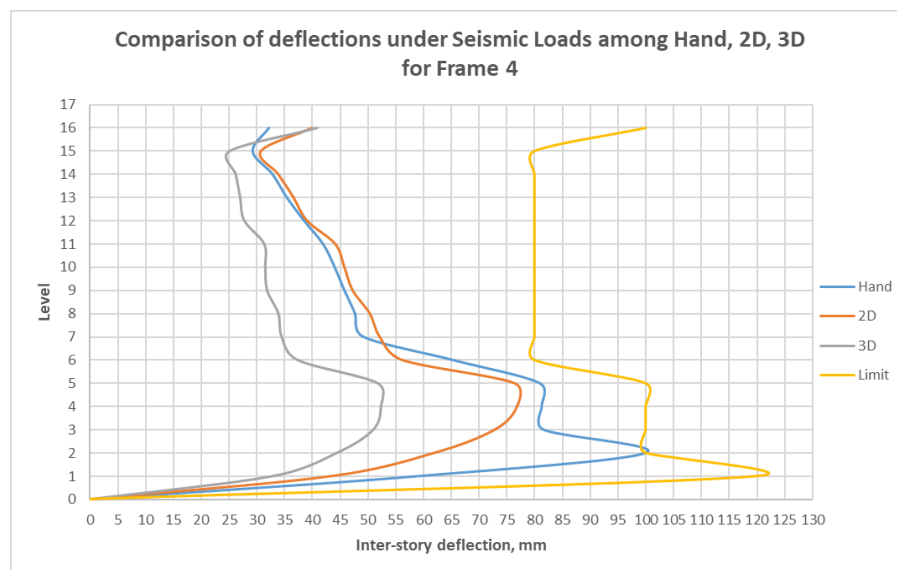
Where  $h_{sx}$  is the story height (e.g., for our building the story height is 6000 mm for floors 1, 5000 for floor 2-5, 16; 4000 mm floors 6-15, and 3500 for floor -1).

For the wind drifts, the inter-story drift was assumed to be  $h/600$ . It should be noted that all the wind inter-story deflections passed the limit check.

The calculated wind and seismic drifts in 3D SAP, 2D SAP and hand calculations for Frames A and 3 can be seen in Tables 9.3-9.8 in Appendix A. The comparisons of the SAP2000 and hand calculations with allowable limits can be seen in Figures 3.3.25 and 3.3.26.



**Figure 3.3.25.** Comparison of Inter-Story drift from wind loads between Hand, 2D and 3D



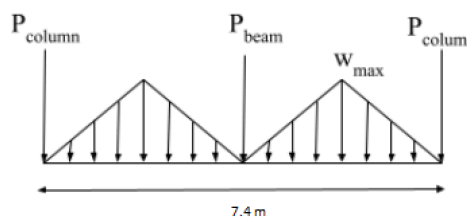
**Figure 3.3.26** Comparison of Inter-Story drift from seismic loads between Hand, 2D and 3D

### 3.3.3. Internal force (M, N, V) analysis under all possible loads for all structural members

#### Internal forces verifications under Dead load

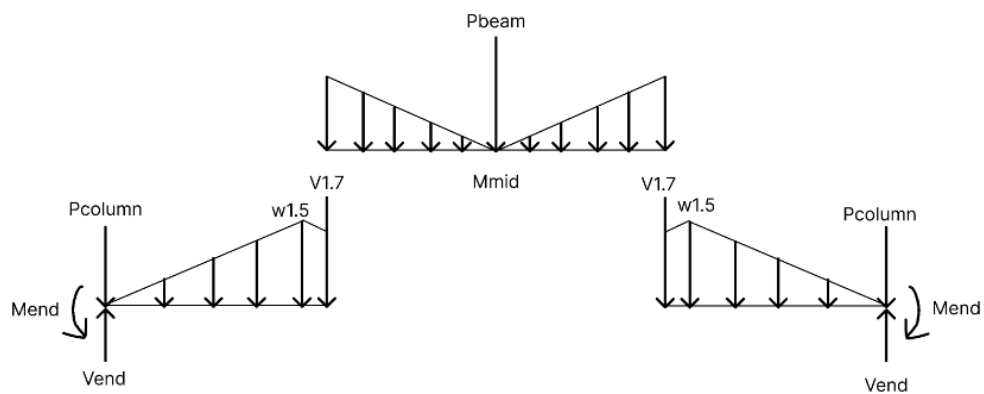
An approximate statically indeterminate analysis method (Hibbeler et al., 2012) was used to determine the internal forces arising from the static load. The inflection point was measured for preliminary calculations at a distance of 0.1 span (0.1L) from each support.

Figure 3.3.27 shows a typical loading configuration for interior beams supporting both floor and roof loads.



**Figure 3.3.27. Load arrangement for internal beams.**

The loading for the approximate analysis is in Figure 3.10.



**Figure 3.3.28. Loading arrangement for approximate analysis method.**

$$L = 7.4 \text{ m,}$$

*Floor dead load*

$$w_{1.5} = 22.03 \text{ kN/m (maximum distributed load)}$$

$$w_{1.7} = 19.1 \text{ kN/m}$$

$$V_{end} = 133.49 \text{ kN}$$

$$M_{mid} = 84.76 \text{ kN} - m$$

$$M_{end} = 121.77 \text{ kN} - m$$

These calculated moments were used to create a bending moment diagram (BMD). Separate moment estimates were needed for the top and bottom of the first level, as it is 6 meters high while the other floors are 4 meters high. The column stiffness is constant between the first and second floors, although the model takes into account differences in bending stiffness. The simplified equations are:

$$M_{top} = \frac{\frac{E_t I_t}{L_t}}{\frac{E_t I_t}{L_t} + \frac{E_b I_b}{L_b}} \cdot M = \frac{\frac{1}{L_t}}{\frac{1}{L_t} + \frac{1}{L_b}} \cdot M \quad (3.6)$$

$$M_{bottom} = \frac{\frac{E_b I_b}{L_b}}{\frac{E_t I_t}{L_t} + \frac{E_b I_b}{L_b}} \cdot M = \frac{\frac{1}{L_b}}{\frac{1}{L_t} + \frac{1}{L_b}} \cdot M \quad (3.7)$$

The calculations for the top and bottom moments:

$$M_{top} = \frac{\frac{1}{5}}{\frac{1}{5} + \frac{1}{4}} \cdot 121.77 = 54.12 \text{ kN} - m$$

$$M_{bottom} = \frac{\frac{1}{6}}{\frac{1}{4} + \frac{1}{6}} \cdot 121.77 = 30.44 \text{ kN} - m$$

*Roof dead load*

The calculations for roof dead load can be found below:

$$V_{1.7} = 48.7 \text{ kN}, V_{end} = 117.28 \text{ kN}$$

$$M_{mid} = 82.39 \text{ kN} - m,$$

The bending moment diagram for the building is shown in Figure 3.3.29.

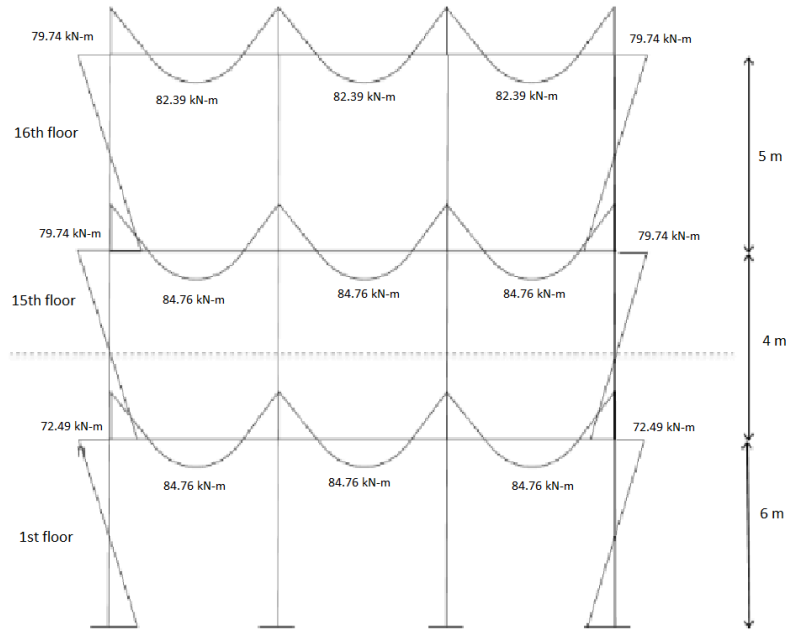


Figure 3.3.29. Moment frame diagram.

To calculate the shear force diagram, the following equation can be used:

$$V = \frac{M_{top} + M_{bottom}}{h_i} \quad (3.8)$$



Figure 3.12. Shear force calculation for one column.

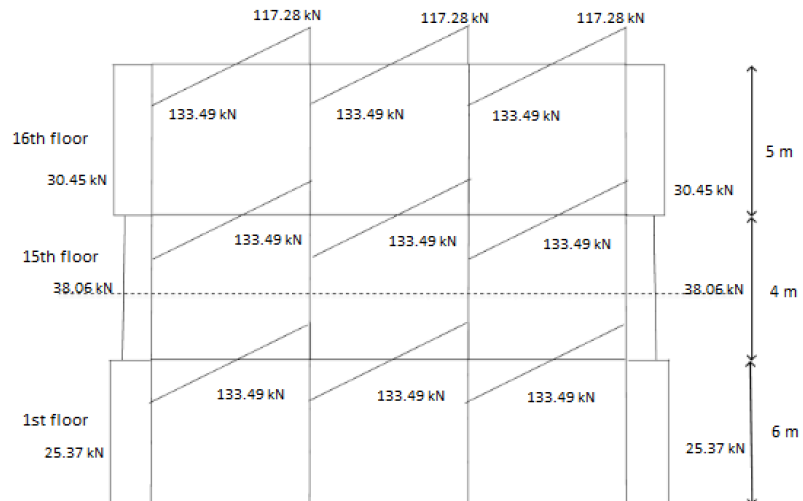
The calculations for columns on different floors were calculated as following:

$$\text{For 16th floor: } V = \frac{72.51 + 79.74}{5} = 30.45 \text{ kN}$$

$$\text{For 15th floor: } V = \frac{79.74 + 72.5}{4} = 38.06 \text{ kN}$$

$$\text{For 1st floor: } V = \frac{121.77 + 30.44}{6} = 25.37 \text{ kN}$$

The shear force diagram for the building is shown in Figure 3.3.30.



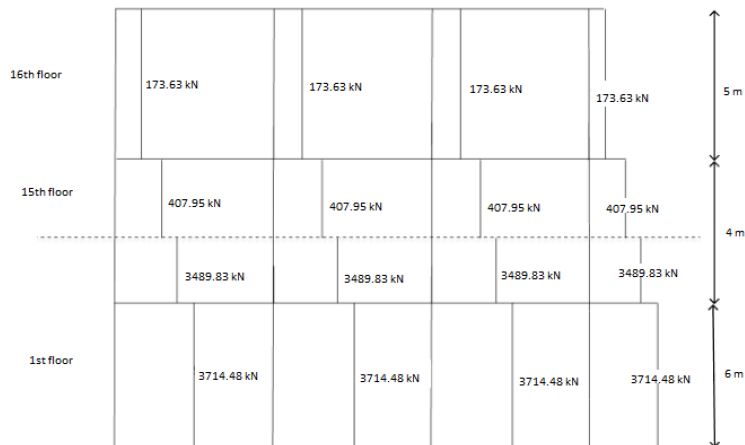
**Figure 3.3.30. Shear force diagram.**

To draw an axial force diagram, the  $V_{end}$  will be used which will increase with each floor:

$$\text{For 20th floor: } V_{end} = 173.63 \text{ kN}$$

$$\text{For 13th floor: } V_{end} = 173.63 + 133.49 + W_{col} = 407.95 \text{ kN}$$

The axial force diagram for the building is shown in Figure 3.3.31.



**Figure 3.3.31. Axial force diagram.**

Floor	Frame 4 under the dead load (External Column)		
	Hand		
	Shear force, kN	Axial force, kN	Moment, kN-m
16	30.45	173.63	79.74
15	38.06	407.95	79.74
14	38.06	642.30	79.74
13	38.06	876.64	79.74
12	38.06	1110.99	79.74
11	38.06	1345.33	79.74
10	38.06	1579.68	79.74
9	38.06	1814.02	79.74
8	38.06	2048.37	79.74
7	38.06	2282.71	79.74
6	38.06	2517.06	79.74
5	30.45	2751.40	79.74
4	30.45	3006.19	79.74
3	30.45	3249.37	79.74
2	30.45	3489.83	79.74
1	25.37	3714.48	72.49

**Table 3.3.10. Hand calculated values for internal forces under dead load**

### Internal forces verifications under Wind load

The internal forces for Frame 4 were determined using the Portal Frame Method, based on the following assumptions:

- A hinge is inserted at the midpoint of each girder, as this location is presumed to have zero moment.
- Similarly, a hinge is placed at the midpoint of each column, where the moment is also assumed to be zero.
- At each floor level, the shear force in the interior column hinges is taken as twice that in the exterior column hinges, reflecting the frame's behavior as a combination of portal units.

The way of solution is provided in Appendix.

Story	h,m	Fxi (kN)	Fx (kN)	V (kN)	a (m)	b (m)
16	5	32.48	32.48	2.71	2.5	3.7
15	4	39.20	71.68	5.97	2	3.7
14	4	34.42	106.10	8.84	2	3.7
13	4	34.03	140.13	11.68	2	3.7
12	4	33.63	173.77	14.48	2	3.7
11	4	33.21	206.98	17.25	2	3.7
10	4	32.77	239.75	19.98	2	3.7
9	4	32.30	272.05	22.67	2	3.7
8	4	31.79	303.84	25.32	2	3.7
7	4	31.25	335.09	27.92	2	3.7
6	4	30.66	365.75	30.48	2	3.7
5	5	33.73	399.48	33.29	2.5	3.7
4	5	36.51	435.99	36.33	2.5	3.7
3	5	35.20	471.20	39.27	2.5	3.7
2	5	33.60	504.80	42.07	2.5	3.7
1	6	34.48	539.28	44.94	3	3.7
0 (Ground)	<b>Ax (kN)</b>	<b>Ay (kN)</b>	<b>Ma (kN-m)</b>	<b>Bx (kN)</b>	<b>By (kN)</b>	<b>Mb (kN-m)</b>
	-44.94	-453.09	134.82	-89.88	0.00	269.64

**Table 3.3.11. Hand calculated values for internal forces under wind load**

<b>1 (kN)</b>	<b>2 (kN)</b>	<b>3 (kN)</b>	<b>4 (kN)</b>	<b>5 (kN)</b>	<b>6 (kN)</b>
-1.83	-29.77	1.83	0.00	-24.36	1.83
-6.52	-35.94	4.69	0.00	-29.40	4.69
-14.53	-31.55	8.01	0.00	-25.81	8.01
-25.62	-31.20	11.09	0.00	-25.53	11.09
-39.76	-30.83	14.14	0.00	-25.23	14.14
-56.91	-30.45	17.15	0.00	-24.91	17.15
-77.03	-30.04	20.12	0.00	-24.58	20.12
-100.09	-29.60	23.05	0.00	-24.22	23.05
-126.03	-29.14	25.94	0.00	-23.84	25.94
-154.81	-28.65	28.78	0.00	-23.44	28.78
-186.38	-28.11	31.57	0.00	-23.00	31.57
-229.47	-30.92	43.09	0.00	-25.30	43.09
-276.51	-33.47	47.04	0.00	-27.38	47.04
-327.59	-32.27	51.08	0.00	-26.40	51.08
-382.55	-30.80	54.95	0.00	-25.20	54.95
-453.09	-31.61	70.55	0.00	-25.86	70.55
<b>Cx (kN)</b>	<b>Cy (kN)</b>	<b>Mc (kN-m)</b>	<b>Dx (kN)</b>	<b>Dy (kN)</b>	<b>Md (kN-m)</b>
-89.88	0.00	269.64	-44.94	0.00	134.82

**Table 3.3.12. Continuation table**

<b>7 (kN)</b>	<b>8 (kN)</b>	<b>9 (kN)</b>	<b>10 (kN)</b>	<b>11 (kN)</b>	<b>12 (kN)</b>	<b>13 (kN)</b>
0.00	-18.95	1.83	0.00	-13.53	1.83	0.00
0.00	-22.87	4.69	0.00	-16.34	4.69	0.00
0.00	-20.08	8.01	0.00	-14.34	8.01	0.00
0.00	-19.85	11.09	0.00	-14.18	11.09	0.00
0.00	-19.62	14.14	0.00	-14.01	14.14	0.00
0.00	-19.37	17.15	0.00	-13.84	17.15	0.00
0.00	-19.12	20.12	0.00	-13.65	20.12	0.00
0.00	-18.84	23.05	0.00	-13.46	23.05	0.00
0.00	-18.55	25.94	0.00	-13.25	25.94	0.00
0.00	-18.23	28.78	0.00	-13.02	28.78	0.00
0.00	-17.89	31.57	0.00	-12.78	31.57	0.00
0.00	-19.68	43.09	0.00	-14.05	43.09	0.00
0.00	-21.30	47.04	0.00	-15.21	47.04	0.00
0.00	-20.54	51.08	0.00	-14.67	51.08	0.00
0.00	-19.60	54.95	0.00	-14.00	54.95	0.00
0.00	-20.11	70.55	0.00	-14.37	70.55	0.00

**Table 3.3.13. Continuation table**

<b>14 (kN)</b>	<b>15 (kN)</b>	<b>16 (kN)</b>	<b>17 (kN)</b>	<b>18 (kN)</b>	<b>19 (kN)</b>
-8.12	1.83	0.00	-2.71	1.83	1.83
-9.80	4.69	0.00	-3.27	4.69	6.52
-8.60	8.01	0.00	-2.87	8.01	14.53
-8.51	11.09	0.00	-2.84	11.09	25.62
-8.41	14.14	0.00	-2.80	14.14	39.76
-8.30	17.15	0.00	-2.77	17.15	56.91
-8.19	20.12	0.00	-2.73	20.12	77.03
-8.07	23.05	0.00	-2.69	23.05	100.09
-7.95	25.94	0.00	-2.65	25.94	126.03
-7.81	28.78	0.00	-2.60	28.78	154.81
-7.67	31.57	0.00	-2.56	31.57	186.38
-8.43	43.09	0.00	-2.81	43.09	229.47
-9.13	47.04	0.00	-3.04	47.04	276.51
-8.80	51.08	0.00	-2.93	51.08	327.59
-8.40	54.95	0.00	-2.80	54.95	382.55
-8.62	70.55	0.00	-2.87	70.55	453.09

**Table 3.3.14. Continuation table**

Story	Internal Column			External Column		
	Axial Force (kN)	Shear Force (kN)	Internal moment (kN-m)	Axial Force (kN)	Shear Force (kN)	Internal moment (kN-m)
16	0.00	5.41	13.53	-1.83	2.71	6.77
15	0.00	11.95	23.89	-6.52	5.97	11.95
14	0.00	17.68	35.37	-14.53	8.84	17.68
13	0.00	23.36	46.71	-25.62	11.68	23.36
12	0.00	28.96	57.92	-39.76	14.48	28.96
11	0.00	34.50	68.99	-56.91	17.25	34.50
10	0.00	39.96	79.92	-77.03	19.98	39.96
9	0.00	45.34	90.68	-100.09	22.67	45.34
8	0.00	50.64	101.28	-126.03	25.32	50.64
7	0.00	55.85	111.70	-154.81	27.92	55.85
6	0.00	60.96	121.92	-186.38	30.48	60.96
5	0.00	66.58	166.45	-229.47	33.29	83.23
4	0.00	72.67	181.66	-276.51	36.33	90.83
3	0.00	78.53	196.33	-327.59	39.27	98.17
2	0.00	84.13	210.33	-382.55	42.07	105.17
1	0.00	89.88	269.64	-453.09	44.94	134.82

**Table 3.3.15. The taken hand calculated values for internal and external columns**

**Comparison of internal forces between 2D, 3D and hand calculations**

Floor	Frame 4 under the dead load (External Column)								
	Hand			2D			3D		
	Shear force, kN	Axial force, kN	Moment, kN-m	Shear force, kN	Axial force, kN	Moment, kN-m	Shear force, kN	Axial force, kN	Moment, kN-m
16	30.45	173.63	79.74	18.07	217.75	48.82	21.62	164.65	56.82
15	38.06	407.95	79.74	31.67	518.99	68.41	45.28	398.75	96.97
14	38.06	642.30	79.74	34.54	824.88	73.76	51.58	641.59	109.31
13	38.06	876.64	79.74	36.51	1133.05	78.29	56.5	889.39	120.71
12	38.06	1110.99	79.74	39.48	1442.5	81.22	63.68	1140.87	130.54
11	38.06	1345.33	79.74	34.55	1751.59	71.27	54.88	1392.4	112.8
10	38.06	1579.68	79.74	36.84	2060.03	78.93	60.61	1643.57	129.92
9	38.06	1814.02	79.74	37.58	2368.24	77.24	63.68	1895.73	129.93
8	38.06	2048.37	79.74	33.58	2675.53	69.32	55.34	2146.84	114.21
7	38.06	2282.71	79.74	35.71	2981.73	73.5	62.16	2396.89	128.43
6	38.06	2517.06	79.74	31.96	3286.8	65.1	55.59	2645.84	111.78
5	30.45	2751.40	79.74	26	3590.52	66.76	44.71	2892.92	114.62
4	30.45	3006.19	79.74	26.33	3918.22	68.44	48.32	3158.63	125.49
3	30.45	3249.37	79.74	22.83	4229.93	58.54	40.35	3411.02	103.46
2	30.45	3489.83	79.74	23.47	4536.49	58.74	43.16	3658.56	111.59
1	25.37	3714.48	72.49	11.23	4821.6	43.33	29.27	3888.31	86.42
-1							52.63		126.46

**Table 3.3.16.** Comparison of Frame 4 under dead load among Hand, 2D and 3D calculations

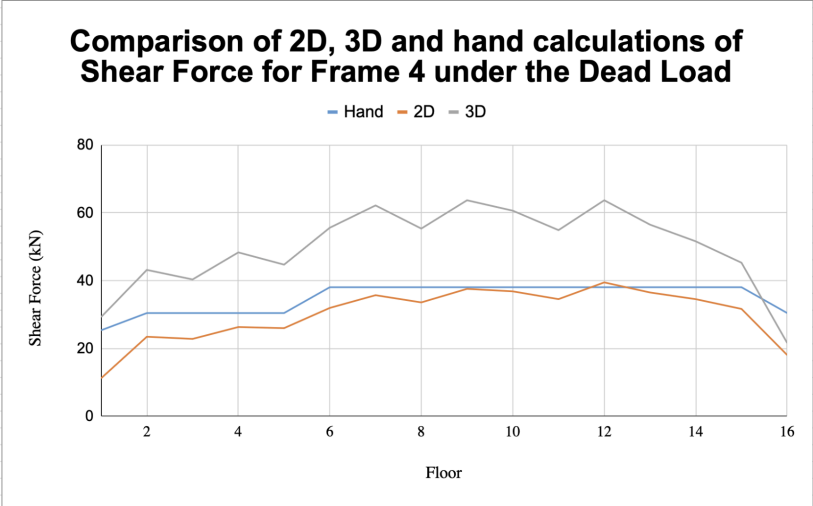


Figure 3.3.32 Shear Force Comparison under dead load

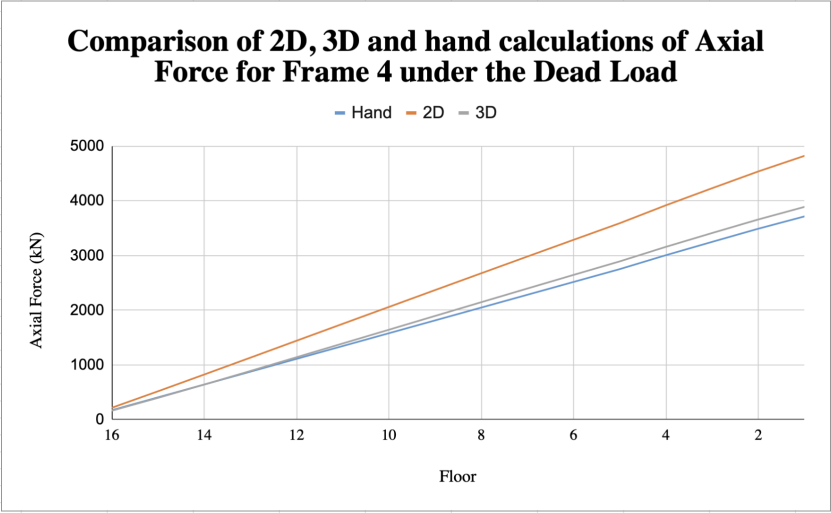


Figure 3.3.33. Axial Force Comparison under dead load

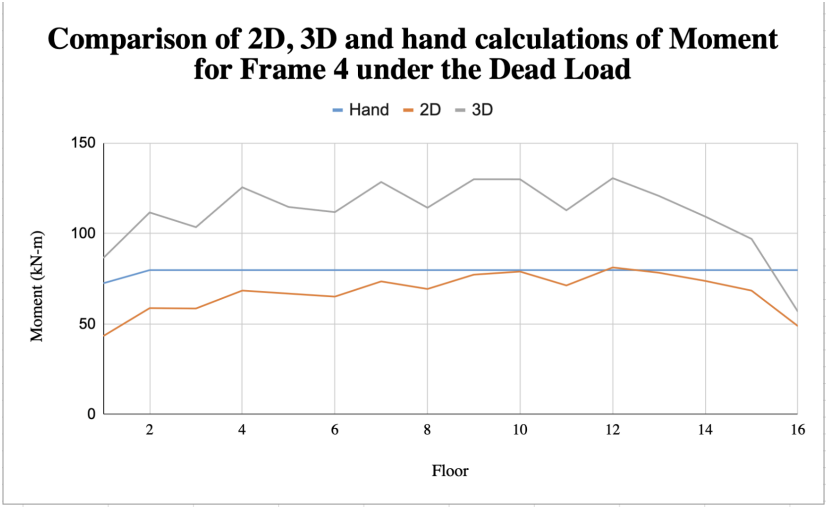
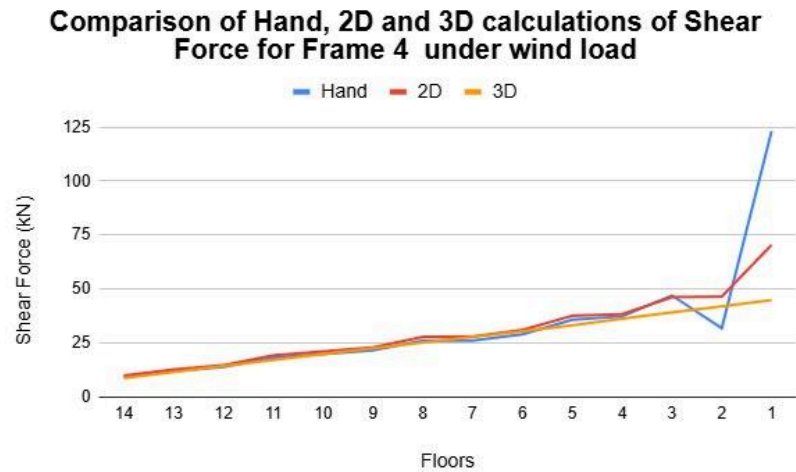


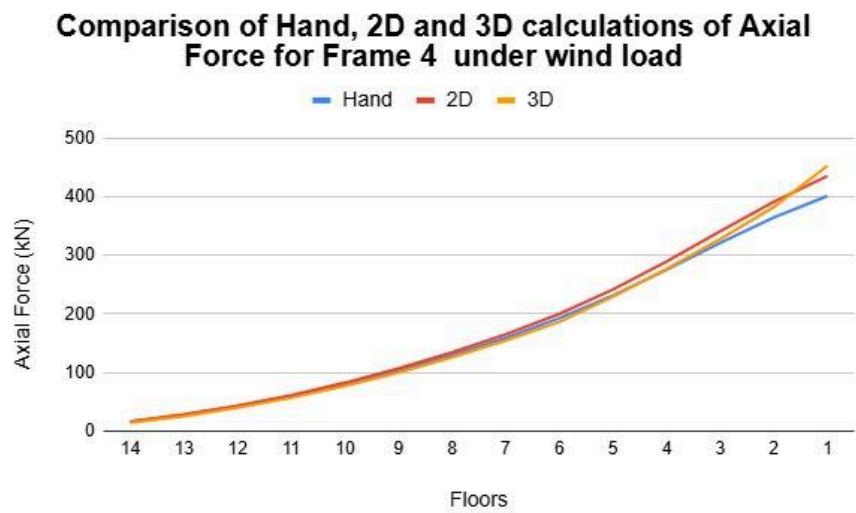
Figure 3.3.34. Moment Comparison under dead load

Floor	Frame 4 under the wind load (External Column)								
	Hand			2D			3D		
	Shear force kN	Axial Force kN	Moment kN-m	Shear Force kN	Axial Force kN	Moment kN-m	Shear Force kN	Axial Force kN	Moment kN-m
16	2.71	1.828	6.77	3.39	2.07	7.62	4.29	2.33	10.19
15	5.97	6.520	11.95	6.58	7.53	11.67	6.85	8.13	12.9
14	8.84	14.529	17.68	9.38	15.93	16.14	10.04	16.66	18.44
13	11.68	25.620	23.36	12.08	27.38	20.33	12.84	28.41	23.42
12	14.48	39.760	28.96	13.97	41.77	21.54	14.85	43.23	25.59
11	17.25	56.911	34.50	18.33	59.04	29.48	19.4	61.15	34.4
10	19.98	77.034	39.96	19.92	79.59	31.62	21.21	82.47	37.81
9	22.67	100.088	45.34	21.7	103.04	30.54	23.04	106.8	38.06
8	25.32	126.029	50.64	26.22	129.34	36.96	27.94	134.15	46.63
7	27.92	154.810	55.85	26.22	159.13	30.52	28.08	165.1	42.56
6	30.48	186.379	60.96	29.18	192.53	22.65	31.14	199.94	36.5
5	33.29	229.467	83.23	35.9	231.51	56.71	37.69	241.48	69.11
4	36.33	276.509	90.83	37.38	275.59	79.45	38.44	289.61	81.29
3	39.27	327.590	98.17	47.1	321.35	144.53	46.32	340.81	121.53
2	42.07	382.545	105.17	31.94	364.78	143.08	46.56	391.53	165.18
1	44.94	453.091	134.82	123.15	401.18	744.41	70.48	435.32	424.38

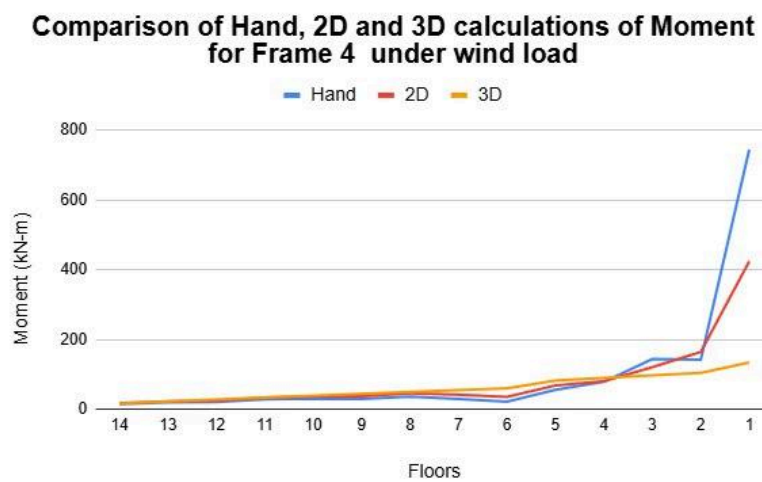
**Table 3.3.17.** Comparison of Frame 4 under wind load (External Column) among Hand, 2D and 3D calculations



**Figure 3.3.35.** Shear Force Comparison of Wind loads



**Figure 3.3.36.** Axial Force Comparison from Wind Loads



**Figure 3.3.37.** Moment Comparison of Wind load

### 3.4. Structural member design (size or check reinforcement) using software

Before sizing the reinforcing of the structural elements, it should be mentioned that correctness of all structural joints and parts of the building must be successfully checked in SAP2000 after running the model.

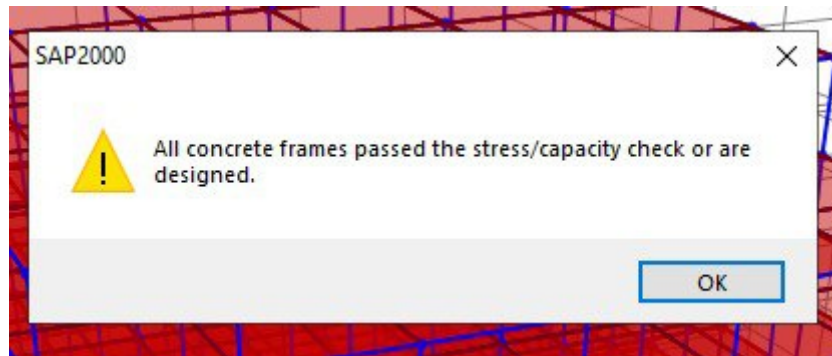


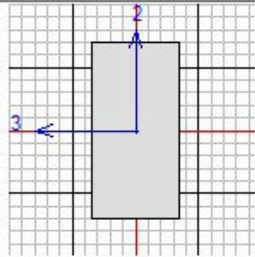
Figure 3.4.1. Structural design check in SAP2000

#### 3.4.1. Two-way Slab

Calculation of reinforcement of slab in our case is done by hand calculation only, as SAP2000 does not provide detailed sizing of the slab, particularly reinforcement. Check section 3.5.1 for hand calculations.

#### 3.4.2. Major Beams

Assuming that the building might have significant loads from the seismic loads, the following load combination was used in calculations:  $1.2D+1.0E+1.0L+0.2S$ , due to having the most critical positive and negative moments among other load combinations. Consequently, maximum moments are observed at the edges of the beams, and therefore, negative moment value is used for determination of the top reinforcement, while positive moment for bottom reinforcement. As a result, 4 different types of beams would be used during the construction of the hotel, particularly classified by the reinforcement on different floor levels. Parameters of the beams are presented below.



Units KN, mm, C

ACI 318-14 BEAM SECTION DESIGN Type:Sway Special Units: KN, mm, C (Summary)

Element : 136 D=600.000 B=300.000 bf=300.000  
 Section ID : Major Beam ds=0.000 dcb=60.000  
 Combo ID : COMB5 E=35.000 fct=0.040 Lt.Wt. Fac.=1.000  
 Station Loc : 7400.000 L=7400.000 fy=0.414 fys=0.414

Phi(Bending): 0.900  
 Phi(Shear): 0.750  
 Phi(Seis Shear): 0.600  
 Phi(Torsion): 0.750

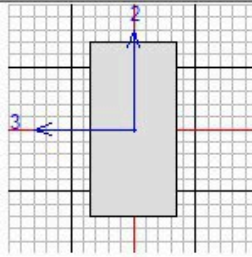
Design Moments, M3	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	335165.015	-670330.031	335165.015	-670330.031

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	3884.817	0.000	3884.817	616.958
Bottom (-2 Axis)	1786.982	1786.982	98.682	616.958

Shear Reinforcement for Shear, V2				
Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
2.237	374.735	0.000	374.735	187.505

Reinforcement for Torsion, T						
Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*TCr	Area Ao	Perimeter Ph	
0.000	0.000	21.580	28208.697	91709.229	1444.400	

Figure 3.4.2. Beam on Slab levels 1-4



ACI 318-14 BEAM SECTION DESIGN Type:Sway Special Units: KN, mm, C (Summary)

Element : 709 D=600.000 B=300.000 bf=300.000  
 Section ID : Major Beam ds=0.000 dct=60.000 dcb=60.000  
 Combo ID : COMB5 E=35.000 fc=0.040 Lt.Wt. Fac.=1.000  
 Station Loc : 7400.000 L=7400.000 fy=0.414 fys=0.414

Phi(Bending): 0.900  
 Phi(Shear): 0.750  
 Phi(Seis Shear): 0.600  
 Phi(Torsion): 0.750

Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	321703.478	-643406.957	321703.478	-643406.957

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	3719.849	0.000	3719.849	616.958
Bottom (-2 Axis)	1709.905	1709.905	0.000	616.958

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
1.992	333.706	0.000	333.706	178.698

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tr	Area Ao	Perimeter Ph
0.000	0.000	0.382	28508.384	91709.229	1444.400

Figure 3.4.3. Beam on Slab levels 5-8

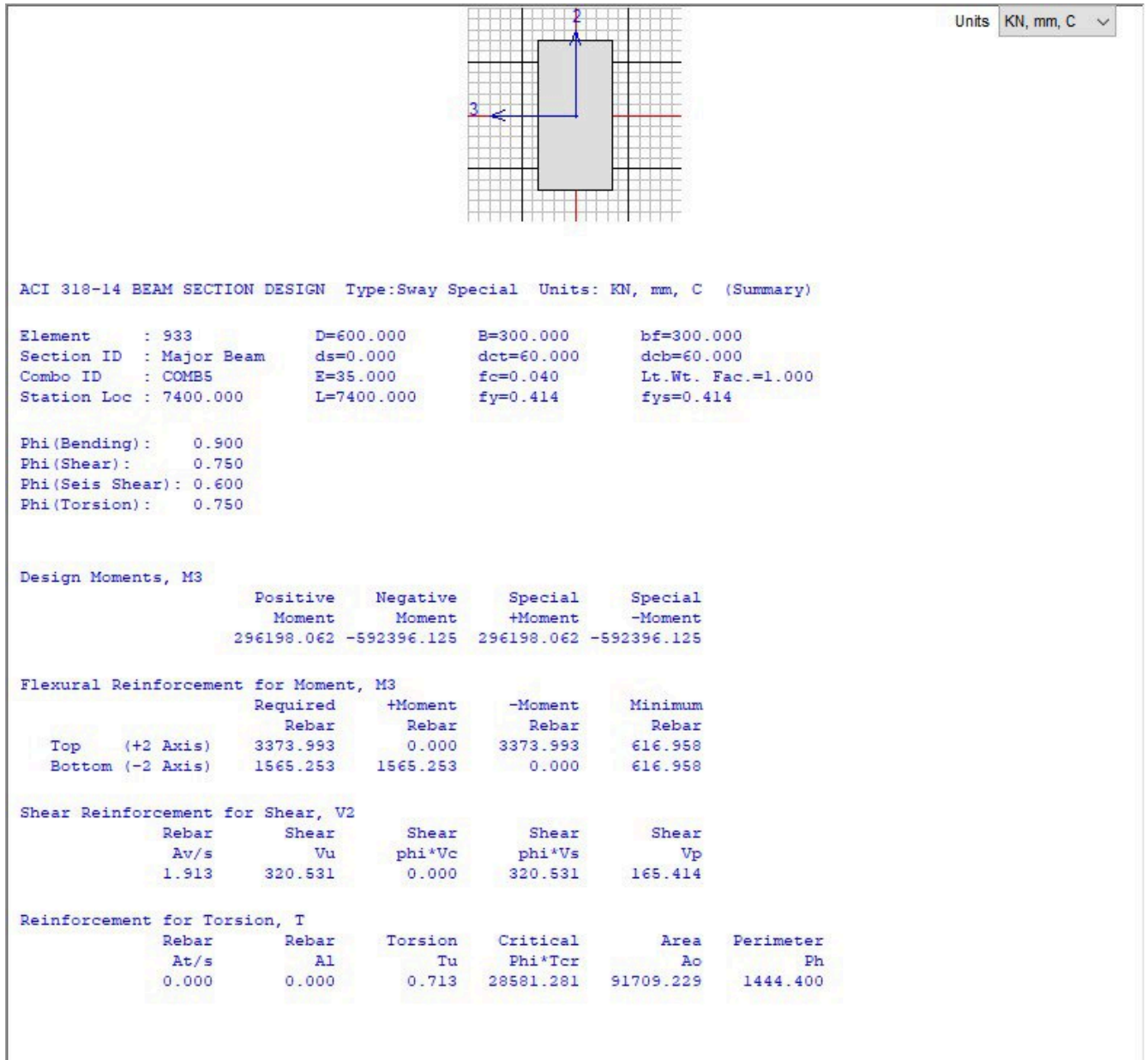


Figure 3.4.4. Beam on Slab levels 9-12

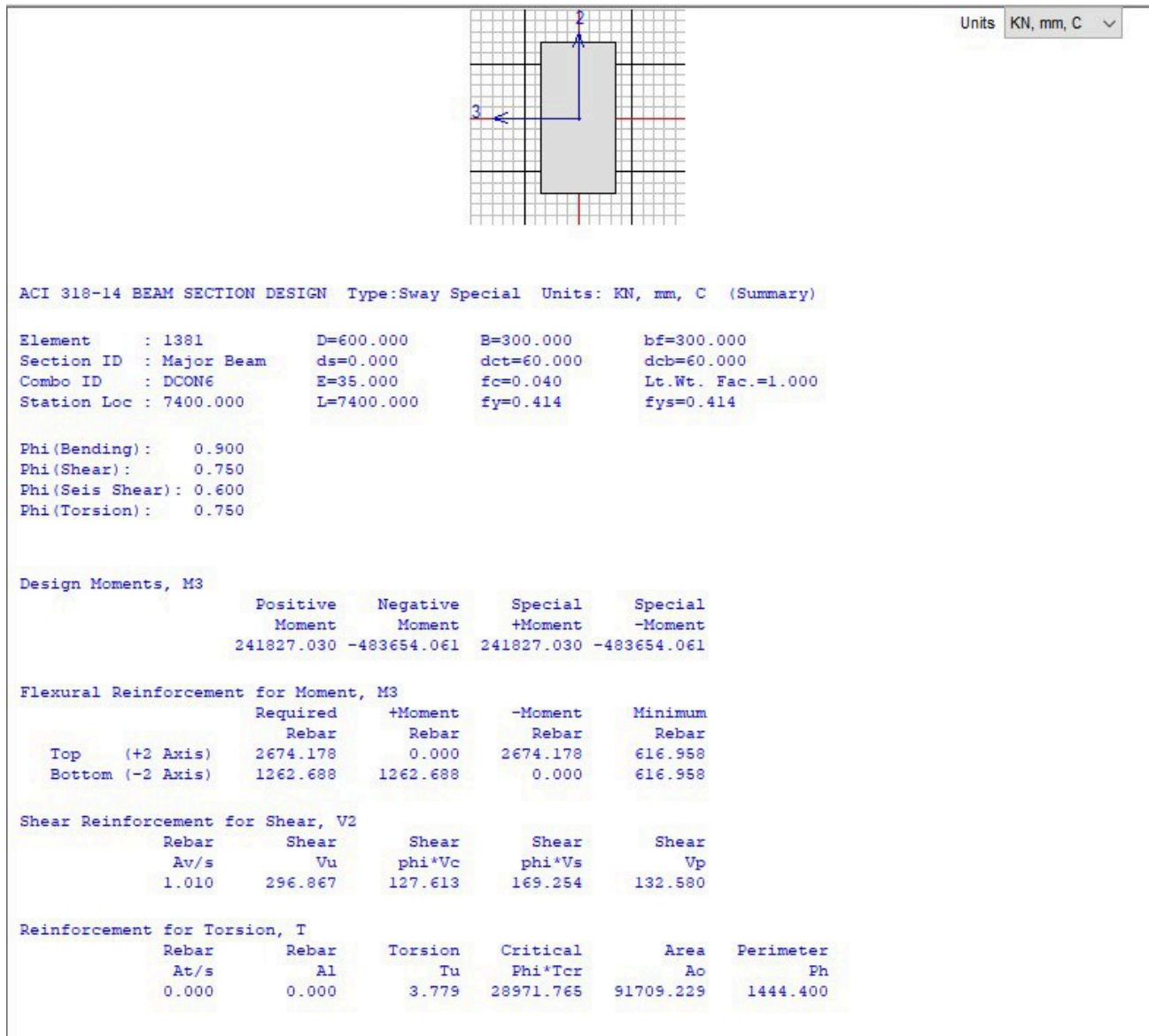
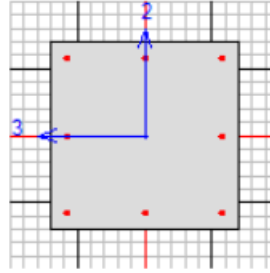


Figure 3.4.5. Beam on Slab levels 13-17

### 3.4.3. Columns

After running a model of the building in SAP2000, it was revealed that the critical load combination is #2, which is  $1.2D + 1.6L + 0.5Lr$ . A column on the lowest floor and floor 3 were chosen for the design. See Figure and Figure .



