DESIGN OF THE HIGH-RISE HOTEL BUILDING IN A HIGH SEISMIC ZONE IN SEATTLE, WASHINGTON USA

(Capstone Project II)

Bachelor of Engineering
(Civil and Environmental)

NAZARBAYEV UNIVERSITY

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2025

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DECLARATION

We hereby declare that this report entitled “Design of the High-Rise Hotel Building in a High Seismic Zone in Seattle, Washington USA” is the result of our own project work except for quotations and citations which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

SIGNATURES

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Date: April 26, 2025

ACKNOWLEDGEMENTS

We would like to express my deepest appreciation to all those who provided us the possibility to complete this report. While writing this report, our team learned that the preliminary design of a construction project is a step-by-step and precise process. At the design stage, numbers are not taken for nothing, all sizes are precisely selected and calculated according to the requirements of local state institutions, as well as according to the comfort for each type of building separately.

ABSTRACT

The project requires designing and planning for a 12-storey high-rise hotel Pacific Skyline Lodge at its location at 211 Fairview Ave N in Seattle Washington. This project arose from technical requirements and its essential design criterion needed to place the structure in an area with high seismic vulnerability. The city of Seattle was selected as the building site because it experiences prominent seismic activity in an evolving urban landscape. The Pacific Skyline Lodge project encompasses architectural design, structural and geotechnical engineering, environmental considerations, and construction management. Every specialized field contributes essential duties toward ensuring that the structure achieves an attractive appearance together with safety standards and environmental sustainability together with operational efficiency. The design team will apply specific attention to developing both seismic-resilience and sustainable construction techniques. The project intends to build a hotel integration within its natural environment which maintains compliance with current building regulations while demonstrating outstanding strength during seismic events. A multidisciplinary system in the design tackles real-world difficulties that support Seattle's urban growth and economic benefits for its cityscape.

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1. Introduction

The goal of this project is to design a 12-storey high-rise hotel in a high seismic zone. The name of the hotel is "Pacific Skyline Lodge" and it will be located at 211 Fairview Ave N, Seattle, WA 98109, USA. Based on the technical requirements, the project consists of architectural, structural, geotechnical, environmental, and construction management aspects. The following table 1.1. illustrates the weight percentage of each of these parts and the corresponding responsible members.

Table 1.1. Job distribution

Part	Weight percentage	Responsible member
Structural & Architectural	40%	Arnur Amangeldy Amirzhan Bitimov
Geotechnical	35%	Nurgul Amangaliyeva Nursaule Kabizhan
Environmental	15%	Rassul Kabdrashitov Zhanna Kussainova
Construction Management	10%	Veronika Ten

All the above-mentioned parts have a particular area to focus on. For instance, the architectural part focuses on general design, 2D and 3D planning of the building using the Revit Autodesk program, while calculation and estimation of loads using SAP2000 simulations, design of appropriate columns, slabs and making the list of materials are related to the structural part. SAP2000 and the geotechnical part emphasises the determination of the soil profile and designing the foundation of the building. The environmental component aims to design a storm sewer system, whereas the scope of construction management is to provide a feasibility study, project scheduling and cost estimation.

2. Ethical Considerations in the Capstone Project

In order to ensure that the design, execution, and results of engineering projects are in line with professional ethics, sustainability principles, and society values, ethical considerations are an essential component. Several criteria were used to assess ethical issues for this capstone project:

1. Health, Safety, and Welfare of the Public

Priority of Safety: The design of the project places a high priority on the security of all parties involved, including the general public, infrastructure users, and construction workers. Risks related to the phases of building, operation, and maintenance were reduced by the implementation of certain measures.
Standards Compliance: To safeguard people and property, the project complies with national and international building codes, environmental laws, and safety standards.

2. Sustainability

Impact on the Environment: The project assesses and reduces its environmental impact. Natural ecosystems were preserved, resource utilization was optimized, and carbon emissions were decreased. The use of sustainable materials and methods is part of this.
Longevity and Resilience: The design takes climate change and harsh weather events into account to make sure the infrastructure can endure long-term difficulties without sacrificing the needs of future generations.

3. Equity and Inclusivity

Accessibility: The project incorporates measures to guarantee universal access, guaranteeing that individuals with disabilities, the elderly, and other marginalized groups are included.
Community Engagement: Stakeholder consultations were conducted to incorporate diverse perspectives, particularly those from underserved communities who may be impacted by the project.

4. Professional Integrity

Transparency: To guarantee accountability and reliability, all choices, computations, and presumptions have been openly recorded.
Preventing Conflicts of Interest: The project team made sure that choices were made on the basis of merit rather than selfish interests by remaining impartial while choosing

contractors, supplies, and techniques.

Recognition of Contributions: All scholars, practitioners, and collaborators whose work impacted the project are appropriately acknowledged.

5. Data Privacy and Security

Information Protection: Sensitive data was treated carefully during the data collecting and analysis stage, and privacy procedures were put in place to safeguard any proprietary or personal information that might be implicated.

Responsible Technology Use: Sophisticated instruments and software were employed in an ethical manner, guaranteeing precise outcomes free from falsification or manipulation.

6. Ethical Environmental Engineering Practices

Prevention of Harm: The project's planning and implementation prevent damage to the environment and biodiversity.

Pollution Control: During the building and operating stages, systems were implemented to control pollutants, manage waste, and avoid contamination.

7. Educational and Professional Development

Knowledge Sharing: To advance the field's collective understanding of civil and environmental engineering, the project's insights and conclusions will be provided to classmates, professors, and other applicable stakeholders.

Ethical Leadership: By highlighting the significance of ethics in engineering practice, this project acts as a model for aspiring engineers.

This capstone project will not only meet its technical objectives but will also adhere to the fundamental principles of Civil and Environmental Engineering because ethical considerations are incorporated into it. By putting safety, sustainability, inclusivity, and integrity first, this project is an outstanding example of responsible engineering that benefits both the environment and society.

3. Architectural Design

3.1. Site Selection and Site Analysis

The project location is 211 Fairview Ave N, Seattle, State of Washington, the United States of America. We decided to choose this location based on several reasons. Firstly, according to the project requirements, our building must be located in a high seismic zone, where the seismic factor S_s is above 1.0. Based on ASCE Hazard Tool reports, Seattle City has been considered a high seismic zone with the value S_s=1.54. Secondly, Seattle is the largest metropolis in the Pacific Northwest, which means that this city is a good place for tourism and hotel business. Generally, the climate of Seattle is quite appealing to tourists. The average temperature in summer is about 24°C, while 8°C in the winter season with light snowfall.

Table 3.1. Weather Conditions in Seattle

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec
Average high temperature in ºF/ºC	47/ 8.33	50/ 10	54/ 12.22	58/14.44	65/ 18.33	70/ 21.11	76/ 24.44	76/ 24.44	71/ 21.67	60/ 15.56	51/ 10.56	46/ 7.78
Average low temperature in ºF/ºC	37/ 2.78	37/ 2.78	39/ 3.89	42/5.56	47/ 8.33	52/11.11	56/13.33	56/ 13.33	52/11.11	46/ 7.78	40/ 4.44	36/ 2.22
Days with precipitation	18	14	16	14	12	9	5	4	7	13	18	17
Hours of sunshine	69	108	178	207	253	268	312	281	221	142	72	52
Average precipitation in in/mm	5.55/ 140.97	3.46/ 87.884	3.70/ 93.98	2.68/ 68.072	1.93 /49.022	1.54/ 39.116	0.67/ 17.018	0.87/ 22.098	1.42 /36.068	3.46/ 87.884	6.54/ 166.116	5.31 /134.874

Humidity (%)	84	83	80	75	72	70	65	65	73	82	84	83

Seattle weather conditions can be described as a temperate oceanic climate. From the table above the weather in winter can be concluded as mild and wet, while the summer season is dry and cool.

According to the (Suncalc.com, 2022), in Figure 2.2 the sun's direction is from the intersection of Fairview Ave N and John St to the opposite corner of Boren Ave N. Total daylight duration is 10.25 hours, and a shadow length at 1 p.m. is 1.69 meters. The shadow will start in the lower right corner and gradually shift toward the upper left corner as the day progresses. It shows that the building will not have direct sunlight from the front and back sides that prevents excessive sun lights and reduces shadows on the site.

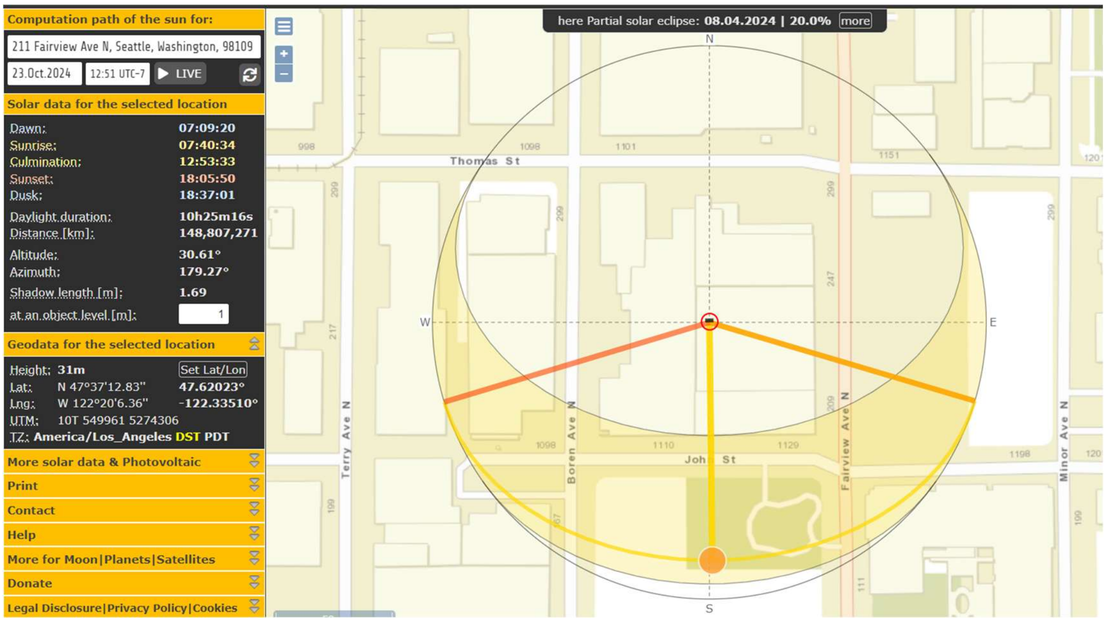


Figure 3.1. Sun direction (Suncalc.com, 2022)

On Figure 2.3, the wind direction is represented via black lines which are directed from north-east to south-west, from Thomas St to John St. It means that the wind direction will encounter the facade at the right sides, the side with the lowest vertical cross-sectional area, which leads to the reduction of the wind load to the facade. Here the red rectangular represents the location of the building.

Figure 3.2. Wind direction (Windfinder.com, 2022)

Based on the Seattle Building Code 2018, the standard parking spaces are 8.5 ft wide and 18 ft long. As we can see from figure 2.4, our building area will consist of inside two-lane traffic, where cars would be able to roundabout motion through the hotel. As for the parking spaces, Seattle Municipal Code (SMC) says that there are no minimum parking requirements for hotel, but having at least 0.5 parking spaces per guest room is recommended. There are 180 guest rooms in hotel, so we need 90 parking spaces as minimum. Surface area will have 100 parking spaces and underground parking with 60*50 metres which would have 68 parking spaces, eventually 168 parking spaces at total.

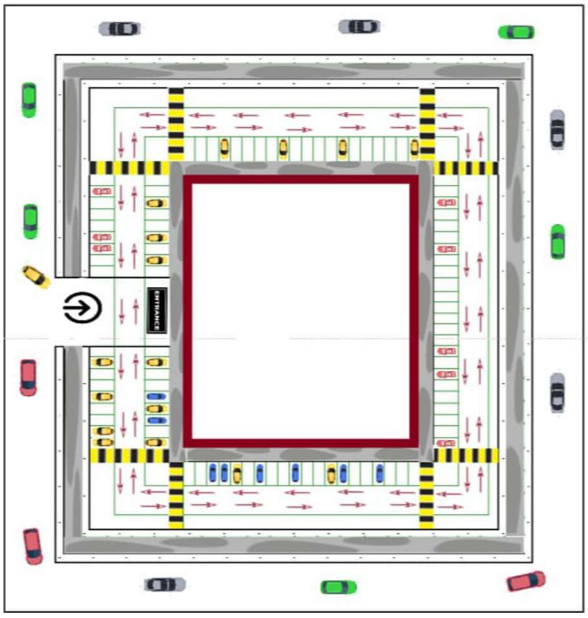


Figure 3.3. Traffic motion and surface parking spaces

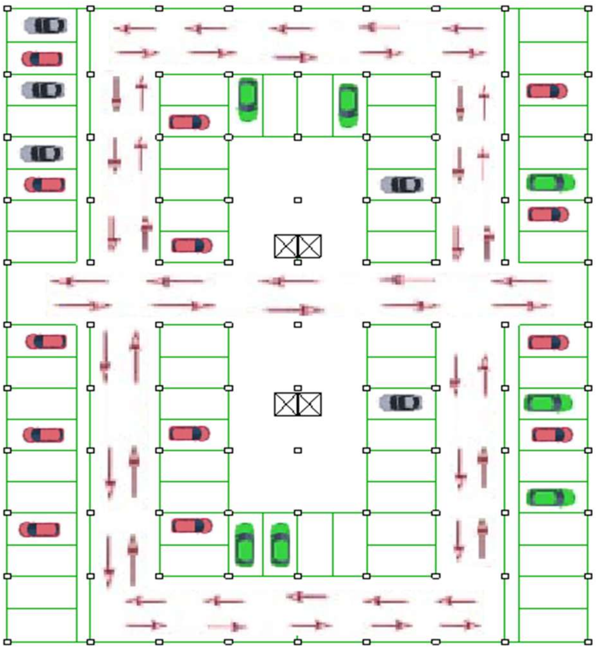


Figure 3.4. Traffic motion and underground parking spaces

Besides the appropriate climate for tourism, there are a lot of famous attractions such as the Space Needle, Seattle Art Museum and Seattle Aquarium.

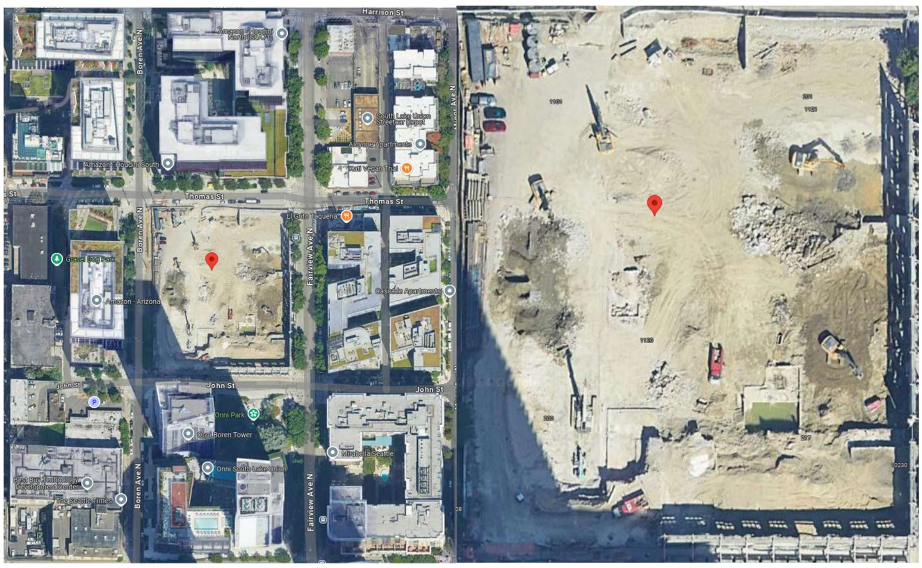


Figure 3.5. Site location and layout (Google Maps)

The selection of the site is based on the soil profile, the proximity to the main highway of the city and the appropriate surrounding infrastructure for the hotel business. The building is located about 450 metres from the I-5 Express, the busiest highway in Washington state. This short distance will improve transportation and accessibility of the building, thereby increasing the hotel's attractiveness and operational efficiency. The surrounding infrastructure is also significant to the site selection since access to services like restaurants, shops, and entertainment centres can provide better comfort for guests. Our project is surrounded by well-developed infrastructure such as Amazon Go, Tha, Japanese and Mexican restaurants, banks and even Gravel Dog Park. The combination of the above factors would ensure high hotel occupancy, which was crucial verification for site selection.