ACI 318-14 COLUMN SECTION DESIGN Type: Sway Special Units: KN, m, C (Summary)

Element : 2428 B=0.750 D=0.750 dc=0.067  
 Section ID : column 3 E=35000000. fc=40000.000 Lt.Wt. Fac.=1.000  
 Combo ID : COMB2 L=5.000 fy=413685.473 fys=413685.473  
 Station Loc : 5.000 RLLF=1.000

Phi(Compression-Spiral): 0.750  
 Phi(Compression-Tied): 0.650  
 Phi(Tension Controlled): 0.900  
 Phi(Shear): 0.750  
 Phi(Seismic Shear): 0.600  
 Phi(Joint Shear): 0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3

Rebar Area	Design Pu	Design M2	Design M3	Minimum M2	Minimum M3
0.006	9853.793	-0.062	371.882	371.882	371.882

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Cm Factor	Delta_ns Factor	Delta_s Factor	K Factor	L Length
Major Bending(M3)	0.400	1.000	1.000	1.000	5.000
Minor Bending(M2)	0.518	1.000	1.000	1.000	5.000

SHEAR DESIGN FOR V2,V3

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
Major Shear(V2)	0.000	0.469	916.102	0.000	0.000
Minor Shear(V3)	0.000	0.073	916.102	0.000	153.921

JOINT SHEAR DESIGN

	Joint Shear Ratio	Shear VuTop	Shear VuTot	Shear phi*Vc	Joint Area
Major Shear(V2)	N/A	N/A	N/A	N/A	N/A
Minor Shear(V3)	N/A	N/A	N/A	N/A	N/A

(6/5) BEAM/COLUMN CAPACITY RATIOS

Major Ratio	Minor Ratio
N/A	N/A

Figure 3.4.6. Parameters of Column 3 Floor Design

### 3.5. Hand calculation verification for structural member design (size or check reinforcement)

#### 3.5.1. Two-way Slab

##### Flexural analysis

The interior and exterior slab panel design was performed according to the Direct Design Method. The slab panel dimensions are 7400mm\*7400mm, and the column size is 850mm\*850mm. The slab thickness is assumed to be 200 mm which have to be checked with the minimum thickness criteria:

Interior panel:

$$\bar{y} = 205.88 \text{ mm}$$
$$I_{beam} = 9.32 * 10^9 \text{ mm}^4$$

The moment of inertia of the slab in the long and short direction is:

$$I_{slab} = 2.47 * 10^9 \text{ mm}^4$$
$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = \frac{9.32*10^9}{2.47*10^9} = 3.78$$

$\alpha_{fm}$  is greater than 2, so the minimum thickness of the slab should be calculated as follows:

$$h_{min} = \frac{l_n(0.8+f_y/200000)}{36+9\beta} = \frac{(7400-850)(0.8+413/1400)}{36+9*1} = 159.4 \text{ mm}$$

Hence, a thickness of 200 mm satisfies the minimum requirement criteria. Next, factored loads have to be calculated:

$$q_d = 9.924 \text{ kN/m}^2$$
$$q_l = 3.83 \text{ kN/m}^2$$
$$q_u = 1.2 * 9.924 + 1.6 * 3.83 = 18.04 \text{ kN/m}^2$$

Now, the total static moments in the both directions can be calculated:

$$M_o = \frac{q_u l_n^2}{8} = \frac{18.04*7.4*(6.55)^2}{8} = 715.79 \text{ kN} - \text{m}$$
$$M_n = 0.65M_o = 465.26 \text{ kN} - \text{m}$$
$$M_p = 0.35M_o = 250.53 \text{ kN} - \text{m}$$

$$\alpha_{f1} = \alpha_s = 3.78 \geq 1.0$$

$$\frac{l_2}{l_1} = 1.0$$

Interior						
M	Column Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
%	75.00	75.00	75.00	25.00	25.00	25.00
new Mu (kN-m)	348.95	187.89	348.95	116.32	62.63	116.32
Mu slab (kN-m)	52.34	28.18	52.34	17.45	9.39	17.45
b	2.49	2.49	2.49	2.49	2.49	2.49
Rn (kPa)	867.84	467.30	867.84	289.28	155.77	289.28
Ro	0.0021	0.0011	0.0021	0.0007	0.0004	0.0007
As (m <sup>2</sup> )	0.00087	0.00047	0.00087	0.00029	0.00015	0.00029
As (in <sup>2</sup> )	1.34722	0.72104	1.34722	0.44517	0.23923	0.44517
Asmin (in <sup>2</sup> )	1.388	1.388	1.388	1.388	1.388	1.388
As final (in <sup>2</sup> )	1.388	1.388	1.388	1.388	1.388	1.388
Bar Selected	5#5	5#5	5#5	5#3	5#3	5#3
Spacing (m)	0.4975	0.4975	0.4975	0.4975	0.4975	2.4975
2h	0.4	0.4	0.4	0.4	0.4	0.4

**Table 3.5.1. Summary of the interior beam-supported panel design.**

Edge panel:

$$\bar{y} = 238.46 \text{ mm}$$

$$I_{beam} = 7.88 * 10^9 \text{ mm}^4$$

The moment of inertia of the slab in the long and short direction is:

$$I_{slab} = 2.18 * 10^9 \text{ mm}^4$$

$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = \frac{7.88*10^9}{2.18*10^9} = 3.61$$

$\alpha_{fm}$  is greater than 2, so the minimum thickness of the slab should be calculated as follows:

$$h_{min} = \frac{l_n(0.8+f_y/200000)}{36+9\beta} = \frac{(7400-850)(0.8+413/1400)}{36+9*1} = 159.4 \text{ mm}$$

Hence, a thickness of 200 mm satisfies the minimum requirement criteria. Next, it is essential to consider the torsional resistance of the effective transverse edge beam, which is reflected by  $\beta t$ :

$$C = \Sigma(1 - \frac{0.63x}{y})(\frac{x^3 y}{3}) = 7.31 * 10^8 \text{ mm}^4$$

$$\beta_t = \frac{E_{cb} C}{2E_{cs} I_s} = 0.17$$

Since  $\beta_t < 2.5$ , the interpolated coefficient should be used for the exterior support.

The reinforcement area estimations for the exterior panel are summarized in Table 3.5.2.

Exterior						
M	Column Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
%	98.33	75.00	75.00	1.67	25.00	25.00
new Mu (kN-m)	457.48	187.89	348.95	7.79	62.63	116.32
Mu slab (kN-m)	68.62	28.18	52.34	1.17	9.39	17.45
b	1.35	1.35	1.35	1.35	1.35	1.35
Rn (kPa)	2102.66	863.60	1603.83	35.78	287.87	534.61
Ro	0.0053	0.0021	0.0040	0.0001	0.0007	0.0013
As (m <sup>2</sup> )	0.00116	0.00047	0.00088	0.00002	0.00015	0.00029
As (in <sup>2</sup> )	1.80093	0.72538	1.36274	0.02968	0.23970	0.44681
Asmin (in <sup>2</sup> )	0.751	0.751	0.751	0.751	0.751	0.751
As final (in <sup>2</sup> )	1.801	0.751	1.363	0.751	0.751	0.751
Bar Selected	6#5	6#5	6#5	5#3	5#3	5#3
Spacing (m)	0.2243	0.2243	0.2243	0.2692	0.2692	0.2692
2h	0.22	0.22	0.22	0.25	0.25	0.25

**Table 3.5.2. Summary of the exterior beam-supported panel design.**

### Shear analysis

Shear should be checked at this point:

$$d_{shear} = 165.075 \text{ mm}$$

$$f'_c = 40 \text{ MPa}$$

$$f_y = 413 \text{ MPa}$$

$$V_u = [l_1 l_2 - (b + d_{avg})(h + d_{avg})] * q_u$$

$$V_u = [3.7 * 3.7 - (0.85 + 0.165075)^2] * 18.04 = 228.34 \text{ kN}$$

$$\lambda_s = \sqrt{\frac{2}{(1+0.004d_{shear})}} = \sqrt{\frac{2}{(1+165.075*0.004)}} \leq 1, \text{ thus use } \lambda_s = 1.0$$

$$\phi V_{c1} = \phi(0.17\lambda_s \sqrt{f'_c} + \frac{N_u}{6A_g})b_w d = 366.47 \text{ kN}$$

$$\phi V_{c2} = \phi(0.66\lambda\lambda_s \rho^{(1/3)} \sqrt{f'_c} + \frac{N_u}{6A_g}) b_w d = 254.92 \text{ kN} \rightarrow \text{controls}$$

$$\phi V_{c2} = 254.92 \text{ kN} > V_u = 228.34 \text{ kN} \rightarrow \text{Shear criteria is satisfied}$$

### 3.5.2. One Major Beam

For the hand calculation verification of the major beam reinforcement, both positive and negative moments obtained from SAP2000 were used. Negative moment was taken to calculate the bottom reinforcement and positive moment for top reinforcement. Load combination #5 is considered (1.2D + 1.0E + 1.0L + 0.2S) due to having critical values.

#### Top Reinforcement

$$M_u = 670 \text{ kN} - m$$

$$b = 300 \text{ mm}, h = 600 \text{ mm},$$

$$d = 600 - 2.5 \text{ in} \cdot 25.4 \text{ mm/in} = 536.5 \text{ mm}$$

$$f'_c = 40 \text{ MPa}, f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{bd^2} = \frac{670}{300 \cdot 536.5^2} = 7759.15 \text{ kN/m}^2$$

$$\rho = \frac{0.85f'_c}{f_y} \left[ 1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right] = 0.021$$

$$A_s = \rho b d = 0.021 \cdot 300 \cdot 536.5 = 3423 \text{ mm}^2$$

$$A_s f_y = 0.85f'_c a b \rightarrow a = \frac{3423 \cdot 413}{0.85 \cdot 40 \cdot 300} = 141 \text{ mm}$$

$$\beta_1 = 0.85 - 0.05 \left( \frac{5801 - 4000}{1000} \right) = 0.76$$

$$c = \frac{a}{\beta_1} = \frac{141}{0.76} = 185.5 \text{ mm}$$

$$\epsilon_t = 0.003 \left( \frac{d-c}{c} \right) = 0.003 \left( \frac{536.5 - 185.5}{185.5} \right) = 0.0057 > 0.005 \rightarrow$$

tension-controlled and ductile

However,  $A_s$  calculated by SAP2000 is 3888 mm<sup>2</sup>. Consequently, the error should be calculated as:

$$\text{Error} = \frac{3888 - 3423}{3888} \times 100\% = 11.96\%$$

$$A_{s,min} = 3 \sqrt{f'_c} b d / f_y \geq 200 b d / f_y = 613 \text{ mm}^2 > 536.5 \text{ mm}^2$$

$$A_s = 3888 \text{ mm}^2 > 613 \text{ mm}^2 \rightarrow \text{OK!}$$

Therefore, the  $A_s = 3888 \text{ mm}^2$  ( $6.03 \text{ in}^2$ ) is the required area for the bar selection. Followingly, at the top reinforcement, 5#10 bars will be chosen ( $A_s = 6.35 \text{ in}^2$ ). Therefore, the minimum edge distance of the beam should be checked. With the assumption that #3 stirrups will be used, MED = 5.25 in, diameter of #10 and #3 bars = 1.27 in and  $\frac{3}{8}$  in, respectively, and therefore, spacing is 2.011 in (51 mm), which meets all code requirements.

### Bottom Reinforcement

$$M_u = 335 \text{ kN} - \text{m}$$

$$b = 300 \text{ mm}, h = 600 \text{ mm},$$

$$d = 600 - 2.5 \text{ in} \cdot 25.4 \text{ mm/in} = 536.5 \text{ mm}$$

$$f'_c = 40 \text{ MPa}, f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{bd^2} = \frac{335}{300 \cdot 536.5^2} = 3880 \text{ kN/m}^2$$

$$\rho = \frac{0.85f'_c}{f_y} \left[ 1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right] = 0.0098$$

$$A_s = \rho b d = 0.0098 \cdot 300 \cdot 536.5 = 1582.85 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \rightarrow a = \frac{1582.85 \cdot 413}{0.85 \cdot 40 \cdot 300} = 65.2 \text{ mm}$$

$$\beta_1 = 0.85 - 0.05 \left( \frac{5801 - 4000}{1000} \right) = 0.76$$

$$c = \frac{a}{\beta_1} = \frac{65.2}{0.76} = 85.76 \text{ mm}$$

$$\varepsilon_t = 0.003 \left( \frac{d-c}{c} \right) = 0.003 \left( \frac{536.5 - 85.76}{85.76} \right) = 0.0158 > 0.005 \rightarrow$$

tension-controlled and ductile

However,  $A_s$  calculated by SAP2000 is  $1787 \text{ mm}^2$ . Consequently, the error should be calculated as:

$$\text{Error} = \frac{1787 - 1582.85}{1787} \times 100\% = 11.42\%$$

$$A_{s,min} = 3 \sqrt{f'_c} b d / f_y \geq 200 b d / f_y = 613 \text{ mm}^2 > 536.5 \text{ mm}^2$$

$$A_s = 1787 \text{ mm}^2 > 613 \text{ mm}^2 \rightarrow \text{OK!}$$

Therefore, the  $A_s = 1787 \text{ mm}^2$  ( $2.77 \text{ in}^2$ ) is the required area for the bar selection. Followingly, at the bottom reinforcement, 3#9 bars will be chosen ( $A_s = 3.00 \text{ in}^2$ ). Therefore, the minimum edge distance of the beam should be checked. With the assumption that #3 stirrups will be used, MED = 5.25 in, diameter of #9 and #3 bars = 1.128 in and  $\frac{3}{8}$  in, respectively, and therefore, spacing is 2.1525 in (55 mm), which meets all code requirements.

### Shear analysis

The maximum  $V_u = 374.2 \text{ kN}$  was chosen from Figure .  $\lambda = 1$  for normal-weight concrete.

$$\phi V_c = \phi \cdot 2\lambda\sqrt{f'_c} b_w d = 0.75 \cdot 2 \cdot \sqrt{40000} \cdot 300 \cdot 536.5 = 48.285 \text{ kN}$$

Since  $V_u > \phi V_c$ , shear reinforcement is needed.

$$V_{c1} = 4\sqrt{f'_c} b_w d = 128.76 \text{ kN}, V_{c2} = 8\sqrt{f'_c} b_w d = 257.52 \text{ kN}$$

$$V_s = \frac{V_u - \phi V_c}{\phi} = 244.43 \text{ kN} < V_{c2}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{244.43}{0.413 \cdot 536.5} = 1.08 \text{ mm}^2/\text{mm}$$

The value of  $A_v/s$  calculated by SAP2000 is  $2.237 \text{ mm}^2/\text{mm}$ .

$$\text{Error} = \frac{2.237 - 1.08}{1.08} \times 100\% = 51\%$$

Although the differences are large, the assumptions of concrete cover and d might have been different in SAP2000, which explains the differences. However, for the design of stirrups, the value of  $A_v/s = 2.237 \text{ mm}^2/\text{mm}$  should be used. For the major beam shear design #3 @ 200 mm stirrups will be used.

### Torsional analysis

Since the design is governed by seismic forces, the torsion should also be checked. From Figure ,  $T_u = 0.02158 \text{ kN-m}$ . Firstly, the section properties should be calculated:

$$A_{cp} = 300 \cdot 600 = 0.18 \text{ m}^2, P_{cp} = 2(300 + 600) = 1.8 \text{ m}$$

$$T_n = \phi\lambda\sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}}\right) = 2.7 \text{ kN} - \text{m} > T_u \rightarrow \text{torsional reinforcement is not needed.}$$

### 3.5.3. One Column

#### Slenderness ratio check

For this example column from Floor 3 (0.8m \* 0.8m) was chosen for hand calculations verification. To begin with, the slenderness ratio of the column should be checked to identify whether the column is short or slender. Since our columns are not braced using the lateral braces or shear walls, the column is identified to be swaying.

From SAP 2000,  $P_u = 9854 \text{ kN}$

$$I_{column} = 0.0239 \text{ m}^4, I_{beam} = 0.00189 \text{ m}^4, L_{column} = 5 \text{ m}, L_{beam} = 7.4 \text{ m}$$

$$\text{Max } \frac{kl_u}{r} = 22$$

1. Interior column (800 mm × 800 mm):

$$\Psi = \frac{\Sigma EI/l_c \text{ of columns}}{\Sigma EI/l_c \text{ of beams}} = 9.35 \rightarrow k = 2.8$$

$$\frac{Kl_u}{r} = 51.3 > 22 \rightarrow \text{slender column}$$

2. Corner column (800 mm × 800 mm):

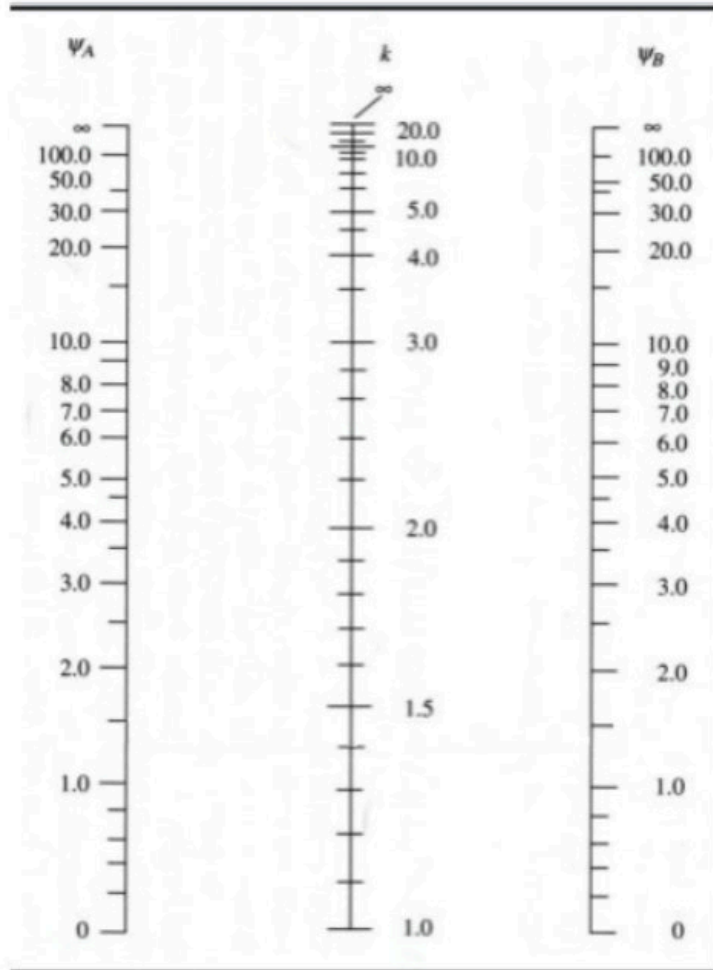
$$\Psi = \frac{\Sigma EI/l_c \text{ of columns}}{\Sigma EI/l_c \text{ of beams}} = 18.71 \rightarrow k = 3.5$$

$$\frac{Kl_u}{r} = 64 > 22 \rightarrow \text{slender column}$$

3. Exterior column (800 mm × 800 mm):

$$\Psi = \frac{\Sigma EI/l_c \text{ of columns}}{\Sigma EI/l_c \text{ of beams}} = 12.47 \rightarrow k = 3.1$$

$$\frac{Kl_u}{r} = 56.8 > 22 \rightarrow \text{slender column}$$



**Figure 3.5.1. k value graph**

As all considered variations of columns are slender, following calculations should be made:

To proceed with Slender Column Design, sway magnification is necessary:

1) Firstly, the column buckling is calculated:  $P_c = \frac{\pi^2 EI}{(kl_u)^2} = 46610 \text{ kN}$

2) After that the sway magnification coefficient is determined:

$$\delta_s = \frac{1}{1 - \frac{\Sigma P_u}{0.75 \Sigma P_c}} = 1.39 < 2.5$$

3) To identify whether the further magnification is required, the following equation must be computed:

$$\frac{l_u}{r} < \frac{35}{\sqrt{P_u / (f_c' A_g)}} = 18.33 < 56.41 \rightarrow \text{No further magnification is required}$$

4) Thus, these are the amplified moments that will be used in further column design:

$$M_1 = \delta_s M_1 = 0.09 \text{ kN} - m$$

$$M_2 = \delta_s M_2 = 518.02 \text{ kN} - m$$

### Axial and moment analysis

The maximum moments and forces acting on the column were found from the SAP2000 software and are equal to:

$$M_u = 518.02 \text{ kN} - m$$

$$P_u = 9854 \text{ kN}$$

$$V_u = 0.469 \text{ kN}$$

$\phi$  for tied columns is 0.65

Using these values the reinforcement ratio could be identified used the following procedure:

$$M_n = \frac{M_u}{\phi} = 796.96 \text{ kN} - m$$

$$P_n = \frac{P_u}{\phi} = 15160 \text{ kN}$$

$$e = \frac{M_n}{P_n} = 0.0526 \text{ m}$$

$$\gamma = 0.825$$

$$K_n = \frac{P_n}{A_g f'_c} = 0.592$$

$$R_n = \frac{P_n e}{A_g f'_c h} = 0.039$$

After that, interaction diagrams for rectangular columns from ACI 318-19 were used to identify the reinforcement ratio:

$$\rho = 0.1 = 1\%$$

$$A_s = 0.1 * A_g = 0.0064 \text{ m}^2 = 9.92 \text{ in}^2$$

Bar selection:

$$8\#10 \text{ bars } (A_s = 10.16 \text{ in}^2)$$

### Shear analysis

In order to check the shear capacity of the column,  $V_c$  should be calculated:

$$V_c = 0.17(1 + \frac{P_u}{14A_g})\lambda\sqrt{f'_c}b_w d = 22812.15 \text{ kN}$$

$$\phi V_c = 0.65 * 22812.15 = 14827.9 \text{ kN}$$

$$\phi V_c / 2 = 7413.95 \text{ kN} > V_u = 0.469 \rightarrow \text{minimum reinforcement required}$$

Spacing for shear reinforcement is minimum of:

$$48 * d_t = 0.4572 \text{ m}$$

$$16 * d_l = 0.406 \text{ m}$$

The least column dimension = 0.4 m

### Biaxial bending analysis

The Bresler Reciprocal Load Method should be used to determine if the column is safe against biaxial bending.

Bending moment values for both axes were obtained using the SAP200 software:

$$M_{ux} = 0.086 \text{ kN} - \text{m}$$

$$M_{uy} = 518.02 \text{ kN} - \text{m}$$

$$P_u = 9854 \text{ kN}$$

The procedure according to Bresler Reciprocal Load Method is summarized below:

$$M_{nx} = \frac{M_{ux}}{\phi} = 0.133 \text{ kN} - \text{m}$$

$$M_{ny} = \frac{M_{uy}}{\phi} = 796.96 \text{ kN} - \text{m}$$

$$P_n = \frac{P_u}{\phi} = 15160 \text{ kN}$$

$$\gamma_x = \gamma_y = 0.82$$

$$e_x = \frac{M_{nx}}{P_n} = 0.000009$$

$$e_y = \frac{M_{ny}}{P_n} = 0.053$$

$$P_{nx} = \frac{K_{nx} * f'_c * A_g}{\phi} = 51200 \text{ kN}$$

$$P_{ny} = \frac{K_{ny} * f'_c * A_g}{\phi} = 27569 \text{ kN}$$

$$P_0 = 0.85 * f'_c * A_g + A_{st} (f_y - 0.85f'_c) = 21762 \text{ kN}$$

Calculating Pn:

$$\frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_0}$$

$$P_n = 101492 \text{ kN}$$

Checking the validity of the method:

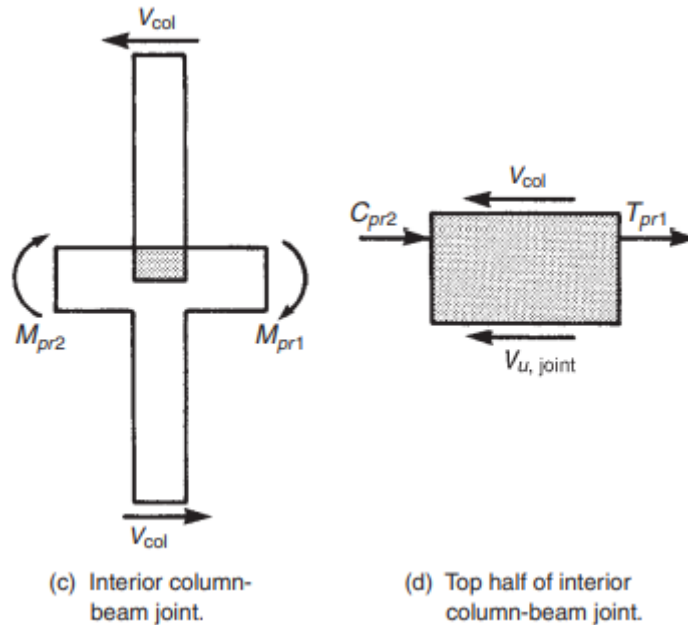
$$P_n = 101492 > 0.1P_0 \rightarrow \text{the method is valid}$$

$$\phi P_n = 65970 > P_u \rightarrow \text{the column is safe against biaxial bending}$$

### 3.6. Structural joint design

#### 3.6.1. Cast-in-place joints

According to Chapter 15 of ACI 318-19, the detailed joint design calculations was provided in the design book by Wight and MacGregor (Wight & MacGregor, 2009). The free body diagram of the interior beam-column joint can be seen in Figure 3.4.8.



**Figure 3.6.1. Free body diagram of the beam-column joint.**

An interior beam-column joint's shear was calculated according to the following equation:

$$V_{u,joint} = T_{pr1} + C_{pr2} - V_{col} \quad (3.10)$$

To calculate the joint shear, the reinforcement details were taken from the designed beam and column in section 3.5.

The values in the above equation were calculated below:

$$T_{pr1} = \alpha A_s f_y = 1.25 \cdot 1935.48 \cdot 413/1000 = 999.20 \text{ kN}$$

$$C_{pr1} = \alpha A_s f_y = 1.25 \cdot 4096.766 \cdot 413/1000 = 2115 \text{ kN}$$

From SAP2000, the column joint:

$$V_{col} = 374.2 \text{ kN}$$

Accordingly, the shear demand on joint:

$$V_{u,joint} = 999.2 + 2115 - 374.2 = 2740 \text{ kN}$$

To find the shear strength of joint, firstly the effective width of joint,  $b_j$  was calculated:

$$b_j = \frac{b_b + b_{col}}{2} = \frac{300 + 800}{2} = 550 \text{ mm} < b_b + h_{col} = 300 + 800 = 1100 \text{ mm}$$

The shear capacity of the joint can be calculated using the following equation:

$$V_n = \gamma \sqrt{f'_c} A_j \quad (3.11)$$

Where  $\gamma = 1.7$  for confined interior joint and  $A_j$  is the product of  $b_j$  and  $h_{col}$  and hence:

$$V_n = \gamma \sqrt{f'_c} A_j = 1.7 \sqrt{40} \cdot 550 \cdot 800 = 4730.77 \text{ kN}$$

The nominal shear strength requirements should be  $\phi V_n \geq V_u$ . The value of  $\phi$  can be taken as 0.85 to account for the strain hardening of the reinforcement.

$$0.85 \cdot 4730.77 = 4021.15 \text{ kN} \geq V_u = 2740 \text{ kN}$$

As a result, the joint is safe against shear forces. The similar procedure can be applied to check the joint's shear capacity in other directions and it was found that it is safe against shear.

The spacing requirements were also calculated according to the ACI 15.4.2 using the following formula:

$$A_v = \max\left(\frac{0.062 \sqrt{f'_c} b_c s}{f_{yt}}, \frac{0.35 b_c s}{f_{yt}}\right) \quad (3.12)$$

$$s = \min\left(\frac{A_v f_{yt}}{0.062 \sqrt{f'_c} b_c}, \frac{A_v f_{yt}}{0.35 b_c}\right) \quad (3.13)$$

Accordingly, the minimum spacing with 2#3 ties:

$$s = \min(187, 209) = 187 \text{ mm}$$

So, 2#3 ties at 180 mm spacing will be provided to meet the code requirements.

### 3.7. Structural Reinforcement detailing

#### 3.7.1. Reinforcement bar design and selection

Component	Top reinforcement	Bottom reinforcement	Stirrups
Major Beam (1-12 floors)	5#10 bars	3#9	#3 at 200 mm (130 mm on the edges L=1690 mm)
Major Beam (13-17 floors)	5#9	2#9	#3 at 200 mm (130 mm on the edges L=1690 mm)
Two-way slab	6#5 (ext.strip), 5#5 (int.strip)	5#3	
		Reinforcement	Ties
Column -1 floors (850 x 850 mm)		8#11	#4 at 200 mm
Column 1-4 floors (800 x 800 mm)		8#10	#3 at 200 mm
Column 5-13 floors (700 x 700 mm)		8#9	#3 at 200 mm
Column 14-16 floors (400 x 400 mm)		8#8	#3 at 200 mm

**Table 3.6.1. Final selected bars.**

#### 3.7.2. Development Length

Development lengths of bars were calculated according to the ACI 318-19. The bar diameters for #9 and #10 are  $d_b = 1.128$  in and  $d_b = 1.27$  in, respectively. The following equation was used to calculate the development length of bars in tension:

$$l_d = \frac{f_y \Psi_t \Psi_e \Psi_g}{20 \lambda \sqrt{f'_c}} d_b \text{ for bars \#7 and larger} \quad (3.15)$$

Since the following conditions were met: a) clear cover = 1.5 >  $d_b$ ; b) clear spacing of bars =  $(300/25.4 - 2 \cdot 1.5)/3 > d_b$ ; c) minimum #3 stirrups were provided. The development length for #11 bars in tension was found to be:

$$l_d = 44.4 \text{ in} > 850/25.4 = 33.46 \text{ in} \rightarrow \text{need to be hooked bars}$$

For hooked bars, the following equation can be used:

$$l_{dh} = \frac{f_y \psi_e \psi_r \psi_o \psi_c}{55 \lambda \sqrt{f'_c}} d_b^{1.5} \quad (3.16)$$

Accordingly, the  $l_{dh} = 16.93$  in or 430 mm for the bars in tension.

Subsequently, the 90° hook's parameters were found:

$$D = 6d_b = 6.77 \text{ in}, r = D/2 = 3.38 \text{ in or } 86 \text{ mm}$$

$$\text{The distance of the hook: } 12d_b = 13.56 \text{ in or } 344 \text{ mm}$$

Furthermore, the stirrups will be provided in the development length at the distance of  $\leq 3d_b = 85 \text{ mm}$ .

For the development length of bars in compression, the following equation should be used:

$$l_{dc} = \left( \frac{f_y \psi_r}{50 \lambda \sqrt{f'_c}} \right) d_b \geq 0.0003 f_y \psi_r d_b \quad (3.17)$$

Accordingly, the development length of bars in compression will be:

$$l_{dc} = 580 \text{ mm} < 800 \text{ mm}$$

### 3.7.3. Lap Splices

Beams in tension:

$$l_{st} = l_{dh} = 430 \text{ mm}$$

For beams in compression:

$$l_{sc} = 0.0005 f_y d_b = 968 \text{ mm}$$

For columns:

$$l_{sc} = 0.0005 f_y d_b = 1074 \text{ mm}$$

Component	Lap splice, mm
Column -1 floor (850 x 850 mm)	1074
Column 1-4 floors (800 x 800 mm)	968
Column 5-13 floors (750 x 750 mm)	860
Column 14-16 floors (400 x 400 mm)	762

**Table 3.7.1. Lap splices for columns.**

### 3.7.4. Seismic Reinforcement

#### Beams

Given the building's location in a high seismicity region, seismic detailing has been incorporated accordingly. The clear span and width-to-depth ratio meet the criteria outlined in Section 18.6 of the ACI 318-19 code. In compliance with ACI Section 18.6.3.1, a minimum of two longitudinal bars were continuously placed at both the top and bottom portions of the beam. The positive moment strength at the joint face exceeds half of the negative moment strength. Hoops were installed along a length equal to twice the member depth from the support face, not surpassing the lesser of  $d/4$ ,  $6d_b$ , and 150 mm. The first hoop was positioned 50 mm away from the support face.

#### Columns

Seismic provisions for columns were addressed in accordance with Section 18.7. The reinforcement area within columns ranges between 1% and 6%. Transverse reinforcement was placed at a distance  $l_0$  from the joint face on both sides. Here,  $l_0$  was determined as one-sixth of the column's clear span, giving  $l_0 = 400$  mm. All additional requirements outlined in Section 18.7 were also incorporated into the design.

#### Joints

At beam-column joints, the longitudinal reinforcement of beams was extended to reach the far face of the column. Where longitudinal bars passed through the column, the column depth  $h$  was maintained greater than  $20d_b$ .

Development lengths of the bars conform to the specified criteria:

$$l_{dh} \geq \begin{cases} \frac{f_y d_b}{65 \lambda \sqrt{f'_c}} \\ 8d_b \\ 6 \text{ in.} \end{cases} \quad (3.18)$$

### 3.8. Structural serviceability design

#### 3.8.1. Vertical deflection

The building safety depends on an evaluation of structural members for their ability to deflect without failure. The minimum required thicknesses for these members can be found in reference tables included in building codes. The minimum thicknesses become the main criterion for determining if deflection checks need to be performed (Hassoun & Manaseer, 2020). For beam members with a yield strength 60 ksi (413 MPa), minimum thickness of the concrete frame's elements can be determined with this formula:

$$h_b = \frac{L \text{ of the span}}{21}$$

Which in our case is:

$$h_b = \frac{7400}{21} = 353 \text{ mm} < 600 \text{ mm}$$

Where 600 mm is the minimum height of the major beam.

Since the selected beam depth exceeds the calculated minimum, further deflection analysis is not necessary.

#### 3.8.2. Crack width

Providing reliable and precise control over cracks remains essential to preserve reinforced concrete structures' safety when exposed to loading scenarios during construction. Building codes provide the permitted crack width standards so assessment becomes crucial to verify that these standards are achieved. The maximum flexural crack width will be computed through the application of this equation according to McCormac and Brown (2016).

$$w = 0.076\beta_h f_s \sqrt[3]{\frac{d}{c} A}$$

Where,

w - the estimated crack width (in),

$\beta_h$  - ratio of the distance from the extreme tension face of the concrete to the neutral axis relative to the distance from the reinforcement centroid (taken as 1.2)

$f_s$  - steel stress,

$d_c$  - cover,

A - effective tension area.

So,

$$d_c = 2.5 \text{ in}$$

$$A = \frac{(2.5)(300)}{25.4^3} = 9.84 \text{ in}^2$$

$$w = 0.076 \cdot 1.2 \cdot (2/3) \cdot 60000 \cdot \sqrt[3]{2.5 \cdot 9.84} = 0.0095 \text{ in} < 0.016 \text{ in}$$

This structure accepts maximum concrete cracks of 0.016 inches in width. The designed measurement passes acceptance criteria since the calculated width remains lower than this specified threshold.

## 4. Geotechnical Design

### 4.1. Site Location, Characterization, and Liquefaction

A multi-story hotel building is proposed to be built in the Los Angeles city, located in the south-west part of California state, USA. With the site being located in the state of California, the geotechnical design is expected to follow the California Building Code and follow the criterias of design published by the American Society of Civil Engineers (ASCE-7).