3.2. Different Views of the building

Figure 3.6. 3D View of the building

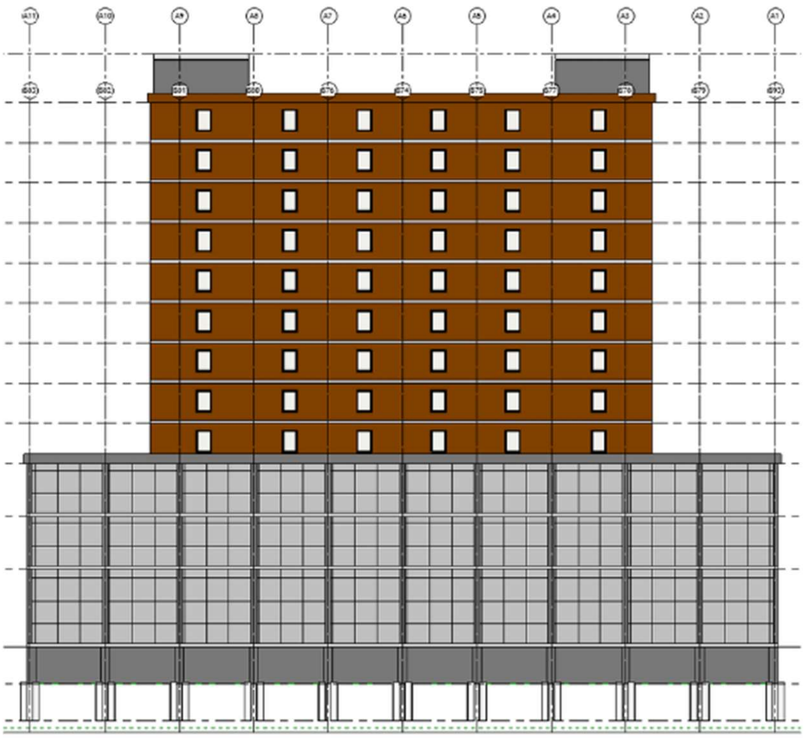


Figure 3.7. North View of the building

Figure 3.8. South View of the building

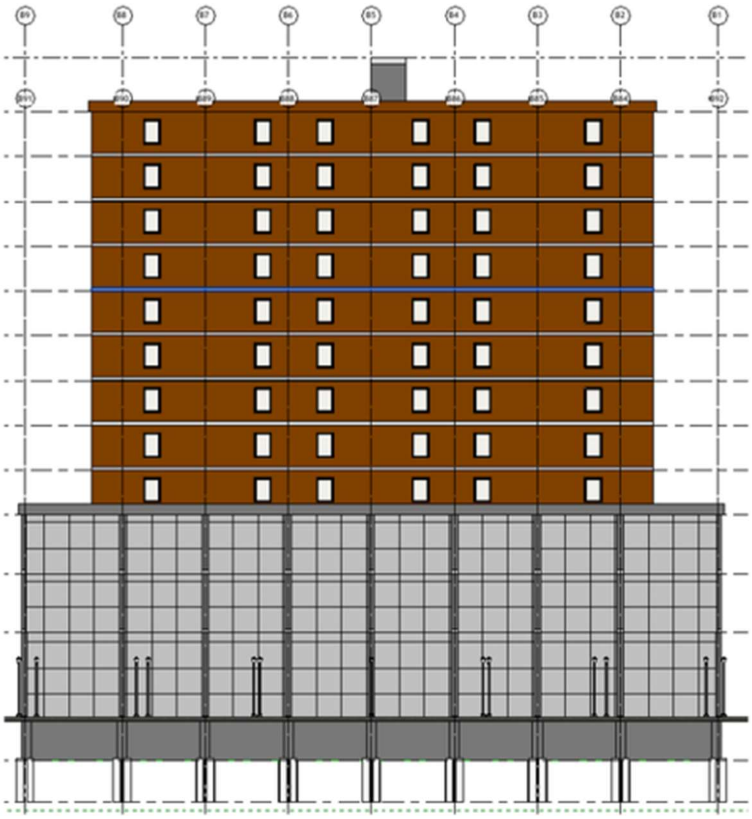


Figure 3.9. East View of the building

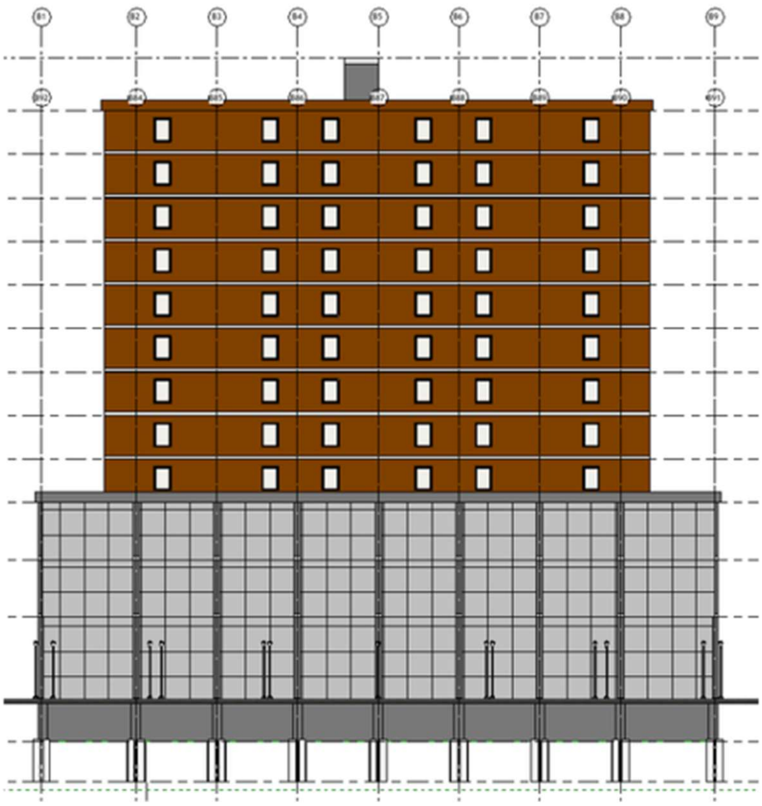


Figure 3.10. West View of the building

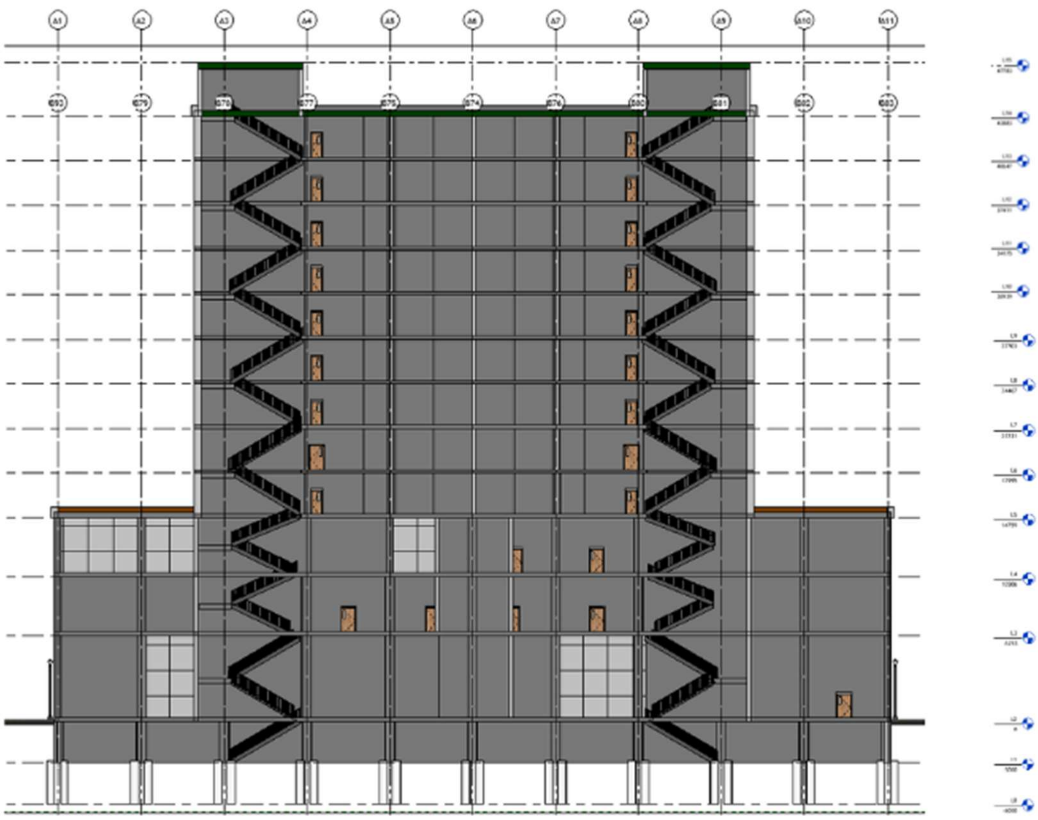


Figure 3.11. Section View of the building

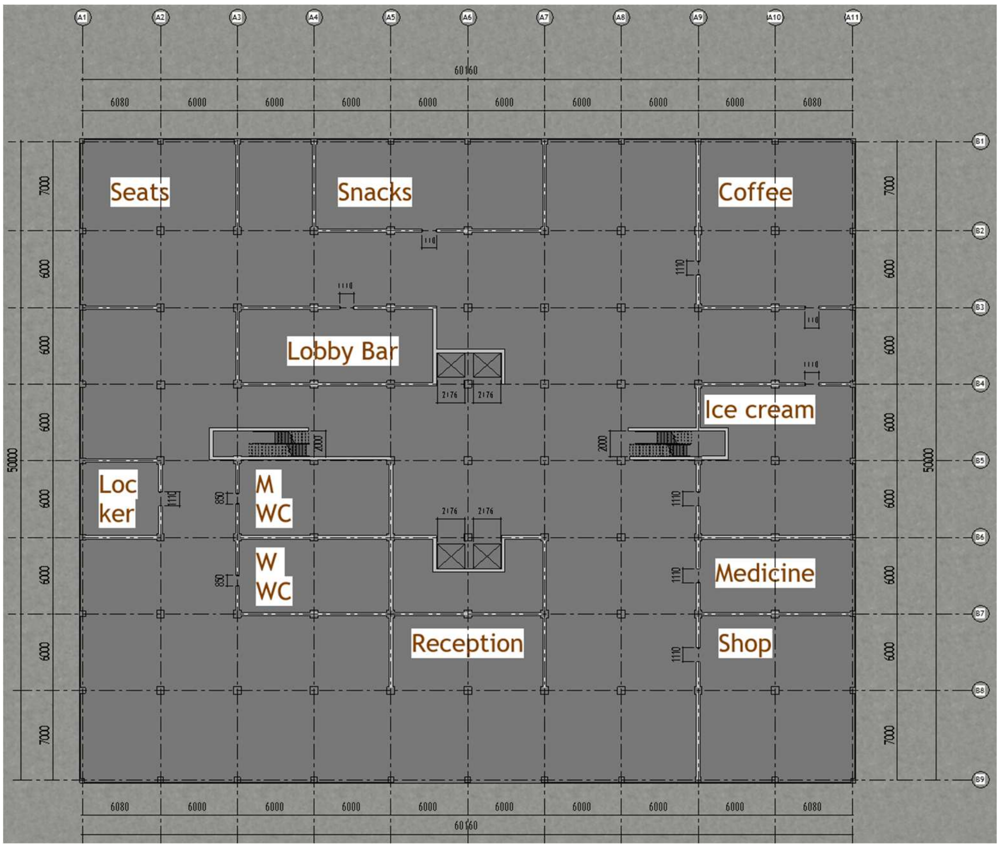


Figure 3.12. 2D View 1st floor

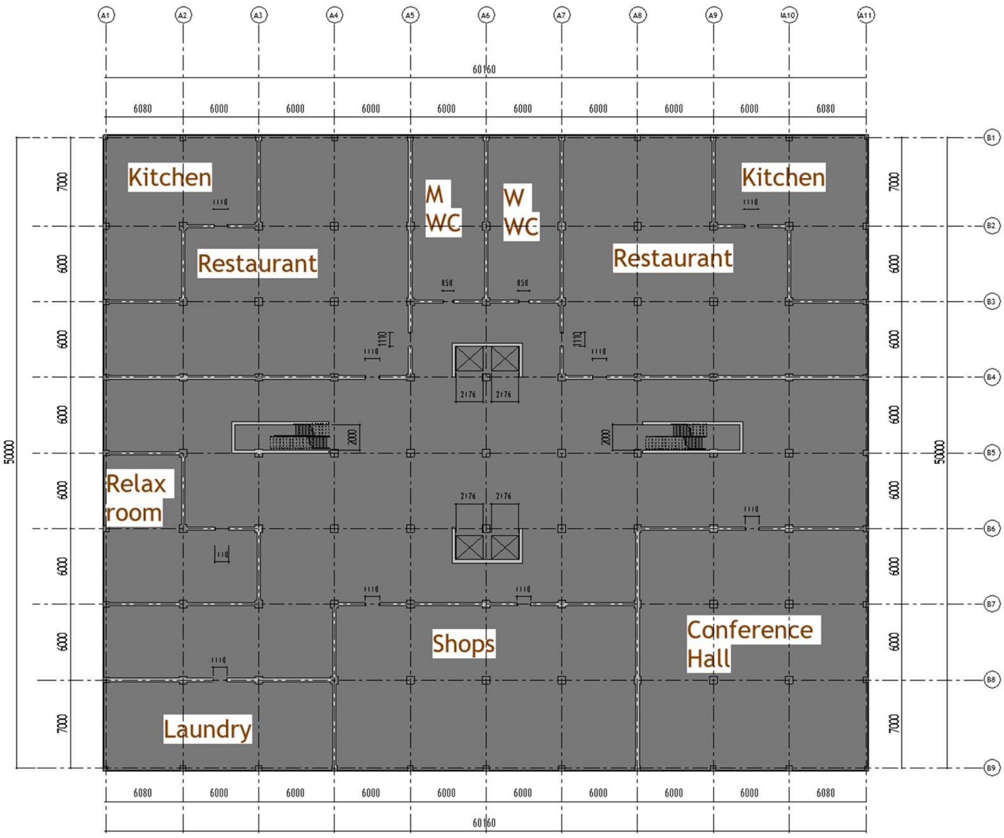


Figure 3.13. 2D View 2nd floor

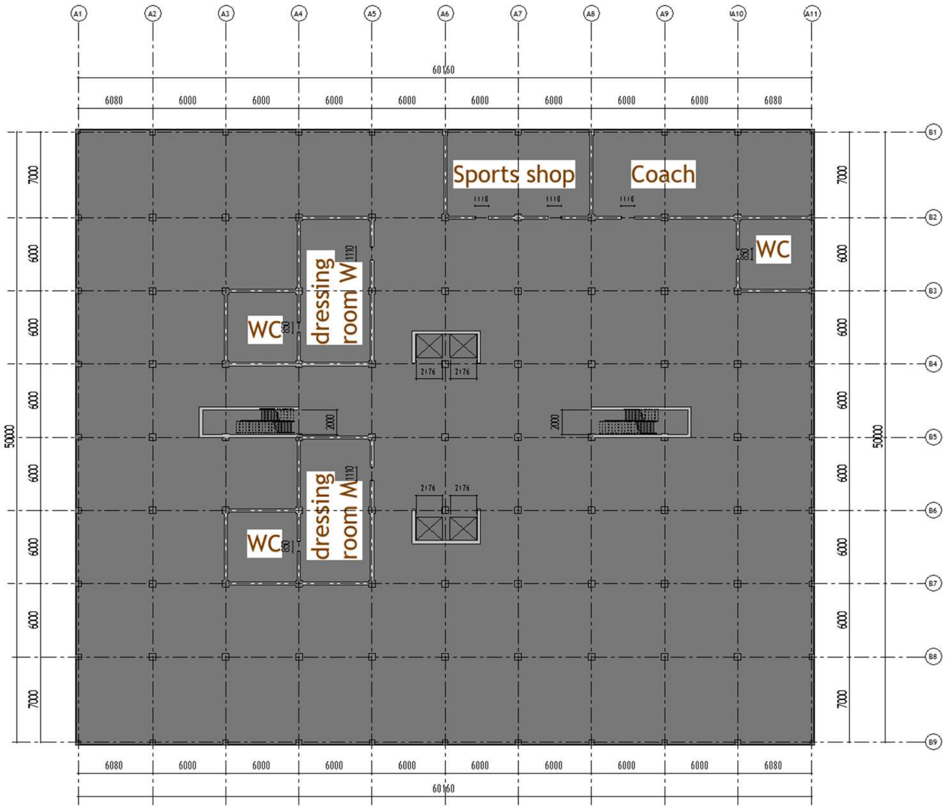


Figure 3.14. 2D View 3rd floor

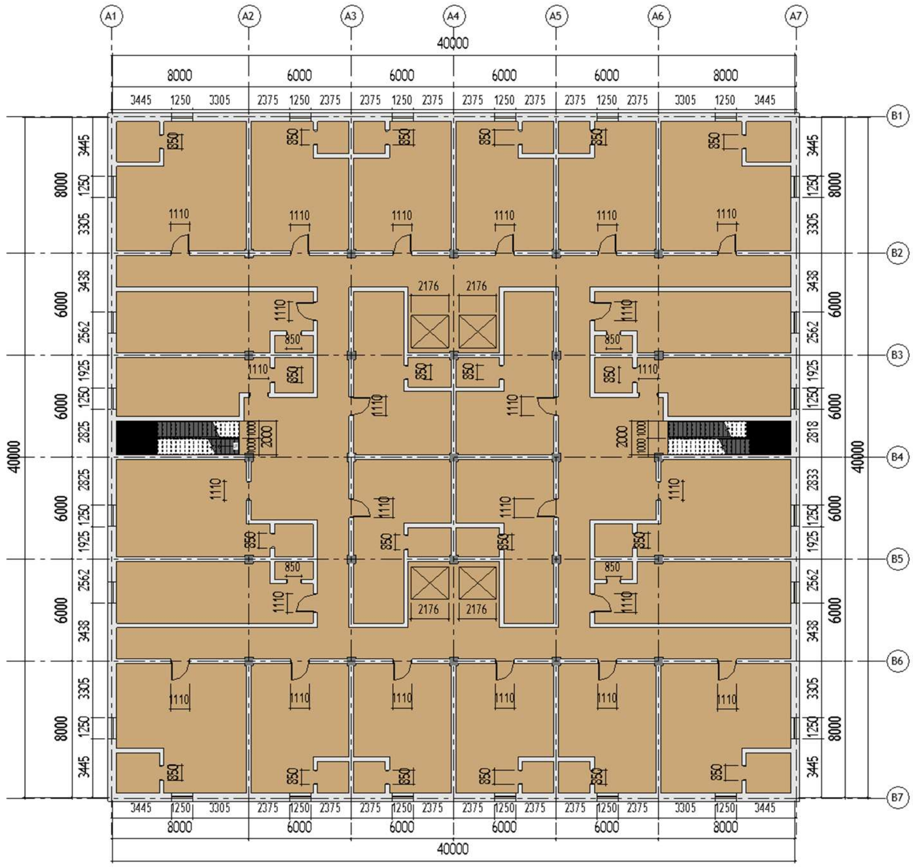


Figure 3.15. 2D View 4-12 floors

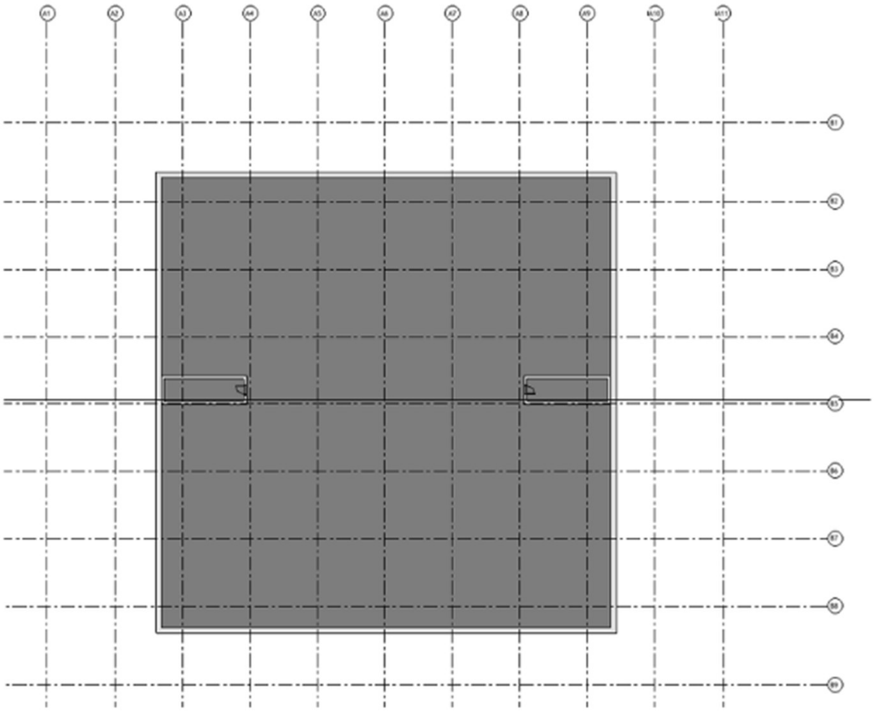


Figure 3.16. Roof layout

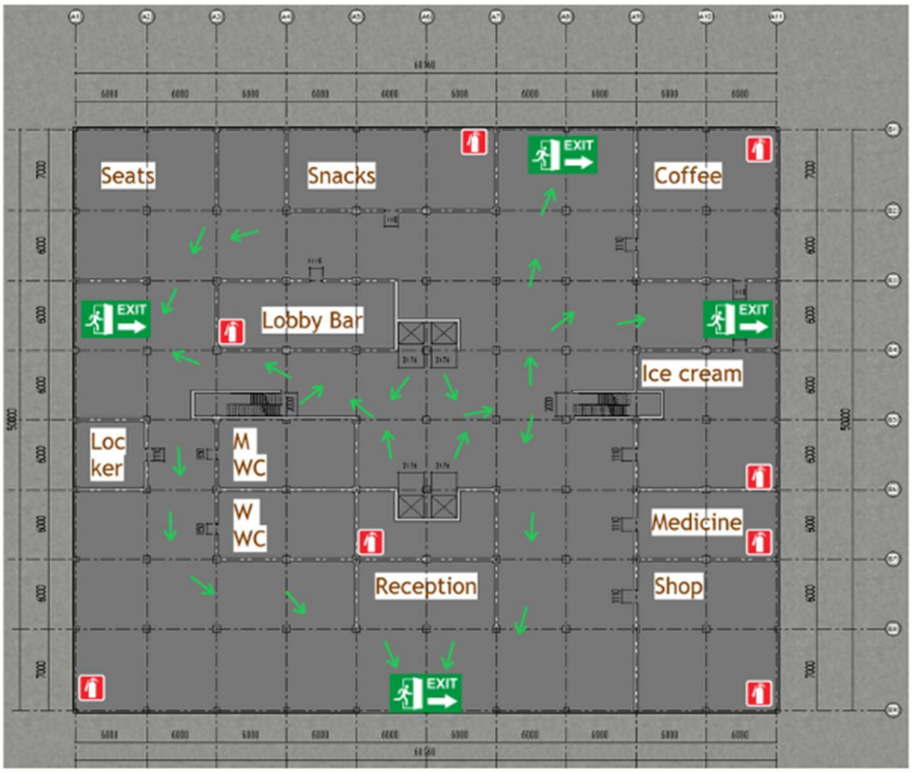


Figure 3.17. Emergency Plan and fire safety 1st floor

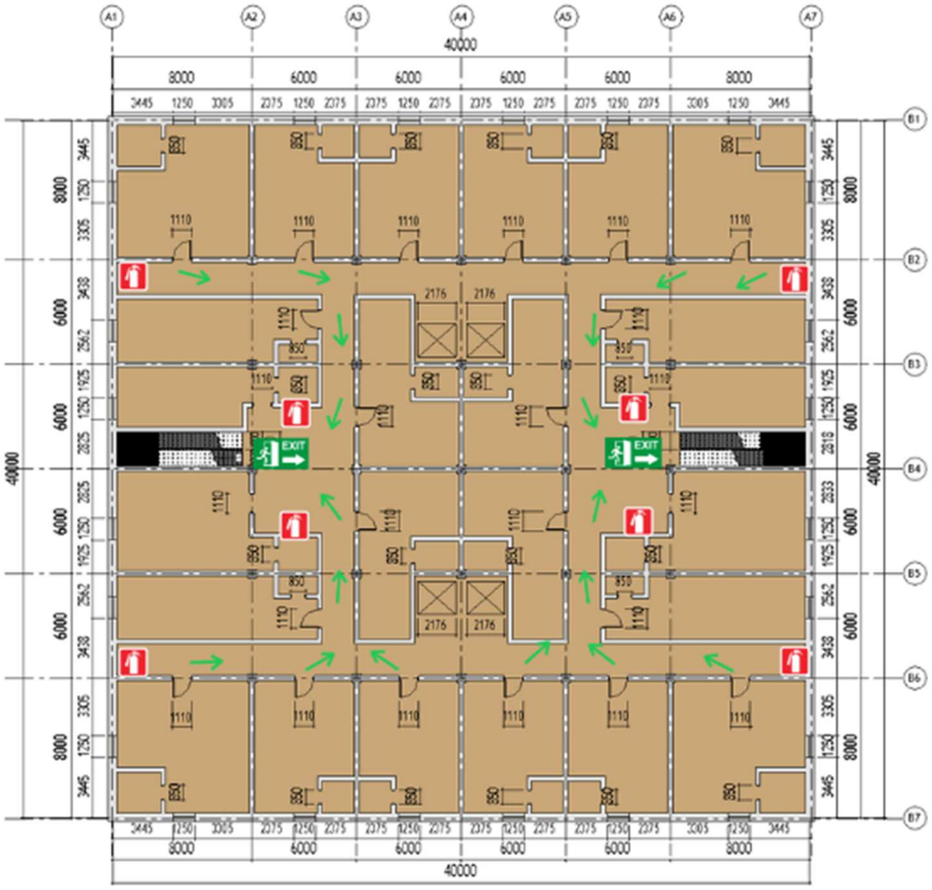


Figure 3.18. Emergency Plan and fire safety typical floor

3.3. Life Cycle Cost

3.3.1. Concrete mixture

The project's concrete mixture is similar to C40 concrete mix (≤ 0.45 w/c ratio) with a w/c ratio of 0.42 and no supplements for the base case and 30% slag for the alternative case.

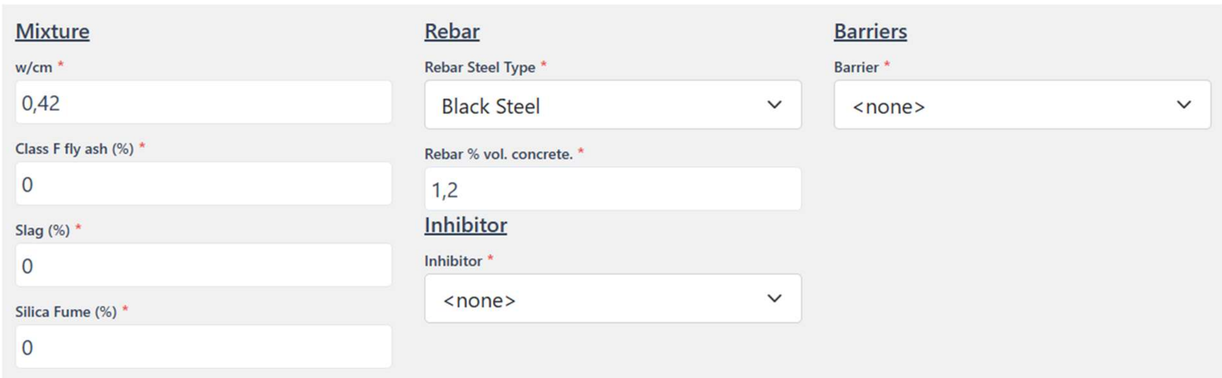


Figure 3.19. Concrete mix ratio for the base case

Furthermore, the Life365 software automatically incorporated chloride exposure and temperature conditions by specifying Seattle City as the project location. The chloride exposure level was set at 0.75% or 18 kg/m^3 by weight of concrete, while the temperature varied between 4.2°C and 18.1°C.

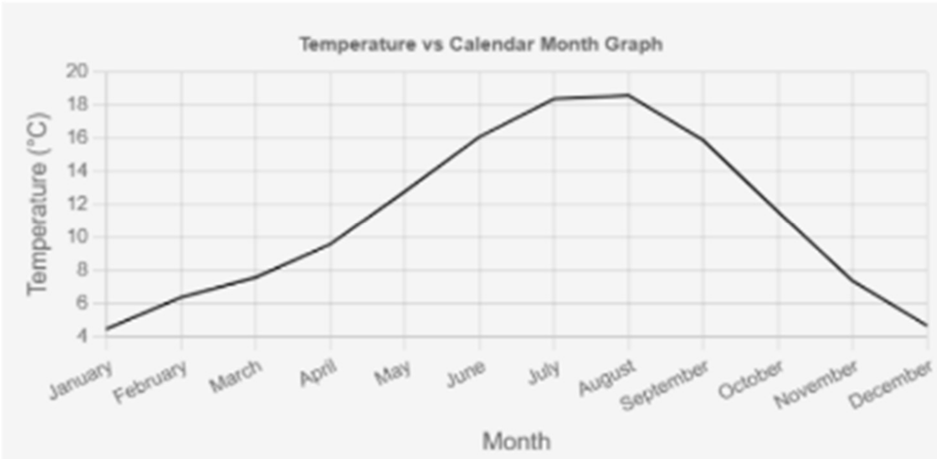


Figure 3.20. Temperature variability in Seattle City

It is important to note that barriers and Life Cycle Costs are directly proportional to each other. Hence, we obtained the following data based on the above-mentioned proportion:

Life Cycle Costs

Name	Construction Cost	Barrier Cost	Repair Cost	Life Cycle Cost
Base case	$166,919	$0	$2,993,436	$3,160,354
Alternative 1	$382,732	$0	$3,651,546	$4,034,279

Figure 3.21. Total life cycle costs

Then, the program automatically calculated cumulative current costs and plotted a graph below. The graph, instead of being a smooth contour, shows steps. That's because the graph is a cumulative cost during this increased event, which can be major renovations or systems upgrades (Blue line- Alternative; Green line- Base case).

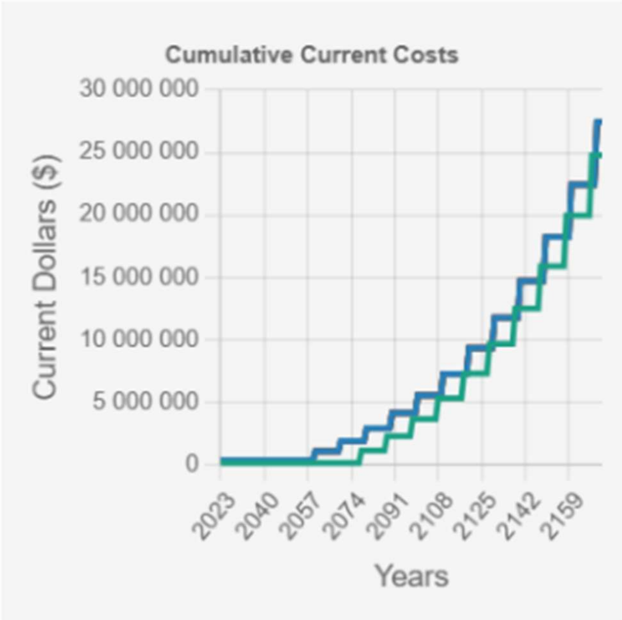


Figure 3.22. Cumulative current costs.

3.4. Corrosion prevention

Since our region is located next to the ocean, it is important to consider anticorrosion elements. The air is laden with moisture and salt, and it makes the metals corrode very much faster. Over time, the corrosion can affect structural safety. To counter all these, we employ corrosion-resistant materials such as stainless steel, galvanised coatings, and marine-grade alloys. Apart from protective coatings and routine maintenance schedules, and drainage, those are construction practices integrated into our anticorrosion strategy. Moreover, 30% of blast-furnace slag is helpful in improving resistance to sulphate attack and alkali-silica reaction. One may also include calcium nitrite as a corrosion inhibitor at a dosage of 5 L/m^3 to arrest corrosion. On top of this, a sealer may be applied to prevent chloride ions from percolating into the structure. Such an anticorrosion overall strategy will extend the life of our buildings and save maintenance costs in the long run while making them safe.

3.5. Leadership in Energy and Environmental Design (LEED)

The Leadership in Energy and Environmental Design (LEED) is an environmental rating of green building practices developed by the U.S. Green Building Council (USGBC) that quantifies construction projects based on the environmental dimension. A project may be submitted for review only after it meets the Minimum Program Requirements of the system.

The LEED certification verifies adherence by a project to sustainable, resilient, and environmentally friendly methods, awarding certification levels based on the scored points attained:

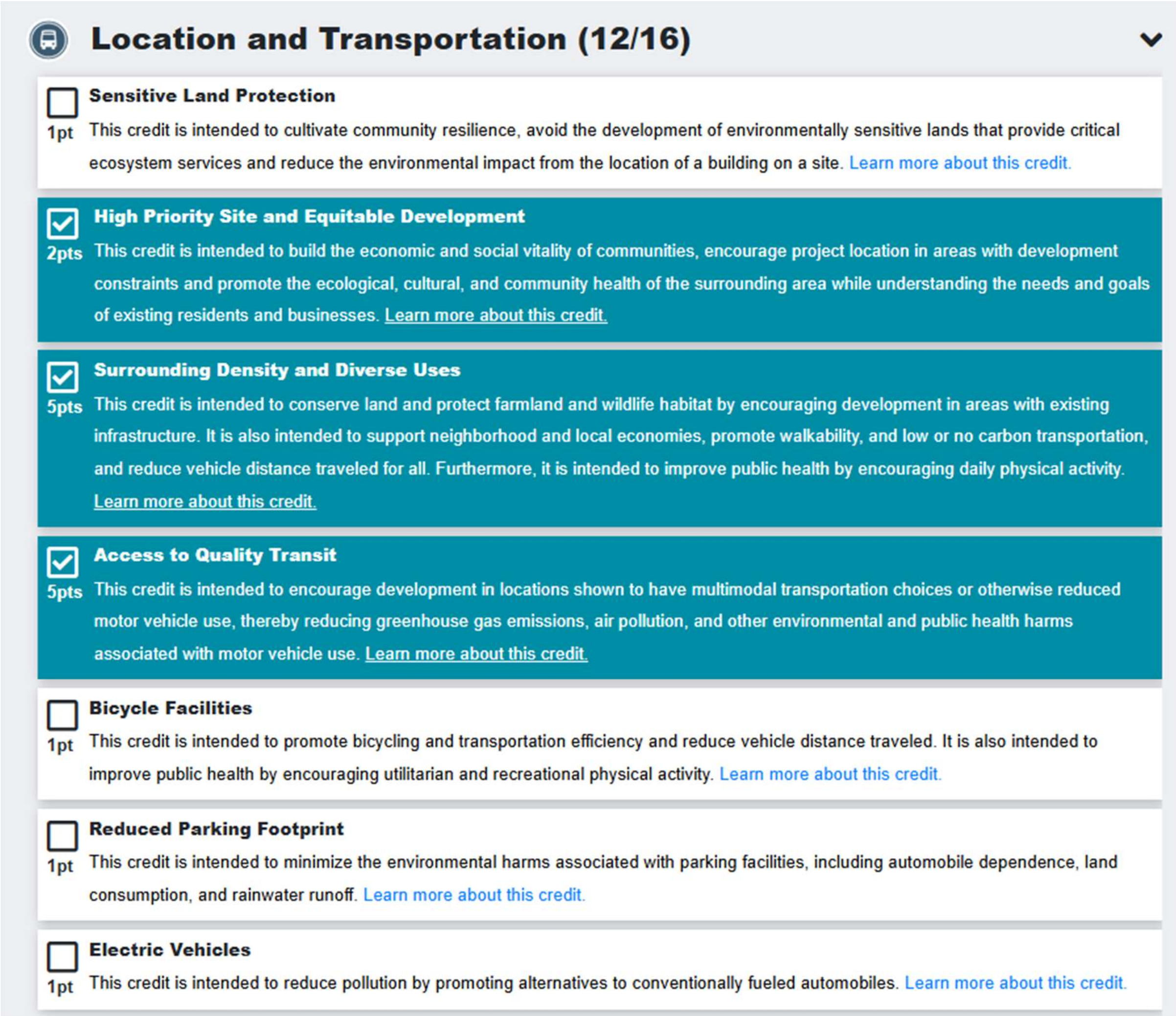
- Certified: 40-49 points
- Silver: 50-59 points
- Gold: 60-79 points
- Platinum: 80+ points

In the state of California, since 2004, the state has mandated LEED certification for certain projects (California Government, n.d.). The LEED rating system assesses buildings on seven primary categories:

- Location and Transportation
- Sustainable Sites
- Indoor Environmental Quality
- Energy and Atmosphere
- Materials & Resources
- Integrative Process
- Innovation
- Regional Priority
- Water Efficiency

3.5.1. Location and Transportation

The site was picked due to its proximity to the city's financial district. The site is pedestrian friendly as residents' jobs will be available in the Downtown area, and it has access to quality public transport systems and bike lanes. The provision of such sustainable means of transportation reduces the use of private cars' greenhouse gas emissions and encourages the short walk by the residents of the complex.