The geotechnical site report by the GeoPentech Company (2020) provided information about the geotechnical conditions of the site and the results of the two boreholes with depths of 31 and 46.5 meters. Modified California samplers and SPT results classified the soil samples according to the Unified Soil Classification System and the complete soil profile values, drawn from the calculations made in Capstone I, are provided in the table below:

Table 4.1.1. Complete Soil Profile

Depth (m)	Soil type	USCS class	$\gamma_{dry}$ , $kN/m^3$	$c'$ , $kN/m^2$	$\phi$ , °	$\sigma'_0$ , $kN/m^2$	$\mu_s$	E, MPa
0-1.524	Sandy Silt with Gravel	ML	15.2	0	15.70	23.2	0.0	15.500
1.524-3.048	Silt with Sand	ML	17.2	0	37.50	49.4	0.3	12.000
3.048-4.572	Well-graded Sand with silt and gravel	SW-SM	17.5	0	37.03	75.6	0.3	8.857
4.572-6.096	Silty Sand	SM	21.6	0	48.29	108.6	0.4	24.652
6.096-10.668	Well-graded sand with silt and gravel	SW-SM	22.9	0	45.49	213.5	0.4	20.643

10.668-1 2.192	Clayey sand with gravel	SC	20.8	0	40.41	245.0	0.3	14.339
12.192-1 3.716	Sand with silt and gravel	SW-SM	21.6	0	41.07	272.6	0.3	14.579
13.716-1 5.240	Silty sand	SM	20.7	0	38.54	299.1	0.3	11.623
15.240-1 8.288	Sandy clay	CL	20.9	39.55	37.52	330.6	0.3	50.000
18.288-2 1.336	Silty clay	CL	20.4	13.31	35.58	362.2	0.3	15.000
21.336-2 4.384	Sandy silt	ML	23.0	0	38.22	426.2	0.3	25.000
24.384-3 0.480	Well-graded sand with silt and gravel	SW-SM	20.7	0	33.26	552.4	0.2	7.163
30.480-3 0.7848	Silt	ML	20.7	0	33.18	558.2	0.2	60.000

Based on the Seismic Hazard Zone information from the Hollywood Quadrangle Zones of Required Investigation Map (CGS, 2014), the site is not located within a liquefaction zone. Site-specific observations confirm that the subsurface consists of dense sands and stiff clays, with no groundwater encountered up to 150 feet below the ground surface. Therefore, the risk of liquefaction is considered negligible.

To further validate this, we calculated the Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR) using Standard Penetration Test (SPT) data, following the method proposed by Idriss and Boulanger (1971). Subsequently, we used the Luna and Frost (1998) formula to compute the Liquefaction Potential Index (LPI), evaluating the soil column up to 30 meters depth, with a weighting factor emphasizing layers closer to the surface. This analysis confirmed no significant liquefaction risk at the site.

The first method to calculate the liquefaction used SPT N values, which were corrected by C factors.

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad (4.1.1)$$

$$a = 0.784 - 0.768 \sqrt{(N_1)_{60}} \quad (4.1.2)$$

From these equations, a value was obtained to be 0.421.

The point of the whole method on deriving the factors of safety for liquefaction:

$$FS = \frac{CRR}{CSR} MSF \quad (4.1.3)$$

where,

CRR - cyclic resistance ratio,

CSR - cyclic stress ratio,

MSF - magnitude scaling factor.

The formulas used to obtain these values are as follows:

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_d} r_d \frac{1}{MSF} \frac{1}{K_\sigma} \quad (4.1.4)$$

$$CRR = \exp\left(\frac{(N_1)_{60cs}}{14.1}\right) + \left(\frac{(N_1)_{60cs}}{126}\right)^2 + \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8 \quad (4.1.5)$$

$$MSF = 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058 \leq 1.8 \quad (4.1.6)$$

$$r_d = \exp(a(z) + \beta(z)) M_w \quad (4.1.7)$$

$$a(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \quad (4.1.8)$$

$$(\beta) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \quad (4.1.9)$$

where,

$K_\sigma$  - overburden correction factor,

$M_w$  - earthquake magnitude, assumed to be 6.7 (U.S. Geological Survey, n.d)

$r_d$  - stress reduction factor,

$z$  - layer depth.

The factor of safety, FS, is used to calculate LPI by the equation provided by Luna and Frost (1998):

$$LPI = \sum w_i F_i H_i, \quad (4.1.10)$$

where,

Liquefaction severity of a layer,  $F_i = 1 - FS_i$  if  $FS_i < 1$  and  $F_i = 0$  for other cases

Weighting factor,  $w_i = 10 - 0.5z_i$  if  $z < 20$  m and  $w_i = 0$  for other cases.

Overall, the following Table 4.7 presents a summary of liquefaction calculation:

Table 4.1.2. Calculation of LPI

Depth (m)	Layer thickness (m)	Soil type	$N_{60}$	FC	$(N_1)_{60cs}$	CR R	FS	$F_i$	$w_i$	LPI
1.524	1.524	Sandy Silt with Gravel	0	0	0	0	0	0	0	0
3.048	1.524	Silt with Sand	13	11	22.48	0.24	1.12	0	8.48	0
4.572	1.524	Well-graded Sand with silt and gravel	16	11	21.33	0.22	1.07	0	7.71	0
6.096	1.524	Silty Sand	60	12	62.39	1.10	5.40	0	6.95	0
10.668	4.572	Well-graded sand with silt and gravel	74	9	50.69	0.90	4.89	0	4.67	0

12.192	1.524	Clayey sand with gravel	51	8	31.81	0.63	3.53	0	3.90	0
13.716	1.524	Sand with silt and gravel	60	9	34.60	1.02	6.00	0	3.14	0
15.240	1.524	Silty sand	50	12	28.17	0.39	2.39	0	2.38	0
18.288	3.048	Sandy clay	52	72	28.90	0.42	2.81	0	0.86	0
21.336	3.048	Silty clay	47	53	23.98	0.27	1.91	0	0	0
24.384	3.048	Sandy silt	75	69	30.78	0.54	4.12	0	0	0
30.480	6.096	Well-graded sand with silt and gravel	50	11	14.50	0.15	1.29	0	0	0
30.7848	0.3048	Silt	50	70	18.25	0.19	1.58	0	0	0

Overall, as it was stated earlier, the building zone according to the Seismic Hazard Zone (CGS, 2014) is not located in the liquefaction zone, and the building area's subsurface consists of dense sands and stiff clays without a groundwater table below the surface area along 150 feet, since minimizing any corresponding risks. This calculation and its result confirmed the analysis.

Negative skin friction of the soil is considered negligible due to the excavation of upper soil layers for the basement construction to the depth of 3.55 meters.

## 4.2. Site Seismicity and Site Response Analysis

### 4.2.1. Site Seismicity Analysis

During the geotechnical investigation of the location by GeoPentech, the specific shear-wave velocity values were calculated to identify the site class for seismic design as it is written in ASCE 7-16.  $V_{s(30)}$  is an average shear wave velocity of 30 meters, calculated between 35 and 135 feet of the site below the ground surface. It was found to be 1700 ft/s or 518 m/s, corresponding to Site Class C of site classification for Seismic Design Category D, while having Risk Category II. The value was obtained by the following formula:

$$V_s = \frac{\sum_{i=1}^n di}{\sum_{i=1}^n \frac{di}{v_{si}}} \quad (4.2.1)$$

where,

$i$  = separate soil/rock layer between 1 and  $n$ ,

$v_{si}$  = shear wave velocity of  $i$ -th layer,

$di$  = thickness of the  $i$ -th layer.

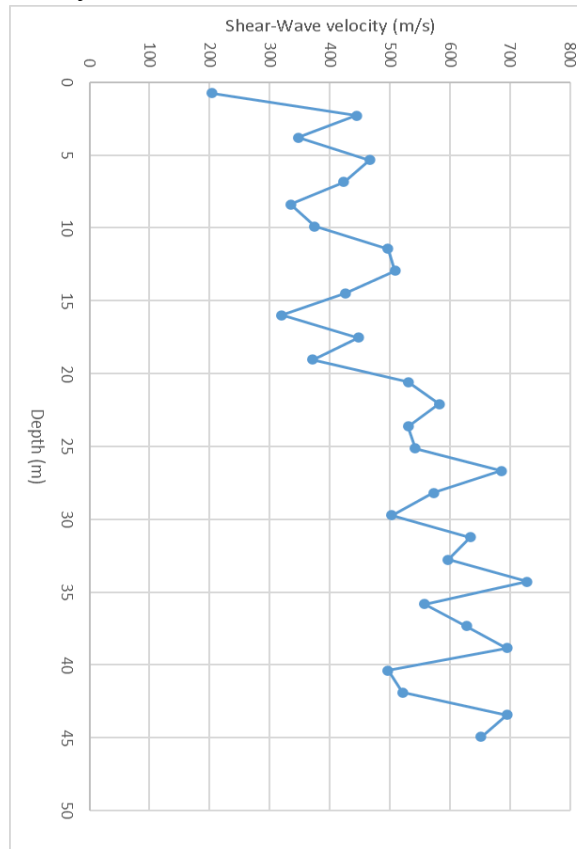


Figure 4.2.1 Shear-wave velocity graph of the location.

Table 4.2.1. Values of shear-wave velocity from 0 to 45m below the ground surface.

Depth, D (m)	SW-velocity, $V_s$ (m/s)
0.76	204
2.29	445
3.81	347
5.33	466
6.86	424
8.38	335
9.91	375
11.43	497
12.95	509
14.48	427
16.00	320
17.53	448
19.05	372
20.57	530
22.10	582
23.62	530
25.15	543
26.67	686
28.19	573
29.72	503
31.24	634
32.77	597
34.29	728
35.81	558

37.34	628
38.86	695
40.39	497
41.91	521
43.43	695
44.96	652

Geotechnical investigation provided by GeoPentech provided information on site-specific design response spectrum via the use of both probabilistic seismic hazard and deterministic seismic hazard analysis methods. Both of the analysis were conducted based on the average shear wave velocity, which provided the final values of MCE (Maximum Considered Earthquake) and the final risk-targeted design response spectrum was found by taking the minimum value from probabilistic and deterministic values, but no less than design-based minimum MCE.

Table 4.2.2. Values of MCE in accordance with time

Period (sec)	Probabilistic MCER (g)	Deterministic MCER (g)	Code-Based Deterministic Min. MCER (ASCE 7-16) (g)	Final Design Specific Design Response Spectrum (g)
0.010	1.058	1.172	0.730	0.705
0.020	1.126	1.203	0.750	0.751
0.030	1.168	1.299	0.809	0.779
0.050	1.408	1.532	0.955	0.945
0.075	1.813	1.909	1.190	1.209
0.100	2.17	2.224	1.386	1.446
0.150	2.523	2.577	1.606	1.682
0.200	2.672	2.832	1.765	1.781
0.250	2.632	2.889	1.800	1.755
0.300	2.443	2.758	1.718	1.628

0.400	2.113	2.437	1.518	1.409
0.500	1.835	2.132	1.329	1.223
0.750	1.306	1.533	0.955	0.871
1.000	0.981	1.134	0.706	0.654
1.500	0.599	0.688	0.429	0.399
2.000	0.422	0.471	0.293	0.281
3.000	0.258	0.294	0.183	0.172
4.000	0.179	0.198	0.124	0.128
5.000	0.143	0.157	0.098	0.103
7.500	0.093	0.090	0.056	0.068
10.000	0.059	0.055	0.034	0.041

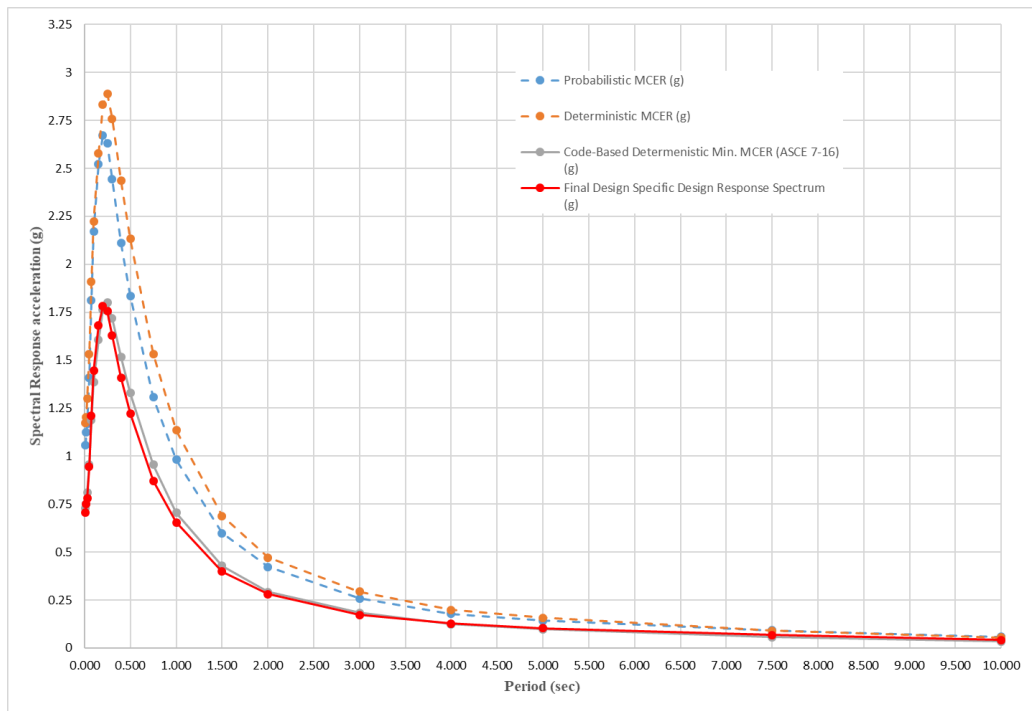


Figure 4.2.2 Site-specific Design Response Spectrum Values

The Figure 4.2.2. Provides information on Design Spectral Response Acceleration for short period (SDS) and Design Spectral Response Acceleration for 1 second period (SD-1) of 1.75 and 0.65, respectively. With  $SDS > 0.5$  and  $SD-1 < 0.75$ , it was concluded that the chosen site belongs to Seismic Category D.

## 4.2.2. Site Response Analysis

Plaxis 2D software was used for observing the soil-structure interaction by performing site response analysis. Strong motion data was retrieved from the National Center for Environmental Information database and was further used in Plaxis 2D software, as shown in the figures below:

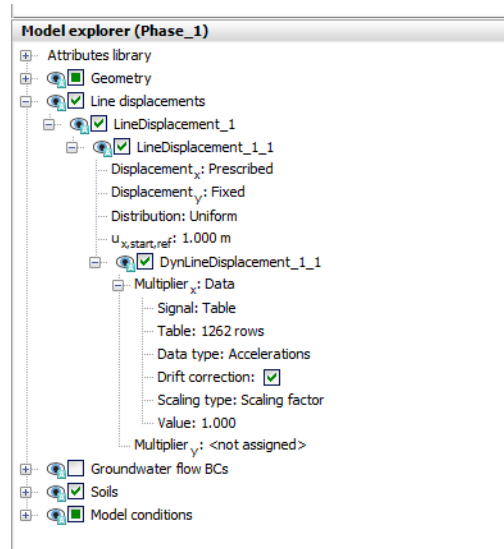


Figure 4.2.2.1. Linear Displacement settings

General	
ID	Phase_1
Start from phase	Initial phase
Calculation type	Dynamic
Loading type	Staged construction
Pore pressure calculation t	Use pressures from
Thermal calculation type	Ignore temperature
Dynamic time interval	20.00 s
First step	1
Last step	1681
Design approach	(None)
Special option	0
Deformation control parameters	
Ignore undr. behaviour (A)	<input type="checkbox"/>
Reset displacements to zero	<input checked="" type="checkbox"/>
Reset small strain	<input checked="" type="checkbox"/>
Reset state variables	<input type="checkbox"/>
Reset time	<input type="checkbox"/>
Updated mesh	<input type="checkbox"/>
Ignore suction	<input checked="" type="checkbox"/>
Cavitation cut-off	<input type="checkbox"/>
Cavitation stress	100.0 kN/m <sup>2</sup>
Numerical control parameters	
Max cores to use	256
Max number of steps store	1
Use compression for result	<input type="checkbox"/>
Use default iter parameter:	<input type="checkbox"/>
Max steps	1681
Time step determination	Manual
Number of sub steps	3
Tolerated error	0.01000
Max unloading steps	5
Max load fraction per step	0.5000
Over-relaxation factor	1.200
Max number of iterations	60
Desired min number of iterz	6
Desired max number of iter	15
Use subspace accelerator	<input type="checkbox"/>
Subspace size	3
Use line search	<input type="checkbox"/>
Use gradual error reductor	<input type="checkbox"/>

Figure 4.2.2.2. Phase configurations for dynamic analysis

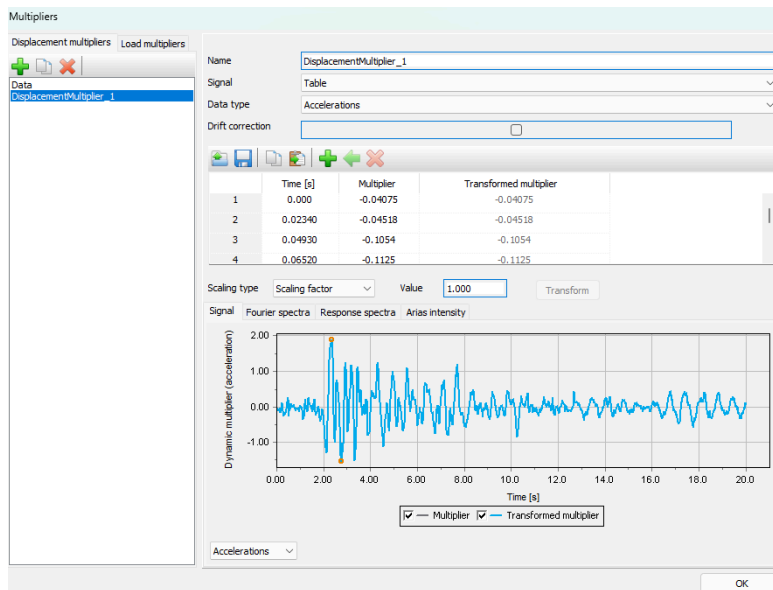


Figure 4.2.2.3. Strong motion data input to Plaxis 2D software

Deformation analysis of the soil gave the calculated value of soil deformation to be 20.78 mm.

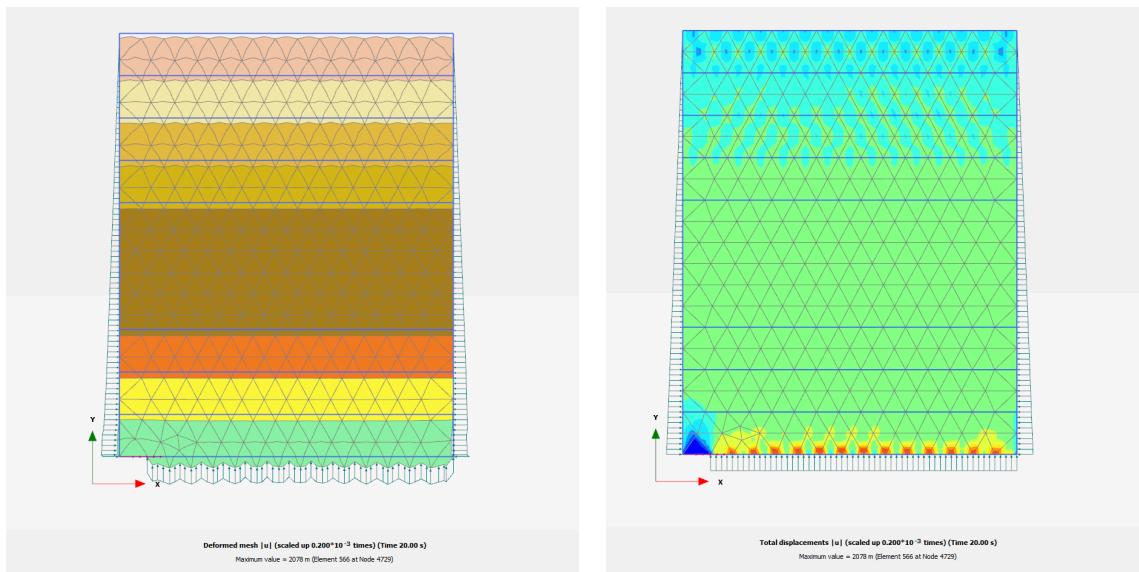


Figure 4.2.2.4. Soil deformation analysis

The remaining figures display the accelerogram for the earthquake data, ground motion amplitude computed via the Fast Fourier transformation of the previous acceleration plot, and the PSA (Peak Spectral Acceleration).

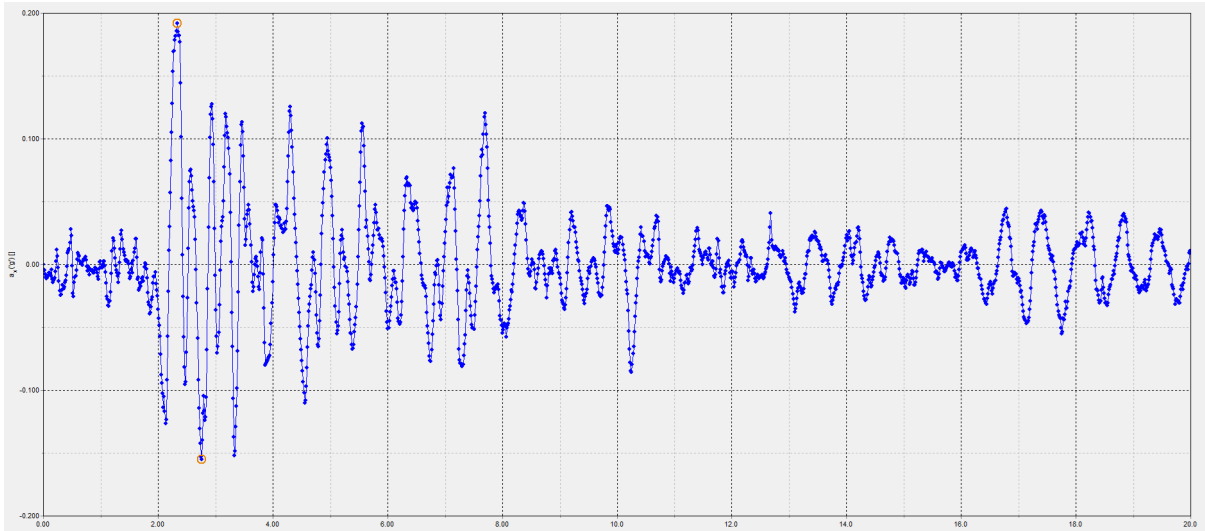


Figure 4.2.2.5. Earthquake accelerogram plot

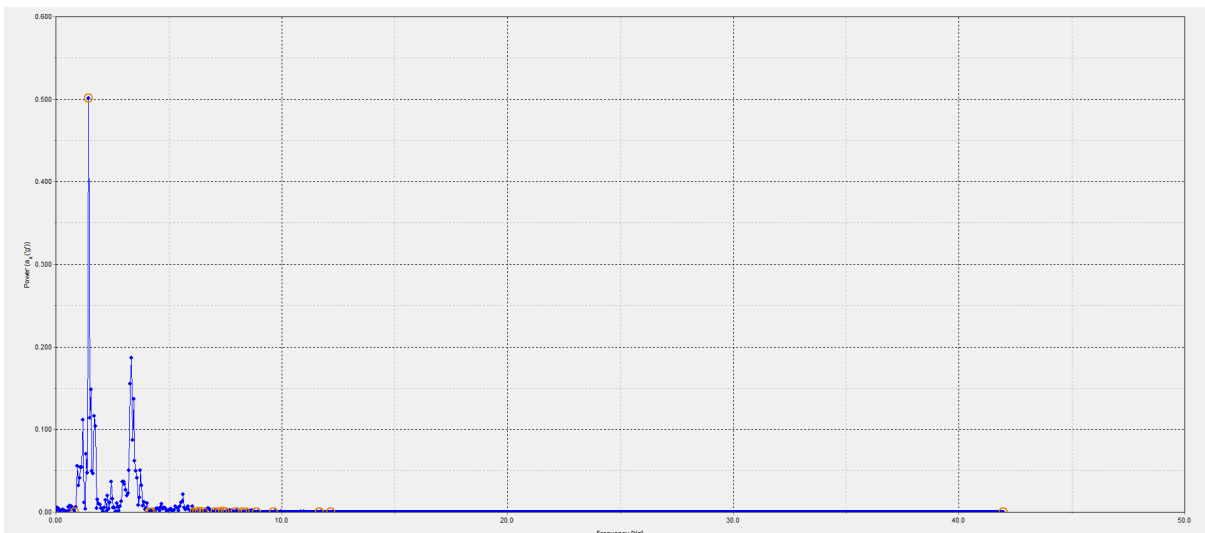


Figure 4.2.2.6. Ground motion amplitude plot

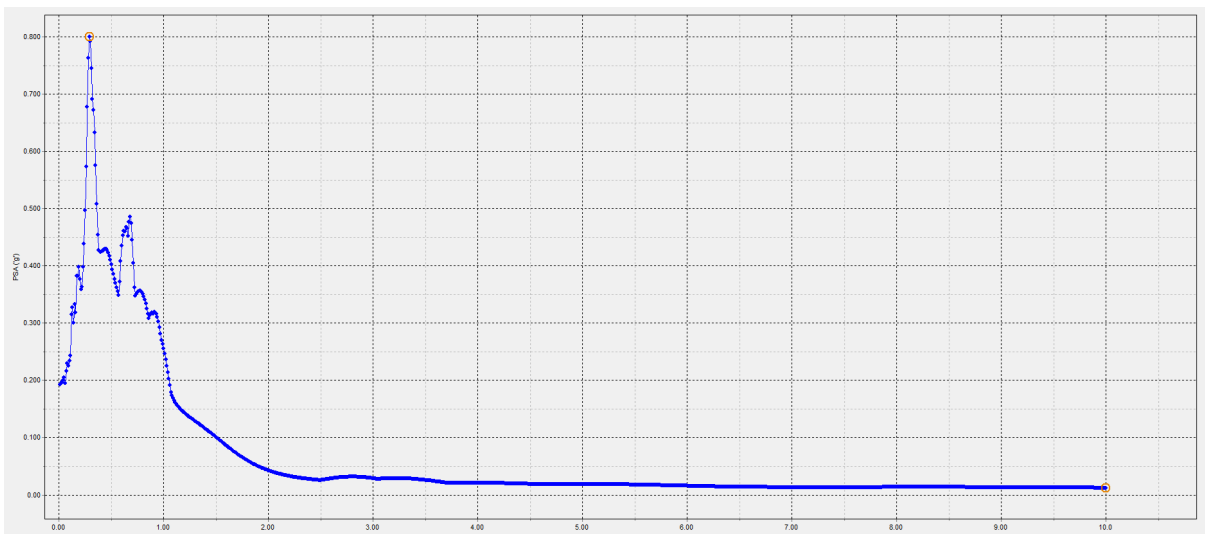


Figure 4.2.2.7. PSA (Peak Spectral Acceleration) plot

### 4.3. Foundation Design

#### 4.3.1. Selected Foundation Type

Foundation of the proposed structure is expected to be able to withstand various loads, including seismic, dead, and live loads, as well as follow the requirements for allowable settlements and deflections. Both shallow foundation and deep foundation designs were tested via hand calculations below:

#### Shallow Foundation:

##### *Pad Foundation*

Three types of columns (exterior, interior, and corner columns) are used for the design of the shallow foundation, with interior columns bearing more load due to their central position, and corner columns bearing the least. Table 4.2.1.1.1 summarizes the column types used and their design load for the hotel.

Table 4.3.1.1. Vertical load summary for the hotel

Column type	Location	Design Load for the hotel, <i>kN</i>
C-1	Interior	13357
C-2	Exterior	6724
C-3	Corner	3408

The design of the shallow foundation requires the calculations of the ultimate bearing capacity for the shear failure. Das & Sivakugan (2019) suggest that shallow foundation structures' safety factor is mostly considered 3. The general bearing capacity calculation equations are as follows:

$$q_{all} = \frac{q_u}{FS} \quad (4.3.1.1)$$

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (4.3.1.2)$$

where,

$c'$  = cohesion, kN/m<sup>2</sup>

$q$  = effective stress at the bottom level of the foundation, kN/m<sup>2</sup>

$\gamma$  = unit weight of the soil, kN/m<sup>2</sup>

$B$  = width of the foundation, m

$F_{cs}, F_{qs}, F_{\gamma s}$  = 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

The embedded depth ( $D_f$ ) is assumed as 2 meters below the ground surface, whereas the foundation itself was assumed to be a square foundation with all sides equal ( $B=L$ ). Since the basement parking has a height of 3.5 meters, the bottom of the shallow foundation is situated in the silty sand layer of the site (layer 4 depth: 4.572 - 6.096 meters below the ground surface).

For the calculations of the bearing capacity factors, the friction angle value was taken as 48.3°, and the factors' equations are as follows:

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right) \times e^{\pi \times \tan \phi'} \quad (4.3.1.3)$$

$$N_c = (N_q - 1) \times \cot \phi' \quad (4.3.1.4)$$

$$N_\gamma = 2 \times (N_q + 1) \tan \phi' \quad (4.3.1.5)$$

The equations for the shape factor:

$$F_{cs} = 1 + \left(\frac{B}{L}\right) \times \left(\frac{N_q}{N_c}\right) \quad (4.3.1.6)$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \times \tan \varphi' \quad (4.3.1.7)$$

$$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B}{L}\right) \quad (4.3.1.8)$$

For all of the columns the assumption of  $\frac{D_f}{b} > 1$  and  $\varphi' > 0$  were made, which leads to the calculation of depth factors with the following formula:

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \times \tan \varphi'} \quad (4.3.1.9)$$

$$F_{qd} = 1 + 2 \tan \varphi' \times (1 - \sin \varphi')^2 \times \tan^{-1} \left(\frac{D_f}{b}\right) \quad (4.3.1.10)$$

$$F_{\gamma d} = 1 \quad (4.3.1.11)$$

The inclination factors with a value of inclination equal to 0 were calculated using the next formula :

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta}{90}\right)^2 \quad (4.3.1.12)$$

$$F_{\gamma i} = \left(1 - \frac{\beta}{\varphi'}\right)^2 \quad (4.3.1.13)$$

The vertical effective stress (q) is calculated using the following equation:

$$q = \gamma \times D_f \quad (4.3.1.14)$$

Taking cohesion as zero due to the sand soil layer and following the calculations according to the above-provided formulas, the following values for the columns' bearing capacities were obtained:

First, the bearing capacity factors for all three types of columns will be calculated and cohesion  $c'$  is taken as 0:

$$N_q = \tan^2 \left(45 + \frac{48.3}{2}\right) \times e^{\pi \times \tan(48.3)} = 233.90$$

$$N_c = (233.90 - 1) \times \cot(48.3) = 207.58$$

$$N_\gamma = 2 \times (233.90 + 1) \tan(48.3) = 527.12$$

The next step is to calculate shape factors for the square foundation (B=L):

$$F_{cs} = 1 + 1 \times \left( \frac{233.90}{207.58} \right) = 2.13$$

$$F_{qs} = 1 + 1 \times \tan(48.3) = 2.12$$

$$F_{\gamma s} = 1 - 0.4 \times 1 = 0.6$$

The depth factor factors for three types of columns are calculated with the width of column, B, unknown, leaving it to the next calculation:

$$F_{qd} = 1 + 2 \tan(48.3) \times (1 - \sin(48.3))^2 \times \tan^{-1}\left(\frac{2}{B}\right) = 1 + 0.1441 \times \tan^{-1}\left(\frac{2}{B}\right)$$

$$F_{cd} = 1 + 0.1441 \times \tan^{-1}\left(\frac{2}{B}\right) - \frac{1 - (1 + 0.1441 \times \tan^{-1}\left(\frac{2}{B}\right))}{207.58 \times \tan(48.3)} = 1 + 0.14472 \times \tan^{-1}\left(\frac{2}{B}\right)$$

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

As the inclination is 0, the calculation of inclination factors proceeds as follows:

$$F_{ci} = F_{qi} = F_{\gamma i} = 0$$

As the bottom of the pad foundation is located in the silty sand layer, the well-graded sand with soil and water and the silty sand's vertical effective stresses were calculated:

$$\gamma_{SW-SM} = 17.5 \text{ kN/m}^3$$

$$\gamma_{SM} = 21.6 \text{ kN/m}^3$$

$$q = (1.52 - 0.5) \cdot 17.5 + 1.52 \cdot 21.6 = 50.88 \text{ kN/m}^3$$

Now, the ultimate bearing capacity equation can be built with a width to equate it with another equation, thus, solving for the B:

$$q_u = 50.88 \cdot 233.90 \cdot 2.12 \cdot (1 + 0.1441 \times \tan^{-1}(\frac{2}{B})) \cdot 1 + \frac{1}{2} \cdot 21.6 \cdot B \cdot 527.12 \cdot 0.6 \cdot 1 \cdot 1$$

$$q_u = 25229.76 + 3635.61 \tan^{-1}(\frac{2}{B}) + 3415.74B$$

To design a pad foundation size, our load per unit area should not exceed the allowable bearing capacity calculated using the safety factor, so it should at least be equal for the time of obtaining the B value. Here, this equation was solved for the interior column first:

$$q_{load} = \frac{Q}{BL} = \frac{Q}{B^2} \quad (4.3.1.15)$$

$$q_{allow} = \frac{q_u}{FS} \quad (4.3.1.16)$$

$$\frac{25229.76 + 3635.61 \tan^{-1}(\frac{2}{B}) + 3415.74B}{3} = \frac{13356.999}{B^2}$$

From this equation, the positive value of B was obtained to be equal to 1.104 m. The same procedures were made with exterior and corner columns and the following table was summarized:

Table 4.3.1.2. Vertical load summary and foundation sizes for columns

Column type	C-1: Interior	C-2: Exterior	C-3: Corner
$Q_{load}, kN$	13357	6724	3405
$q_w, kN/m^2$	35619	34153	33384
$q_{alb}, kN/m^2$	11873	7123	3565
$q_{load}, kN/m^2$	10959	10747	10637
$B = L, m$	1.104	0.791	0.566

So, as Table 4.9 suggests, the designed pad foundation would not provide a safe construction as it is seen that the bearing capacity per unit area exceeds the allowable bearing capacity for exterior and corner columns while almost reaching the value for the interior column.

### *Mat Foundation*

Designing a mat foundation has a different approach than a pad foundation, as there is a set of columns to support the load of the building, thus it covers a larger area, according to Das & Sivakugan (2019). The assumption for embedded depth,  $D_f$ , is made to be equal to 1 m from the ground surface. Dimensions of the hotel are 45\*45 m, so the total area of the building is  $2025 \text{ m}^2$ . Therefore, the mat foundation's dimensions will be the same to cover the occupied area. An interior column, which supports the largest amount of load, which is equal to 13357 kN, was taken as a base value to distribute the loads uniformly. Overall, 49 columns are supporting the building, therefore having a tributary area of  $43.09 \text{ m}^2$ ,  $q_{load}$  will be equal to  $310.01 \text{ kN/m}^2$ . The general bearing capacity equation was used to proceed with the design of the mat foundation. Due to the  $D_f$  being equal to 1m, the layer for the foundation was chosen to be in well-graded sand with silt and gravel (layer No. 3). Following values were considered while designing the mat foundation:

$$\gamma_{SW-SM} = 17.5 \text{ kN/m}^3$$

$$\phi' = 37^\circ$$

$$c' = 0$$

So, as it was designed for the pad, the same procedures apply to the mat foundation except for several differences. Bearing capacity factors are calculated as follows:

$$N_q = \tan^2\left(45 + \frac{37}{2}\right) \times e^{\pi \times \tan(37)} = 43.09$$

$$N_c = (43.09 - 1) \times \cot(37) = 55.79$$

$$N_\gamma = 2 \times (43.09 + 1) \tan(37) = 66.52$$

Shape factors calculation, where our mat foundation dimensions apply, as stated earlier (45\*45):

$$F_{cs} = 1 + \frac{45}{45} \times \left( \frac{43.09}{55.79} \right) = 1.77$$

$$F_{qs} = 1 + \frac{45}{45} \times \tan(37) = 1.75$$

$$F_{\gamma s} = 1 - 0.4 \times \frac{45}{45} = 0.6$$

.While calculating the depth factors, we will assume that  $\frac{D_f}{b} < 1$   $\phi' > 0$ :

$$F_{qd} = 1 + 2 \times \tan(37) \times (1 - \sin(37))^2 \times \tan^{-1}\left(\frac{1}{45}\right) = 1.005$$

$$F_{cd} = 1.005 - \frac{1-1.005}{55.79 \times \tan(37)} = 1.005$$

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

As the inclination is 0, the calculation of inclination factors proceeds as follows:

$$F_{ci} = F_{qi} = F_{\gamma i} = 0$$

Effective vertical stress is calculated for the layer of the foundation only:

$$q = (1.524 - 0.5) \cdot 17.5 = 17.92 \text{ kN/m}^3$$

Having put all the inputs into the general equation, the value of the ultimate bearing capacity was obtained:

$$q_u = 17076 \text{ kN/m}^2$$

The next step is to calculate net ultimate capacity:

$$q_{net.u} = q_u - q = 17058 \text{ kN/m}^2$$

As it was done with the pad, the factor of safety was taken as 3 to provide for the safety of the structure and calculate for allowable net capacity:

$$q_{net.allow} = \frac{17058}{3} = 5686 \text{ kN/m}^2$$

From here it is observed that our previous  $q_{load} = 310 \text{ kN/m}^2$  is much smaller than allowable net bearing capacity. In addition, according to Terzaghi and Peck (1948), there is a

settlement that needs to be included in the calculation to not exceed the 50mm of settlement as stated by them:

$$q_{net} = \frac{N_{60}}{0.08} F_d \frac{S_e}{25} \quad (4.3.1.17)$$

$$F_d = 1 + 0.33 \frac{D_f}{B} \quad (4.3.1.18)$$

So, substituting the values to the equation will give us  $q_{net} = 377.75 \text{ kN/m}^2$ . From this:

$$q_{net.allow} = \frac{377.75}{3} = 125.92 \text{ kN/m}^2$$

As we included the settlement, it can be concluded from here that  $q_{net.allow} < q_{allow}$ , which means it would not provide a safe structure. So, a mat foundation for such a structure is not a suitable and safe choice.

### **Pile Foundation:**

#### *Single Pile: Bearing Capacity*

The bearing capacity for the designed single-pile foundation is a summation of the point-bearing capacity and the frictional resistance of the soil, specifically sand, and clay, according to the following formula:

$$Q_u = Q_p + Q_s \quad (4.3.1.19)$$

Where,

$Q_u$  - ultimate bearing capacity of the pile,  $kN$ .

$Q_p$  - point-bearing capacity of the pile,  $kN$ ;

$Q_s$  -frictional resistance,  $kN$ .

The calculations for the piles were done assuming that the pile diameter is 0.4 meters.

#### **The point-bearing capacity of the Sand layers:**

Five methods were used for the point-bearing capacity calculations: Meyerhof's method, Vesic's method, Coyle and Castello's method, Meyerhof's method (using SPT values), and Briaud's method (using SPT values).

#### Method 1: Meyerhof's method for granular soils:

$$Q_p = A_p q_p = A_p q' N_q^* \leq A_p q_l \quad (4.3.1.20)$$

$$q_l = 0.5 p_a N_q^* \tan \phi' \quad (4.3.1.21)$$

The values of  $N_q^*$  were obtained from the interpolated table values presented in the textbook by Das & Sivakugan in Table 12.6 (2019). Additionally, the calculated values of sand's point-bearing capacity should not exceed the boundary values calculated using eq. 4.2.2.1.1.3.

Method 2: Vesic's method:

Vesic's method for estimating the point bearing capacity of sand layers is based on the cavilite's expansion theory to evaluate stress parameters (Das & Sivakugan. 2019).

$$Q_p = A_p q_p = A_p \overline{\sigma_0'} N_{\sigma}^* \quad (4.3.1.22)$$

Where,

$$\overline{\sigma_0'} = \left( \frac{1+2K_0}{3} \right) q' - \text{mean effective normal stress, } kN/m^2;$$

$$K_0 = 1 - \sin \phi' - \text{earth pressure coefficient at rest;}$$

$$N_{\sigma}^* - \text{bearing capacity factor.}$$

The reduced rigidity index is used for the calculations of the bearing capacity factor with the following formulas:

$$I_{rr} = \frac{I_r}{1+I_r \Delta} \quad (4.3.1.23)$$

Where,

$$I_r = \frac{E_s}{2(1+\mu_s)q' \tan \phi'} - \text{rigidity index;}$$

$$E_s = m * p_a - \text{modulus of elasticity with } m=200, \text{ MPa;}$$

$$\mu_s = 0.1 + 0.3 \left( \frac{\phi' - 25}{20} \right) - \text{Poisson's ratio for } 25^\circ \leq \phi' \leq 45^\circ;$$

$$\Delta = 0.005 \left( 1 - \frac{\phi' - 25}{20} \right) \frac{q'}{p_a} - \text{average volumetric strain below the pile point.}$$

Method 3: Coyle and Castello's method:

$$Q_p = A_p q' N_q^* \quad (4.3.1.24)$$

Where,

$Q'$  - effective virtual stress at the pile tip;

$N_q^*$  - bearing capacity factor depended on the embedment ratio

Method 4: Meyerhof's method (using SPT values):

$$Q_p = A_p q_p = A_p * 0.4 p_a N_{60} \frac{L}{D} \leq 4 p_a N_{60} \quad (4.3.1.25)$$

Where,

$N_{60}$  - average value of standard number near pile point (10D above and 4D below the pile point)

Method 5: Briaud's method (using SPT values):

For Briaud's method for point-bearing capacity, the correction factor of 19.7 and the  $N_{60}$ (standard penetration resistance) values were used.

$$Q_p = A_p q_p = A_p * 19.7 p_a (N_{60})^{0.36} \quad (4.3.1.26)$$

The calculated values for the point-bearing capacity of a single pile using all five methods are shown in the table below:

Table 4.3.1.3. Point-bearing capacity in Sand for Pile lengths of 7-11 meters (d=0.4m)

Pile length, m	Depth, m	Layer	Meyerhof, kN	Vesic, kN	Coyle and Castello, kN	Meyerhof (SPT), kN	Briaud (SPT), kN	The average value, kN
7	10.5	Well - graded Sand with Silt and Gravel	6505.35	3018.45	2895.50	2491.20	1049.45	3192.07
8	11.5	Clayey Sand with Gravel	2074.88	2030.58	2898.20	2815.72	1097.82	2183.44
9	12.5	Sand with Silt and Gravel	2273.50	2100.24	2990.24	2739.36	1086.77	2238.02
10	13.5	Sand with Silt and Gravel	2273.50	1676.31	3037.01	2739.36	1086.77	2162.59
11	14.5	Silty Sand	1566.25	2300.96	2696.39	2656.64	1074.58	2058.96

The average value from all five methods was calculated and chosen to be the final point-bearing capacity of the sand layers.

**The point-bearing capacity of Clay layers:**

Method 1: Meyerhof's method

$$Q_p \approx N_c^* c_u A_p = 9c_u A_p \quad (4.3.1.27)$$

Where,

$c_u$  - undrained cohesion of soil beneath the tip of the pile

Method 2: Vesic's method:

$$Q_p = A_p c_u N_c^* \quad (4.3.1.28)$$

$$I_{rr} = I_r = \frac{E_s}{3c_u} \quad (4.3.1.29)$$

Table 4.3.1.4 Point-bearing capacity in Clay for Pile lengths of 12-17 meters (d=0.4m)

Pile length, m	Depth, m	Layer	Meyerhof, kN	Vesic, kN	$Q_{s,ave}$ , kN
12	15.55	Sandy CLAY	44.21	58.72	51.71
13	16.55	Sandy CLAY	44.21	58.72	51.71
14	17.55	Sandy CLAY	44.21	58.72	51.71
15	18.55	Silty CLAY	15.05	58.72	17.39
16	19.55	Silty CLAY	15.05	58.72	17.39
17	20.55	Silty CLAY	15.05	58.72	17.39

**The frictional resistance of Sand layers:**

Method 1: Coyle and Costello's method:

$$Q_s = K \overline{\sigma_0}' \tan(\delta') pL \quad (4.3.1.30)$$

K – effective earth pressure coefficient from Figure 12.2 given by Das & Sivakugan (2019)

$\overline{\sigma_0}'$  – average effective overburden pressure,  $kN/m^2$ ;

$\delta' = 0.8\phi'$  – soil-pile friction angle;

p - perimeter of the pile

Method 2: General skin friction formula method:

$$Q_s = \sum fp\Delta L = \sum K\sigma'_o \tan(\delta')p\Delta L \quad (4.3.1.31)$$

Where,

$K = 1.5$  – effective earth pressure coefficient for precast concrete piles;

$\sigma'_o$  – effective vertical stress,  $kN/m^2$ ;

$\Delta L$  – thickness of soil layer, m.

The frictional factor is calculated by two following equations:

$$\text{For } z=0 \text{ to } L': f = K\sigma'_o \tan(\delta') \quad (4.3.1.32)$$

$$\text{For } z=L' \text{ to } L: f = f_{z=L'} \quad (4.3.1.33)$$

Both Meyerhof's and Briaud's method of calculating the frictional resistance of sand layers uses the following equation with different average unit frictional resistance:

$$Q_s = pLf_{ave} \quad (4.3.1.34)$$

Method 3: Meyerhof's method (using SPT values):

For this method, the average frictional resistance is calculated using the equation below:

$$f_{ave} = 0.02p_a(N_{60})_{ave} \quad (4.3.1.35)$$

Method 4: Briaud's method (using SPT values):

For this method, the average frictional resistance is calculated using the equation below:

$$f_{ave} = 0.224p_a((N_{60})_{ave})^{0.29} \quad (4.3.1.36)$$

Table 4.3.1.5. Frictional resistance in Sand for Pile lengths of 7-11 meters ( $d=0.4m$ )

Pile length, m	Depth, m	Layer	Coyle and Costello, kN	General equation, kN	Meyerhof, kN	Briaud, kN
7	10.55	Well-graded SAND with Silt and Gravel	2577.79	1542.69	824.67	601.35
8	11.55	Clayey SAND with Gravel	2596.89	2125.90	947.19	688.25
9	12.55	SAND with Silt and	3055.43	2428.35	1106.94	782.88

		Gravel				
10	13.55	SAND with Silt and Gravel	3580.91	2706.78	1229.93	869.87
11	14.55	Silty SAND	3592.47	2543.84	1343.42	954.90

The general skin friction formula method is a more reliable method of calculating the skin friction of sand layers due to a more accurate approach with the use of various soil characteristics and the subjectivity of the SPT values.

**The frictional resistance of Clay layers:**

The frictional resistance of clay could be determined by three methods:

$\alpha -$ ,  $\beta -$ ,  $\lambda -$  methods. The frictional resistance is calculated using three methods, namely  $\lambda -$ ,  $\alpha -$ ,  $\beta -$  methods, and the average of the three methods will yield the final frictional resistance of each clay layer.

**$\lambda$  method**

$$f_{av} = \lambda(\overline{\sigma'_0} + 2c_u) \tag{4.3.1.37}$$

$$Q_s = pL f_{av} \tag{4.3.1.38}$$

$\lambda$  – frictional coefficient dependent on the embedment length;

$\overline{\sigma'_0}$  – mean effective vertical stress for the entire embedment length;

$c_u$  – mean undrained shear.

**$\alpha$  method**

$$Q_s = \sum \alpha c_u p \Delta L \tag{4.3.1.39}$$

$\alpha$  - empirical adhesion factor;

$c_u$  - undrained shear strength.

**$\beta$  method**

$$Q_s = \sum (1 - \sin\phi'_R) \tan\phi'_R \sigma'_o * p * \Delta L \tag{4.3.1.40}$$

It is important to mention that the frictional resistance of the pile is the summation sum of skin frictions of both sand clay layers.

Table 4.3.1.6. Frictional resistance in Clay for Pile lengths of 12-17 meters (d=0.4m)

Pile Length, m	Depth, m	Layer	$\lambda$ method, kN	$\alpha$ method, kN	$\beta$ method, kN	$Q_{s, ave}$ , kN
12	15.55	Sandy CLAY	4015.01	3254.33	4601.51	3956.95
13	16.55	Sandy CLAY	4082.33	3291.49	4765.93	4046.58
14	17.55	Sandy CLAY	4146.93	3328.65	4938.13	4137.90
15	18.55	Silty CLAY	4184.14	3361.39	5473.42	4339.65
16	19.55	Silty CLAY	4210.97	3377.70	5660.81	4416.50
17	20.55	Silty CLAY	4246.95	3394.02	7041.61	4894.19

Combining all of the above-calculated values of frictional resistances, point-bearing capacities, and the factor of safety of 3, the following table sums up the calculation of the single pile foundation:

Table 4.3.1.7 Single pile foundation bearing capacity summary (d=0.4m)

Pile Length, m	Depth, m	Layer	$Q_p$ , kN	$Q_s$ , kN	$Q_u$ , kN	$Q_{all}$ , kN
7	10.55	Well-graded SAND with Silt and Gravel	3192.07	1542.69	4734.76	1578.25
8	11.55	Clayey SAND with Gravel	2183.44	2125.90	4309.34	1436.45
9	12.55	SAND with Silt and Gravel	2238.02	2428.35	4666.37	1555.46
10	13.55	SAND with Silt and Gravel	2162.59	2706.78	4859.37	1623.12
11	14.55	Silty SAND	2058.96	2543.84	4602.80	1534.27
12	15.55	Sandy CLAY	51.71	3956.95	4008.67	1336.22
13	16.55	Sandy CLAY	51.71	4046.58	4098.30	1366.10

14	17.55	Sandy CLAY	51.71	4137.90	4189.62	1396.51
15	18.55	Silty CLAY	17.39	4339.65	4357.04	1452.35
16	19.55	Silty CLAY	17.39	4416.50	4433.89	1477.96
17	20.55	Silty CLAY	17.39	4894.19	4911.59	1637.20

As it may be observed from the allowable bearing capacities of single piles of various lengths, the single pile foundation is not strong enough to carry the weight of the hotel, thus the group pile method should be considered as a foundation of the building.

### *Single Pile: Settlement*

The elastic settlement of the single pile is a summation of three settlement components:

- $Se_{(1)}$  - Elastic shortening of the pile;
- $Se_{(2)}$  - Settlement of pile due to working load at the pile point;
- $Se_{(3)}$  - Settlement of pile due to working load along the pile shaft.

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \quad (4.3.1.41)$$

$$S_{e(2)} = \frac{q_{wp} D}{E_s} (1 - \mu_s^2) I_{wp} \quad (4.3.1.42)$$

$$S_{e(3)} = \frac{Q_{ws} D}{pLE_s} (1 - \mu_s^2) I_{ws} \quad (4.3.4.43)$$

Since it was already confirmed that the single pile is not suitable to be chosen as this building's foundation, the following table of single pile calculation includes only 5 iterations:

Table 4.3.1.8. Single pile foundation settlement for pile length of 7-11 meters (d=0.4m)

Pile length, m	$Se_{(1)}$ , mm	$Se_{(2)}$ , mm	$Se_{(3)}$ , mm	$Se$ , mm
7	0.0009	0.0130	0.0033	0.0172
8	0.0012	0.0140	0.0054	0.0205
9	0.0015	0.0140	0.0057	0.0213
10	0.0018	0.0170	0.0059	0.0247
11	0.0019	0.0038	0.0055	0.0112

The given calculation resulted in a decision to select a group of piles as a foundation for the multi-story hotel as it was found to be suitable according to its bearing capacity and settlement values.

Group of piles consists of individual pile foundations that act as one when combined into a group. Although the foundations can be manufactured using various materials available in the market (steel, concrete, timber, or their combinations), for this project the concrete piles were chosen. Concrete piles are more cost efficient when compared to steel piles and more durable than timber piles. Taking into account the corrosive resistance and easier installation and joint connection techniques, concrete piles are the best option available in the market.

Regarding the installation technique of concrete piles, they can be cast on site or be prefabricated prior to installation. Precast driven piles were chosen for the design of the foundation. Bored piles were not chosen for this building due to their limitation in low load-bearing capacity and overall suitability for cohesive soils, opposite to the chosen site's soil type.

#### **4.3.2. Foundation Design under Axial Loading**

##### **4.3.2.1. Hand calculations of the axial bearing capacity**

In the previous part of the capstone work, it was found that the multi-story hotel building requires the group of piles to have enough bearing capacity. The following table provides the information on structural load of each pile group type.

Table 4.3.2.1.1. Vertical load summary for the hotel

Column type	Location	Design Load for the hotel, kN
C-1	Interior	13356.99
C-2	Exterior	6724.01
C-3	Corner	3407.52

To determine the configuration of the group of piles, it is important to calculate the efficiency of each configuration of the group of piles. The efficiency of the group will determine whether the group will act as a block pile or as a single pile.

$$\eta = \left[ \frac{2(n_1+n_2-2)d+4D}{\pi n_1 n_2} \right] \quad (4.3.2.1.1)$$

Where,

$n_1, n_2$  – number of piles in a group;

$d = 3.5D$  – center-to-center pile spacing (Das & Sivakugan, 2019);

$D$  - diameter of the pile;

$p$  – pile perimeter.

Additionally, if  $\eta < 1$ , then piles act as a single block with  $Q_{g(u)} = \eta \sum Q_u$  and if  $\eta \geq 1$ , then all of the piles are considered to be individual piles and the load bearing capacity is  $Q_{g(u)} = \sum Q_u$ . Lastly, the allowable bearing capacity is calculated using the load bearing capacity and factor of safety of 3.00.

$$Q_{g(all)} = \frac{Q_{g(u)}}{FS} \quad (4.3.2.1.2)$$

$Q_{g(all)}$  – allowable bearing capacity of the group pile;

$Q_{g(u)}$  – ultimate bearing capacity of the group pile.

After several iterations, the diameter of 0.4 meters was chosen for the diameter of exterior, interior, and corner columns, and the group of piles was found to act as a single pile according to the calculated efficiency values of:

$$\eta_{interior} = \left[ \frac{2(3+3-2) \times 1.4 + 4 \times 0.4}{\pi \times 0.4 \times 3 \times 3} \right] = 1.13$$

$$\eta_{exterior} = \left[ \frac{2(3+2-2) \times 1.4 + 4 \times 0.4}{\pi \times 0.4 \times 3 \times 2} \right] = 1.33$$

$$\eta_{corner} = \left[ \frac{2(2+2-2) \times 1.4 + 4 \times 0.4}{\pi \times 0.4 \times 2 \times 2} \right] = 1.43$$

For  $\eta \geq 1$ ,  $Q_{g(u)} = \sum Q_u$

For  $\eta < 1$ ,  $Q_{g(u)} = \eta \sum Q_u$

The pile dimensions for the group of piles is calculated per the minimum spacing between the piles and their diameter.

$$L_g = (n_1 - 1)d + 2(D/2) \quad (4.3.2.1.3)$$

$$B_g = (n_2 - 1)d + 2(D/2) \quad (4.3.2.1.4)$$

Configurations of the interior, exterior, and corner columns are provided in the table below:

Table 4.3.2.1.2. Group of piles dimensions summary

	Interior Column	Exterior Column	Corner Column
Pile diameter, m	0.4	0.4	0.4
Pile length, m	8	8	8
Pile spacing, m	1.40	1.40	1.40
$n_1$	3	3	2
$n_2$	3	2	2
$L_g$	3.2	3.2	1.8
$B_g$	3.2	1.8	1.8

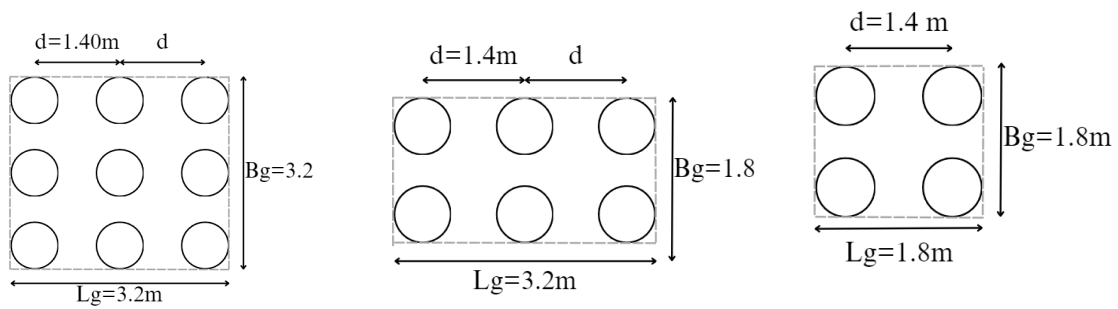


Figure 4.3.2.1.1. Group Pile configurations with calculated dimensions

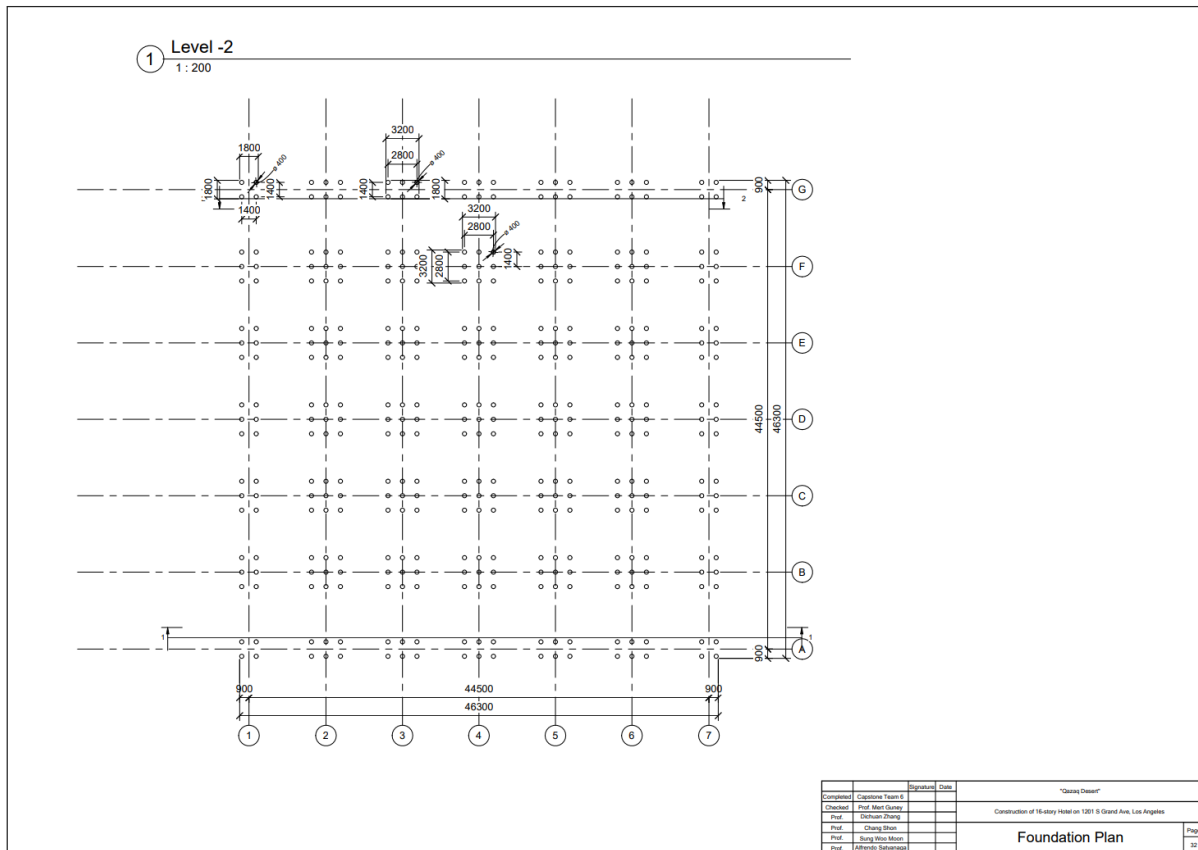


Figure 4.3.2.1.2. Technical drawing of the foundation plan

With the chosen pile diameters and lengths, the bearing capacity of the groups of piles is presented in the following table under each of the column types:

Table 4.3.2.1.3. Bearing capacity of the groups of piles

	Pile length, m	$Q_p$ , kN	$Q_s$ , kN	$Q_u$ , kN	$\Sigma Q_u$ , kN	$Q_{all}$ , kN	$Q_{design}$ , kN
Interior	8	3192.07	1542.69	4734.76	42612.88	14638.94	13356.99
Exterior	8	2183.44	2125.90	4309.34	28408.59	11436.67	11057.84
Corner	8	2183.44	2125.90	4309.34	18939.06	8234.40	8042.06

#### 4.3.2.2. Hand calculations of the settlement

Vesic's method for settlement calculations was used to determine the group pile's elastic settlement in response to the applied loads. Equation 4.3.2.2.1 shows that

according to Vesic's equation the elastic settlement for individual piles equals the sum of the elastic shortening (Equation 4.3.2.2.2), settlement due to working load at the point (Equation 4.3.2.2.3), and settlement due to working load along the pile shaft (Equation 4.3.2.2.4), whereas the settlement of group piles is calculated using Equation 4.3.2.2.2

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)} \text{ (individual piles)} \quad (4.3.2.2.1)$$

$$S_{g(e)} = \sqrt{\frac{B_g}{D}} \times S_e \text{ (group of piles)} \quad (4.3.2.2.2)$$

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws}) \times L}{A_p \times E_p} \quad (4.3.2.2.3)$$

Where,

$$Q_{wp} = \frac{Q_p / FS}{n_1 \times n_2} \text{ - working load at the pile point;}$$

$\xi$  - constant of 0.67;

$$Q_{ws} = \frac{Q_p / FS}{3} \text{ - working load at the pile point;}$$

L - Pile length;

$A_p$  - cross-section area of the pile;

$$E_p = 4700 \times \sqrt{f'_c} \text{ - modulus of elasticity of the pile material.}$$

$$S_{e(2)} = \frac{q_{wp} \times D}{E_s} \times (1 - \mu_s^2) \times I_{wp} \quad (4.3.2.2.4)$$

Where,

$$q_{wp} = \frac{Q_{wp}}{A_p} \text{ - working load at the pile point;}$$

D - pile diameter;

$E_s$  - modulus of elasticity of soil below the pile point;

$\mu_s$  - Poisson's Ratio of the soil;

$I_{wp}$  - influence factor.

$$S_{e(3)} = \frac{Q_{ws} \times D}{p \times L \times E_s} \times (1 - \mu_s^2) \times I_{ws} \quad (4.3.2.2.5)$$

Meyerhof's method can also be used for the calculations of the elastic settlement of a group of piles:

$$S_{g(e)} = \frac{0.96q\sqrt{B_g I}}{N_{60}} \quad (4.3.2.2.6)$$

$$q = Q_g / (L_g B_g) \quad (4.3.2.2.7)$$

$$I = 1 - L / 8B_g \geq 0.5 \quad (4.3.2.2.8)$$

Where,

Table 4.3.2.2.1. Elastic settlement for the group of piles

	L, m	Q <sub>g</sub> , kN	L <sub>g</sub> , m	B <sub>g</sub> , m	q, kN/m <sup>2</sup>	N <sub>60</sub>	I	S <sub>g</sub> , mm	S <sub>e</sub> , mm
Interior	8	8707.7	3.2	3.2	1570.7	48	0.727	30.006	15.097
Exterior	8	12193.2	3.2	1.8	2181.5	48	0.514	22.732	10.861
Corner	8	5805.16	1.8	1.8	2792.4	48	0.514	29.097	12.357

#### 4.3.2.3. Software analysis of pile groups

Pile reaction to the axial loading was tested using Plaxis 3D software with constructing the piles through embedded beams and modelling the pile caps with plates. Mohr-Coulomb model was used for building the soil profile and all of the necessary input values such as unsaturated unit weight, saturated unit weight, friction angle, and others were entered to the software, in accordance with the soil profile information.

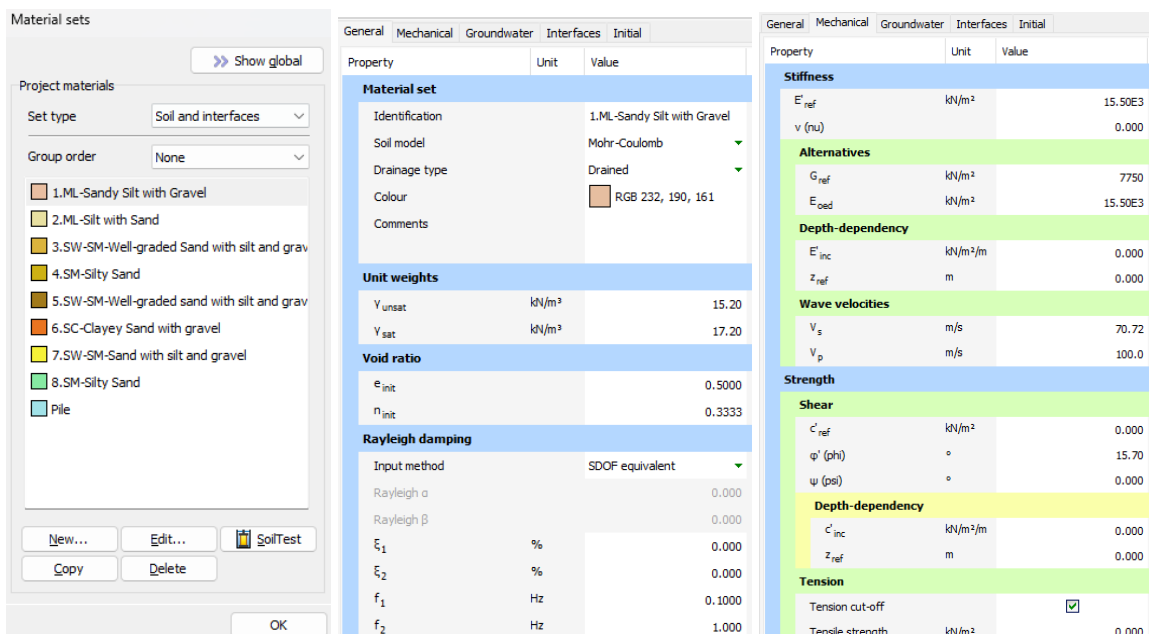


Figure 4.3.2.3.1. Soil data input in Plaxis 3D

Additionally, GEO5 software package was also used to perform the numerical analysis of the pile groups and, similarly to Plaxis 3D, all the necessary soil values were imputed into GEO5.

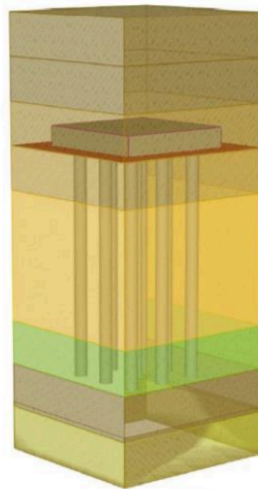


Figure 4.3.2.3.2. Example of pile group placed in soil in GEO5 (Interior pile group)

Load combination for each column type were calculated by structural engineers and are provided in the table below:

Table 4.3.2.3.1. Most critical load combinations for columns

Column type	Load Combination	Fx, kN	Fy, kN	Fz, kN	Mx, kN*m	My, kN*m	Mz, kN*m
Corner	1.2D+1.0E+L	381.98	40.76	162.45	-44.73	748.34	-0.97
Exterior	1.2D+1.0E+L	725.66	0.06	324.90	-0.06	796.27	0.00
Interior	1.2D+1.6L+0.5Lr	-0.08	0.12	13413.08	-0.11	-0.07	0.00

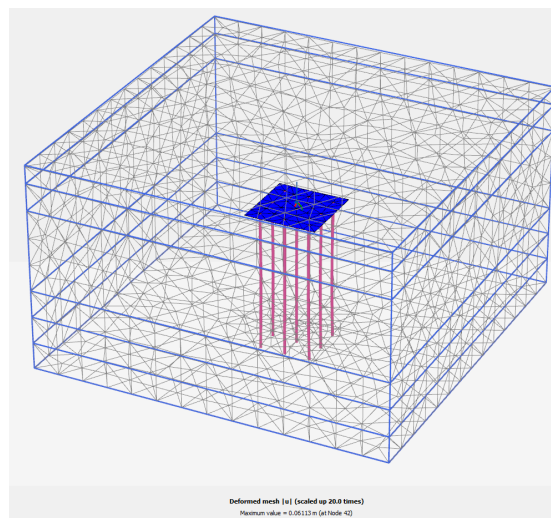


Figure 4.3.2.3.3. Example of pile group deformation mesh in Plaxis 3D (Interior pile group)

The following table compares the hand calculated settlement and software analysis settlement values of three types of piles. It can be observed that the settlement values of the GEO5 software were significantly lower than the Plaxis 3D values and hand calculation. This difference occurred due to the difference in the code calculations (Eurocode design code use causes lower values).

Table 4.3.2.3.2. Settlement values obtained by using various calculation methods

Column type	Pile configuration	Sg, mm	Se, mm	GEO5 Sg,mm	Plaxis 3D Sg, mm
Interior	3 x 3	30.006	15.097	10.800	61.130
Exterior	3 x 2	22.732	10.861	7.400	48.710
Corner	2 x 2	29.097	12.357	7.200	35.620

### 4.3.3. Foundation Design under Lateral Loading

#### 4.3.3.1. Hand calculations of the lateral bearing capacity

Piles frequently experience lateral loads in addition to axial forces. To address this, the most severe lateral loading scenarios were identified, and calculations were conducted to determine the lateral load acting on an individual pile. It was found that corner columns experienced the highest lateral load values. These lateral load results are presented in Table 4.3.3.1.1:

Table 4.3.3.1.1. Calculations of Lateral Load.

Column Type	Load Combination	$F_{x'}$ , kN	$F_{y'}$ , kN	Number of piles	Lateral load, kN
Corner	1.2D+1L+1Ey	-0.652	510.784	4	127.53
Exterior		25.121	495.855	9	57.87
Interior		7.901	436.577	9	49.39

The constant modulus of subgrade reaction ( $\eta_h$ ) and the subgrade modulus at a particular depth ( $z$ ,  $k_z$ ) were determined, with their values presented in Table 4.3.3.2.

Table 4.3.3.1.2 Calculation of constant modulus of subgrade and subgrade modulus.

Layer	Layer depth, m	Soil Type	$\eta_h$	$k_z$
-------	----------------	-----------	----------	-------

1	1.52	Sandy Silt with Gravel	6250	9500
2	1.52	Silt with Sand	6250	19000
3	1.52	Well-graded Sand with Silt and Gravel	6250	28500
4	1.52	Silty Sand	16500	53580
5	4.57	Well-graded Sand with Silt and Gravel	16500	128985
6	1.52	Clayey Sand with Gravel	16500	154065
7	1.52	Sand with Silt and Gravel	16500	179145
8	1.52	Silty Sand	16500	204225

The lateral bearing capacity of the pile was determined according to Brom's methodology (Das & Sivakugan, 2019). Initially, an elastic approach was applied for categorizing the piles as either rigid or long. To distinguish between short and long piles, the characteristic length was computed based on the following expression:

$$T = \sqrt[5]{\frac{E_p I_p}{\eta_h}} \quad (4.3.3.1.1)$$

Where,

$E_p$  – piles' modulus of elasticity, MPa;

$I_p$  – pile's moment of inertia,  $m^4$ ;

$\eta_h$  – horizontal subgrade reaction's constant modulus.

The pile's modulus of elasticity was determined by employing the following formula:

$$E_p = 4700 \sqrt{f'_c} \quad (4.3.3.1.2)$$

Where,

$f'_c$  – represents the concrete compressive strength, taken as 35 MPa, which corresponds to the minimum required compressive strength for precast concrete piles used in areas designated as seismic design category D (LA Building Code, 2022).

The moment of inertia was identified using the following formula:

$$I_p = \frac{\pi D^4}{64} \quad (4.3.3.1.3)$$

The constant modulus of horizontal subgrade reaction ( $\eta_h$ ) was determined to be 16500, a typical value for dense, dry sand conditions (Das & Sivakugan, 2019).

$$T = \sqrt[5]{\frac{27805.6 \cdot 10^3 \cdot 0.0031}{16500}} = 1.392 \text{ m}$$

The pile length for each loading scenario and column type was established at 8 meters. Given that (L) exceeds (5T), these piles fall into the category of long piles.

To proceed with evaluating the lateral bearing capacity, the Rankine passive earth pressure coefficient was first determined:

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right) \quad (4.3.3.1.4)$$

After that, pile's section modulus was calculated with the formula:

$$S = \frac{\pi D^3}{32} \quad (4.3.3.1.5)$$

The pile's yield moment was obtained next:

$$M_y = SF_y \quad (4.3.3.1.6)$$

Where,

$F_y$  – pile material's yield stress, MPa.

$$M_y = 35 \cdot 10^3 \cdot 0.0135 = 472.5 \text{ kN} \cdot \text{m} \text{ for the interior columns.}$$

The ultimate lateral resistance value was determined by referencing the graph shown below:

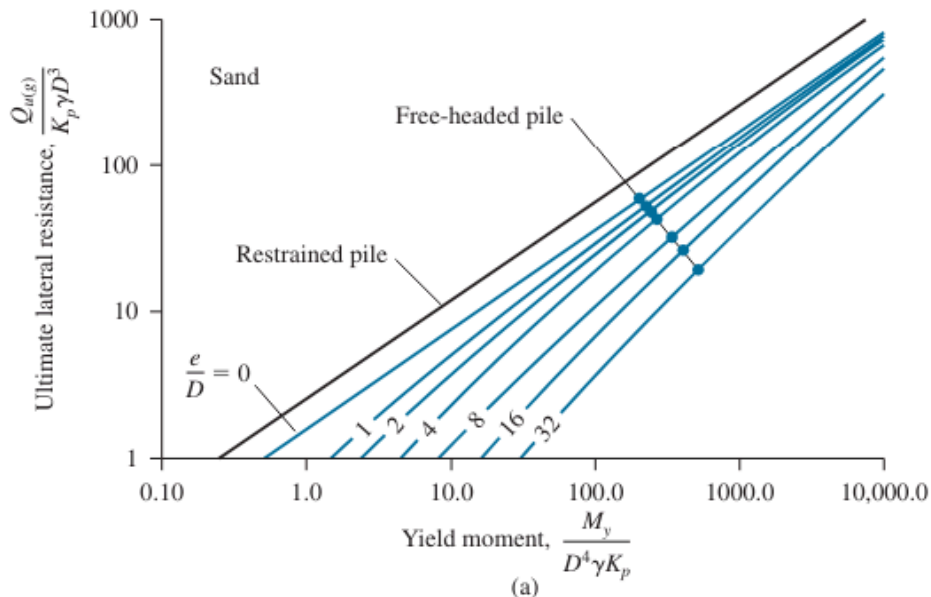


Figure 4.3.3.1.1. Ultimate lateral resistance values along yield moment for long piles in sandy soils (Das & Sivakugan, 2019).

So, as the value of  $Q_{ug}$  is determined from the figure,  $Q_{all}$  can be obtained with the following formula:

$$Q_{all} = \frac{Q_{ug}}{FS} \quad (4.3.3.1.7)$$

The factor of safety was determined as 3 for the following calculations (Das & Sivakugan, 2019).

The calculated lateral load-bearing capacities of piles for various column types are summarized and presented in Table 4.3.3.3 below.

Table 4.3.3.1.3. Comparison of Calculated Lateral Load-Bearing Capacities and Applied Column Lateral Loads

Column type	$Q_{ug}$ , kN	$Q_{all}$ , kN	$Q_{lateral}$ , kN
Interior	593.75	197.92	49.39
Exterior	397.53	132.51	57.87
Corner	472.84	157.61	127.53

As indicated by Table 4.3.3.1.3, the lateral bearing capacity of each pile exceeds the applied lateral load. Therefore, the design can be deemed acceptable.

#### 4.3.3.2 Hand calculations and software analysis of the deflection

Lateral deflection was evaluated using an elastic approach as described by Das and Sivakugan (2019). Initially, the deflection at the pile head was computed using the equation provided below:

$$\eta = \sqrt[5]{\frac{\eta_h}{E_p I_p}} \quad (4.3.3.2.1)$$

The deflection at any given depth along the pile was then determined using the subsequent equation:

$$x_z(z) = A_x \frac{Q_g T^3}{E_p I_p} + B_x \frac{M_g T^2}{E_p I_p} \quad (4.3.3.2.2)$$

Coefficients  $A_x$  and  $B_x$  were acquired from tabulated data presented by Das and Sivakugan (2019). The resulting deflections at various depths are summarized in Table 4.3.3.2.1

Table 4.3.3.2.1. Individual pile's Lateral Deflection.

Layer number	Depth, m	$A_x$	$B_x$	$x_z$ (corner) mm	$x_z$ (interior) mm	$x_z$ (exterior) mm
0	1.52	2.435	1.623	28.03	25.89	27.78
1	1.52	0.962	0.364	10.01	8.96	9.89
2	1.52	0.142	-0.07	0.49	0.46	0.48
3	1.52	-0.075	-0.089	-1.15	-1.09	-1.12
4	1.52	-0.05	-0.028	-0.62	-0.52	-0.59
5	4.57	-0.009	0	-0.07	-0.05	-0.06
6	1.52	0	0	0	0	0
7	1.52	0	0	0	0	0
8	1.52	0	0	0	0	0

Figure 4.3.3.2.1. below shows the pile's deflection values.

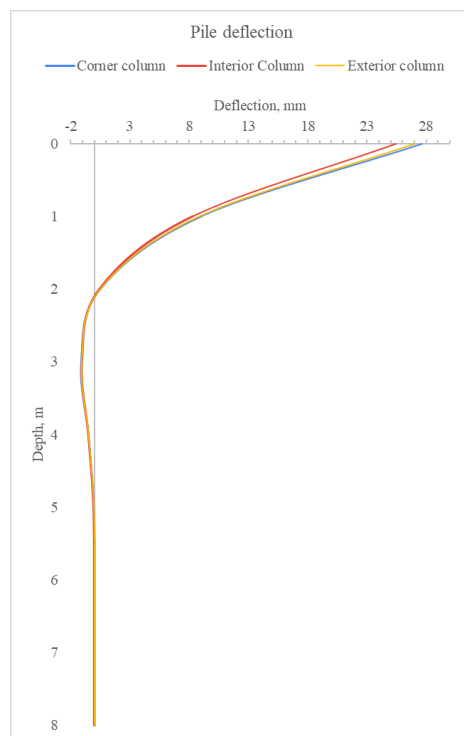


Figure 4.3.3.2.1. Values of deflection for each pile.

The pile head deflection was determined using Brom's approach. Initially, the dimensionless length was computed as follows:

$$\text{Dimensionless length} = \eta \cdot L \quad (4.3.3.2.3.)$$

Subsequently, the deflection value was obtained from the graphical representation provided in Figure 4.3.3.2.2.