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.

- [x] **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.

- [x] **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.

- [x] **Access to Quality Transit** — 5pts
  This credit is intended to encourage development in locations shown to have multimodal transportation choices or otherwise reduced motor vehicle use, thereby reducing greenhouse gas emissions, air pollution, and other environmental and public health harms associated with motor vehicle use. Learn more about this credit.

- [ ] **Bicycle Facilities** — 1pt
  This credit is intended to promote bicycling and transportation efficiency and reduce vehicle distance traveled. It is also intended to improve public health by encouraging utilitarian and recreational physical activity. Learn more about this credit.

- [ ] **Reduced Parking Footprint** — 1pt
  This credit is intended to minimize the environmental harms associated with parking facilities, including automobile dependence, land consumption, and rainwater runoff. Learn more about this credit.

- [ ] **Electric Vehicles** — 1pt
  This credit is intended to reduce pollution by promoting alternatives to conventionally fueled automobiles. Learn more about this credit.

Figure 3.23. Location and transportation scoreboard.

3.5.2. Sustainable Sites

For our project, we selected Protect or Restore Habitat, Rainwater Management, and Light Pollution Reduction because they are directly applicable to our site's natural resources and sustainability goals. The site already contains existing green areas, so habitat restoration is possible and effective. Stormwater management is sensible considering the area's rainfall regime, to prevent flooding and protect surrounding communities. Light pollution reduction also benefits local wildlife and creates a healthier, more enjoyable nighttime environment for future residents.

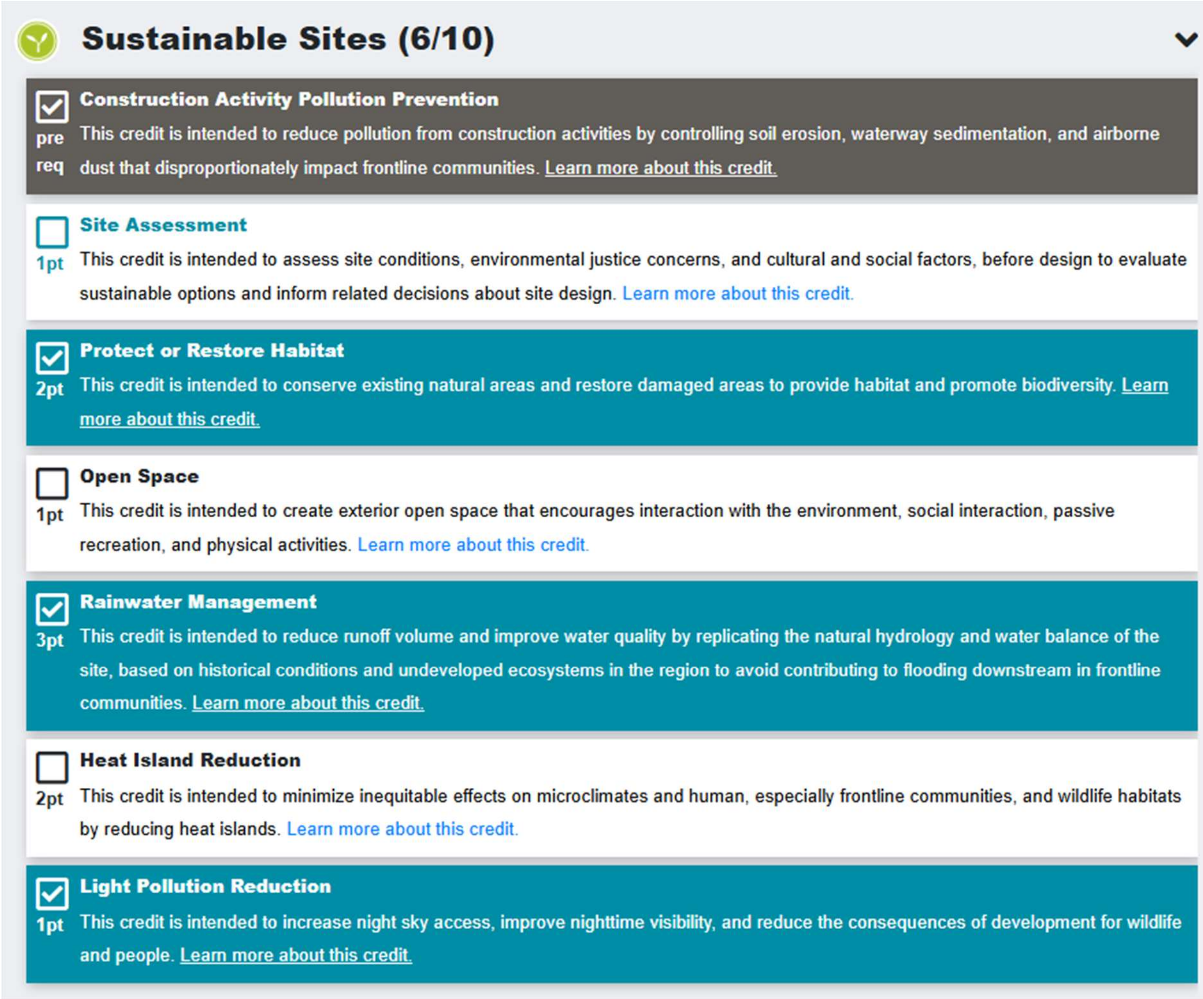


Figure 3.24. Sustainable Sites scoreboard.

3.5.3. Water Efficiency

We prioritized water efficiency by taking Outdoor Water Use Reduction, Indoor Water Use Reduction, and Water Metering, for a total of 9 of 11 points. These credits were prioritized since our site is situated in a water-scarce area and therefore conservation becomes essential. Conserving both outdoor and indoor potable water usage reduces environmental impact and operating expenses considerably. Water metering enables us to track consumption, detect inefficiencies, and facilitate long-term water management that supports our sustainability objectives.

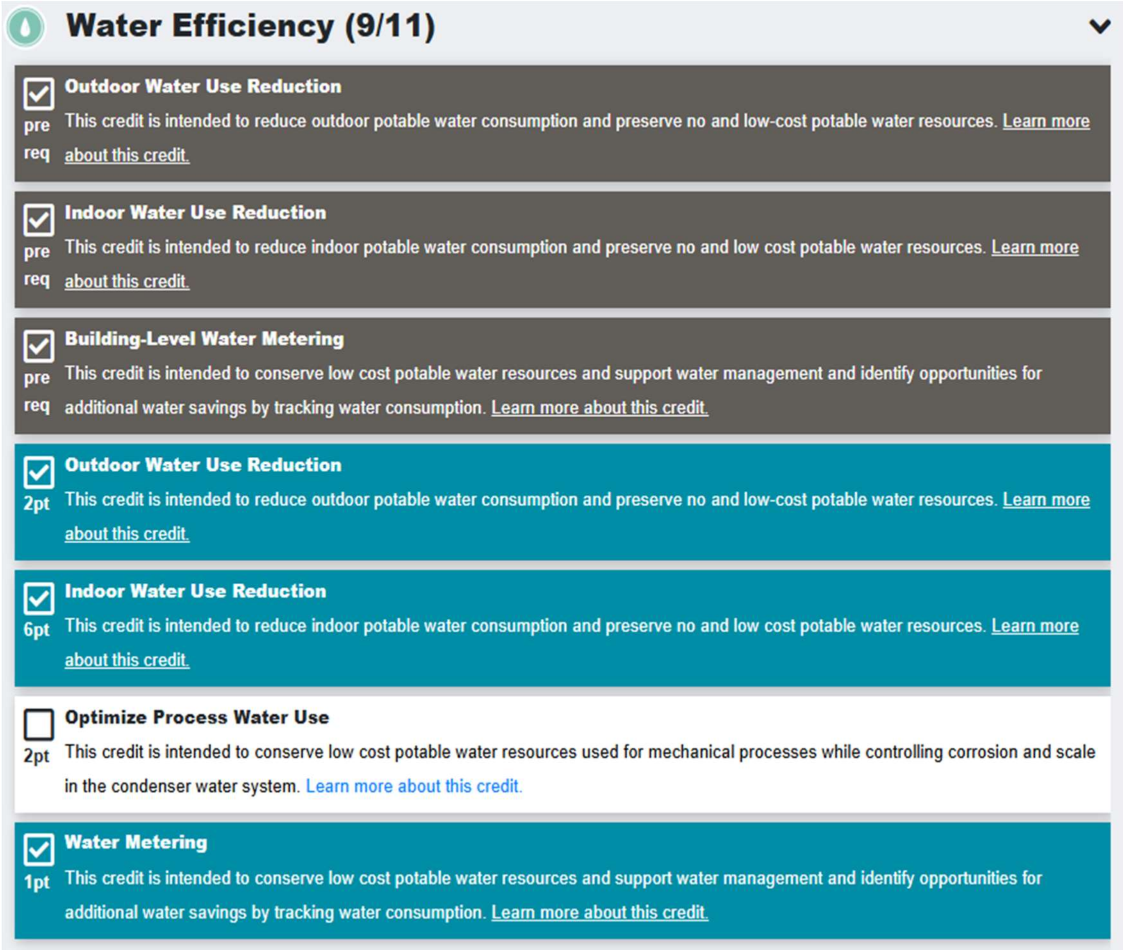


Figure 3.25. Water Efficiency scoreboard.

3.5.4. Energy and Atmosphere

These choices from the Figure 3.26. reflect our commitment to minimizing environmental impacts across the building lifecycle. We employed long-lasting, low-impact materials, promoted transparency of environmental product performance, and reduced landfill waste by reusing and recycling. This aligns with our sustainability objectives and responsible use of resources.

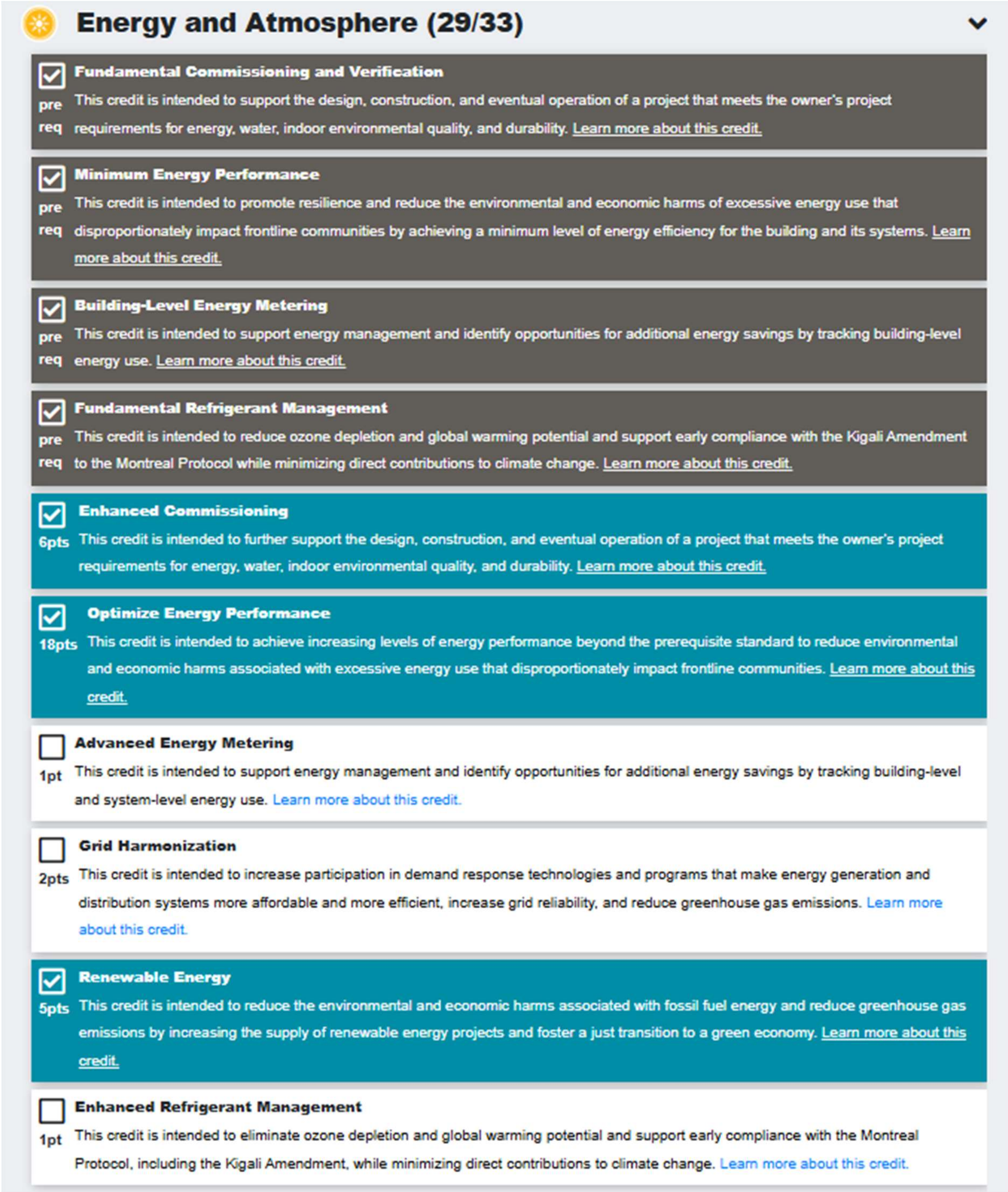


Figure 3.26. Energy and Atmosphere scoreboard.