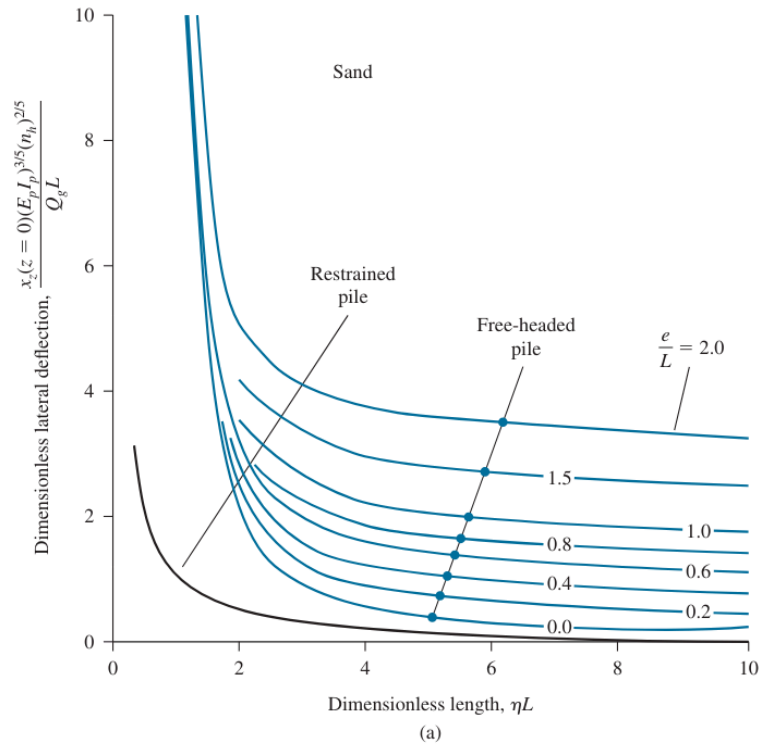


Figure 4.3.3.2.2. Values of Deflection at the pile head for long piles located in sand (Brom's method).

Moreover, pile head deflections were calculated using LPile software (Excel version). The computed deflection values at the pile head are summarized in Table 4.3.3.2.2.

Table 4.3.3.2.2. Values of pile head's deflection.

Deflection at the pile head ( $z = 0$ ), mm								
Corner Column			Interior Column			Exterior Column		
Elastic	Brom's	LPile	Elastic	Brom's	LPile	Elastic	Brom's	LPile
28.03	12.97	24.84	25.89	14.02	22.59	27.78	12.07	24.11

The pile deflection results obtained from all analytical and numerical methods remained within acceptable limits. Overall, the displacement values did not surpass the permissible threshold, indicating that the design meets performance requirements.

#### 4.3.4. Group Pile Design

A group of multiple piles is linked together by a structural component known as a pile cap, which plays a crucial role in transferring structural loads into the soil. Concrete alone lacks sufficient strength to effectively resist bending and shear forces, making it necessary to incorporate reinforcement. The reinforcing details for both the piles and the pile cap were designed following guidelines provided by Ray (1994).

##### 4.3.4.1. Pile Cap Reinforcement

Initially, the dimensions of the pile cap are determined using the equation below. Since the piles in the group are configured in a square pattern, the length and width dimensions are identical, hence  $L = B$ :

$$L = B = d(n_{1,2} - 1) + 2 * 1.5D \quad (4.3.4.1.1)$$

$$h_{min} = 1.5D \quad (4.3.4.1.2)$$

The loading analysis applied to the pile cap is summarized in the table provided. The assumed unit weights were  $24 \text{ kN/m}^3$  for concrete and  $17.2 \text{ kN/m}^3$  for backfill material.

Table 4.3.4.1.1. The load analysis for the interior group of piles.

DL ( $\text{kN/m}^2$ )	LL ( $\text{kN/m}^2$ )	$W_{pc}$ ( $\text{kN/m}^2$ )	$W_{bf}$ ( $\text{kN/m}^2$ )	$W_{sc}$ ( $\text{kN/m}^2$ )
155.32	114.87	$h * \gamma_{concr} = 18$	$h_{bf} * \gamma_{soil} = 8.6$	270.45

The load combinations outlined below were utilized in the structural design:

$$LC_1 = 1.2D + 1.6L$$

$$LC_2 = 1.2D + 1L + 1W$$

$$LC_3 = 1.2D + 1L + 1E_x$$

$$W = W_{pc} + W_{bf} + W_{load} = 18 + 8.6 + 270.45 = 297.05 \text{ kN/m}^2 \quad (4.3.4.1.3)$$

Load factors used:  $1.2W = 356.46 \text{ kN/m}^2$  and  $1.4W = 415.87 \text{ kN/m}^2$

$$\text{for } 1.2W: M'_{11} = M'_{22} = \frac{4.5 * 356.46 * 1.925^2}{2} = 2972 \text{ kN} - m \quad (4.3.4.1.4)$$

$$\text{for } 1.4W: M'_{11} = M'_{22} = \frac{4.5 * 415.87 * 1.925^2}{2} = 3467.38 \text{ kN} - m \quad (4.3.4.1.5)$$

The reaction forces were calculated using the following equations:

$$Q = \frac{P}{R} \pm \frac{M_{xx}y}{I_{xx}} \pm \frac{M_{yy}x}{I_{yy}} \quad (4.3.4.1.6)$$

Where,

$P = (1.2 \text{ or } 1.4)N + W_{pc} + W_{bf}$  - vertical load on the group piles;

$M_{xx} = M_x + Ne_y + H_y h + M_x^*$  - moment of x-axis;

$M_{yy} = M_y + Ne_x + H_x h + M_y^*$  - moment of y-axis;

$I_{xx} = \sum y^2$  about x-axis ( $y=1.5$  m);

$I_{yy} = \sum x^2$  about y-axis ( $x=1.5$  m).

The method used to calculate bending moments in the pile cap arising from the reaction forces of piles is detailed as follows:

$$M''_{11} = 1.4(Q_3 + Q_4) = 1.4(1322.11 + 1324.25) = 3704.9 \text{ kNm} \quad (4.3.4.1.7)$$

$$M''_{22} = 0.5(Q_1 + Q_2 + Q_3) = 1984.5 \text{ kNm} \quad (4.3.4.1.8)$$

The combined bending moments were found to be:

$$M_{11} = M'_{11} + M''_{11} = 3467.38 + 3704.9 = 7172.28 \text{ kNm}$$

$$M_{22} = M'_{22} + M''_{22} = 3467.38 + 1984.5 = 5451.88 \text{ kNm}$$

For reinforcement design, bars with diameters of 32 mm and 25 mm were selected for the x and y directions, respectively. Concrete with a compressive strength of 35 N/mm<sup>2</sup> was chosen, while the yield strength of the reinforcing steel bars was set at 460 N/mm<sup>2</sup>. A minimum concrete cover of 75 mm was specified according to ACI-318-19 standards for structures continuously exposed to the ground. Consequently, the reinforcement area for the pile cap was calculated using the following method:

$$d_x = h_{pile\ cap} - cover - 0.5d_{bar} = 750 - 75 - 0.5 * 32 = 663 \text{ mm} \quad (4.3.4.1.9)$$

$$K = \frac{M_{11}}{f_{cu} b d^2} = \frac{7172.28}{35 * 3000 * 663^2} = 0.155 \quad (4.3.4.1.10)$$

$$z = d(0.5 + \sqrt{(0.25 - \frac{K}{0.9})}) \leq 0.95d \quad (4.3.4.1.11)$$

$$z = 663(0.5 + \sqrt{0.25 - \frac{0.155}{0.9}}) = 516 \leq 0.95d = 629.85$$

$$A_{st} = \frac{M_{11}}{0.87f_y z} = \frac{7172.28}{0.87 * 460 * 516} = 34732.05 \text{ mm}^2 \quad (4.3.4.1.12)$$

$$\#of\ bars = \frac{A_{st}}{A_{bar}} = \frac{31640.74}{804.2} = 43.2 \quad (4.3.4.1.13)$$

$$Spacing = \frac{4500\ mm}{44} = 102.27\ mm$$

**use 44 #34 mm at 102.5 mm spacing**

Similar procedures were carried out for determining the reinforcement design in the y-direction of the pile cap:

$$d_y = h_{pile\ cap} - cover - 0.5d_{bar} = 755 - 75 - 32 - 0.5 * 26 = 635\ mm$$

$$K = \frac{M_{11}}{f_{cu}bd^2} = \frac{7172.28}{35*4500*630.5^2} = 0.114$$

$$z = d(0.5 + \sqrt{(0.25 - \frac{K}{0.9})}) \leq 0.95d$$

$$z = 635(0.5 + \sqrt{(0.25 - \frac{0.114}{0.9})}) = 540.50 \leq 0.95d = 603.25$$

$$A_{st} = \frac{M_{11}}{0.87f_y z} = \frac{7172.28}{0.87*460*540.50} = 22610.51\ mm^2$$

$$\#of\ bars = \frac{A_{st}}{A_{bar}} = \frac{22610.51}{491.44} = 46$$

$$Spacing = \frac{4500\ mm}{46} = 97.83\ mm$$

**use 46 #26 mm at 97.83 mm spacing**

Reinforcement design calculations specific to each pile in the three-pile group are detailed in the subsequent table.

Table 4.3.4.1.2. The pile cap design reinforcement.

Group Piles	$h$ , mm	$d_x$ , mm	$d_y$ , mm	Design in x	Design in y
Exterior	610	510	481.5	44 #34mm @ 92mm	42 #26mm @ 90mm
Interior	755	663	635	44 #34mm @ 105mm	46 #26mm @ 98mm
Corner	670	588	557.5	40 #34mm @ 74mm	20 #26mm @ 158mm

#### 4.3.4.2. Single Pile Reinforcement

Every pile must be properly reinforced to withstand the loads it experiences. The first step involves evaluating the pile's slenderness. If the pile is classified as slender, its design moment needs to be increased accordingly.

$$l_e = \beta l_o = 1.2 * 9 = 10.8 \text{ m} \quad (4.3.4.2.1)$$

Where,

$l_o$  - unsupported length of the pile (m);

$l_e$  - effective length (m);

$\beta = 1.2$  - for piles (fixed by a pile cap).

$$\frac{l_e}{h} = \frac{10.8}{0.5} = 21.6 > 10 \rightarrow \text{slender pile} \quad (4.3.4.2.2)$$

$$a = \frac{l_e^2}{2000h} K = \frac{9.0^2}{2000*0.5} * 1 = 0.117 \text{ m} \quad (4.3.4.2.3)$$

$$M_{add} = Q_{min} * a = 1175.62 \text{ kN} * 0.117 \text{ m} = 137.55 \text{ kN} - \text{m} \quad (4.3.4.2.4)$$

$$M_{mag} = M + M_{add} = 105.2 + 137.55 = 242.75 \text{ kN} - \text{m} \quad (4.3.4.2.5)$$

$$e = \frac{M_{mag}}{Q_{min}} = \frac{242.75}{1175.62} = 0.206 \text{ m} \quad (4.3.4.2.6)$$

$$\frac{e}{R} = \frac{0.206}{0.25} = 0.824 \quad (4.3.4.2.7)$$

$$\frac{M}{h^3} = \frac{242.75}{0.5^3} = 1.942 \text{ kN/m}^2 \quad (4.3.4.2.8)$$

$$k = \frac{h_s}{h} = 0.75$$

A minimum reinforcement ratio of 1.6% is required to meet the structural demands:

$$A_{st} = \frac{A_c * 1}{100} = \frac{\pi * 0.25^2 * 1.6}{100} = 3142 \text{ mm}^2 \quad (4.3.4.2.9)$$

Use **12 #20mm** bars (3769.92 mm<sup>2</sup>)

To assess whether shear reinforcement is necessary, a specific verification procedure must be conducted:

$$\frac{M_{max}}{Q_{max}} = \frac{242.75}{1536.44} = 0.158 < 0.6h = 0.3 \rightarrow \text{no shear check is necessary} \quad (4.3.4.2.10)$$

The parameters used for designing and reinforcing an individual pile with a length of 9 meters, within a group of three piles, are summarized in the following table. A concrete cover of 75 mm was adopted.

Table 4.3.4.2.1. The reinforcement specifications for a single pile.

Group Piles	D, m	$l_e/h$	$M_{add}$	$M_{mag}$	$M/h^3$	$\rho$ , %	$A_{sc}$ , mm <sup>2</sup>	Design

Exterior	0.4	26	180.14	188.21	3.22	3.1	4398	14 #20 mm
Interior	0.4	20	137.52	141.14	1.22	1.9	3170	12 #20 mm
Corner	0.4	22	258.22	271.19	3.12	3.3	4771	16 #20 mm

The reinforcement layout for piles located at the exterior, interior, and corner positions within the group was evaluated using Geo5 software.



Figure 4.3.4.2.1 The reinforcement patterns for single piles depend on their group location (exterior, interior, or corner).

#### 4.3.4.3. Software Analysis of Pile Groups

Plaxis 3D Software was used to analyze the pile deformations under the effect of an axial and lateral loadings. The layout of the soil and building foundation was built as shown in the Figures 4.3.4.3.1 and Figure 4.3.4.3.2. with all the necessary pile caps and sheets piles included and the piles themselves being built by using embedded beams. This resulted in a deformation of 15.23 mm and the cross-section of the deformed mesh in Plaxis 3D is shown in Figure 4.3.4.3.3.

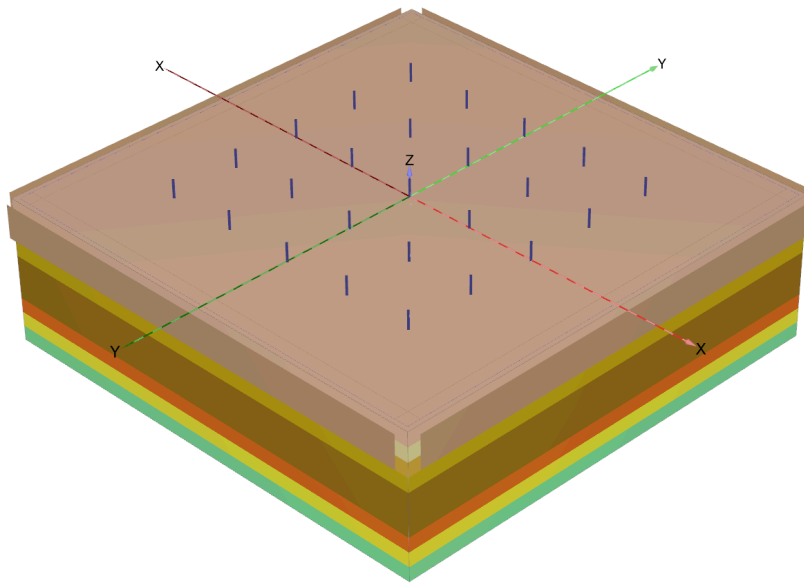


Figure 4.3.4.3.1. Soil Layout in Plaxis 3D

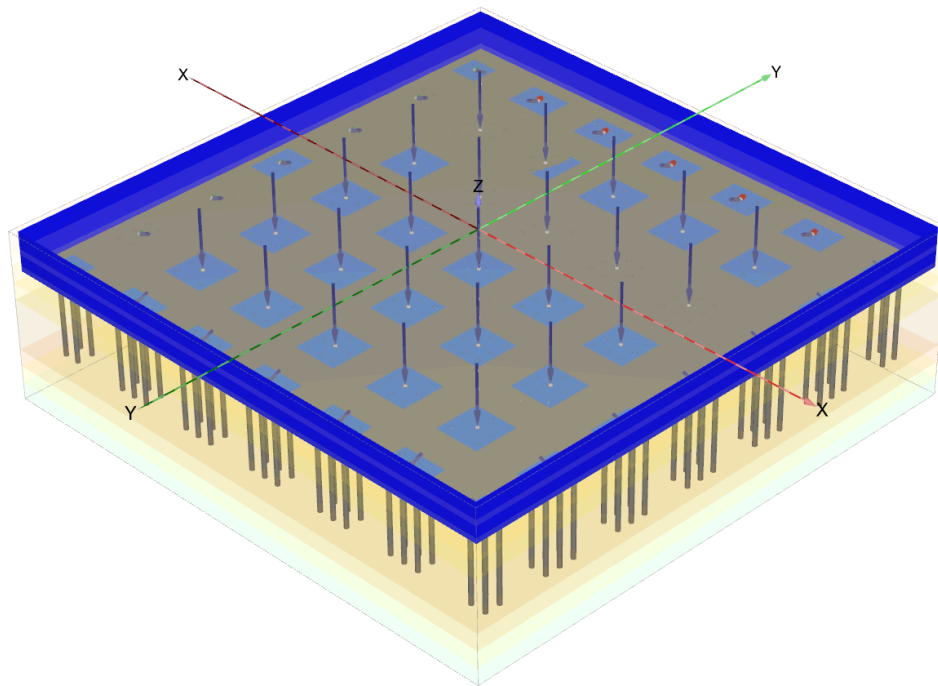


Figure 4.3.4.3.2. Pile group Layout in Plaxis 3D

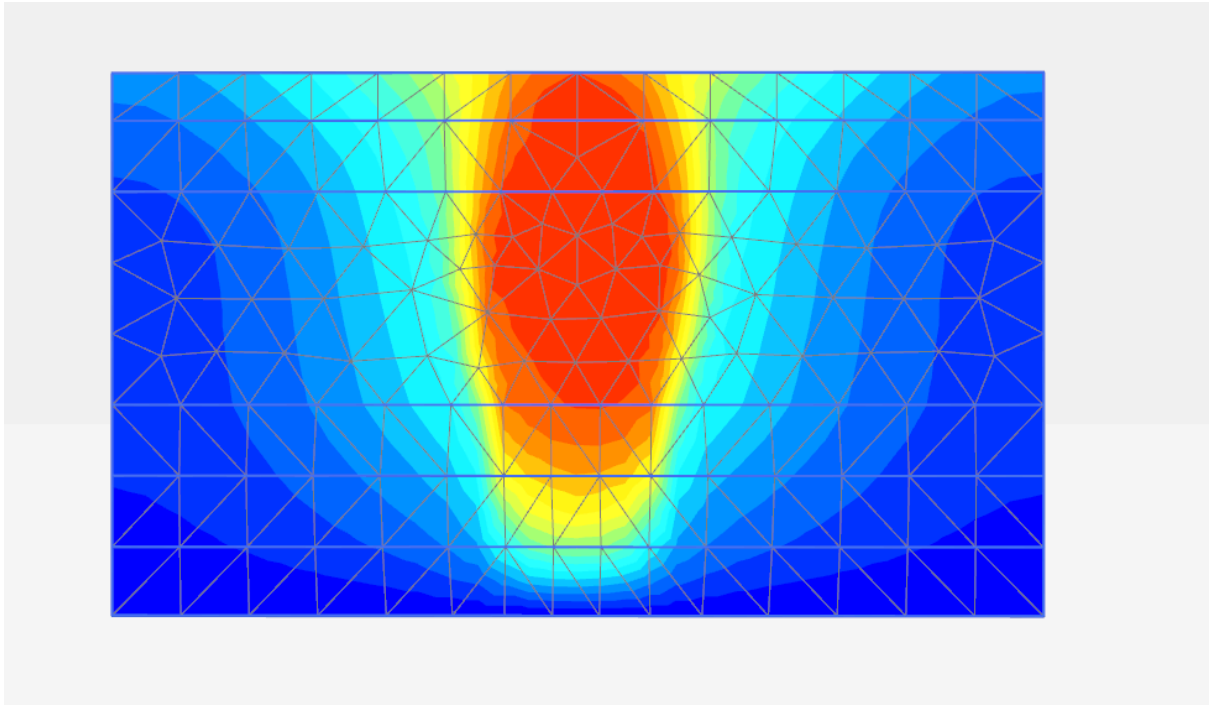


Figure 4.3.4.3.3. Cross-section of the deformed mesh in Plaxis 3D

#### 4.4. Sheet Pile Design

To support the lateral load that is put in the area of the basement of the building, the sheet piles were chosen to be lateral load-bearing structures. Although several material types, such as wood, precast concrete, steel, or aluminum, could be used for sheet-pile manufacture, precast sheet piles were chosen for the current project due to their ability to withstand permanent stress to which the structure will be subjected (Das & Sivakugan, 2019).

##### 4.4.1 Hand calculations of sheet piles

To calculate the dimensions of piles we will start with Rankine's active and passive earth pressure coefficients:

$$K_a = \tan^2\left(45 - \frac{\varphi'}{2}\right) \quad (4.4.1.1)$$

$$K_p = \tan^2\left(45 + \frac{\varphi'}{2}\right) \quad (4.4.1.2)$$

The basement floor is designed to be 3.5m in depth, so it will be the minimum length of the sheet pile. For the following calculations, we will use  $\gamma = 17.5 \text{ kN/m}^3$  and  $\varphi' = 37^\circ$ . So:

$$K_a = \tan^2\left(45 - \frac{37}{2}\right) = 0.25$$

$$K_p = \tan^2\left(45 + \frac{37}{2}\right) = 4.02$$

$K_{p(\text{design})}$  is calculated taking into account the factor of safety:

$$K_{p(\text{design})} = \frac{K_p}{FS} \quad (4.4.1.3)$$

$$K_{p(\text{design})} = \frac{4.02}{2} = 2.01$$

As there is no water table in the designated area we calculate active pressure at a depth of 3.5m:

$$\sigma'_2 = \gamma L K_a \quad (4.4.1.4)$$

$$\sigma'_2 = 17.5 \cdot 3.5 \cdot 0.25 = 15.23 \text{ kN/m}^2$$

The depth  $L_3$  where the net pressure becomes zero was calculated as shown below:

$$L_3 = \frac{L K_a}{K_{p(\text{design})} - K_a} \quad (4.4.1.5)$$

$$L_3 = \frac{3.5 \cdot 0.25}{2.01 - 0.25} = 0.49 \text{ m}$$

We used the following formula to calculate the net pressure  $\sigma'_5$ :

$$\sigma'_5 = \gamma L K_{p(\text{design})} + \gamma L_3 (K_{p(\text{design})} - K_a) \quad (4.4.1.6)$$

$$\sigma'_5 = 17.5 \cdot 3.5 \cdot 2.01 + 17.5 \cdot 0.49 \cdot (2.01 - 0.25) = 138.34 \text{ kN/m}^2$$

Then the area of the pressure diagram is calculated:

$$P = \frac{1}{2} \sigma'_2 L + \frac{1}{2} \sigma'_5 L_3 \quad (4.4.1.7)$$

$$P = \frac{1}{2} \cdot 15.56 \cdot 3.55 + \frac{1}{2} \cdot 15.56 \cdot 0.49 = 30.40 \text{ kN/m}$$

The centroid,  $\bar{z}$ , of this pressure diagram is now calculated:

$$\bar{z} = \frac{L(2K_a - K_{p(\text{design})})}{3(K_{p(\text{design})} - K_a)} \quad (4.4.1.8)$$

$$\bar{z} = \frac{3.5(2 \cdot 0.25 - 2.01)}{3(2.01 - 0.25)} = 1.63 \text{ m}$$

Then, the values of A are calculated using these formulas:

$$A'_1 = \frac{\sigma'_5}{\gamma(K_{p(\text{design})} - K_a)} \quad (4.4.1.9)$$

$$A'_2 = \frac{8P}{\gamma(K_p - K_a)} \quad (4.4.1.10)$$

$$A'_3 = \frac{6P[2\bar{z}\gamma(K_p - K_a) + \sigma'_5]}{\gamma^2(K_p - K_a)^2} \quad (4.4.1.11)$$

$$A'_4 = \frac{P[6\bar{z}\sigma'_5 + 4P]}{\gamma^2(K_p - K_a)^2} \quad (4.4.1.12)$$

Values of  $L_4$  and  $D_{theory}$  are calculated using:

$$L_4^4 + A'_1 L_4^4 - A'_2 L_4^2 - A'_3 L_4 - A'_4 = 0 \quad (4.4.1.13)$$

$$D_{theory} = L_3 + L_4 \quad (4.4.1.14)$$

According to Das and Sivakugan (2019), the actual depth of penetration has increased by 20-30%. For the next, we calculate:

$$\sigma'_3 = L_4(K_p - K_a)\gamma \quad (4.4.1.15)$$

$$\sigma'_4 = \sigma'_5 + \gamma L_4(K_p - K_a) \quad (4.4.1.16)$$

$$L_5 = \frac{\sigma'_3 L_4 - 2P}{\sigma'_3 + \sigma'_4} \quad (4.4.1.17)$$

$$z' = \sqrt{\frac{2P}{\gamma(K_p - K_a)}} \quad (4.4.1.18)$$

$$M_{max} = P(\bar{z} + z') - \left[ \frac{1}{2} \gamma z'^2 (K_p - K_a) \right] \left( \frac{1}{3} \right) z' \quad (4.4.1.19)$$

Section modulus of the sheet pile is finally calculated with:

$$S = \frac{M_{max}}{\sigma_{all}} \quad (4.4.1.20)$$

where the allowable stress,  $\sigma_{all}$ , was used  $170 \text{ MN/m}^2$  due to the basic grade of ASTM A-328, which is assumed to satisfy the working stresses.

The following table represents the results of these calculations according to the construction building:

Table 4.4.1.1. Results of calculation for sheet pile design

Variable	Value
$A'_1$	4.178
$A'_2$	7.92
$A'_3$	46.14
$A'_4$	47.89
$L_4$ , (m)	3.31
$L_5$ , (m)	0.80
$\sigma'_3$ , ( $kN/m^2$ )	105.78
$\sigma'_4$ , ( $kN/m^2$ )	258.74
$z'$ , (m)	1.38
$D_{theory}$ , (m)	3.79
$M_{max}$ , ( $kN \cdot m/m$ )	80.56
$S$ , ( $m^3/m$ )	$4.69 \cdot 10^{-4}$

As a result, from the final section of modulus of  $4.69 \cdot 10^{-4} \text{ m}^3/\text{m}$ , a dimensions of the sheet pile can be estimated:

Section designation: PZ-22

$H = 235.0 \text{ mm}$

$L = 558.8 \text{ mm}$

$f = 9.53 \text{ mm}$

$w = 9.53 \text{ mm}$

#### 4.4.2. Software Analysis for Sheet Pile Design

Here, a comprehensive analysis was made using Geo5 software to calculate the sheet pile's bending moments, shear forces, and dimensions. Figure 4.4.2.1. shows the final analysis of the software where the bending moment was 76.01 kNm/m and the shear force was 129.23 kN/m. The material chosen for the sheet pile was S235. No inclination is considered on top of the surface of a terrain as well as the water table, noted by design. For the sheet pile cross-section, I an type was used.

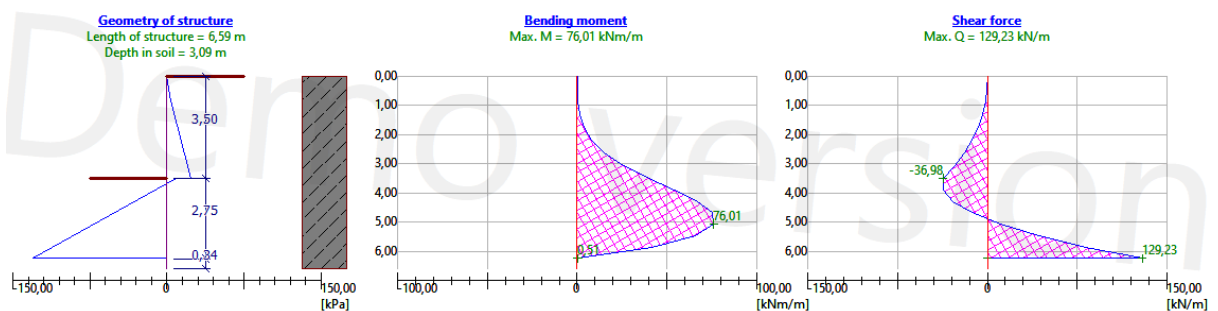


Figure 4.4.2.1. Geometry, Bending Moment, and Shear Force analysis of the sheet pile

Max. value of shear force	= 129,23 kN/m
Max. value of moment	= 76,01 kNm/m
Required depth of structure in soil	= 3,09 m
Overall length of structure	= 6,59 m

Figure 4.4.2.2. Results of main sheet pile analysis

Overall, Figure 4.4.2.3 shows that this analysis was satisfactory with the given conditions of soil and properties of the sheet pile.

Results	
<b>BENDING :</b>	<b>SATISFACTORY</b> (53,9%)
<b>SHEAR :</b>	<b>SATISFACTORY</b> (33,9%)

Figure 4.4.2.3. Satisfaction analysis results

In addition, the Bishop method was chosen for the analysis of the slope stability satisfaction. Figure 4.4.2.4. Presents positive results for the analysis of slope:

Slope stability verification (Bishop)	
Sum of active forces :	$F_a = 310,61 \text{ kN/m}$
Sum of passive forces :	$F_p = 883,79 \text{ kN/m}$
Sliding moment :	$M_a = 2792,38 \text{ kNm/m}$
Resisting moment :	$M_p = 7222,96 \text{ kNm/m}$
Utilization :	38,7 %
<b>Slope stability ACCEPTABLE</b>	

Figure 4.4.2.4. Results of slope stability analysis

#### 4.5. Foundation Construction Procedure

For a successful installation of the piles as a foundation of the building, there is a need to prepare the site for a construction procedure. It is important to mention that all of the preparation on the site should comply with the timeline requirements and agree with the construction engineer's plan of the project. Site preparation consists of the 7 following steps:

- 1) Initial site preparation step: this includes the clearing of the site from former building structures, if there are any, and clearing the site from the construction and demolition waste;
- 2) Geotechnical site surveying step: this step is carried out to ensure that the prior geotechnical investigation, if there is any, is accurate and applicable for a new building construction, and to carry out the zoning of the construction site;
- 3) Site soil testing step: general soil characterization is done and this data will be further used to check its compatibility with future foundation type;
- 4) Site investigation step: in-depth collection of geotechnical data for the future foundation design;
- 5) Sheet pile installation step: sheet piles are driven into the soil to the depth calculated based on the depth of the basement of a future building and are interlocked with each other until the whole perimeter of the building is covered;
- 6) Upper layer soil excavation step: upper layers of the soils are excavated in accordance with the future footings or basement design;

7) Pile Installation step: driven piles are installed into the soil via the use of either diesel hammer device or adjustable hydraulic device.

Overall quality of the foundation construction procedure should always be checked by construction engineers and managers and supervision by the geotechnical engineers should also be kept during construction works.

## **5. Environmental Engineering**

### **5.1. Regulations and Policies**

To create a storm sewer system for a 17-story hotel in a designated area (at 1201 South Grand Avenue in Los Angeles (LA)), several local regulations and policies must be considered. These regulations aim to ensure proper stormwater management, reduce pollution, and mitigate flood risks. Below is a general outline of the several key regulations that should be considered for compliance.

#### ***Low Impact Development (LID) Requirements***

One of the most important regulations is LA's Low Impact Development Ordinance, which applies to new construction and redevelopment projects. The goal is to reduce stormwater runoff and promote on-site infiltration and retention of water. The LID plan should include details on how the project will capture and manage the first 3/4 inches of rainfall. The LID ordinance encourages the use of Best Management Practices (BMPs) like rain gardens, permeable pavements, bioswales, and infiltration basins to manage stormwater on-site.

Based on the given regulations, water-permeable materials/pavement are going to be used for around 50 percent of the designated area. Water-permeable pavements are designed with pores, an open structure, or partially permeable materials to allow water to flow through or around them and into the underlying soil. This offers several benefits, including enabling rainwater to soak into the ground, replenishing groundwater supplies, and reducing the burden on sewer systems.

#### ***National Pollutant Discharge Elimination System (NPDES)***

Depending on the size of our project (110\*45= 4950 ) and the potential for stormwater pollution, we need an NPDES permit from the Los Angeles Regional Water Quality Control Board. Compliance with federal, state, and local water quality standards, particularly for projects near water bodies or those with significant runoff potential should also be considered.

#### ***Regional Planning and Building Codes***

Compliance with LA Building Codes, and the Los Angeles County Flood Control District guidelines related to water management and flood control. In addition, public works coordination such as the Los Angeles Department of Public Works and potentially LA Sanitation & Environment (LASAN) are required as the building's stormwater system will be involved in the regional city's infrastructure such as public storm drains.

## 5.2. Site analysis

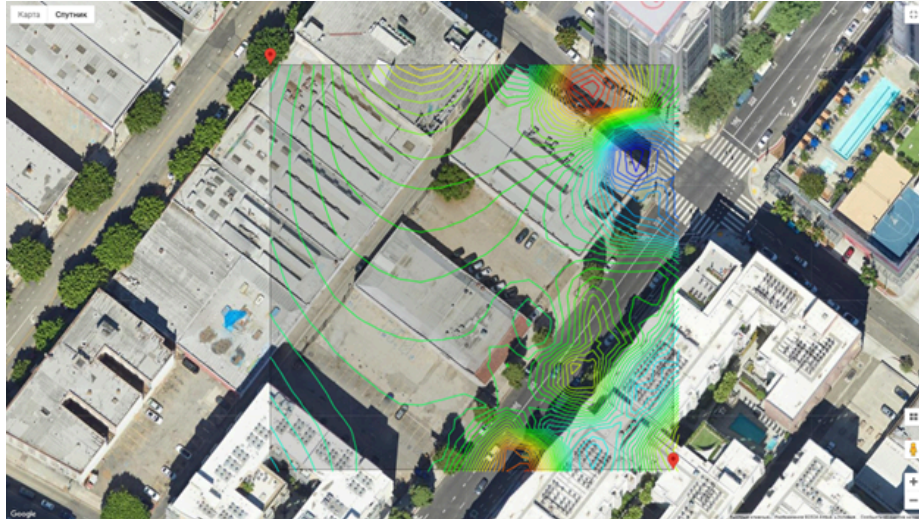
### 5.2.1. Topography

The topography of the project area is generally characterized by relatively flat terrain with slight variations in elevation (Figure 5.2.1.1). However, it is surrounded by a more complex topography that influences the city's infrastructure, including its drainage systems. Downtown LA itself is situated on a relatively flat plain with elevations ranging from about 60 to 120 meters above sea level and for our projected area the max elevation is equal to 74 meters, while the minimum elevation is 72 meters (Figure 5.2.1.2). This flat topography is ideal for urban development but requires careful drainage management to prevent flooding during heavy rains.



Figure 5.2.1.1. Topographic map

Natural drainage in downtown LA generally flows toward the Los Angeles River, which runs along the eastern edge of downtown and those flow directions also can be seen in Figure 5.2.1.1. The river, which was once a natural watercourse, has been channelized to control flooding.



Sampling	Plot Options	Save/Load Cookie	Other Options
North West corner Latitude: [34.0394] Longitude: [-118.263]	<input checked="" type="radio"/> Number of levels: [35]	save data in cookie	clear map ▾
South East corner Latitude: [34.0404] Longitude: [-118.264]	<input type="radio"/> Custom levels [m]: [380,400,420]	load data from cookie ▾	change resolution ▾
Sampling Point: N-S axis: [20] W-E axis: [20]	<input type="radio"/> Level Interval [m]: [5]	remove cookie ▾	
	<input type="checkbox"/> Plot sampling points		
	Units: <input checked="" type="radio"/> m <input type="radio"/> ft		
	Rounding for legend (decimal places): [0]		

Instructions	Elevation Data
Go to the desired location in the map, set two markers by clicking the map to define a rectangle (or enter coordinates manually). Click the button [get	73.16544342041016 min: 72 m 73.04304504394531 max: 74 m

Figure 5.2.1.2. Elevation data for the designed area

Elevations in drainage system design ensure that water flows efficiently under gravity, prevents flooding, and optimize the slope of pipes and channels. Proper use of elevation data ensures the functionality and effectiveness of the entire drainage network. In summary, while the downtown LA area is largely flat with moderate elevations, its proximity to surrounding hills and mountains influences its drainage and water management needs. The slight slope of the land and the presence of the Los Angeles River are key factors in managing stormwater runoff in the area.

### 5.2.2. Soil Type

Sandy soils have large, well-connected pores, which allow for high water infiltration rates. This means that water can percolate into the ground faster compared to clayey or silty soils. The reduced runoff may allow for smaller storm sewer pipes, especially for areas where

sandy soil dominates. The high infiltration rate may reduce the need for large stormwater retention or detention systems, as the soil will naturally absorb a significant portion of the rainwater. The storm sewer system pipes may require additional structural support, such as bedding layers made of compacted gravel or crushed stone to prevent shifting or sinking in the sandy soil. Reinforced concrete could be a good option for pipe material, also it was already used for this location (Figure 5.2.2.1).

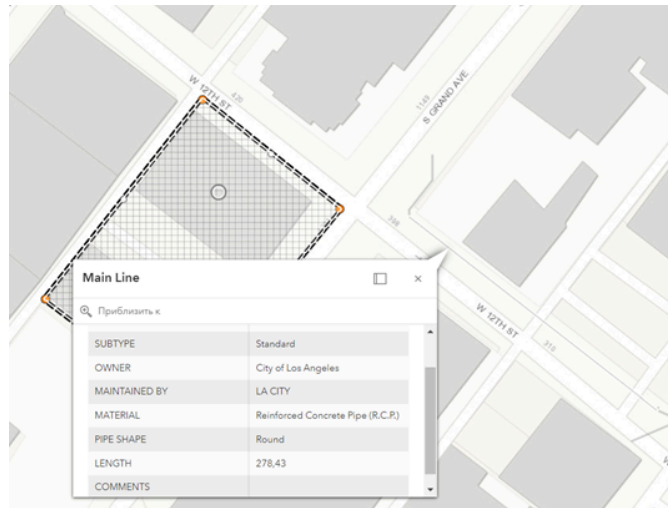


Figure 5.2.2.1. Pipe characteristics

### 5.2.3. Existing Infrastructure

The existing infrastructure in downtown Los Angeles has a significant impact on the design and performance of the storm sewer system. The dense urban environment, with its well-established roads, buildings, and utilities, creates both challenges and opportunities for stormwater management. The vast number of impervious surfaces in downtown LA increases stormwater runoff, as water cannot naturally infiltrate into the ground. High-density development leaves limited room for new stormwater infrastructure. This can restrict the ability to install large underground storm sewer pipes or retention systems. Existing infrastructure for our location includes various underground utilities (water lines, gas lines, electric conduits, telecommunications), which complicates the installation or upgrading of storm sewer systems (Figures 5.2.3.1 and 5.2.3.2).



Figure 5.2.3.1 Storm Drain System in the projected area



Figure 5.2.3.2 Sewer System in the projected area

This feature class represents current stormwater conduits within the City of Los Angeles. The GeoHub (Figure 5.2.3.1, 5.2.3.2, 5.2.2.1) is the City's public platform for visualizing, exploring, and downloading location-based Open Data. By using this platform, we can analyze near pipelines, and get information about physical characteristics such as their shape, materials used, and length (Figure 5.2.3.2).

### 5.3. Preliminary Horizontal Layout and Section Breakdown

The primary map was created and maintained by the Bureau of Engineering / GIS Mapping Division. GIS data contains the storm pipes and the storm drain inlets. A black rectangle shape is the projected area for our construction and light blue lines are the pipelines, while dark blue lines are the existing stormwater drain system (Figure 5.3.1). The longest line is the main line, which is the primary pipe in the drainage system, responsible for transporting large volumes of stormwater from multiple connector lines to a discharge point. Main lines are larger in diameter and are typically located deeper underground. There will be connectors between them, also known as a lateral line, which are smaller pipes that connect

local drainage points (e.g., street drains, building downspouts, catch basins) to the main drainage line. The hotel area from the figure can be considered as the roof for the roof area of our construction buildings, and an arrow drawn means the direction of the rainfall flow. Hidden gutter systems will be constructed on one sloped side of the buildings, and stormwater will flow through the pipes and reach the catch basins. After water will be transmitted through the connectors to the main line. This primary line will be connected to the public stormwater sewer system design (Figure 5.6). The parking area is located at the crossroads of South Grand and West 12 streets. The arrow on the figure shows the direction of the water flow, also it could be considered as the slope direction. A 2 percent slope should be constructed in this projected area as it is a requirement that depends on state regulations, in our case, it is the 2020 City of Los Angeles Building Code (2 Volumes). Channel drains along the parking area's width will be used to collect all stormwater and connect the drain system to the lateral pipeline along S Grand Street.



Figure 5.3.1 Preliminary layout of the storm sewer system

#### 5.4 Drainage and grading plan

Grading and drainage plans are vital for environmental engineering, particularly in construction sites. It involves the planned implementation of landscaping and drainage in such a way that the surface discharge of water will be controlled adequately without harming the natural ecosystem and ensuring functionality and aesthetics.