3.5.5. Materials and Resources

These selected credits promote lifecycle thinking, transparency, and responsible use of resources. They show a strategic push toward circular economy habits, sustainable procurement, and supply chain environmental burden reduction. Data-driven decision-making is facilitated by Environmental Product Declarations and Waste Management, while closed-loop material flow through Lifecycle Impact Reduction aims for optimum long-term environmental performance. This selection further improves our project's linkage to LEED goals and demonstrates a high level of commitment to sustainable building practice and supply chain stewardship

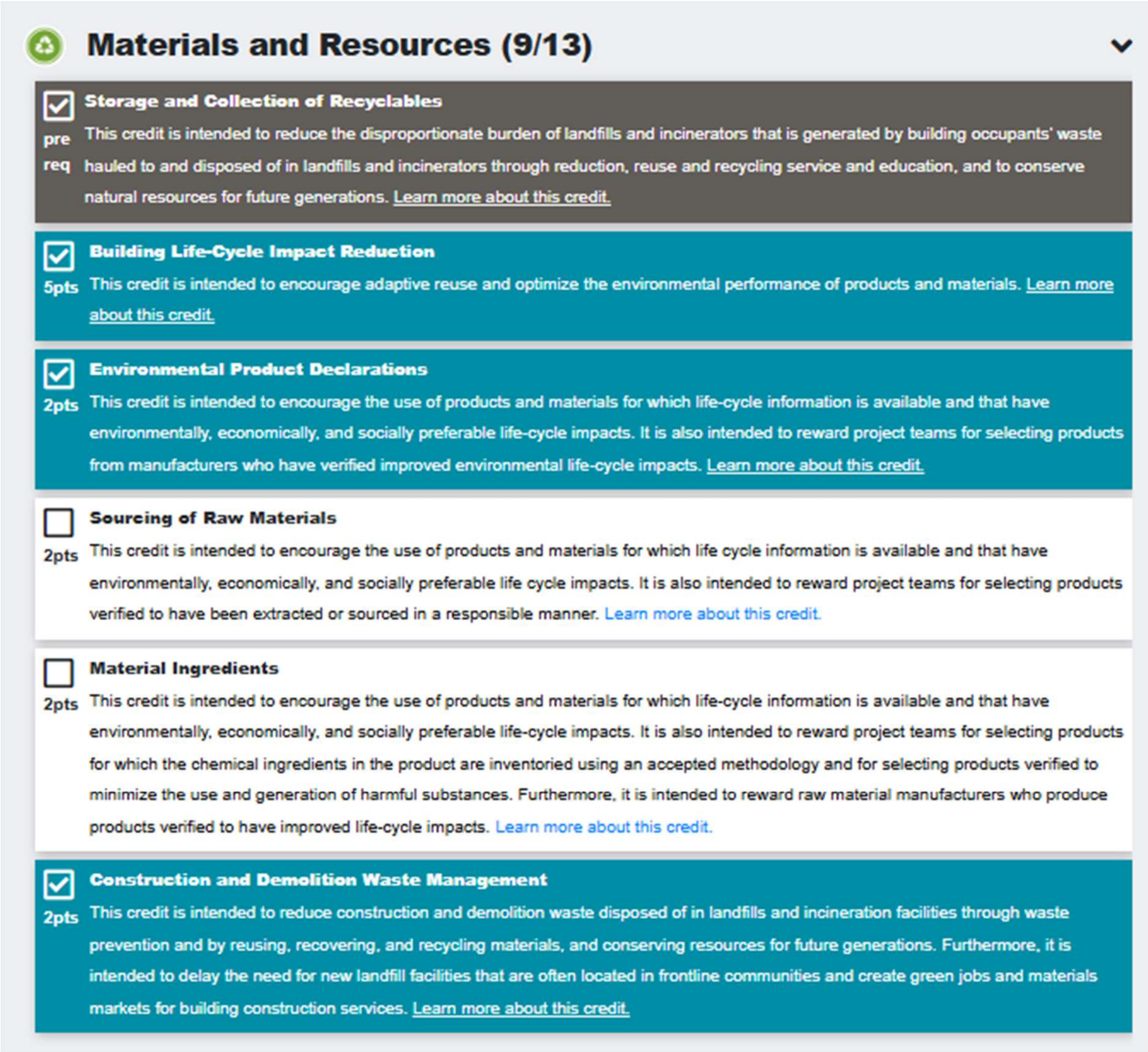


Figure 3.27. Materials and Resources scoreboard.

3.5.6. Indoor Environmental Quality

Our chosen Indoor Environmental Quality (IEQ) credits emphasize occupant health, well-being, and productivity through treatment of the air quality issue on a balanced, multi-component basis, involving active pollution control and air quality testing. By combining thermal comfort, interior lighting, and low-emitting products, our building expresses an open-ended strategy for interior well-being. The attention given to both construction-phase and occupancy-phase air quality control illustrates thoughtfulness in being proactive in response to long-term threats to health as well as providing a healthy, people-centric interior environment.

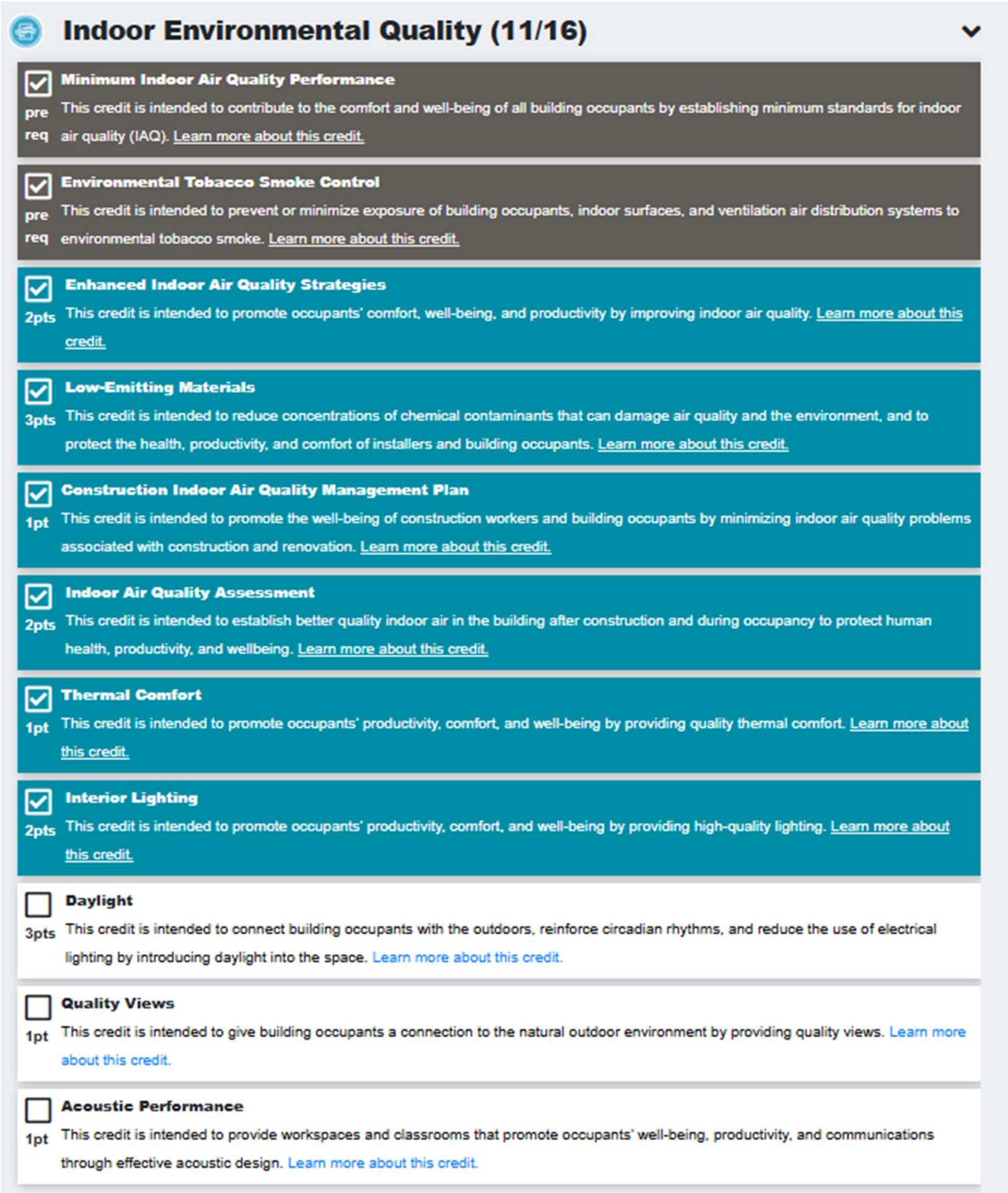


Figure 3.28. Indoor Environmental Quality scoreboard.

3.5.7. Integrative Process

We selected the Integrative Process credit to ensure all the systems in the building are considered as a whole at the beginning. This minimizes inefficiency, restricts loss of resources, and aids in cost-efficient decisions. Interrelationships considered upfront allow us to offer assistance for sustainable outcomes that lead to LEED objectives, which will provide a solid foundation for subsequent design and construction activities.

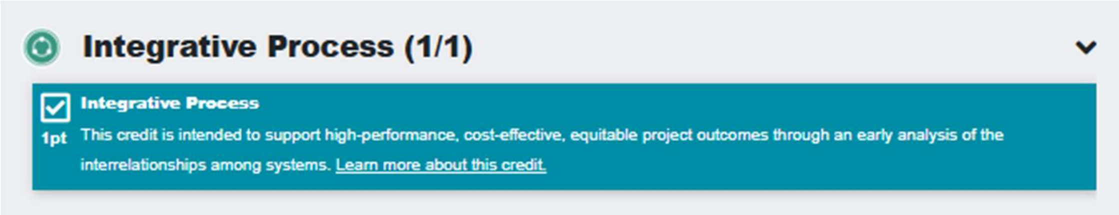


Figure 3.29. Integrative Process scoreboard.

3.5.8. Innovation

All Innovation credits were chosen to push beyond standard design and show commitment to creative thinking. The project includes strategies that go beyond standard LEED requirements and includes a LEED Accredited Professional to head the team. These credits focus on environmental leadership, encourage innovative solutions, and ensure successful collaboration, which is essential to optimizing performance, equity, and sustainability throughout the building's lifecycle.

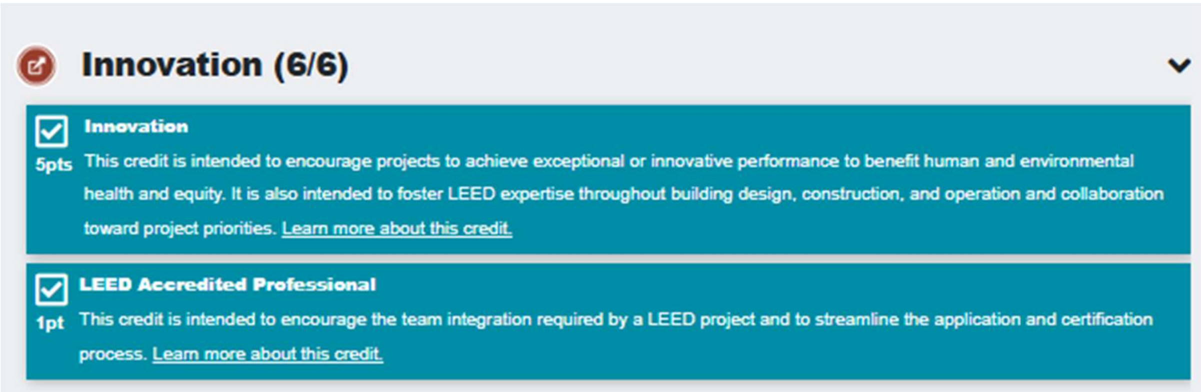


Figure 3.30. Innovation scoreboard.

3.5.9. Regional Priority

We pursued all available Regional Priority credits to prioritize the special social and environmental imperatives of the project's geographic region. These credits cause the building to have positive impacts on local health, equity, and environmental concerns. In connecting our design strategies with regional imperatives, we not only enhance our LEED score but address local community and climate-specific concerns.

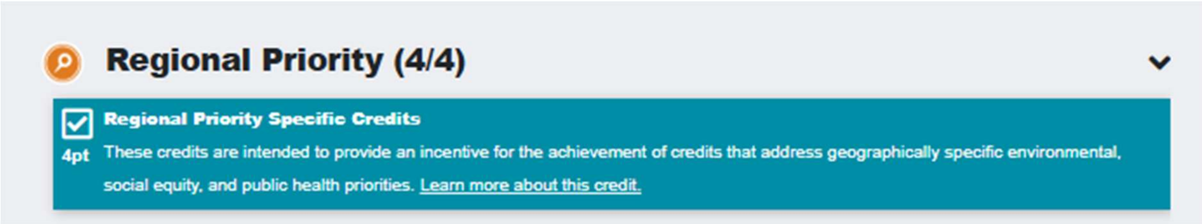


Figure 3.31. Regional Priority scoreboard.

For this project, the LEED score obtained was 87, and Platinum certification was obtained. This is a high score and indicates the good sustainability and environmentally friendly nature of the project.

3.6. Materials Selection

The location of our building is the city of Seattle, which is distinguished by its rainy climatic conditions. Different aspects of the climate can significantly affect the construction process. Let’s discuss possible scenarios:

In Seattle, temperature fluctuates between 45 to 60 ºF on average. The mild temperature in winter is beneficial for the curing of the concrete and installation of the asphalt. In our hotel, we use concrete for floor and ceiling units, and asphalt layer for pavement, so the construction of these members of the structure would be easier and favourable in winter. On the other hand, the relatively cool summers could create problems while working with materials that require higher temperatures to properly cure and set. Materials that require higher temperatures for curing are concrete, asphalt, mortar and grout, adhesives, paints and adhesives. It would require proper planning and scheduling of the project, to overcome such problems.

The main material for the exterior walls of the hotel is brick. The brick as a material is subjected to the thermal expansion and contraction effect. It expands when heated and contracts when cooled. The overall mild temperature throughout the year reduces this effect on the brick wall, making the material proper for the temperature conditions of the city. Also, mortar and brick have different thermal expansion coefficients, which can cause joint stress, but this is a less concern in Seattle’s mild climate.

Moreover, the mild temperature is favourable for workers' health and safety. It creates good conditions to work, preventing risks of heat exhaustion in high temperatures.

Humidity is high throughout the year in Seattle, the minimum being 65% in July and August, and reaching its maximum of 85% in December and January. The main problems that could be caused by the high humidity rate are extended curing time of the materials, corrosion and mold. Drying and curing times for concrete and mortar are extended in such humidity conditions. To prevent delays from the schedule, and to prevent reducing the strength of the materials, special additives such as accelerators (calcium chloride, non-chloride accelerators), water reducers for concrete and waterproofing agents, and plasticizers for mortar can be used. To solve the corrosion problems, protective coatings such as galvanised and epoxy coatings for the steel members could be used. Referring to the mold problem, proper ventilation system, dehumidifiers and sealing against water intrusion are the proper options to mitigate risks.

Overall number of days with precipitation is 147. For the construction process, these conditions bring 2 main difficulties, construction work delays and storage problems. The high level of precipitation leads to the saturation of the ground, which creates problems with working on foundation. Materials sensitive to the moisture effect should be stored and handled appropriately to prevent material degradation. Moreover, high levels of precipitation in the city would require a proper drainage system to handle frequent rains and to prevent accumulation of water on the roof, basement floors and surrounding areas of the building.

Table 3.2. Roof Materials

Roof Materials		
Material	**Layer thickness(mm)**	**Description**
RC concrete, stone	125	Our roof is flat and should be durable to the climate conditions of our site location. Therefore, the tar and gravel option is suitable. Purpose of the gravel layer on top is to reflect UV rays on sunny days. Purpose of bituminous layers is to resist water leakage. Mineral wool is used primarily as a
Stone Concrete Screed	40	
Bituminous	0	
Mineral wool	101.6	
Four-ply felt	100	
Gravel	50	

		thermal insulation but also repels water as a second protection from water damage.

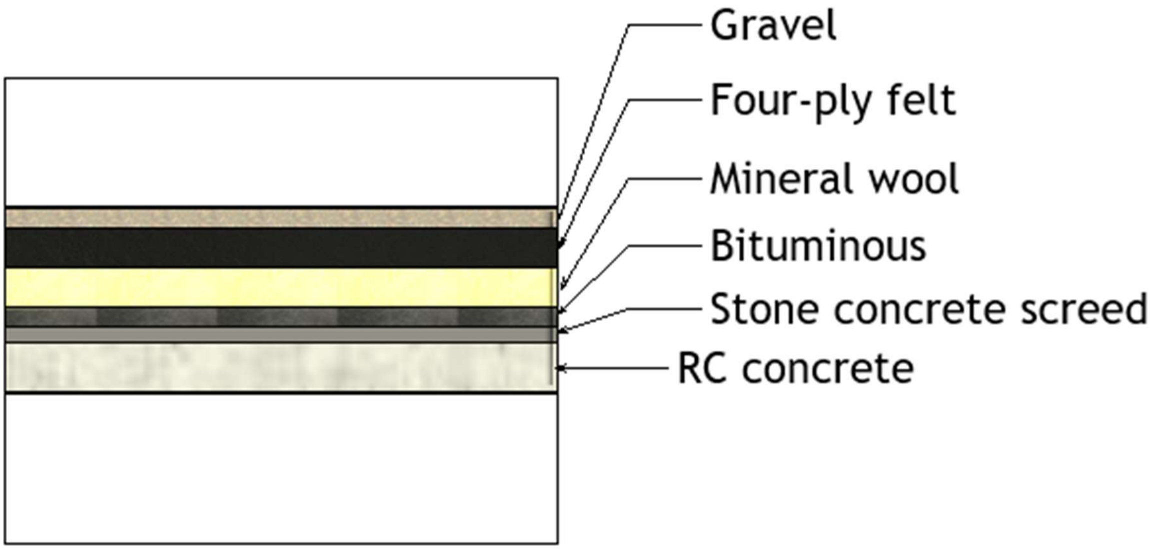


Figure 3.32. Roof Materials outline

Table 3.3. Ceiling Materials

Ceiling Materials Floors 2-13		
Material	Layer thickness	Description
Mechanical duct allowance	0	Our building is public. Therefore, it is necessary to consider the ventilation and air conditioning of all rooms. In this regard, it is a good idea to leave a special mechanical duct allowance for future ventilation ducts and air conditioners. Purpose of the suspended steel channel system is to create conditions for reliable
Suspended steel channel system	50	
Gypsum wall board	12	

		fastening of gypsum wall boards.

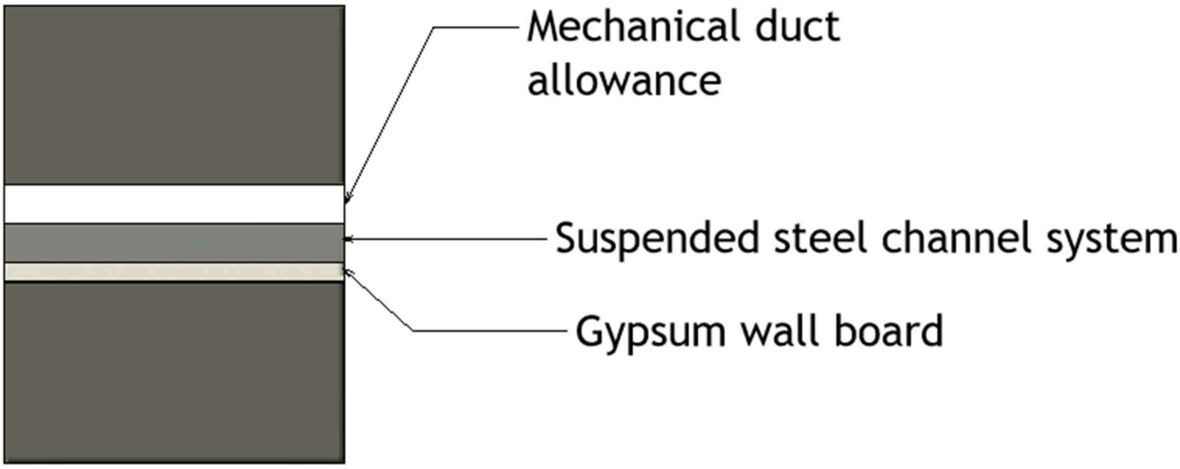


Figure 3.33. Ceiling Materials outline

Table 3.4. Floor Materials in Apartments

Floor Materials Floors 5-13		
Material	Layer thickness(mm)	Description
RC concrete, stone	125	The floor screed is used to align the floor to the same level. To ensure the silence between the floors rigid insulation is used and for water resistance a single ply sheet is necessary. The building is public, so it is necessary to ensure the resistance of materials to all conditions. Hardwood is an ideal solution; it is resistant to water and can withstand a heavy load.
Rigid insulation	13	
Single ply sheet	19	
Stone concrete screed	40	
Fibreboard	5	
Hardwood	22	

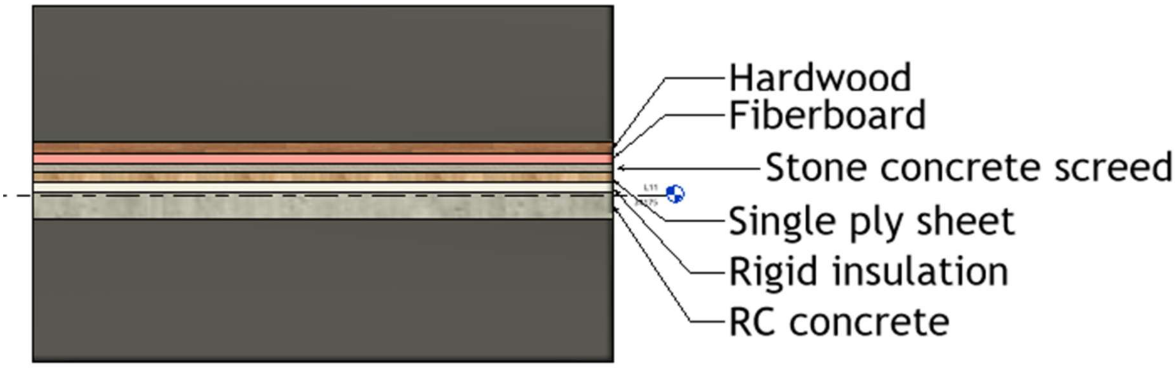


Figure 3.34. Floor Materials in Apartments outline

Table 3.5. Exterior Wall Materials in Apartment

Exterior Wall Materials Floors 5-13		
Material	Layer thickness	Description
Cement Screed	5	For the resistance of the walls to moisture and frost, an air layer is used to allow water to drain; one mineral wool for insulation and against water; vapour permeable membrane final protection from moisture. Thus, the walls are resistant to all weather conditions in the region of our building
Brick	380	
Vapour permeable membrane	0.45	
Mineral wool	101.6	
Metal stud layer	1.3715	
Air	0	
Granite panel	15	

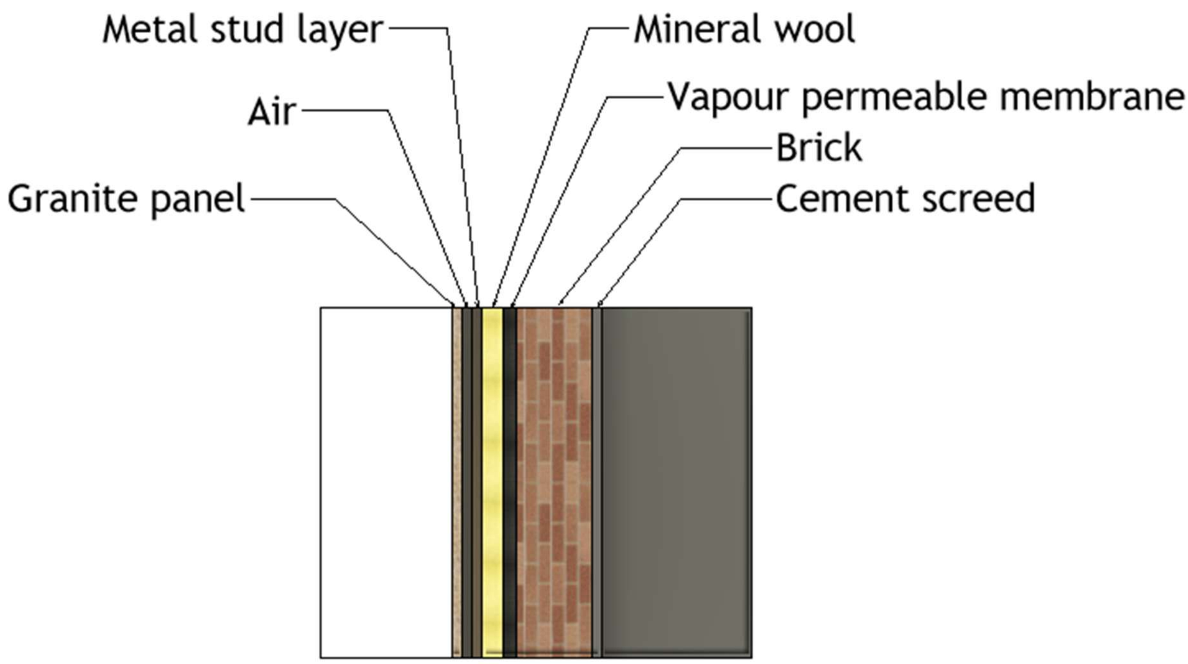


Figure 3.35. Exterior Wall Materials in Apartments outline

Table 3.6. Interior Wall Materials

Interior Wall Materials Floors 2-13		
Material	Layer thickness	Description
Cement Screed	5	Fiberglass batt for insulation and also against noises; Fiberglass batt is used. The wall screed is used to align the same level throughout the span.
Fiberglass batt	50	
Autoclaved aerated concrete blocks	150	
Fiberglass batt	50	
Cement Screed	5	