Land grading involves modifying the surface of the land to create topography capable of directing water appropriately while supporting the proper structural load. This ensures that gardens, parking lots, roadways, and green spaces are created to look and feel good in use. Grading minimizes puddles and washes away dirt to ensure that proper construction techniques can be performed in the long run.

Drainage planning is important for ensuring compliance with relevant legislation and supporting water retention and conservation objectives. This will reduce the risk of flooding, which can do serious damage to property and the environment. We can protect ourselves and the environment from irreversible harm, with adequate drainage plans.

### ***Importance in Construction and Environmental Engineering***

- Grading and drainage are important parts of construction and environmental engineering projects. They are also critical for reducing soil displacement, reducing water pooling, and keeping the landscape permanent.
- Grading reduces soil displacement while preserving the natural landscape, and proper drainage prevents water from pooling and damaging infrastructures and the environment. Groundwater recharge can help lessen demand for artificially made water systems and be an environmentally friendly solution by implementation through natural infiltration techniques.
- Knowing local regulations and standards for grading and drainage is essential for whether you will meet legal requirements and get permittance. Grading and Drainage plan design that's well thought out adds to the visual attractiveness and function of a construction site. Effective Grading and Drainage Plan Design
- Incorporating these principles within construction projects aligns with the goals of sustainable development and minimizes environmental consequences while providing long-term environmental sustainability.

### 5.4.1 Existing grades

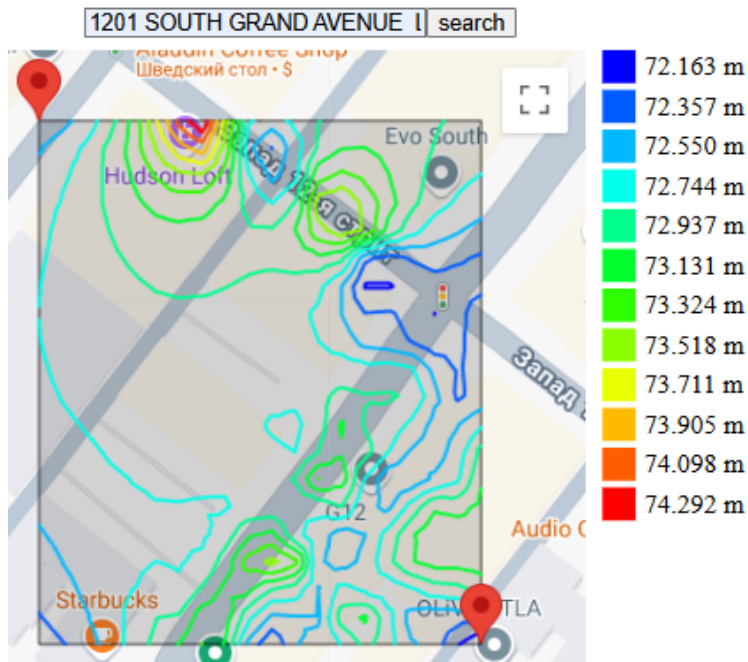


Figure 5.4.1.1 Existing contours and elevation values

Figure 5.4.1.1 is a topographic map showing elevations in an area as contour lines. As shown by the color-coded legend to the right, the contour lines represent different elevations:

- Color gradient: The colors vary from blue (topographically lowest elevations, approximately 72.163 m) to red (topographically highest elevations, approximately 74.292 m).
- Grade Lines: Each faint grey line represents a height above sea level, the closer the lines are to each other the steeper the slope of that terrain. On the other hand, wider spacing suggests a softer slope.
- Land Features: Contour patterns can be interpreted as peaks, valleys, or flat areas. For instance, the closely spaced contours in red indicate a higher, steeper section, while the bluish-green areas indicate lower elevations.

### 5.4.2 Proposed grades

This figure represents a grading and drainage drawing for a site that includes a hotel and a parking area.

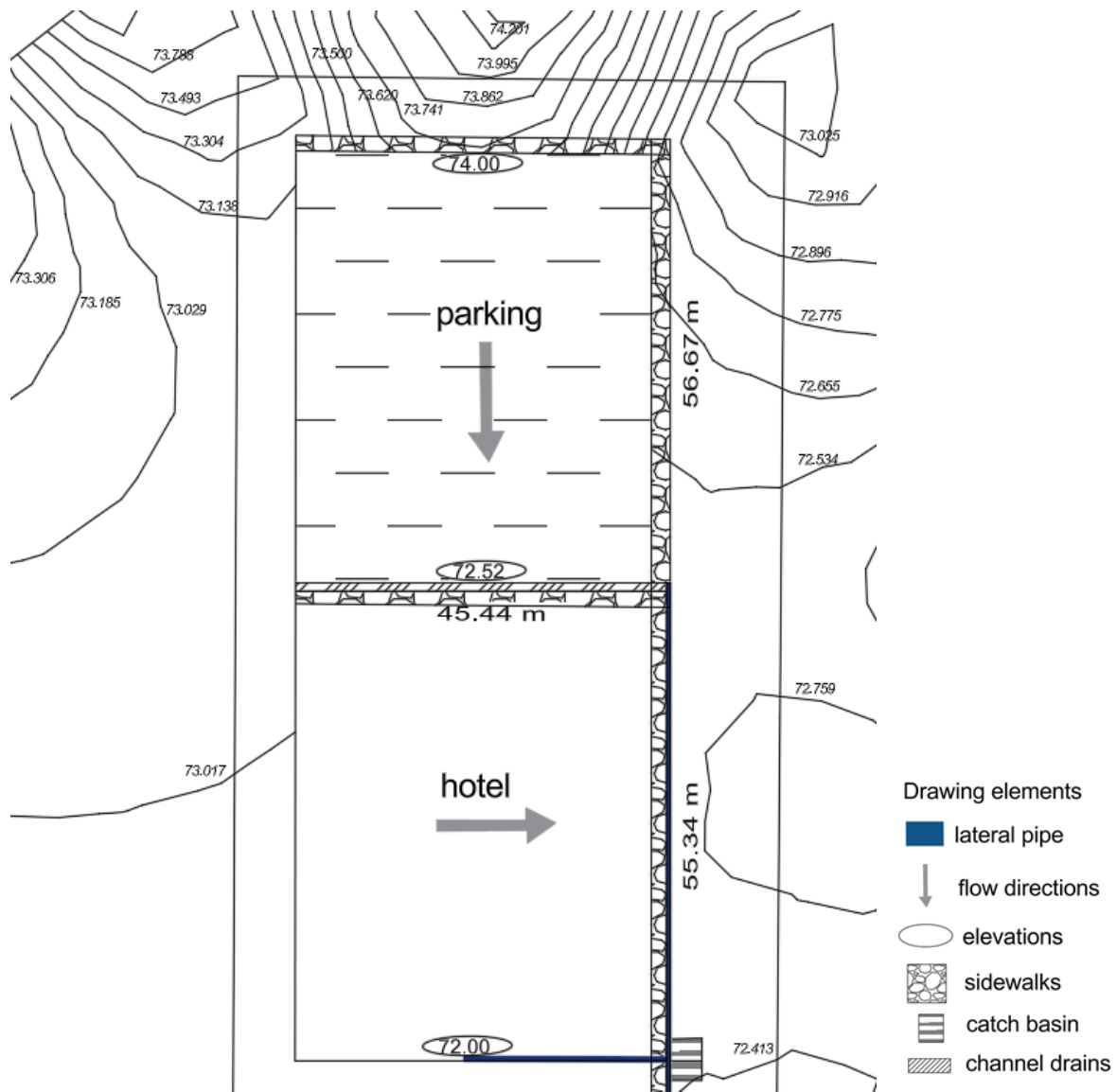


Figure 5.4.2.1 A grading and drainage drawing

The components of the drawing are:

*Grading Details:*

Contour Lines: These show the elevation change across the site, the lowest point around 72.00 m and the highest being around 74.00 m, the parking area is performed at a height in comparison to the hotel and thus results in the fall in the level will allow water to flow towards a discharge point. Grade the site to provide the proper slope and surface water runoff from the building.

*Drainage Elements:*

Lateral Pipe: A blue pipe runs through the site to collect and channel the water, allowing it to flow to an in-stream outlet.

Channel Drains: These are strategically placed to catch surface water and prevent it from pooling.

Catch Basins: These are used to collect runoff water from the graded surface and channel it to the drainage system.

Flow Direction — Water flow is shown with arrows for moving water from higher areas (eg parking) to lower areas and into drainage infrastructure

Figure 5.4.2.2 illustrates the layout and elevation details for designing a parking area with proper drainage.

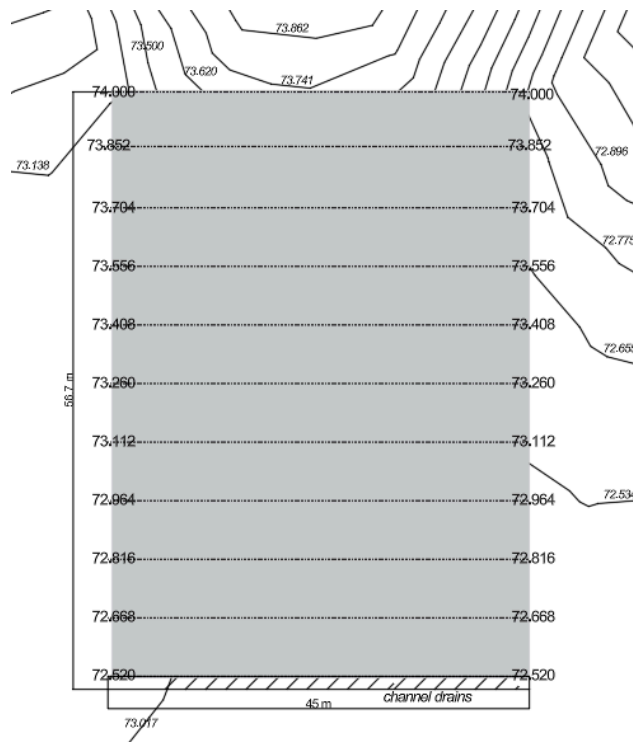


Figure 5.4.2.2. Grading plan for a parking lot

Elevation levels (70.000 m lowest, 72.520 m highest) These lines also represent the incline of the parking lot and are intended to be high enough to divert the water. This parking lot surface also slopes in a stepwise manner (image top to bottom– decreasing elevation). At the bottom edge (elevation 72.520 m) channel drains are located, accordingly draining the

water runoff from the slope falling down parking lot. It is 45 meters wide and includes it lengthwise slop for over a 56.7 meter stretch in that direction.

These drawings help guarantee sites are oriented correctly so that water will drain responsibly and not cause flooding or wash away parts of a site in addition to keeping spaces safe and functional.

### **5.4.3 Ground improvements**

#### *Regions Requiring Excavation (Cut)*

Southern and Central Areas: In Figure 5.4.2.1, the elevations in the southern and central portions of the site are above 72.50 m, especially in areas where the contour lines are close to each other indicating steep slopes. Cutting these areas will reduce the ground level of 72.50 m at the north edge of the parking lot and 72.00 m for the hotel building to reach the design elevation. These excess materials should be excavated to have a uniform surface for the parking area and associated infrastructure.

#### *Regions Requiring Filling:*

Northern Zone: Areas below 74.00 m (for instance; 73.620 m and 73.500 m) necessitate earthwork operations to elevate the land above the necessary height. This is true of any depressions or low points in the center of the site where the grading design demands uniformity.

During this ground improvement, excavated material from one area will be reused to fill the lower northern area, minimizing the need for importing or disposing of soil. Moreover, the final surface slopes as per the grading plan, will direct water from higher areas (74.000 m) to the drainage system (72.520 m).

### **5.4.4 Drainage elements**

- Channel drains

Channel drains, also referred to as trench drains, represent an essential component of drainage solutions for parking areas. These linear drainage systems are designed to effectively collect and guide surface water away from the parking area, ensuring safety, durability, and adherence to environmental regulations.

Channel drains consist of long, narrow trenches embedded in the surface of the parking lot, typically having a U- or V-shape and equipped with a grate cover. The grate allows water to flow into the trench, which is then directed towards an appropriate outlet such as a storm sewer, detention pond, or another drainage system.

The main channel drains` advantages are prevention of water pooling, protection of pavement, and enhanced safety. Heavy rainfall can cause standing water on parking lots, making them hazardous for vehicles and pedestrians. To prevent this, channel drains are used to efficiently collect and direct water away from the parking space. Moreover, excessive water can damage asphalt and concrete, causing cracks and potholes in the parking lot surface. Channel drains play a crucial role in preserving the structural integrity of the pavement. In addition, by quickly removing water from the parking area, channel drains help reduce the risk of hydroplaning and improve traction for both vehicles and pedestrians, ensuring a safer environment.



Figure 5.4.4.1. Channel drainst

- Catch basin

Catch basins are underground structures designed to collect water from a grated surface. These structures prevent debris, such as leaves and sediment, from entering the water supply by trapping them in a sump beneath the grate. The clean water then flows into connected pipes or storm drains, ensuring that only clean water enters the system.

The importance of catch basins is significant. They help to divert runoff from parking lots, which can cause water to pool at the edges of the lot. This runoff can also lead to soil

erosion in nearby areas. By controlling this runoff, catch basins help to prevent both problems. Additionally, they help reduce the risk of flooding by managing large volumes of water more efficiently.



Figure 5.4.4.2. Catch basin

- Swales

A swale next to a building is a shallow, gently sloping channel designed to collect and direct water away from the foundation of the structure. These swales can be lined with various materials such as grass, vegetation, or gravel, depending on the type of soil and aesthetic preferences. This helps to enhance the natural filtration and absorption of water.

The importance of swales near building sides is significant. They serve several purposes:

- Foundation protection: By directing water away from the foundation, swales prevent water seepage and basement flooding, which can cause structural instability due to saturated soil.
- Erosion prevention: Swales slow down the flow of water and reduce the risk of erosion around the building's surroundings, protecting foundations and maintaining landscaping.
- Runoff management: Swales channel water runoff from rain or snowmelt towards designated drainage areas, such as retention ponds or storm drains, preventing pooling and flooding near the building.



Figure 5.4.4.3. Swalest

## 5.5 Rainfall Runoff

Managing stormwater runoff is vital for urban infrastructure, especially for significant projects like a 17-story hotel in Los Angeles. It's important to understand and estimate stormwater runoff volumes to create an effective management system that meets local regulations and reduces the risk of flooding.

Stormwater runoff refers to the surplus rainwater that moves across hard surfaces like roads, rooftops, and parking lots rather than soaking into the ground. In cities, unmanaged runoff can cause problems such as flooding, erosion, and water pollution, as it transports debris, chemicals, and sediments into drainage systems and natural water bodies (U.S. Environmental Protection Agency, 2023). Proper stormwater management is essential to address these challenges and promote the sustainability of urban areas.

Hydraulic design is crucial for managing stormwater, as it ensures that drainage systems can efficiently collect, transport, and release runoff without leading to structural or environmental harm. Well-designed hydraulic systems help protect water quality, minimize property damage, and meet regulatory requirements. In Los Angeles, stormwater management strategies must include Low Impact Development (LID) techniques, like green roofs and permeable pavements, to decrease runoff volumes and enhance infiltration (LADPW, 2023). A rainwater collection system can effectively minimize stormwater runoff and offer an alternative water source for non-potable applications like irrigation and cooling.

These systems generally consist of catchment surfaces, gutters, storage tanks, and filtration units. By incorporating rainwater harvesting in a large-scale project such as a hotel, it is possible to greatly reduce dependence on municipal water supplies and support sustainability objectives (California Stormwater Quality Association, 2022).

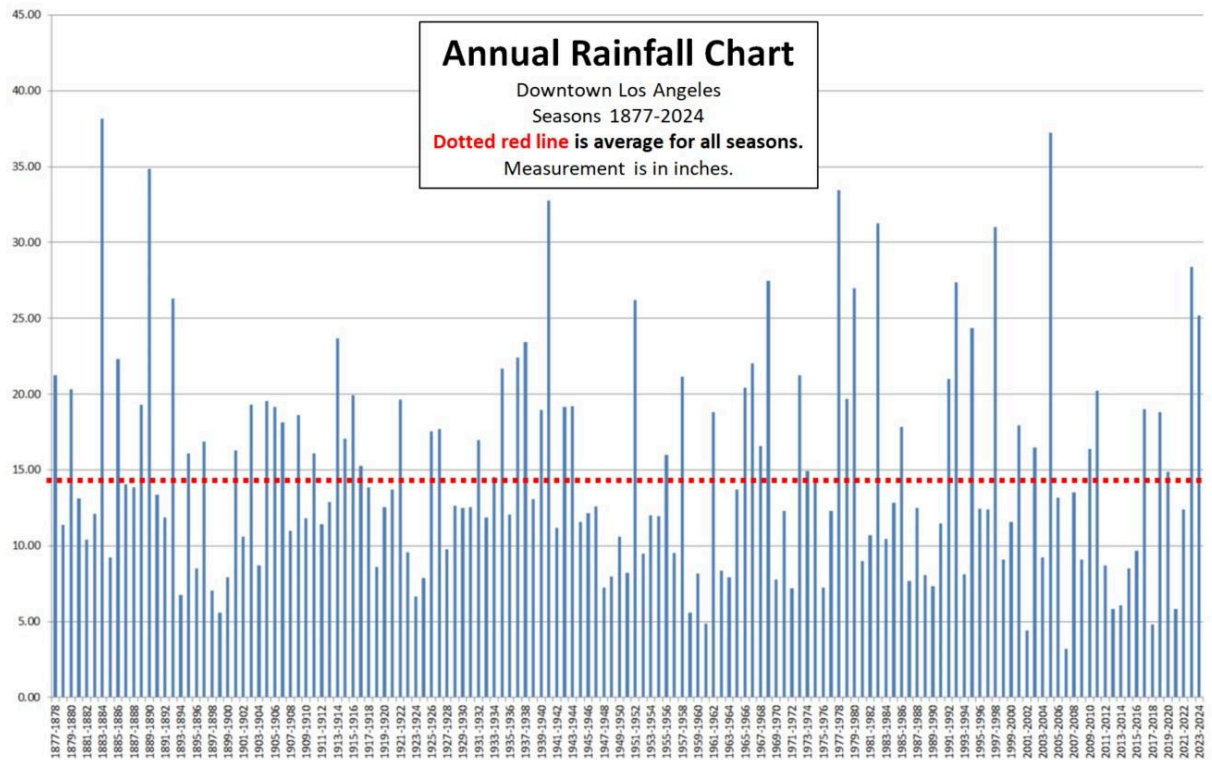


Figure 5.5.0.1. Annual Rainfall Chart

The chart displays the annual rainfall in Downtown Los Angeles from the 1877-78 season to the 2023-24 season, measured in inches. The blue bars represent the total rainfall for each season, while the dotted red line indicates the historical average across all seasons. The data shows significant variability, with some years experiencing extreme rainfall while others remain well below average.

### 5.5.1 The Rational Method

Designing stormwater drainage systems in Los Angeles County involves converting rainfall into runoff volumes and flow rates. The Department of Public Works utilizes two primary methods for this conversion: the Rational Method and the Modified Rational Method (MODRAT), depending on site conditions.

$$Q=C \times I \times A \tag{5.5.1.0.1}$$

where

Q- the estimated design discharge in cubic feet per second (cfs)

C- the composite runoff coefficient (a unitless factor indicating surface permeability)

I- the rainfall intensity in inches per hour for the specific design storm

A- is the watershed area in acres.

### 5.5.1.1 Time of Concentration

Since MODRAT relies on the TC to determine rainfall intensities for the rational equation, its calculation is crucial. The Public Works Department accepts the kinematic wave theory as a standard method for determining the TC. This approach divides the longest flow path into two components: overland flow and conveyance flow by kinematic wave theory:

$$T_c = t_o + t_c \quad (5.5.1.1.1)$$

Where

$T_c$  - Time of concentration in minutes

$t_o$  - Overland flow travel time in minutes

$t_c$  - Sum of all conveyance travel times in minutes.

Conservation of mass and the momentum equation are used to determine the time associated with overland flow.

$$t_o = \frac{0.94 * L_o^{0.6} * n_o^{0.6}}{I_x^{0.4} * S_o^{0.3}} \quad (5.5.1.1.2)$$

$$I_x = C * I \quad (5.5.1.1.3)$$

Where

$L_o$  - Overland flow length in feet

$n_o$  - Roughness for overland flow surface, dimensionless

$I_x$  - Rainfall excess in in/hr

$S_o$  - Slope of overland flow in ft/ft.

The kinematic wave approach applies to channel flow as well as overland flow. The Manning equation is a form of kinematic wave theory for channels. The Manning equation is used to determine the average velocity in the channel.

$$V = 3.28kS_p^{0.5} \quad (5.5.1.1.4)$$

$$t_c = \left(\frac{1}{60}\right)\left(\frac{L_c}{V}\right) \quad (5.5.1.1.5)$$

Where

k - intercept coefficient

$S_p$  - Slope in percent

$L_c$  - Conveyance flow length in feet

V - Average velocity ft/sec.

### 5.5.1.2 Coefficients (Runoff, Roughness, Intercept)

The runoff coefficient C is determined by assessing the surface characteristics of the contributing watershed. It ranges from 0 to 1.0, with higher values indicating a greater potential for runoff. The coefficient is calculated based on the proportion of different land uses within the drainage area, accounting for variations in permeability across pervious and impervious surfaces. Standard reference tables provide values for different surface types to aid in accurate estimation.

<b>Land type</b>	<b>Runoff coefficient</b>	<i>Value #1</i>	<i>Value #2</i>	<i>Value #3</i>
Building surface, concrete, or asphalt pavement road	0.85~0.95	0.85	0.90	0.95
Large rubble paved road, or gravel road with asphalt surface	0.55~0.65	0.55	0.60	0.65
Gradation macadam road	0.40~0.50	0.40	0.45	0.50
Masonry brick or gravel road	0.35~0.40	0.35	0.375	0.40
Unpaved soil road	0.25~0.35	0.25	0.275	0.35
Garden or green land	0.10~0.20	0.10	0.15	0.20
Water area	0	0	0	0

Figure 5.5.1.2.1. Runoff Coefficients Based on Land Types

Roughness coefficients are essential parameters in hydrology and hydraulics, representing the resistance to flow caused by surface characteristics. These coefficients are primarily used in Manning’s equation to estimate water flow velocity in open channels, stormwater drainage systems, and overland flow. Surfaces with lower roughness coefficients, such as smooth asphalt and concrete, allow water to flow more easily with minimal resistance. In contrast, surfaces with higher roughness values, like dense turf or rural landscapes, create greater friction, slowing down the flow. Understanding roughness coefficients is crucial for accurate flood modeling, stormwater management, and designing efficient drainage systems.

<b>Surface Cover<sup>5</sup></b>	<b><math>n_o</math></b>
Smooth Asphalt	0.012
Concrete Paving	0.014
Packed Clay	0.030
Light Turf	0.250
Dense Turf	0.350
Industrial/Commercial	0.014
Residential	0.040
Rural	0.060

Figure 5.5.1.2.2. Manning Coefficient of Roughness

<b>Material Type</b>	<b>Manning Coefficient n</b>
Soil	0.025
Concrete	0.015
Gravel	0.020
Wood	0.013
Flat steel	0.012
Corrugated steel	0.025
Glass	0.010

Figure 5.5.1.2.3. Typical Value of Manning Coefficient of Roughness

In the context of hydrology and flow resistance, the intercept coefficient may be relevant when developing empirical relationships for runoff estimation or roughness coefficient adjustments. However, roughness coefficients themselves do not usually have an intercept term unless they are part of a regression model predicting flow characteristics.

<b>Surface Description</b>	<b>Intercept Coefficients, k</b>
Paved area	0.619
Unpaved	0.491
Grassed waterway	0.457

Figure 5.5.1.2.4. Intercept Coefficient for Shallow Concentrated Flow

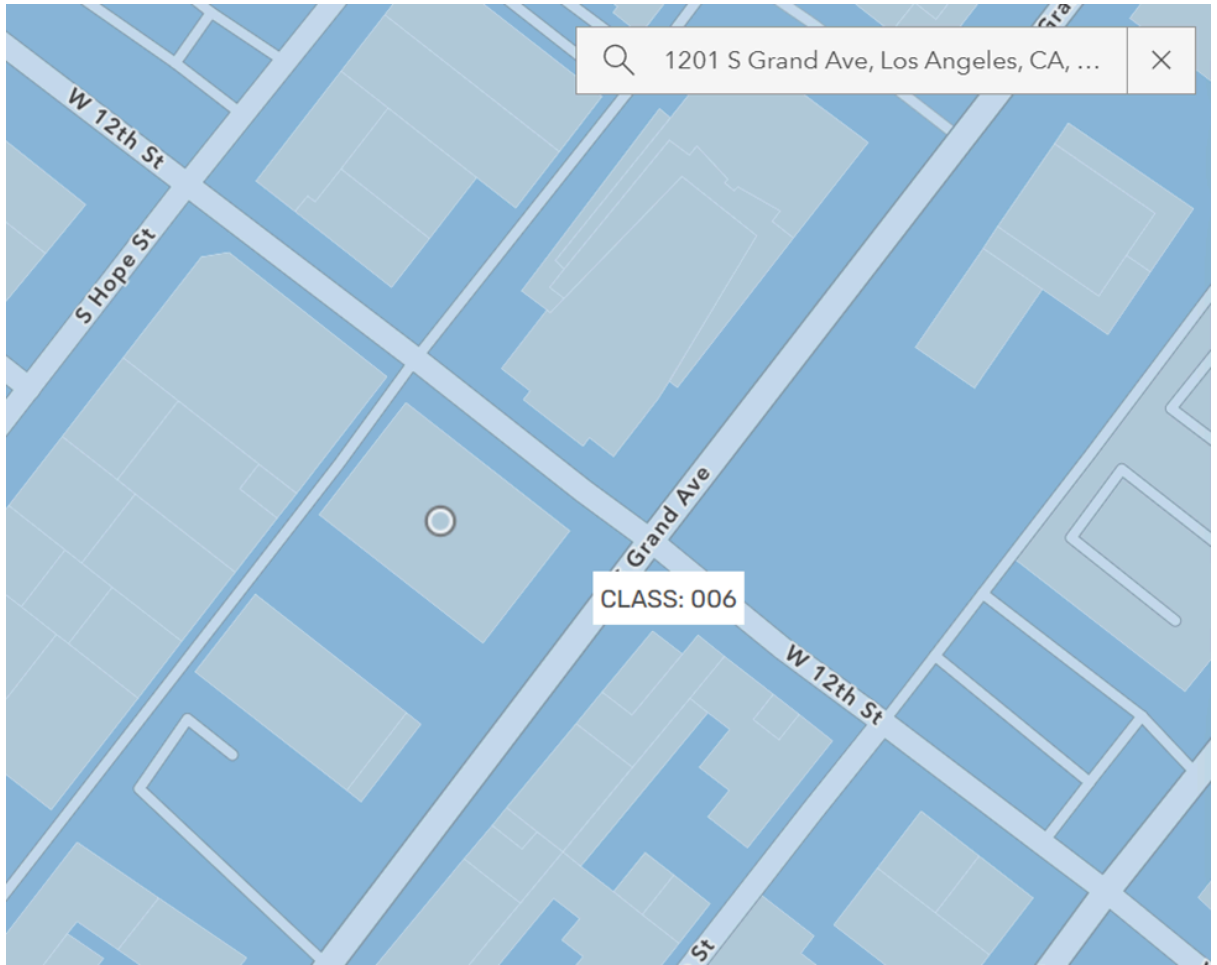


Figure 5.5.1.3.1. Soil Types Feature Layer

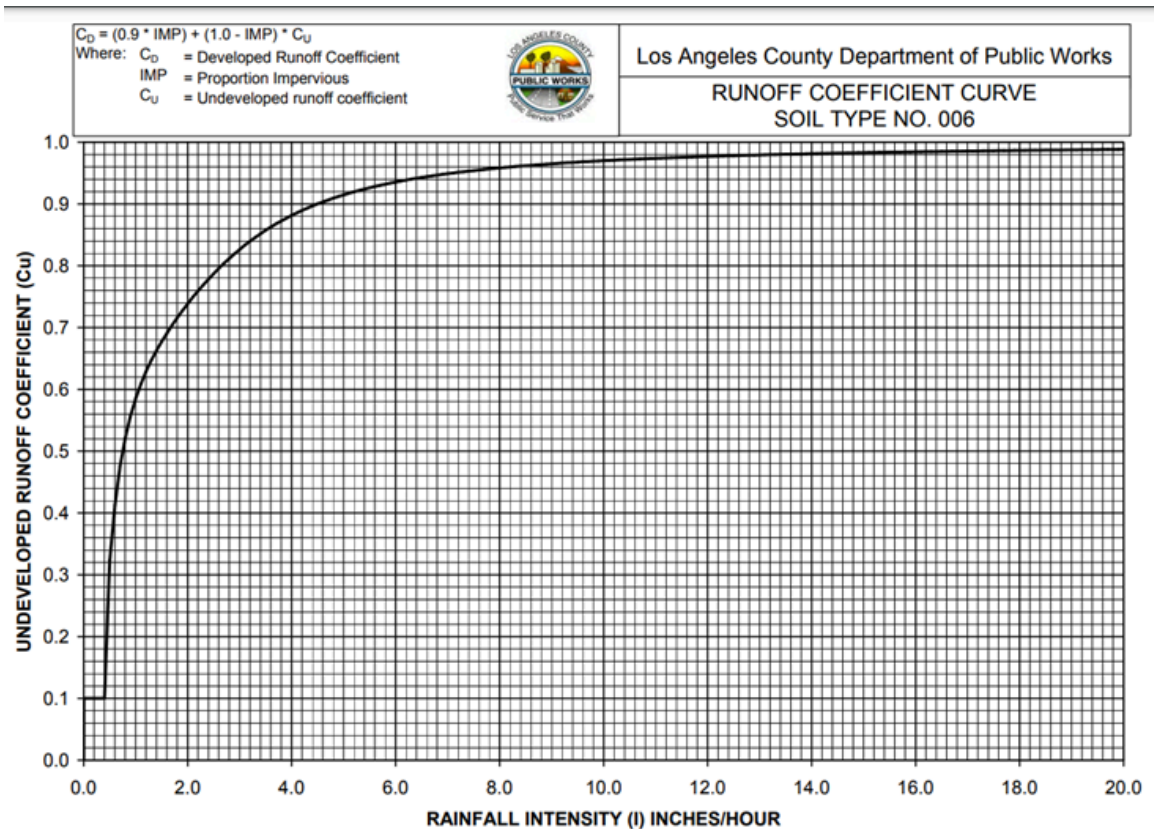


Figure 5.5.1.3.2. Runoff Coefficient

These figures, provided by the Los Angeles County Department of Public Works, presents a Runoff Coefficient Curve for Soil Type No. 006. The runoff coefficient represents the fraction of rainfall that becomes surface runoff rather than infiltrating into the ground.

### 5.5.1.3 Stormwater Quantity Estimation

Rainfall intensity is the hourly rainfall amount for the 10-year, 15-minute precipitation intensity, which was assumed as 0.639. Those data is based on the National Oceanic and Atmospheric Administration. It is a U.S. government agency responsible for monitoring and studying the oceans, atmosphere, and weather. NOAA provides critical data on weather forecasting, climate change, ocean conditions, and environmental monitoring.

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches) <sup>1</sup>										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.152 (0.128-0.184)	0.195 (0.163-0.235)	0.252 (0.210-0.306)	0.302 (0.249-0.369)	0.372 (0.297-0.472)	0.430 (0.335-0.557)	0.491 (0.373-0.653)	0.557 (0.410-0.763)	0.651 (0.459-0.932)	0.729 (0.495-1.08)
10-min	0.219 (0.183-0.264)	0.279 (0.233-0.337)	0.362 (0.301-0.439)	0.432 (0.357-0.529)	0.534 (0.425-0.676)	0.616 (0.480-0.798)	0.703 (0.534-0.935)	0.798 (0.588-1.09)	0.933 (0.658-1.34)	1.04 (0.710-1.55)
15-min	0.264 (0.221-0.319)	0.337 (0.282-0.408)	0.438 (0.365-0.530)	0.523 (0.432-0.639)	0.645 (0.514-0.818)	0.745 (0.580-0.965)	0.850 (0.646-1.13)	0.965 (0.711-1.32)	1.13 (0.796-1.62)	1.26 (0.859-1.88)

Figure 5.5.1.3.1. NOAA Atlas 14 (Precipitation-Frequency)

Then by following the procedure above and by calculations based on all figures and equations of part 5, stormwater quantities were estimated by using Excel tools, and design planning was drawn in AutoCAD software.

	Parking Lots	Roof:Terracce 1	Roof:Terracce 2	Roof:Lounge Bar1	Roof:Lounge Bar2	Roof:Lounge Bar3	Area 1	Area 2	Area 3	Area 4
I (in/hr)=	2,556	2,556	2,556	2,556	2,556	2,556	2,556	2,556	2,556	2,556
Overland flow travel time										
t0 (min)=	3,788654077	1,353359007	1,298479459	6,085370393	5,451135276	2,586165251	1,117993	0,81028	3,83364	0,68182
Conveyance flow travel time										
tc(min)=	1,223851741	0,42554704	0,71373396	1,065240789	0,321031471	0,255365943	1,223852	3,434	0,10235	3,434
Time of concentration										
T(min)=	5,012505818	1,778906047	2,012213418	7,150611182	5,772166747	2,841531193	2,341845	4,24428	3,93599	4,11583
Projected area										
A(ac)=	0,678205144	0,029282231	0,04583843	0,322895289	0,081001818	0,018594835	0,088706	0,08822	0,04958	0,06616
Stormwater runoff										
Q(cfs)=	1,43394487	0,067360845	0,105446724	0,123798054	0,031056097	0,00712926	0,187552	0,18652	0,10483	0,13989
Q calculator=	1,4674									

Figure 5.5.1.3.2. Stormwater Runoff Volumes

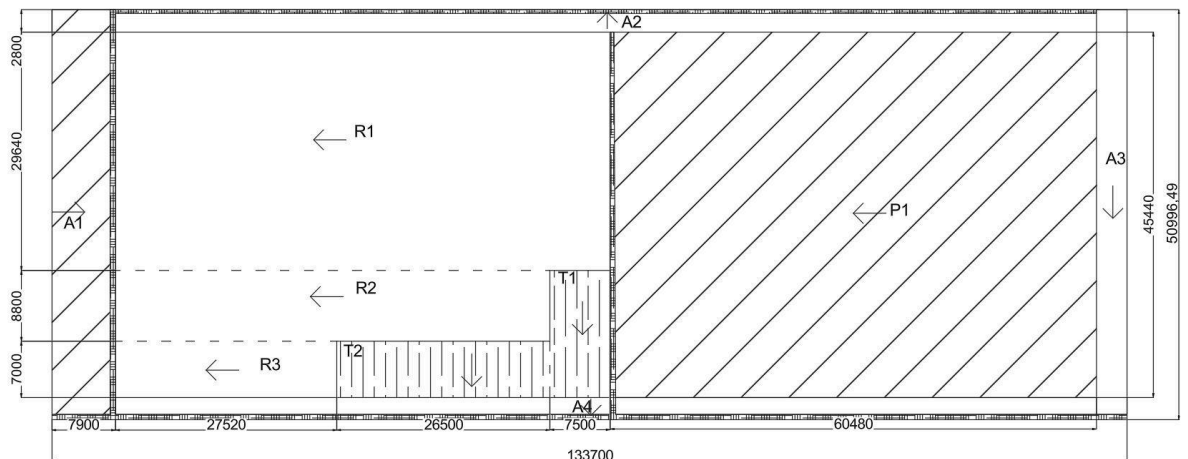


Figure 5.5.1.3.3. Storm Drain System Design

## 5.6 Hydraulic Design

Effective stormwater management begins with a sound hydraulic design that ensures safe and efficient conveyance of runoff generated from impervious surfaces such as rooftops and parking lots. For urban infrastructure projects like hotels, proper sizing of gutters and storm sewer pipes is essential to prevent localized flooding, erosion, and property damage. This design relies on established hydraulic principles and local regulatory standards to guide key parameters such as pipe diameter, slope, and flow capacity. One of the most widely used tools in open-channel and pipe flow analysis is the Manning formula, which relates flow rate to channel geometry, surface roughness, and slope. Additionally, design criteria provided by local authorities, such as the Los Angeles County Department of Public Works, offer valuable standards to ensure compliance with regional drainage and sustainability goals. By integrating these methodologies, the drainage system can effectively manage peak stormwater events while supporting the project's long-term resilience and environmental performance.

### 5.6.1 Material selection

Reinforced concrete pipe (R.C.P.) was chosen for the stormwater drainage system of the proposed hotel and surrounding parking lots because of its exceptional structural performance, long-term durability, and conformity to current municipal infrastructure standards. Both engineering dependability and regulatory compliance are guaranteed by this choice.

- *Exceptional Structural Strength*

R.C.P. is built to handle heavy earth loads and live loads, including the weight of vehicles and construction equipment. This makes it perfect for placement under roads, driveways, and deep trenches—common scenarios in urban hotel projects.

- Rigid Pipe with Load-Bearing Capability

Unlike flexible pipes like HDPE or PVC, concrete pipes don't depend on the surrounding soil for support. Their inherent rigidity allows them to keep their shape and function even in tough soil or compaction situations.

- Long Service Life

With a lifespan that can exceed 75 to 100 years, R.C.P. is a cost-effective choice over time and is highly resistant to weathering, corrosion, and chemical damage—especially crucial in areas like Los Angeles, where soil pH can vary and runoff contaminants may be a concern.

- Hydraulic Reliability

While they may not be as smooth as plastic options, concrete pipes still deliver dependable hydraulic performance. Their larger diameters and consistent shape promote stable flow conditions, and modern casting methods can create lined or treated interiors to minimize roughness and enhance hydraulic capacity.

- Resistance to Fire, UV, and Abrasion

Concrete won't melt, warp, or break down from UV exposure, unlike some plastics. It also stands up well to abrasion from sediment-heavy flows, which is vital during high-flow or debris-laden storm events.

- Environmental Benefits

Concrete is crafted from natural, locally sourced materials, making it a more sustainable and recyclable option compared to many synthetic pipe materials. Plus, it doesn't release microplastics or break down into harmful substances.

- Smooth integration with existing infrastructure

As the site plan suggests, the City of Los Angeles already utilizes R.C.P. in the nearby storm drain system. Compatibility of the new system material with current public utilities allows for standardized connections, reduces transition-related failure, and makes permitting and inspection more efficient.

- Versatility in Shape and Size

Concrete pipes are available in an excellent range of diameters and may be made up as round, elliptical, or box culverts based on hydraulic and spatial purposes. This proves convenient with small city parcels like downtown Los Angeles is reputed to have.



Figure 5.2. Concrete Pipe

### 5.7 Hydraulic Design Elements' dimensioning

*Open Channel Flow:* It is the flow of water in a channel whose surface is exposed to the atmosphere. A few examples include rivers, streams, canals, and drainage ditches. The first figure shows how to determine flow velocity for open channels using Manning's equation, considering the geometry of the channel (hydraulic radius), slope, and roughness.

*Pipe Flow:* This is water flow in a closed pipe or conduit. The third illustration is a rearranged version of Manning's equation that can be used for the calculation of flow velocity for a flowing-full pipe. In this case, the pipe diameter is used instead of the hydraulic radius.

Since pipes make up the majority of a rainfall sewer system, the concepts of pipe flow are directly relevant. To effectively convey stormwater, engineers use equations such as the one in the third image, which is derived from Manning's equation, to determine the proper pipe sizes and slopes. Principles of open channel flow are applicable in a number of ways: inlets frequently serve as a transitional point between pipe and overland (open channel) flow; certain sections of some sewer systems might have culverts or open channels, particularly in places with flat terrain or where open channels are already in place.

To guarantee adequate energy dissipation and stop erosion, open channel flow principles must be taken into account when designing outfalls, which are locations where the sewer system empties into receiving water bodies.

For water flow in trench drains, Manning equation in open channels equation was used (5.7.1). Coefficient of 1.49 means that U.S. units applied. As material selection was concrete, Manning coefficient was given as 0.013. For ease of calculation, drainage elements were divided into small rectangular part for each projected area and were calculated separately as given in an Excel file (Table 5.7.1 and Figure 5.7.2). Numerous iterations were proceed to find out the closest matching flow rate value.

$$Q = \frac{1.49}{n} * A * R^{2/3} * S^{1/2} \quad (5.7.1)$$

Where:

Q- flow rate,  $ft^3/s$  or *CFS*

n- Manning's roughness coefficient

A- Cross sectional area of flow,  $ft^2$

R- Hydraulic radius, ft

P- Wetted perimeter, ft

S- Channel/pipe slope, ft/ft

Cross-section	Optimum width $B$	Optimum cross-sectional area $A$	Optimum wetted perimeter $P_w$	Optimum hydraulic diameter $D_H$
Rectangular	$2d$	$2d^2$	$4d$	$2d$
Trapezoidal	$\frac{2}{\sqrt{3}}d$	$\sqrt{3}d^2$	$2\sqrt{3}d$	$2d$
Semi-circle	$2d$	$\frac{\pi}{2}d^2$	$\pi d$	$2d$

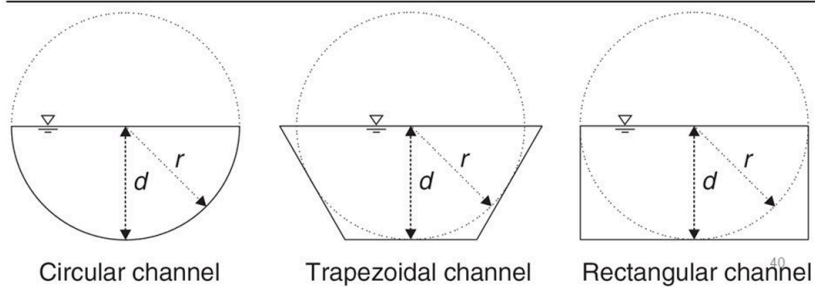


Figure 5.7.1. Hydraulic radius for different channel shapes

Table 5.7.1. Flow rates for different sections of the projected area

	P1	A1+A2+Rtotal	A2	A3+A4+P1+Ttotal	Total
Q, CFS	1.43394	0.04696	0.01698	1.47049	2.95139

P1:	n	s	R or d, m	A, ft^2	Q, cfs
0,013	0,01	0,01	0,1	0,169079	0,921841
			0,105	0,18641	1,049931
			0,11	0,204586	1,188602
			0,115	0,223607	1,338187
			0,116	0,227513	1,369442
			0,117	0,231452	1,40115
			0,118	0,235426	1,433313
A2:	n	s	R or d, m	A, ft^2	Q, cfs
0,013	0,01	0,01	0,02	0,006763	0,012611
			0,022	0,008183	0,01626
			0,024	0,009739	0,020506
A1+A2+Rtotal:	n	s	R or d, m	A, ft^2	Q, cfs
0,013	0,01	0,01	0,03	0,015217	0,03718
			0,032	0,017314	0,044163
			0,033	0,018413	0,047939
			0,034	0,019546	0,051912
A3+A4+P1+Ttotal:	n	s	R or d, m	A, ft^2	Q, cfs
0,013	0,01	0,01	0,118	0,235426	1,433313
			0,119	0,239433	1,465934
			0,1191	0,239835	1,469221
			0,1192	0,240238	1,472513

Figure 5.7.2. Open channel elements' dimensions

Similar method was used to find out proper dimensions for the concrete pipe that will be connected to the open channels and existing drainage infrastructure (Figure 5.7.3). Main difference is in the hydraulic radius. Flow rate equation looks exactly the same as for the open channel, but cross-sectional area and radius will be found by the following formulas:

$$A = \frac{\pi D^2}{4} \quad (5.7.2)$$

$$R = \frac{D}{4} \quad (\text{for full circular flow}) \quad (5.7.3)$$

Table 5.7.2. Flow rates for different sections of the projected area 2

	P1	A1+A2+Rtotal	A2	A3+A4+P1+Ttotal
Q, CFS	1.433945	0.046956	0.016984	1.47049

n	s	R or d, m	A, ft <sup>2</sup>	Q, cfs
0,013	0,01	0,07	0,66	2,848878
		0,0702	0,67	2,870636
		0,0704	0,67	2,892497
		0,0706	0,67	2,914462
		0,0708	0,68	2,93653
		0,071	0,68	2,958703
		0,0712	0,69	2,98098

Figure 5.7.3. Closed channel elements' dimensions

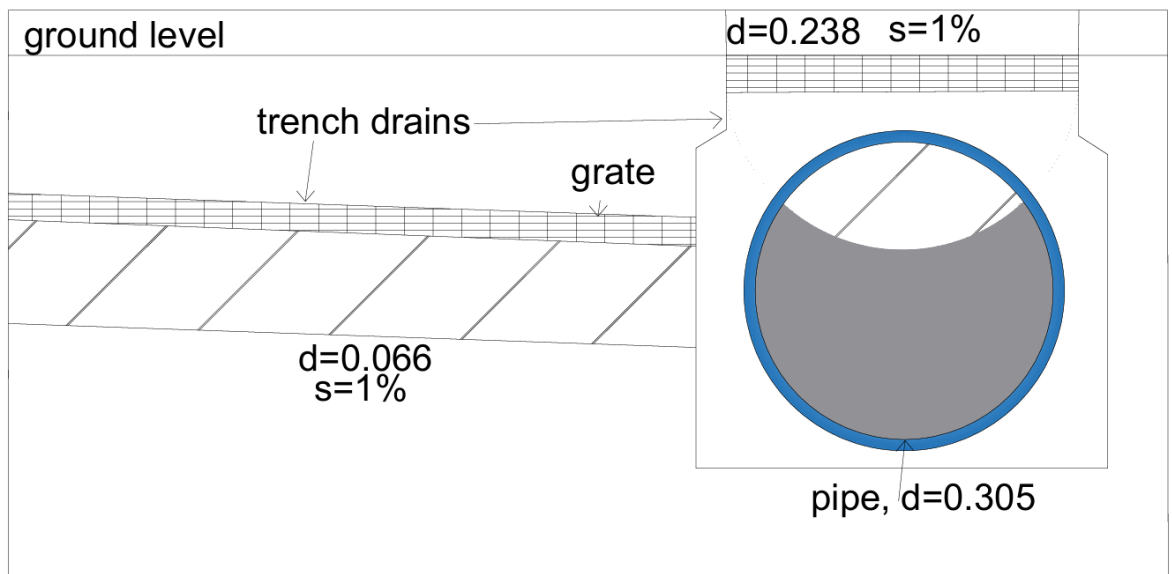


Figure 5.7.4. Cross-sectional view

## 5.8 Leadership in Energy and Environmental Design (LEED)

Leadership in Energy and Environmental Design (LEED) is one of the oldest and the most widely recognized and used green building rating systems around the world. LEED certification has now become a symbol of sustainability achievement and is used for all types of buildings and projects including construction, interior fit-outs, operations, and maintenance upgrades. It is proposed by the United States Green Building Council (USGBC)

and aims to provide a holistic system to “create healthy, highly efficient, cost-saving green buildings”.

Currently, LEED certification offers six pathways with a rating system for different types and scales. Each of them is also broken into nine categories by sustainable improvements. These are Integrated Process, Location and Transportation, Suitable sites Water Efficiency, Energy and Atmosphere, Materials and Resources, Indoor Environmental Quality, Innovation, and Reginal Priorities.

The project should adhere to the particular level of requirements to get a LEED certification. Achieved number of points will identify one of the four certifications: Certified (40-49), Silver (50-59), Gold (60-79), and Platinum (80+ points).



### 1. Location and Transportation

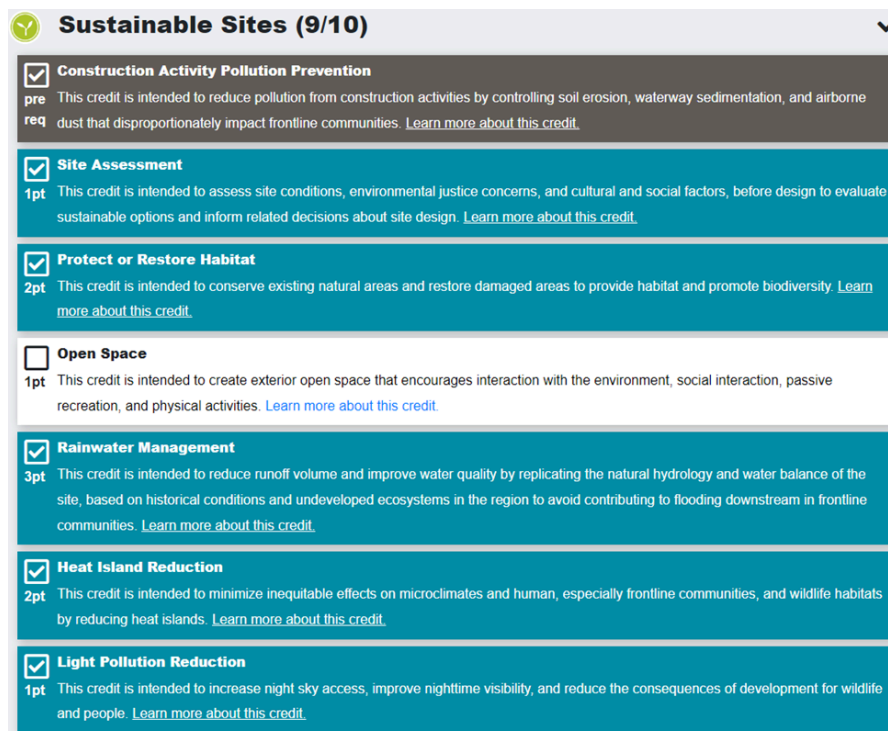
Our site is located at 1201 South Grand Ave, at the intersection of South Grand Ave and West 12<sup>th</sup> Street. As it is a downtown area, the site is considered walkable, promoting transportation choices (public buses and bicycle lanes), and encouraging the development of local economies. These credits are intended to encourage developments in location and sustainability by reducing greenhouse gas emissions, low-carbon transportation, and improving public health. In addition, the installation of electrical vehicle supply equipment in parking spaces will reduce pollution as it will provide an alternative to fueled vehicles. The score for this category is provided in Figure below.

**Location and Transportation (12/16)**

- Sensitive Land Protection**  
1pt This credit is intended to cultivate community resilience, avoid the development of environmentally sensitive lands that provide critical ecosystem services and reduce the environmental impact from the location of a building on a site. [Learn more about this credit.](#)
- High Priority Site and Equitable Development**  
2pts This credit is intended to build the economic and social vitality of communities, encourage project location in areas with development constraints and promote the ecological, cultural, and community health of the surrounding area while understanding the needs and goals of existing residents and businesses. [Learn more about this credit.](#)
- Surrounding Density and Diverse Uses**  
5pts This credit is intended to conserve land and protect farmland and wildlife habitat by encouraging development in areas with existing infrastructure. It is also intended to support neighborhood and local economies, promote walkability, and low or no carbon transportation, and reduce vehicle distance traveled for all. Furthermore, it is intended to improve public health by encouraging daily physical activity. [Learn more about this credit.](#)

## 2. Sustainable Sites

Before the site design and its sustainability options, site conditions, and environmental concerns should be assessed. An existing natural area with its biodiversity will be protected and restored. Moreover, there will be a roof drainage rainwater management system that allows replicating the water balance of the area and its natural hydrology cycle. Decreasing heat islands to minimize negative effects on microclimates can be done by providing structures such as roofs with high solar reflectance value. As the neighboring buildings are low-rise, there are no issues with light pollution for the 12-story hotel.



The image shows a screenshot of a LEED credit checklist for 'Sustainable Sites (9/10)'. The checklist is organized into several rows, each representing a different credit. Each row includes a checkbox, a credit name, a point value, and a brief description. The credits listed are: Construction Activity Pollution Prevention (1pt), Site Assessment (1pt), Protect or Restore Habitat (2pt), Open Space (1pt), Rainwater Management (3pt), Heat Island Reduction (2pt), and Light Pollution Reduction (1pt). All credits except 'Open Space' are marked as completed with a checked box.

Check	Credit Name	Points	Description
<input checked="" type="checkbox"/>	Construction Activity Pollution Prevention	1pt	This credit is intended to reduce pollution from construction activities by controlling soil erosion, waterway sedimentation, and airborne dust that disproportionately impact frontline communities. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	Site Assessment	1pt	This credit is intended to assess site conditions, environmental justice concerns, and cultural and social factors, before design to evaluate sustainable options and inform related decisions about site design. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	Protect or Restore Habitat	2pt	This credit is intended to conserve existing natural areas and restore damaged areas to provide habitat and promote biodiversity. <a href="#">Learn more about this credit.</a>
<input type="checkbox"/>	Open Space	1pt	This credit is intended to create exterior open space that encourages interaction with the environment, social interaction, passive recreation, and physical activities. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	Rainwater Management	3pt	This credit is intended to reduce runoff volume and improve water quality by replicating the natural hydrology and water balance of the site, based on historical conditions and undeveloped ecosystems in the region to avoid contributing to flooding downstream in frontline communities. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	Heat Island Reduction	2pt	This credit is intended to minimize inequitable effects on microclimates and human, especially frontline communities, and wildlife habitats by reducing heat islands. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	Light Pollution Reduction	1pt	This credit is intended to increase night sky access, improve nighttime visibility, and reduce the consequences of development for wildlife and people. <a href="#">Learn more about this credit.</a>

## 3. Water Efficiency

Building level water metering, outdoor and indoor potable water consumption reduction, and low-cost water resources are the requirements for LEED. In our chosen location the officials and policies control water management systems. Automated plumbing systems will be implemented to regulate water consumption by automatically controlling flow rates. For example, low-flow toilets, automatic sensor faucets, and so on. In addition, greywater will be treated and reused for non-potable purposes. This kind of recycling of wastewater will significantly reduce potable water consumption.

**Water Efficiency (6/11)**

- Outdoor Water Use Reduction**  
 pre This credit is intended to reduce outdoor potable water consumption and preserve no and low-cost potable water resources. [Learn more about this credit.](#)
- Indoor Water Use Reduction**  
 pre This credit is intended to reduce indoor potable water consumption and preserve no and low cost potable water resources. [Learn more about this credit.](#)
- Building-Level Water Metering**  
 pre This credit is intended to conserve low cost potable water resources and support water management and identify opportunities for additional water savings by tracking water consumption. [Learn more about this credit.](#)
- Outdoor Water Use Reduction**  
 2pt This credit is intended to reduce outdoor potable water consumption and preserve no and low-cost potable water resources. [Learn more about this credit.](#)
- Indoor Water Use Reduction**  
 6pt This credit is intended to reduce indoor potable water consumption and preserve no and low cost potable water resources. [Learn more about this credit.](#)
- Optimize Process Water Use**  
 2pt This credit is intended to conserve low cost potable water resources used for mechanical processes while controlling corrosion and scale in the condenser water system. [Learn more about this credit.](#)
- Water Metering**  
 1pt This credit is intended to conserve low cost potable water resources and support water management and identify opportunities for additional water savings by tracking water consumption. [Learn more about this credit.](#)

#### ***4. Energy and Atmosphere***

The building should satisfy all the requirements such as minimum energy performance, primary commissioning to support construction, energy management, and decrease economic harms of disproportionate energy consumption. Automatic lighting systems will be considered as an approach to advanced energy metering as it intends to support energy savings. For instance, detecting sensors for turning light and controlling light intensity depends on natural daylight. Besides, all data will be remotely accessible and reported in exact periods. Moreover, heating, ventilation, air conditioning, and service water heating system combinations will be optimized.

* Energy and Atmosphere (25/33)	
<input checked="" type="checkbox"/>	<b>Fundamental Commissioning and Verification</b> pre This credit is intended to support the design, construction, and eventual operation of a project that meets the owner's project requirements for energy, water, indoor environmental quality, and durability. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	<b>Minimum Energy Performance</b> pre This credit is intended to promote resilience and reduce the environmental and economic harms of excessive energy use that disproportionately impact frontline communities by achieving a minimum level of energy efficiency for the building and its systems. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	<b>Building-Level Energy Metering</b> pre This credit is intended to support energy management and identify opportunities for additional energy savings by tracking building-level energy use. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	<b>Fundamental Refrigerant Management</b> pre This credit is intended to reduce ozone depletion and global warming potential and support early compliance with the Kigali Amendment to the Montreal Protocol while minimizing direct contributions to climate change. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	<b>Enhanced Commissioning</b> 6pts This credit is intended to further support the design, construction, and eventual operation of a project that meets the owner's project requirements for energy, water, indoor environmental quality, and durability. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	<b>Optimize Energy Performance</b> 16pts This credit is intended to achieve increasing levels of energy performance beyond the prerequisite standard to reduce environmental and economic harms associated with excessive energy use that disproportionately impact frontline communities. <a href="#">Learn more about this credit.</a>
<input checked="" type="checkbox"/>	<b>Advanced Energy Metering</b> 1pt This credit is intended to support energy management and identify opportunities for additional energy savings by tracking building-level and system-level energy use. <a href="#">Learn more about this credit.</a>
<input type="checkbox"/>	<b>Grid Harmonization</b> 2pts This credit is intended to increase participation in demand response technologies and programs that make energy generation and distribution systems more affordable and more efficient, increase grid reliability, and reduce greenhouse gas emissions. <a href="#">Learn more about this credit.</a>
<input type="checkbox"/>	<b>Renewable Energy</b> 5pts This credit is intended to reduce the environmental and economic harms associated with fossil fuel energy and reduce greenhouse gas emissions by increasing the supply of renewable energy projects and foster a just transition to a green economy. <a href="#">Learn more about this credit.</a>
<input type="checkbox"/>	<b>Enhanced Refrigerant Management</b> 1pt This credit is intended to eliminate ozone depletion and global warming potential and support early compliance with the Montreal Protocol, including the Kigali Amendment, while minimizing direct contributions to climate change. <a href="#">Learn more about this credit.</a>

## 5. Material and Resources

The use of materials and their ingredients for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts are encouraged. Raw materials will be carefully sorted and selected based on life-cycle analysis to ensure environmental responsibility; a portion of cement is replaced with supplementary materials such as fly ash and silica fume, moreover, the use of CobiX technology will allow for a reduction in concrete volume, achieving sustainability goals without sacrificing structural integrity. Additionally, those substitutions will significantly lower carbon dioxide emissions associated with concrete production, which is considered a sustainable approach. A demolition waste management plan will be implemented throughout the project. During the construction process, all waste is disposed of in compliance with local standards for landfill and recycling, also points will be achieved by waste prevention and diversion. Those approaches optimize environmental performance in this category.

**Materials and Resources (8/13)**

- Storage and Collection of Recyclables**  
 2pts This credit is intended to reduce the disproportionate burden of landfills and incinerators that is generated by building occupants' waste hauled to and disposed of in landfills and incinerators through reduction, reuse and recycling service and education, and to conserve natural resources for future generations. [Learn more about this credit.](#)
- Building Life-Cycle Impact Reduction**  
 5pts This credit is intended to encourage adaptive reuse and optimize the environmental performance of products and materials. [Learn more about this credit.](#)
- Environmental Product Declarations**  
 2pts This credit is intended to encourage the use of products and materials for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts. It is also intended to reward project teams for selecting products from manufacturers who have verified improved environmental life-cycle impacts. [Learn more about this credit.](#)
- Sourcing of Raw Materials**  
 2pts This credit is intended to encourage the use of products and materials for which life cycle information is available and that have environmentally, economically, and socially preferable life cycle impacts. It is also intended to reward project teams for selecting products verified to have been extracted or sourced in a responsible manner. [Learn more about this credit.](#)
- Material Ingredients**  
 2pts This credit is intended to encourage the use of products and materials for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts. It is also intended to reward project teams for selecting products for which the chemical ingredients in the product are inventoried using an accepted methodology and for selecting products verified to minimize the use and generation of harmful substances. Furthermore, it is intended to reward raw material manufacturers who produce products verified to have improved life-cycle impacts. [Learn more about this credit.](#)
- Construction and Demolition Waste Management**  
 2pts This credit is intended to reduce construction and demolition waste disposed of in landfills and incineration facilities through waste prevention and by reusing, recovering, and recycling materials, and conserving resources for future generations. Furthermore, it is intended to delay the need for new landfill facilities that are often located in frontline communities and create green jobs and materials markets for building construction services. [Learn more about this credit.](#)

## 6. Indoor Environmental Quality


The building's indoor environmental quality has been designed to meet high standards, focusing on occupants' comfort, productivity, and well-being. Minimum indoor air quality standards will be enforced, with controls in place for environmental tobacco smoke. Enhanced air quality strategies, including the use of low-emitting materials, will reduce chemical contaminants and support healthier indoor spaces. Regular monitoring and assessments will ensure that air quality, lighting, and temperature are maintained at optimal levels, aligning with the building's commitment to a safe and comfortable environment for all. The HVAC systems will be optimized to maintain air quality, while thermal comfort will be ensured through effective insulation and temperature controls. High-quality interior lighting will incorporate smart technologies, such as automatic lighting systems that adjust based on occupancy and natural daylight levels. The design also prioritizes ample daylight access and quality views to connect occupants with the outdoor environment.

**Indoor Environmental Quality (15/16)**

- 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 air quality (IAQ). [Learn more about this credit.](#)  
 req
- 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 environmental tobacco smoke. [Learn more about this credit.](#)  
 req
- 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.](#)

**Your LEED Scorecard**

You've achieved a Gold rating. Just 5 points will get you to Platinum!



## 6. Construction Management

### 6.1. Project Charter

The Project Charter identifies the general overview of a project including its description, scope, objectives, deliverables, risks, issues, assumptions, and stakeholders. The charter is

meant to provide an overview against which the project is to be progressed, preliminary roles, and objectives established. Table 6.1 presents the charter for this project.

Table 6.1.1. Project Charter

<b>Project Title</b>	Design of multi-story hotel in Los Angeles, California, USA		
<b>Project Description</b>	This project deals with the design and construction of a 16-story hotel at 1201 South Grand Avenue, Los Angeles, California, 90015, USA. The site is situated in a bustling commercial area surrounded by office complexes, shopping malls, and restaurants, making it an ideal location for a hotel. Additionally, the high population density in the area highlights the demand for accommodation services for both short-term and long-term visitors.		
<b>Project Start Date</b>	August 2024	<b>Project End Date</b>	March 2026
<b>Project Manager</b>	Kamila Bertebayeva	<b>Project Sponsor</b>	Mert Guney
<b>Purpose</b>	<p><b>Goal:</b> To design a multi-story hotel with a basement in a high seismic zone that follows the latest structural, geotechnical, and LEED standards</p> <p><b>Benefits:</b></p> <ul style="list-style-type: none"> <li>● The hotel will provide comfortable, well-appointed rooms for its guests.</li> <li>● The construction practices to be observed will be environment-friendly.</li> <li>● Increased commercial activities that spill over into the downtown area.</li> </ul>		
<b>Project Scope</b>	<ul style="list-style-type: none"> <li>● Creation of a design unique in focusing on integrating engineering aspects</li> <li>● The management and control of this process of construction requires</li> <li>● Supply of materials, waste management</li> <li>● Continuous facility maintenance</li> </ul>		

<b>Project Title</b>	Design of multi-story hotel in Los Angeles, California, USA		
<b>Project deliverables</b>	<ol style="list-style-type: none"> <li>1. 16-story hotel</li> <li>2. Underground parking lot</li> </ol>		
<b>Project risks</b>	<p>Financial risk:</p> <ul style="list-style-type: none"> <li>● Budget overruns</li> </ul> <p>Schedule risk:</p> <ul style="list-style-type: none"> <li>● Construction delays</li> </ul> <p>Safety risk:</p> <ul style="list-style-type: none"> <li>● On-site accidents</li> </ul> <p>Legal and compliance risk:</p> <ul style="list-style-type: none"> <li>● Permitting and regulatory delays</li> </ul>		
<b>Project assumptions</b>	<ul style="list-style-type: none"> <li>● No act of God will cause the project to stop</li> <li>● There are enough construction crew employees to complete the project</li> <li>● Construction subcontractors will complete their work on time</li> </ul>		
<b>Estimated budget</b>	\$26 112 548		
<b>Project Start Date</b>	Stage	Deadline	Status
	Project Initiation and Planning	14.10.2024	Finished
	Design and Permits	31.01.2025	In progress
	Site Preparation	31.01.2025	Not started
	Foundation and Structure	29.08.2025	Not started
	Utilities and Systems	14.01.2026	Not started
	Project Closure	14.05.2026	Not started
<b>Project team</b>	Azamat Dossov		

<b>Project Title</b>	Design of multi-story hotel in Los Angeles, California, USA
	Kamila Bertebayeva Anuar Dyussenbekov Gulnaz Islyambekova Bagdat Seiilkhan Shynggys Seilkhan

## 6.2. Feasibility study

Every major construction must be preceded by rigorous analysis. The 16-story high-rise hotel is one of the massive and complex developments, hence requiring a comprehensive feasibility study to make sure the project would be viable. The feasibility study below discusses each critical aspect of the forthcoming project.

### Site Analysis:

The site chosen is in the middle of downtown Los Angeles, surrounded by Shopping centers, restaurants, and corporate offices. The location is strategic in terms of the hotel's demand from business people and leisure travelers. The construction site will be big enough to accommodate all construction work, storage of materials, and logistics. After construction is completed, the site will have a parking area and a landscaped entryway. The hotel structure will be placed toward the south of the site to provide easy vehicular entry and exit onto the main streets.

### Design and Development Plan:

From the feasibility study, a detailed design concept was needed for the hotel that supports modern and inviting structures and is aligned with future expansion opportunities. The 3D designs shall be prepared to meet California and Los Angeles building codes, and approvals needed for seismic safety, energy efficiency, and LEED certification markings. To ensure the quality of the demanded structure in terms of safety and durability because of the seismic-prone area, great emphasis has been put on structural, geotechnical, and sustainable building practices.

**Environmental Impact Assessment:**

Seeing that Los Angeles is within an earthquake-prone area, all construction decisions concerning the environment and safety should be guided by the California Environmental Quality Act. A preliminary environmental impact assessment study has been carried out to identify ecological threats and therefore include measures that mitigate such occurrences. The construction will manage wastes that are produced, with a plan for material recycling and waste disposal. Similarly, it will retain environmentally friendly practices in the waste produced by guests and day-to-day operations.

**Legal Analysis:**

The hotel will need approvals for all building codes, environmental laws, and zoning regulations of all relevant local authorities. Designs and environmental reports will be submitted for initial review; this needs professional consultation to meet regulations and stay within the letter of the law in every regard. Contractor contracts, property leases, permits, and related agreements have rigorous management and input at all levels regarding worker safety, labor laws, and property rights.

**Economic Impact Analysis:**

The under-construction 16-story hotel will provide both direct and indirect employment opportunities to people through architects, construction teams, engineers, project managers, and suppliers. It is regarded as a step towards development in the infrastructure of the place, which could boost the local economy. The hotel is expected to attract a good number of guests, thus contributing to the local economy. The pricing models show that room rates are to vary according to facilities and category. Revenue projections for the sale of rooms, coupled with other facilities on site, should provide a good cash flow that aspires to a payback period several years after opening.

**Project Schedule and Timeline:**

The scope of the project was identified, coupled with an efficient work breakdown structure that organized every stage of the construction. The time all expires for weather conditions, public holidays, and other days that are not considered working in construction are included. According to the results of the feasibility study, this 16-story hotel project can be viable in downtown Los Angeles, serving high accommodation that caters to the needs of both business and leisure travelers. This will no doubt make quite

positive contributions toward the local economy and urban landscape. A decisively chosen location with comprehensive planning in every minute detail, and confirmation of regulatory requirements on all counts, speaks volumes for its success, giving investors and stakeholders reason to be confident.

### 6.3. Cost Estimations

The cost estimate for the hotel was developed using RSMeans online software, with all expected costs provided a line item allowance. The approximate total construction cost as reflected in Figure 6.1 is \$47 266 384, while the detailed construction cost, which is presented in Table 6.2 is \$26 112 548.



Figure 6.3.1. Estimated values from RSMeans

Table 6.3.1. Detailed Cost Estimation from RS Means

		Quantity	Cost per SF	Cost
A	Substructure		\$60.38	\$1,316,140.98
A1010	Standard Foundations		\$0.47	\$10,238.42
	Pile caps, 6 piles, 8' - 6" x 5' - 6" x 40", 80 ton capacity, 19" column size, 936 K column	1.55	\$0.16	\$3,501.08
	Pile caps, 8 piles, 8' - 6" x 7' - 9" x 44", 80 ton capacity, 22" column size, 1243 K column	2.17	\$0.31	\$6,737.34

A1020	Special Foundations		\$27.76	\$605,136.83
	Steel H piles, 100' long, 800K load, end bearing, 5 pile cluster	1.55	\$2.32	\$50,474.72
	Steel H piles, 100' long, 1200K load, end bearing, 8 pile cluster	2.17	\$5.20	\$113,398.89
	Grade beam, 30' span, 52" deep, 14" wide, 12 KLF load	1938	\$20.24	\$441,263.22
A1030	Slab on Grade		\$0.41	\$8,863.81
	Slab on grade, 4" thick, non-industrial, reinforced	1362.31	\$0.41	\$8,863.81
A2010	Basement Excavation		\$0.31	\$6,698.76
	Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on-site storage	1362.31	\$0.31	\$6,698.76
A2020	Basement Walls		\$31.44	\$685,203.16
	Foundation wall, CIP, 12' wall height, pumped, .591 CY/LF, 28.79 PLF, 16" thick	1938	\$31.44	\$685,203.16
B	Shell		\$625.24	\$13,628,361.58
B1010	Floor Construction		\$293.92	\$6,406,522.47
	Cast-in-place concrete column, 26" square, tied, 1000K load, 12' story height, 667 lbs/LF, 4000PSI	12775.77	\$144.89	\$3,158,223.06
	Cast-in-place concrete column, 12",	7868.28	\$29.81	\$649,749.97

	square, tied, minimum reinforcing, 150K load, 10'-14' story height, 135 lbs/LF, 4000PSI			
	Cast-in-place concrete column, 16", square, tied, minimum reinforcing, 300K load, 10'-14' story height, 240 lbs/LF, 4000PSI	7868.28	\$40.62	\$885,417.55
	Cast-in-place concrete column, 20", square, tied, minimum reinforcing, 500K load, 10'-14' story height, 375 lbs/LF, 4000PSI	7868.28	\$57.89	\$1,261,777.05
	Cast-in-place concrete beam and slab, 7.5" slab, two-way, 12" column, 25'x25' bay, 40 PSF superimposed load, 149 PSF total load	20434.68	\$19.64	\$428,097.51
	Flat slab, concrete, with drop panels, 6" slab/2.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 PSF total load	1362.31	\$1.07	\$23,257.33
B1020	Roof Construction		\$1.18	\$25,750.09
	Roof, concrete, beam and slab, 25'x25' bay, 40 PSF superimposed load, 20" deep beam, 9" slab, 152 PSF total load	1362.31	\$1.18	\$25,750.09
B2010	Exterior Walls		\$282.87	\$6,165,779.56
	Exterior wall, precast concrete, flat, 8" thick, 10' x 10', white face, 2"	99225.6	\$282.87	\$6,165,779.56

	rigid insulation, low rise			
B2020	Exterior Windows		\$41.45	\$903,556.94
	Windows, aluminum, awning, insulated glass, 4'-5" x 5'-3"	1078.53	\$41.45	\$903,556.94
B2030	Exterior Doors		\$0.35	\$7,559.20
	Door, aluminum & glass, without transom, narrow stile, with panic hardware, 3'-0" x 7'-0" opening	0.48	\$0.09	\$2,041.00
	Door, aluminum & glass, without transom, narrow stile, double door, hardware, 6'-0" x 7'-0" opening	0.58	\$0.19	\$4,087.69
	Door, steel 18 gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening	0.48	\$0.07	\$1,430.51
B3010	Roof Coverings		\$5.12	\$111,702.77
	Roofing, single-ply membrane, EPDM, 60 mils, loosely laid, stone ballast	1362.31	\$0.12	\$2,552.91
	Insulation, rigid, roof deck, extruded polystyrene, 40 PSI compressive strength, 4" thick, R20	1362.31	\$0.28	\$6,016.18
	Roof edges, aluminum, duranodic, .050" thick, 6" face	1938	\$2.86	\$62,393.43
	Flashing, aluminum, no backing sides, .019"	1938	\$0.63	\$13,644.10
	Gravel stop, aluminum, extruded, 4",	1938	\$1.24	\$27,096.15

	mill finish, .050" thick			
B3020	Roof Openings		\$0.34	\$7,490.55
	Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs	6	\$0.34	\$7,490.55
C	Interiors		\$55.73	\$1,214,672.75
C1010	Partitions		\$16.60	\$361,823.65
	Concrete block (CMU) partition, lightweight, hollow, 6" thick, no finish	1937.51	\$1.15	\$24,958.24
	Metal partition, 5/8" fire-rated gypsum board face, no base, 3 -5/8" @ 24" OC framing, same opposite face, sound attenuation insulation	17437.6	\$5.81	\$126,695.67
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"	99225.6	\$5.65	\$123,178.66
	Add for the following: taping and finishing	99225.6	\$3.99	\$86,991.08
C1020	Interior Doors		\$14.92	\$325,264.52
	Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"	242.18	\$14.92	\$325,264.52
C2010	Stair Construction		\$2.36	\$51,345.32
	Stairs, steel, pan tread for conc in-fill, picket rail, 16 risers w/ landing	3	\$2.36	\$51,345.32

C3010	Wall Finishes		\$10.24	\$223,188.65
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	31000.17	\$1.59	\$34,730.74
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	99225.6	\$5.10	\$111,166.41
	Ceramic tile, thin set, 4-1/4" x 4-1/4"	7750.04	\$3.55	\$77,291.50
C3020	Floor Finishes		\$6.28	\$136,855.60
	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 35 oz	17437.6	\$4.52	\$98,532.55
	Vinyl, composition tile, maximum	2179.7	\$0.35	\$7,646.82
	Tile, ceramic natural clay	2179.7	\$1.41	\$30,676.23
C3030	Ceiling Finishes		\$5.33	\$116,195.01
	Gypsum board ceilings, 5/8" fire-rated gypsum board, painted and textured finish, 1-5/8" metal stud furring, 24" OC support	21797	\$5.33	\$116,195.01
D	Services		\$100.40	\$2,188,421.62
D1010	Elevators and Lifts		\$7.30	\$159,077.41
	Traction geared freight, 4000 lb., 15 floors, 10' story height, 200FPM	0.04	\$1.37	\$29,900.64
	Traction, geared passenger, 3500 lb, 15 floors, 10' story height, 2 car group, 350 FPM	0.24	\$5.93	\$129,176.77

D2010	Plumbing Fixtures		\$21.53	\$469,242.05
	Water closet, vitreous china, bowl only with flush valve, wall hung	54.49	\$9.18	\$200,036.52
	Urinal, vitreous china, wall hung	1.21	\$0.10	\$2,116.60
	Lavatory w/trim, vanity top, PE on CI, 20" x 18"	54.49	\$4.07	\$88,784.36
	Kitchen sink w/trim, countertop, stainless steel, 33" x 22" double bowl	0.38	\$0.05	\$1,016.40
	Service sink w/trim, PE on CI, wall hung w/rim guard, 22" x 18"	1.45	\$0.28	\$6,054.19
	Bathtub, recessed, PE on CI, mat bottom, 5' long	54.49	\$7.61	\$165,790.98
	Shower, stall, baked enamel, terrazzo receptor, 36" square	1.21	\$0.18	\$3,842.25
	Water cooler, electric, wall hung, wheelchair type, 7.5 GPH	0.72	\$0.07	\$1,600.75
D2020	Domestic Water Distribution		\$0.22	\$4,787.74
	Gas-fired water heater, commercial, 100< F rise, 500 MBH input, 480 GPH	0.16	\$0.22	\$4,787.74
D2040	Rain Water Drainage		\$0.27	\$5,796.04
	Roof drain, CI, soil, single hub, 5" diam, 10' high	0.38	\$0.06	\$1,278.21
	Roof drain, CI, soil, single hub, 5" diam, for each additional foot add	54.25	\$0.21	\$4,517.83

D3010	Energy Supply		\$3.32	\$72,324.52
	Commercial building heating system, fin tube radiation, forced hot water, 1mil SF, 10 mil CF, total 5 floors	23976.7	\$3.32	\$72,324.52
D3030	Cooling Generating Systems		\$16.20	\$353,052.55
	Packaged chiller, water-cooled, with fan coil unit, medical centers, 60,000 SF, 140.00 ton	21797	\$16.20	\$353,052.55
D4010	Sprinklers		\$4.96	\$108,151.83
	Wet pipe sprinkler systems, steel, light hazard, 1 floor, 50,000 SF	15257.9	\$2.45	\$53,385.71
	Wet pipe sprinkler systems, steel, light hazard, each additional floor, 50,000 SF	20271.21	\$2.45	\$53,485.39
	Standard High Rise Accessory Package 16 story	0.04	\$0.06	\$1,280.73
D4020	Standpipes		\$6.39	\$139,326.05
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor	0.14	\$0.12	\$2,615.49
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, additional floors	20.34	\$4.60	\$100,179.81
	Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM	1	\$1.49	\$32,535.10
	Fire pump, electric, for jockey pump system, add	1	\$0.18	\$3,995.65

D5010	Electrical Service/Distribution		\$21.84	\$476,041.12
	Underground service installation, includes excavation, backfill, and compaction, 100' length, 4' depth, 3 phase, 4 wire, 277/480 volts, 2000 A	2	\$5.36	\$116,733.00
	Feeder installation 600 V, including RGS conduit and XHHW wire, 60 A	100	\$0.11	\$2,373.87
	Feeder installation 600 V, including RGS conduit and XHHW wire, 200 A	100	\$0.25	\$5,490.25
	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A	400	\$10.40	\$226,700.40
	Switchgear installation, incl switchboard, panels & circuit breaker, 277/480 V, 2000 A	2	\$5.72	\$124,743.60
D5020	Lighting and Branch Wiring		\$12.76	\$278,051.31
	Receptacles incl plate, box, conduit, wire, 10 per 1000 SF, 1.2 W per SF, with transformer	22232.94	\$4.77	\$104,073.50
	Wall switches, 5.0 per 1000 SF	21797	\$1.63	\$35,486.82
	Miscellaneous power, to .5 watts	21797	\$0.19	\$4,062.74
	Central air conditioning power, 4 watts	26592.34	\$0.94	\$20,530.88
	Motor installation, three-phase, 460 V, 15 HP motor size	10	\$1.43	\$31,063.05

	Motor feeder systems, three-phase, feed to 200 V 5 HP, 230 V 7.5 HP, 460 V 15 HP, 575 V 20 HP	500	\$0.32	\$7,056.89
	Motor connections, three-phase, 200/230/460/575 V, up to 5 HP	1	\$0.01	\$171.30
	Motor connections, three-phase, 200/230/460/575 V, up to 100 HP	1	\$0.03	\$719.05
	Fluorescent fixtures recess mounted in the ceiling, 0.8 watt per SF, 20 FC, 5 fixtures @32 watt per 1000 SF	21797	\$3.44	\$74,887.08
D5030	Communications and Security		\$5.28	\$115,152.87
	Communication and alarm systems, fire detection, addressable, 100 detectors, includes outlets, boxes, conduit, and wire	0.55	\$2.35	\$51,119.55
	Fire alarm command center, addressable with voice, excl. wire & conduit	0.14	\$0.09	\$1,874.33
	Communication and alarm systems, include outlets, boxes, conduit and wire, intercom systems, 100 stations	0.17	\$1.26	\$27,486.86
	Communication and alarm systems, include outlets, boxes, conduit and wire, master TV antenna systems, 100 outlets	0.12	\$0.99	\$21,494.13
	Internet wiring, 2 data/voice outlets per 1000 S.F.	16.34	\$0.60	\$13,178.00

D5090	Other Electrical Systems		\$0.34	\$7,418.13
	Generator sets, w/battery, charger, muffler, and transfer switch, diesel engine with fuel tank, 500 kW	32.79	\$0.34	\$7,418.13
E	Equipment & Furnishings		\$62.39	\$1,359,986.69
E1090	E1090		\$62.39	\$1,359,986.69
E1090 D1010 150200 0	2.00-Traction gearless elevators, passenger, 3000 lb, 10 floors, 200 FPM	2	\$40.76	\$888,503.00
E1090 D1010 150380 0	1.00-Traction gearless elevators, passenger, 5000 lb, 10 floors, 200 FPM	1	\$20.81	\$453,692.50
E1090 265213 100500	32.00-Emergency lighting units, lead battery operated, twin sealed beam light, 25 W, 6 V each	32	\$0.56	\$12,186.24
E1090 284611 275200	16.00-Detection system, heat detector, smoke detector, ceiling type, excl. wires & conduit	16	\$0.19	\$4,068.00
E1090 282313 102000	1.00-Closed-circuit television system (CCTV), surveillance, one station (camera & monitor)	1	\$0.07	\$1,536.95
	SubTotal		\$904.14	\$19,707,583.62

	Contractor Fees (GC, Overhead, Profit)		\$226.04	\$4,926,895.91
	Architectural Fees		\$67.81	\$1,478,068.77
	<b>Total Building Cost</b>		<b>\$1,197.99</b>	<b>\$26,112,548.30</b>

Table 6.2 above outlines the total net cost, encompassing materials, contractor fees, architectural fees, labor, and overhead expenses. Future work will involve incorporating additional materials and determining their actual costs for 2024 in Los Angeles. Since the available version of RSMeans uses data from 2018, a comprehensive literature review on material costs, various fees, and labor rates will be necessary. This will be followed by an analysis and comparison with updated cost estimates to reflect current pricing. Consequently, some changes in the overall cost of the commercial building are anticipated. It is also worth noting that while RSMeans offers several advantages, the absence of certain specialized construction activities and engineering materials affects the accuracy of the cost estimation.

#### **6.4 Work Breakdown Structure**

The project scope involves defining the work to be done in delivering the desired outcomes, which also includes the processes that handle changes bound to happen so the project can stay on schedule and within budget. The scope is established with the WBS and incorporates changes through formal procedures. WBS is of great importance when it comes to the planning of the general structure of any building process. WBS also delineates and outlines the entire project scope in a product-oriented, graphical format. Since the WBS belongs to the most important tools within the project, it requires proper planning and thoughtfulness during its development to reduce an impact on later phases. WBS provides a basis for project initiation and planning, design and permits, site preparations, foundation and structure, utilities and systems, and project closure. A well-structured WBS goes a long way in increasing the chances of completing the project.

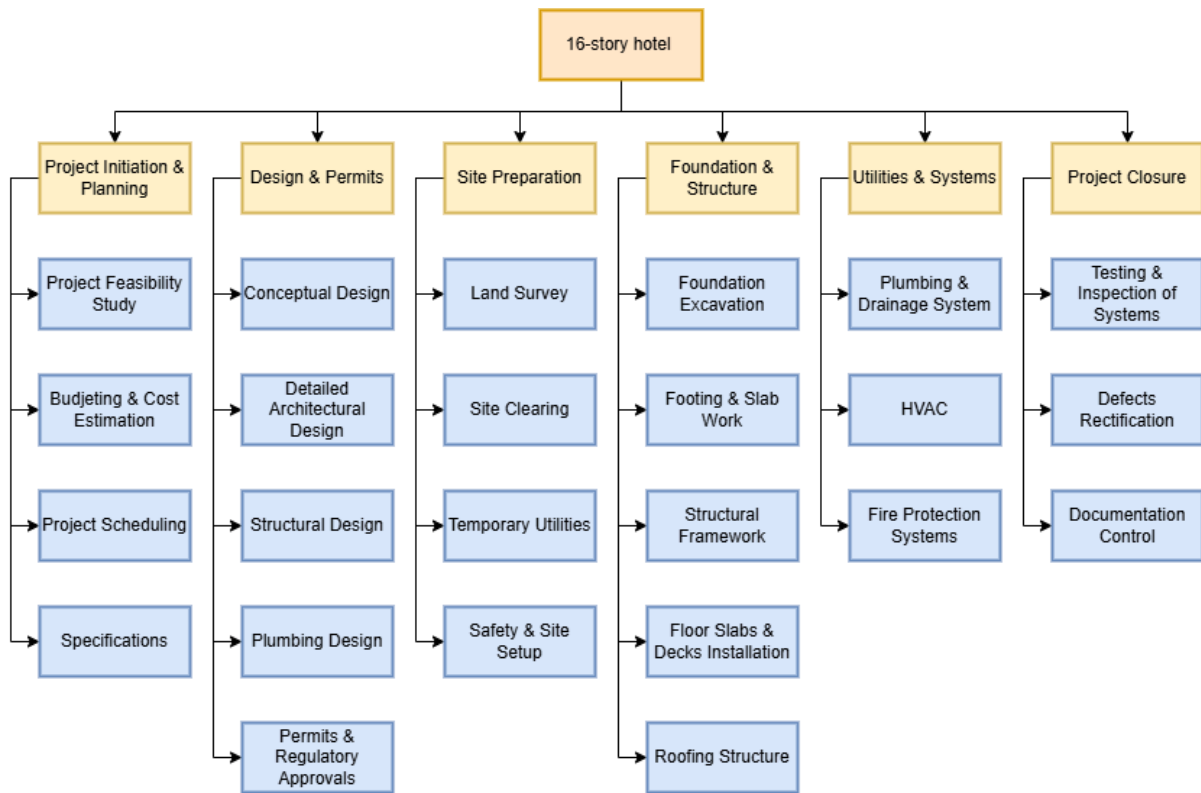


Figure 6.4.1. Work Breakdown Structure

## 6.5 Scheduling

The project schedule was prepared on Primavera P6 Professional in the form of a Gantt chart, ranging from the planning phase up to the project's submission. Summarily, the activities include site preparation, laying the foundation, doing structural work, and finishing. This schedule will help in managing the resources optimally with the possibility of attempting various tasks concurrently to ensure proper management of the project. According to the chart, the total duration of the project shall be 435 working days, with a completion date expected on 31.03.2026.

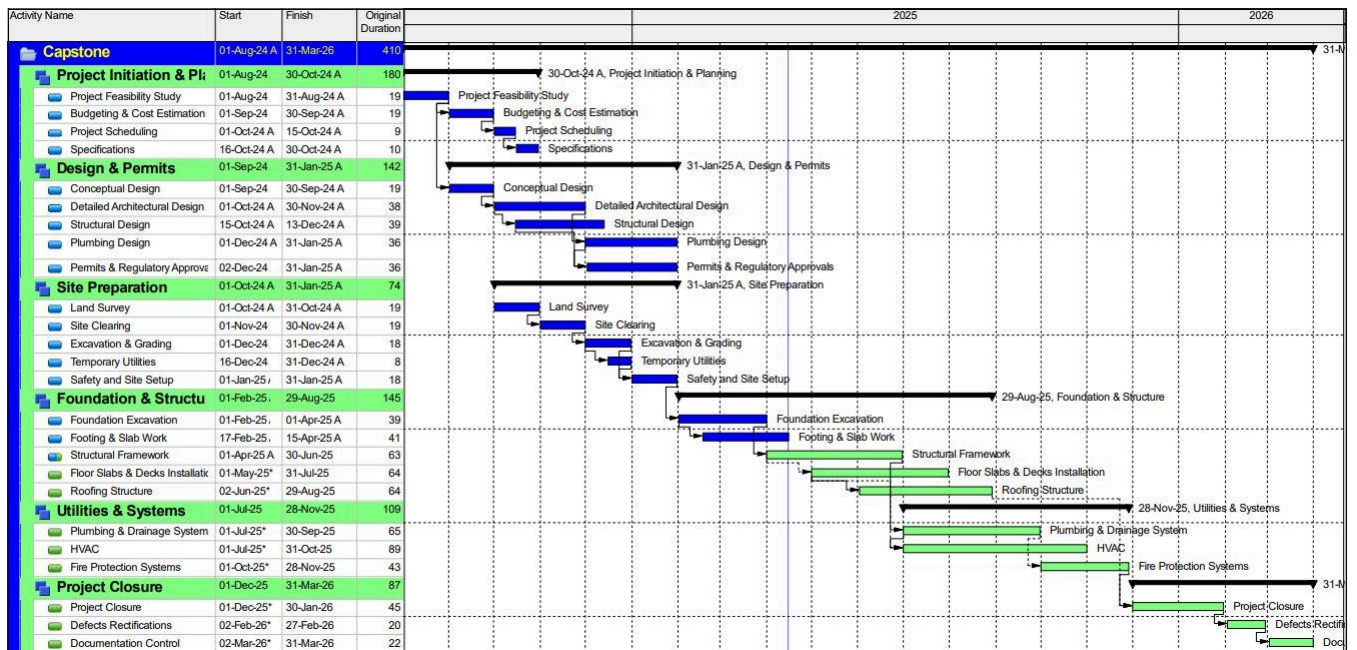


Figure 6.5.1. Gantt Chart

## 6.6 Risk Management

Every construction project involves some inherent risk and thus the need for sufficient preparation especially between the contractors and the owners. Such a process of construction involves conducting a critical risk management analysis to avoid financial loss or even project delay. As far as the industry of construction is concerned, the identification of risks concerns the categorization of risks, analysis of their characteristics, and estimation of the chance of occurrence. Effective risk management is not limited to addressing the issue during or after the construction itself, but rather proactive measures can be taken to show the mitigation of any adverse impacts by comprehensive analysis of the scenario. The analysis will help the decision-maker in making the most optimal decisions using various analytical and mathematical tools at their disposal for the same purpose. By taking into consideration all the parameters and probabilities of risks, investors can decide whether to retain the risks, be neutral towards them, or avoid them.

The risk types, description, likelihood and impact, risk score, and mitigation strategies are identified in Table 6.3. The risk score should be calculated by the formula below.

$$\text{Risk Score} = \text{Likelihood} \times \text{Impact}$$

Table 6.6.1. Risk Assessment Matrix

Type of risk		Description of risk	Likelihood	Impact	Risk Score	Mitigation measures
Financial	F1	Cost Overruns	6	8	48	Have the budgets reviewed regularly, and consider contingency amounts. Ensure a very accurate initial cost estimate by setting up cost-tracking software to monitor expenses.
	F2	Financing Delays	4	7	28	Arrange financing early in the project. Communicate with lenders clearly; have backup financing lined up.
Schedule	Sc 1	Delays in Permits and Approvals	5	7	35	Early application for permit and approval: The more prepared one is in advance with their documentation, the more one can keep in contact with the local authorities.
	Sc 2	Weather-Related Delays	3	6	18	Check weather forecasts and schedule critical construction phases when the weather is mild. Allow in your project schedule for contingencies related to weather delays.

Environmental	E1	Soil and Geological Issues	4	8	32	The geotechnical survey is in detail before the commencement of the construction. If defects are identified, the design modification must be done accordingly.
	E2	Environmental Compliance Violations	3	9	27	Fully adhere to all environmental legislations. Consultations from experts in the field of environment will be sought on how best to comply with the local, state, and federal standards.
Safety	S1	Construction Site Accidents	5	9	45	Participate in ongoing training concerning safety issues and adherence to OSHA regulations. Ensure safety equipment is provided, with routine inspections.
	S2	Fire and Hazards	3	7	21	Install the fire safety plan at work and install fire suppression systems. Conduct fire drills and train workers on emergency procedures.
Operational	O1	Equipment Failures	4	6	24	Calling for routine equipment maintenance and preparation of backup or

						rental equipment if needed. Schedule preventative maintenance for key machinery.
	O2	Labor Shortages	6	8	48	Establish solid contracts and secure sources for skilled labor. If there is a need for more workers, flexible labor arrangements should be utilized.
Legal	L1	Contractual Disputes	4	7	28	Use full, comprehensive, and transparent contracts and engage the services of a legal expert when preparing a contract. Establish machinery for quickly resolving conflict in case of a breakdown.
	L2	Regulatory Changes	3	6	18	Keep abreast of regulatory changes and work with legal advisors to meet new regulations of whatever kind.
Supply Chain	SC 1	Material Delays	5	8	40	Diversify suppliers and hold a proper inventory at the site of the most crucial inputs. Relation building with different types of vendors, not just one, should be made so that when one supplier is

						late, others can compensate for it.
	SC 2	Price Increases of Key Materials	4	7	28	Benefits might also be realized by considering price-lock contracts or negotiating long-term contracts with suppliers that avoid price increases.
Stakeholder	St1	Community Opposition	4	6	24	Hold community engagement meetings so issues can be raised. Keep the lines of communication open to keep community support and plans are adapted where possible.
	St2	Client Scope Changes	5	8	40	Establish a process for managing changes. The cost and schedule impacts are to be highlighted for any changes requested. This will help in the reduction of scope creep.
Design	D1	Design Errors	6	8	48	Conduct several design reviews and audits. Use experienced design staff, including structural engineers on key analyses as required. Establish a formal design change approval process.

	D2	Design Changes During Construction	5	7	35	Limit the changes by freezing the designs as early as possible and making sure everyone concerned is on the same page. Establish a change management system where any design changes should be checked for their impact before considering approval of such changes.
	D3	Structural Design Failures	4	9	36	Ensure all the designs are congruent with the local building codes, in particular, those to do with seismic activity. Engage highly qualified engineers for site-checking of structural stability and allow for redundancy in design.

Upon establishing the risks, a risk management matrix, also known as a risk control matrix, was developed. The matrix quantifies each risk in terms of its probability as well as estimates its probable damage. The distribution and severity of all risks identified are illustrated in the following figure, categorized into three categories: high, medium, and low. The risks are largely in a category with a medium impact.

		Impact		
		Low	Medium	High
Likelihood	High			
	Medium		F2, Sc1, O1 L1, SC2, St1 D2	F1, E1, S1 O2, SC1, St1 D1, D3
	Low		Sc2, E2, S2 L2	E2

Figure 6.6.1. Risk Management Matrix

## 6.7 Quality Management

Construction quality management verifies that all construction processes are in accordance with performance, safety, and regulatory specifications. Quality management plays a major role in defect reduction, efficiency improvement, and customer satisfaction. Quality control is one of the vital components of quality management that involves monitoring and evaluation through systematic steps. The following steps are followed to maintain construction quality:

### 1. Corrective & Preventive Actions

To prevent defects and enhance efficiency, preventive measures are taken from the beginning of the project. Corrective measures are taken for any deviations found to maintain adherence to construction standards.

### 2. Equipment Control

Ensure high-quality and properly maintained equipment is utilized. Regular inspections, routine maintenance, and performance testing of equipment avoid breakdowns during operations.

### 3. Labor Quality Management

Skilled labor is a key element of quality construction. Workers undergo training programs, safety orientations, and regular performance reviews to maintain high standards of workmanship.

4. Design Controls All the designs are thoroughly checked for structural strength, functionality, and adherence to regulations. Design proposals are reviewed before implementation to detect and address possible weaknesses.

#### 5. Supplier Evaluation

Material and equipment suppliers are selected based on quality, reliability, and conformity to industry standards. Continual assessment and comparative analysis of suppliers help to ensure material quality consistency.

#### 6. Document & Record Controls

Detailed documentation of inspections, tests, and remedial works ensures transparency and accountability. Record keeping enables monitoring project progress and compliance with regulatory requirements.

#### Inspection & Test Plans

A systematic inspection and testing plan is conducted to ensure construction quality at every stage. Material inspections ensure the durability and compliance of raw materials with project specifications, and task inspections ensure that all construction activities fulfill quality standards. Various tests, including structural integrity, system functionality, and safety testing, are conducted to verify the reliability of the project before completion. The planned approach helps ensure consistency, detect defects early, and ensure overall project success.

By integrating these quality control procedures, construction projects reach high standards, reduce risks, and ensure long-term safety and durability.



Figure 6.7.1. Construction Control Quality Procedure

### 6.8 Safety management

Ensuring construction safety is crucial for both workers and the overall execution of construction processes, making it a top priority for all stakeholders in a project. In residential building construction, compliance with safety regulations and standards, such as OSHA guidelines, is essential. Initially, a dedicated team must develop a comprehensive safety plan, and workers should be informed about potential hazards, risks, and safety measures. Additionally, the safety plan should be regularly updated throughout the construction phases. Workers must also be equipped with personal protective gear and trained in hazard prevention and management. Beyond worker safety, all site visitors must adhere to safety protocols, and construction activities should not pose risks to public safety. This includes proper material transportation and storage, noise control, and other related factors (Akinlolu et al., 2020).

Here is a table that presents likely hazards on site with ratings. Fall from height is at a high level of risk, with structural collapse, fire and explosions as least at risk out of all.

Table 6.8.1. Assessment of Risks on-site

Risk Identification	Likelihood (1-5)	Severity (1-5)	Risk Rating (1-25)
Fall from Height	4	5	20
Heavy Equipment Accident	3	5	15
Slips, Strips and Falls	5	3	15
Structural Collapse	2	5	10
Exposure to Hazardous Materials	3	4	12
Fire and Explosions	2	5	10
Electrical Hazards	3	5	15
Manual Handling Injuries	4	3	12
Noise and Vibration Exposure	4	3	12
Weather - Related Risks	3	4	12

## 6.9 Construction Site Planning

Construction planning is one of the most important steps in planning the construction process, as it specifies the location and layout of the temporary facilities and equipment utilized at various stages of construction. Such facilities and equipment include material storage areas, routes for machinery, site offices, waste and excavated ground disposal areas and motor park, all of which depend on the availability of space and the construction stage requirements.

Special planning was needed for this high-rise hotel development at 1201 S Grand Avenue, Downtown Los Angeles, in view of the urban location and surrounding roadways. As in Figure 6.6, circulation routes, stockpile areas, machine placement, and the major building footprint were all accommodated in a compact and efficient manner in the site layout.

The construction footprint takes over the western half of the site, and the eastern half will be held in reserve for major temporary purposes, including storage of construction

machinery, reinforcement stockpiles, and zones of holding construction material. A construction access road zigzags through the site to link these zones to the site entry and office cabins to enable the safe and efficient traffic of vehicles and personnel.

Firstly, a specific area of ground that was excavated when the underground operations were going on will be used later. Once the underground work is done and the earth has been cleared from the site, the area will be freed and utilized for key construction activities, i.e., tower crane installation, assembly of the formwork, and further material staging. This space usage in stages enhances the site's efficiency and versatility.

To ensure safety and order, the location will be entirely fenced in, and clear signage, pedestrian pathways, and emergency areas will be instituted according to safety guidelines. This systematic structure allows for efficient logistics, reduced disruption of the public, and a safe working condition for construction workers.

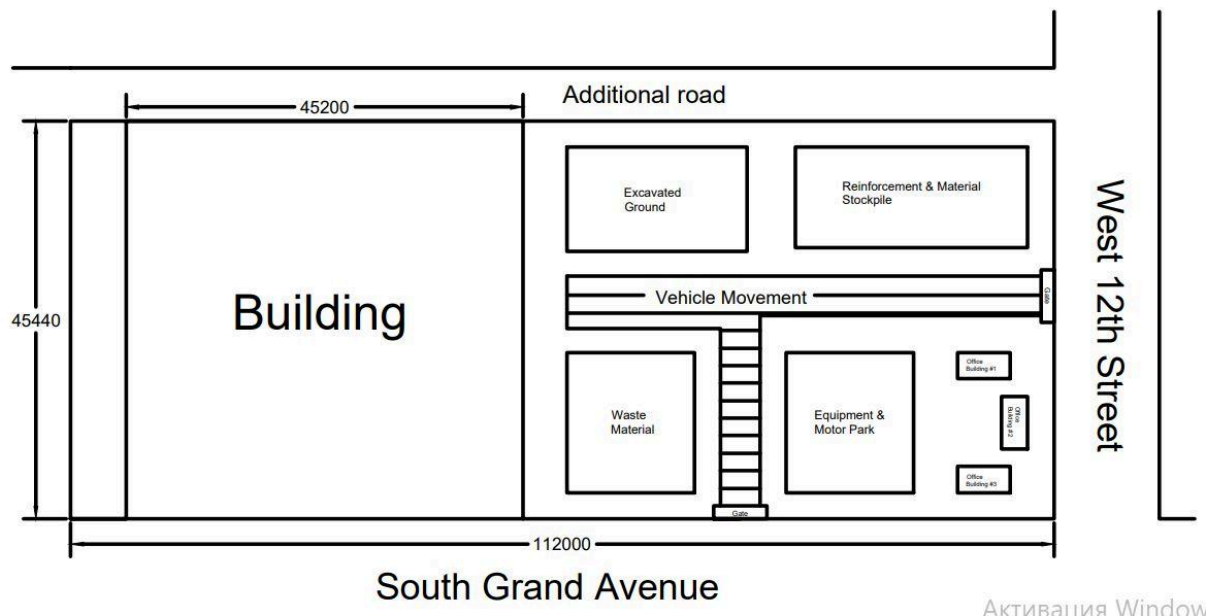


Figure 6.9.1. The construction site layout planning

## 6.10 Stakeholder Analysis

Stakeholder analysis is a critical element of project planning in construction that requires the identification of all the major parties, the understanding of their roles to play, and the management of their interests throughout the entire project duration. According to the Project Management Institute (2017), success in massive construction projects largely depends on accurate stakeholder identification and ongoing interaction.

For the building of the high-rise hotel the stakeholders involve the client (owner), the general contractor, the subcontractors, the design and engineering teams, external

consultants, government agencies, and indirectly, the neighboring community. Each of them has a specific role and different levels of control over the development of the project and its outcomes.

Every construction task has a specific lead party assigned to its execution, management, and coordination. This responsibility assignment provides clarity, prevents duplication of efforts, and encourages accountability. The delegation of responsibility throughout different stages and parties is reflected in Figure 6.7, which identifies who controls each significant element of the project.

To support the above, Table 6.5 summarizes all the stakeholders, including their particular tasks, level of influence (high, medium, and low), and recommended engagement strategies. This systematic approach facilitates better communication, reduces the likelihood of conflicts, and ensures the project stays in line with time, budget, and quality targets.

As observed by Olander (2007), stakeholder analysis also comes into play to identify the risks in the project at the earliest and to create mitigation strategies to keep all the concerned parties aligned.

Table 6.10.1. Stakeholder matrix

Position of stakeholder	Role in project	Interest	Influence
Project Director	Manages the whole project; makes the final decision	High	High
Project Manager	Oversees day-to-day operations and coordination	High	High
Planning Engineer	Creates timeline, tracks progress	Moderate	Moderate
Design Lead	Coordinates all the design elements and consultants	High	High
Architect	Prepares conceptual and architectural design	High	Moderate
Structural Engineer	Creates and maintains structural integrity	High	Moderate
MEP Engineer	Plans and organizes mechanical, electrical, and plumbing systems	High	Moderate
Site Engineer	Manages the on-site operation and labor	Moderate	High

Safety Officer	Ensures safety protocol and regulation compliance	Moderate	Moderate
QA/QC Officer	Performs inspections and quality control	High	Moderate
Documentation Specialist	Manages all closeout, contracts, and documentation	Moderate	Low
HVAC Specialist	Installs the HVAC systems and verifies operation	Moderate	Low
Surveyor	Performs land and layout surveys	Low	Low
Financial Controller	Manages the budget, tracking costs, and financial risks	High	High
City/Permit Authorities	Issues construction permits and conducts inspections	Low	High
Hotel Investors/Owners	Invest capital, anticipate profitability	Low	High
Local Community	Impacted by construction activities	Moderate	Low

<b>Project Initiation &amp; Planning</b>	
Project Feasibility Study	PM.Project Manager
Budgeting & Cost Estimation	PM.Project Manager
Project Scheduling	PD.Project Director
Specifications	PM.Project Manager
<b>Design &amp; Permits</b>	
Conceptual Design	A.Architect
Detailed Architectural Design	A.Architect
Structural Design	SE.Structural Engineer
Plumbing Design	MEPMEP Engineer
Permits & Regulatory Approvals	PM.Project Manager
<b>Site Preparation</b>	
Land Survey	S.Surveyor
Site Clearing	SIE.Site Engineer
Excavation & Grading	SIE.Site Engineer
Temporary Utilities	MEPMEP Engineer
Safety and Site Setup	SO.Safety Officer
<b>Foundation &amp; Structure</b>	
Foundation Excavation	SIE.Site Engineer
Footing & Slab Work	SIE.Site Engineer
Structural Framework	SE.Structural Engineer
Floor Slabs & Decks Installation	SE.Structural Engineer
Roofing Structure	SIE.Site Engineer
<b>Utilities &amp; Systems</b>	
Plumbing & Drainage System	MEPMEP Engineer
HVAC	HVAC.HVAC Specialist
Fire Protection Systems	MEPMEP Engineer
<b>Project Closure</b>	
Project Closure	QA/QC.QA/QC Officer
Defects Rectifications	QA/QC.QA/QC Officer
Documentation Control	DS.Documentation Specialist

Figure 6.10.1. Responsibilities

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## 8. Appendix

### 8.1 Appendix A

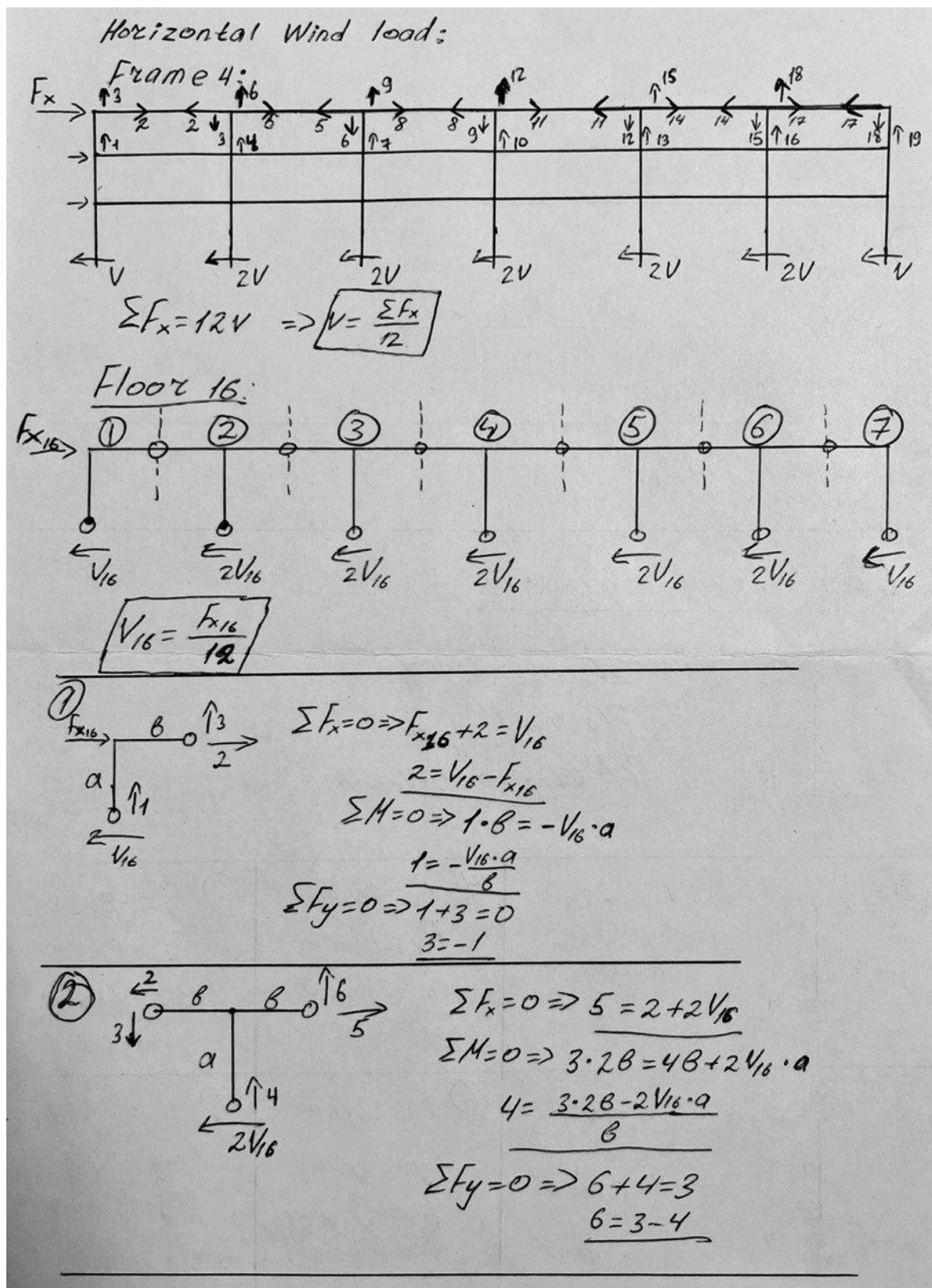


Figure 8.1. Hand calculations for Internal Forces under Wind Load (Part 1)

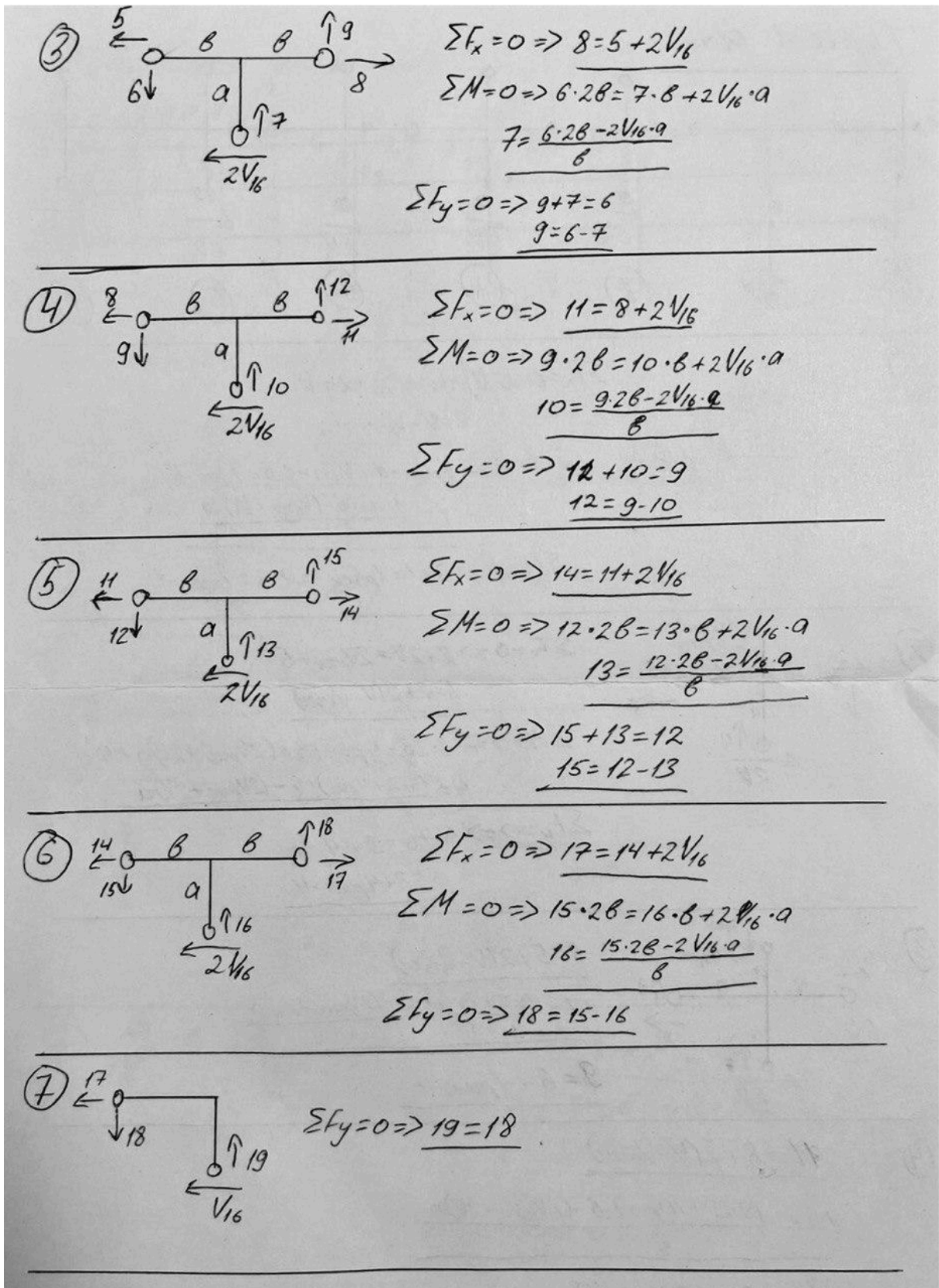


Figure 8.2. Hand calculations for Internal Forces under Wind Load (Part 2)

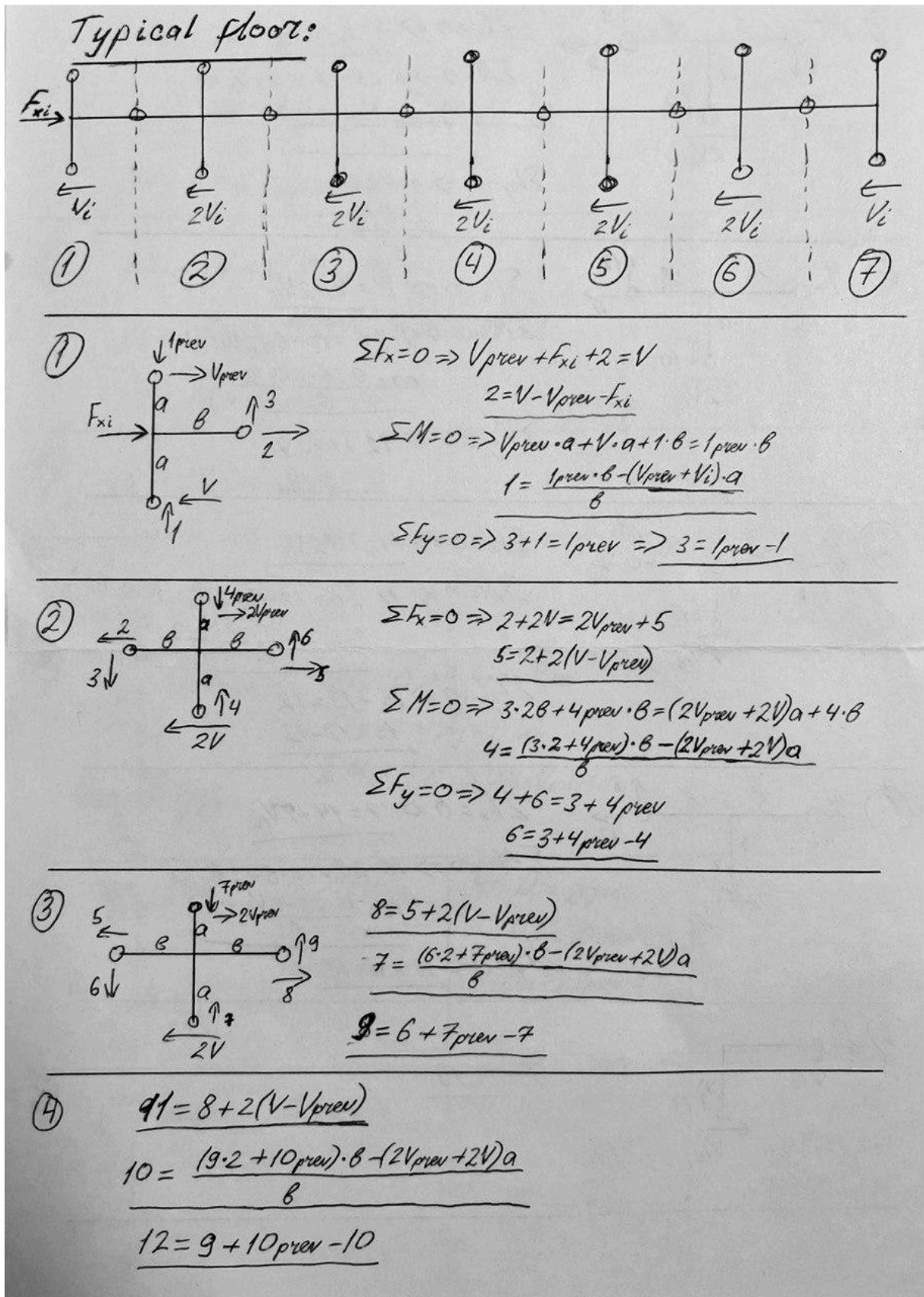


Figure 8.3. Hand calculations for Internal Forces under Wind Load (Part 3)

$$\textcircled{5} \quad 14 = 11 + 2(V - V_{prev})$$

$$13 = \frac{(12 \cdot 2 + 13_{prev})b - (2V_{prev} + 2V)a}{b}$$

$$15 = 12 + 13_{prev} - 13$$

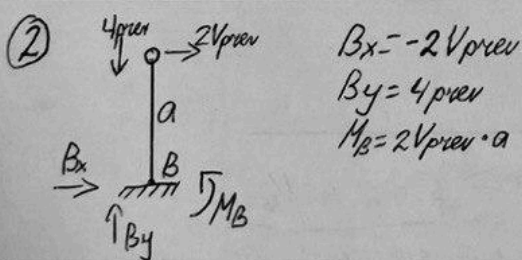
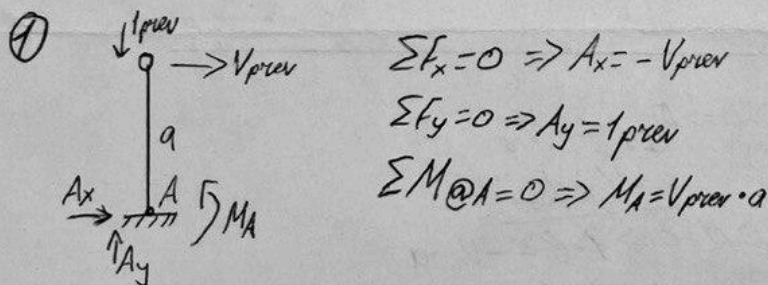
$$\textcircled{6} \quad 17 = 14 + 2(V - V_{prev})$$

$$16 = \frac{(15 \cdot 2 + 16_{prev})b - (2V_{prev} + 2V)a}{b}$$

$$18 = 15 + 16_{prev} - 16$$

$$\textcircled{7} \quad 19 = 18 + 19_{prev}$$

Load to the ground:



$$\textcircled{4} \quad D_x = -2V_{prev}$$

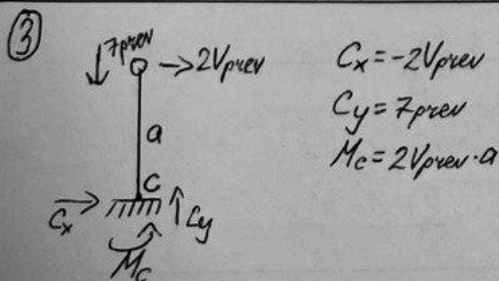
$$D_y = 10_{prev}$$

$$M_D = 2V_{prev} \cdot a$$

$$\textcircled{6} \quad F_x = -2V_{prev}$$

$$F_y = 16_{prev}$$

$$M_F = 2V_{prev} \cdot a$$



$$\textcircled{5} \quad E_x = -2V_{prev}$$

$$E_y = 13_{prev}$$

$$M_E = 2V_{prev} \cdot a$$

$$\textcircled{7} \quad G_x = -V_{prev}$$

$$G_y = 19_{prev}$$

$$M_G = V_{prev} \cdot a$$

Figure 8.4. Hand calculations for Internal Forces under Wind Load (Part 4)