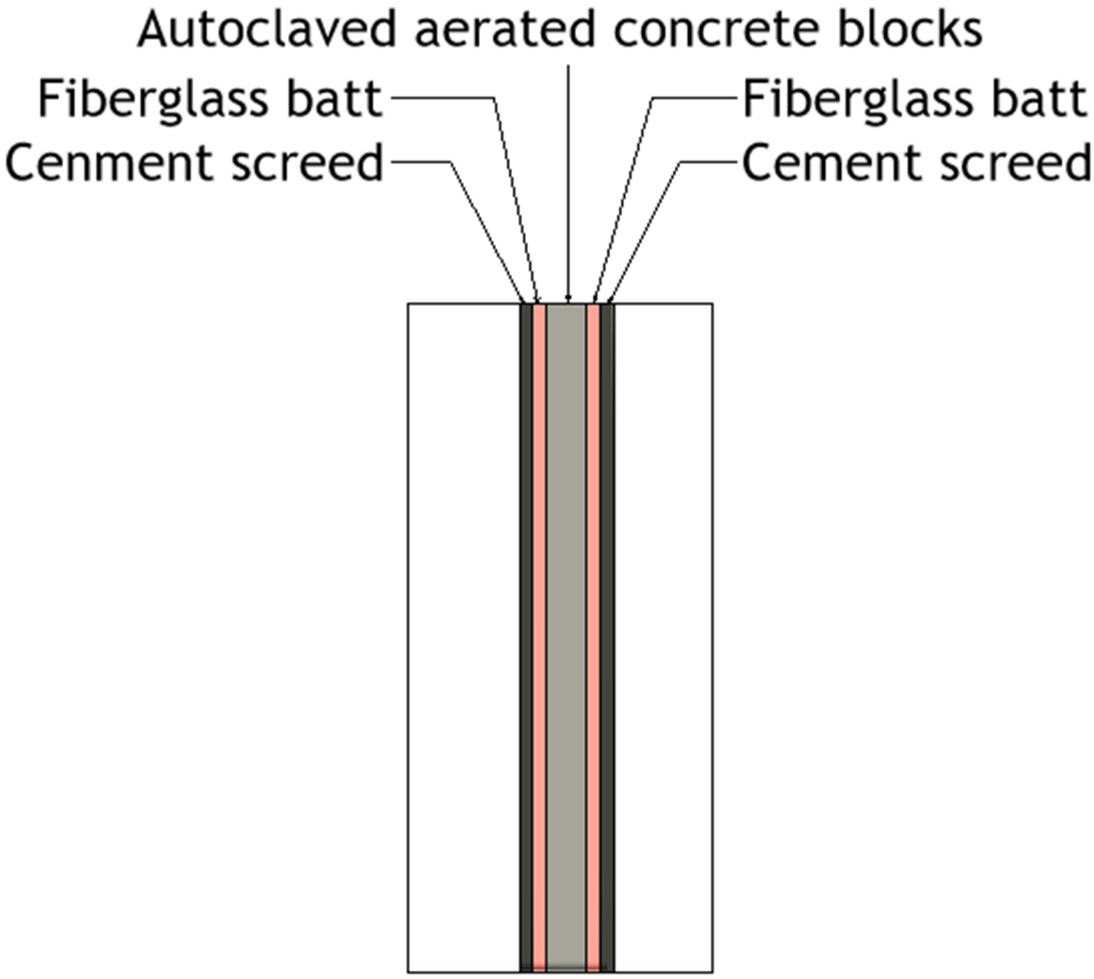


Figure 3.36. Interior Wall Materials outline

Table 3.7. Floor Materials in Public spaces

Floor Materials Floors 2-4		
Material	Layer thickness(mm)	Description
RC concrete, stone	125	The floor screed is used to align the floor to the same level. To ensure the silence between the floors rigid insulation is used and for water resistance a single ply sheet is necessary. The building is public, so it is necessary to ensure the
Rigid insulation	13	
Single ply sheet	19	
Stone concrete screed	40	
Mortar bed	25	

Ceramic tile	19	resistance of materials to all conditions. Ceramic tile is an ideal solution, it is resistant to water, scratches and each tile can withstand a heavy load.

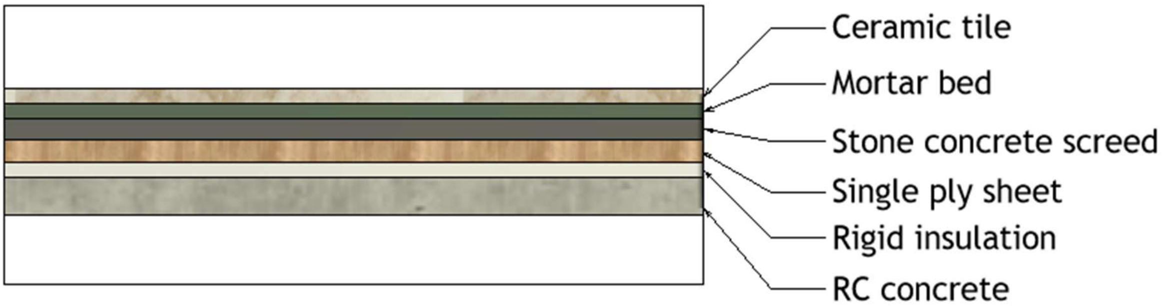


Figure 3.37. Floor Materials in Public spaces outline

Table 3.8. Exterior Wall Materials in Public spaces

Exterior Wall Materials Floors 2-4		
Material	Layer thickness	Description
Glass	6	For the resistance of the walls to moisture and frost, an air layer is used to allow water to drain; one thermal control layer for insulation and against water; Glass walls are applied for modern appearance and attraction of people.
Thermal control layer	12	
Air	0	
Glass	6	

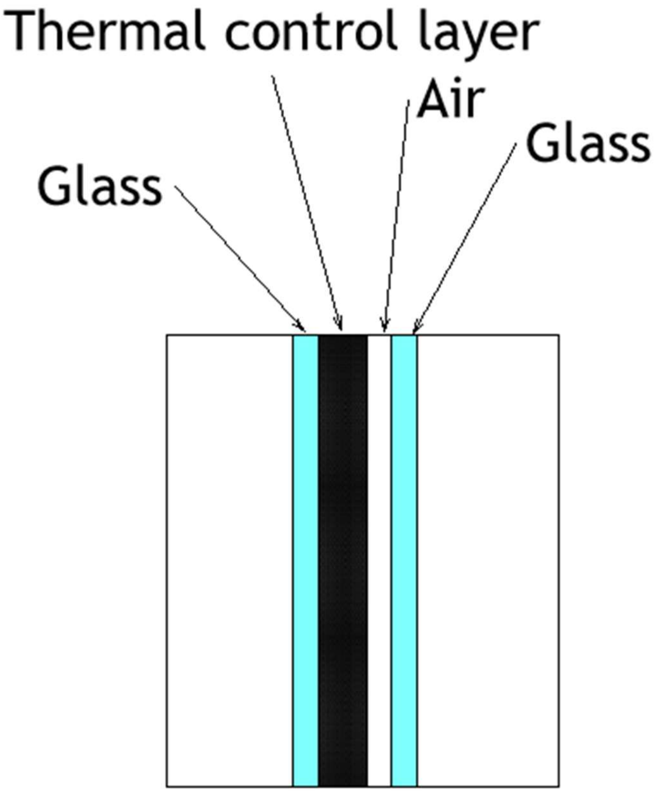


Figure 3.38. Exterior Wall Materials in Public spaces outline

4. Structural Design

4.1. Analysis and Design of Gravity Load Resisting System (GLRS)

4.1.1. Calculation of Dead, Live, and Snow Loads

4.1.1.1. Calculation of Dead Loads

Table 4.1. Floor Dead Load for Level 2-4

Corridors/First floor			
Component	**Material**	**Thickness (mm)**	**Load (kN/m^2)**
Cover	Ceramic tile	19	1.1
	Mortar bed	25	
Leveling	Stone concrete screed (per mm thickness)	40	0.023
Water-proofing	Single-ply sheet	19	0.03
Insulation	Rigid insulation	13	0.04
Structural slab	RC concrete, stone (including gravel) (per 1000 mm thickness)	125	23.6
Mechanical duct allowance			0.19
Suspended steel channel system			0.1
Gypsum board(per mm thickness)		12	0.008
Total		253	5.426

Table 4.2. Floor Dead Load for Apartments Level 5-13

Apartments			
	Material type	**Thickness (mm)**	**Load (kN/m^2)**
Cover	Hardwood	22	0.19
Membrane	Fiberboard(per mm thickness)	5	0.0028
Leveling	Stone concrete screed (per mm thickness)	40	0.023
Water-proofing	Single-ply sheet	19	0.03
Insulation	Rigid insulation	13	0.04
Structural slab	RC concrete, stone (including gravel) (per 1000 mm thickness)	125	23.6
Mechanical duct allowance			0.19
Suspended steel channel system			0.1

Gypsum board(per mm thickness)	12	0.008
Total	236	4.53

Table 4.3. Roof Dead Load

Roof			
	Material type	**Thickness (mm)**	**Load (kN/m^2)**
Cover	Gravel	50	0.26
	Four-ply felt	100	
Insulation	Mineral wool(per 1000 mm thickness)	101.6	2.2
Water-proofing	Bituminous, smooth surface		0.07
Leveling	Stone concrete screed (per mm thickness)	40	0.023
Structural slab	RC concrete, stone (including gravel) (per 1000 mm thickness)	125	23.6
Mechanical duct allowance			0.19
Suspended steel channel system			0.1
Gypsum board(per mm thickness)		12	0.008
Total		428.6	4.80952

Table 4.4. Exterior Wall Dead Load for Level 2

Exterior Walls							
Material	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m^3)**	**Density (kg/m^3)**	**Mass (kg)**	**Weight (kN)**
Glass	6	6000	6000	0.216	2500	540	5.3
Thermal control layer	12			0	0	0	0
Glass	6			0.216	2500	540	5.3
Total	24			0.432		1080	10.59

Table 4.5. Exterior Wall Dead Load for Level 3-4

Exterior Walls							
Material	**Thickness**	**Height**	**Span**	**Volume (m^3)**	**Density (kg/m^3)**	**Mass (kg)**	**Weight (kN)**

	(mm)	(mm)	(mm)				
Glass	6			0.144	2500	360	3.53
Thermal control layer	12	4000	6000	0	0	0	0
Glass	6			0.144	2500	360	3.53
Total	24			0.288		720	7.06

Table 4.6. Exterior Wall Dead Load for Level 5-13

Exterior Walls							
Material	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m^3)**	**Density (kg/m^3)**	**Mass (kg)**	**Weight (kN)**
Granite panel	15			0.27	2700	729	7.15
Air	0			0	0	0	0
Metal stud layer	1.3716			0.45	7820	193	1.89
Mineral wool	101.6	3000	6000	1.8288	50	91.44	0.89
Vapour permeable membrane	0.45			0.0081	300	2.43	0.024
Brick	380			6.84	2300	15732	154.33
Cement Screed	5			0.09	2100	189	1.85
Total	503.4			9.06		16937	166.15

Table 4.7. Interior Wall Dead Load for Level 2

Interior Walls							
Material	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m^3)**	**Density (kg/m^3)**	**Mass (kg)**	**Weight (kN)**
Cement Screed	5	6000	6000	0.18	2700	486	4.77

Fiberglass Batt	50			1.8	32	57.6	0.565
Autoclaved aerated concrete blocks	150			5.4	520	2808	27.55
Fiberglass Batt	50			1.8	32	57.6	0.565
Cement Screed	5			0.18	2700	486	4.77
Total	260			9.36		3895	38.2

Table 4.8. Interior Wall Dead Load for Level 3-4

Interior Walls							
Material	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume** (m^3)	**Density** (kg/m^3)	**Mass (kg)**	**Weight (kN)**
Cement Screed	5	4000	6000	0.12	2700	324	3.18
Fiberglass Batt	50			1.2	32	38.4	0.377
Autoclaved aerated concrete blocks	150			3.6	520	1872	18.36
Fiberglass Batt	50			1.2	32	38.4	0.377
Cement Screed	5			0.12	2700	324	3.18
Total	260			6.24		2597	25.475

Table 4.9. Interior Wall Dead Load for Level 5-13

Interior Walls							
Material	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume** (m^3)	**Density** (kg/m^3)	**Mass (kg)**	**Weight (kN)**

Cement Screed	5	3000	6000	0.09	2700	243	2.38
Fiberglass Batt	50			0.9	32	28.8	0.283
Autoclaved aerated concrete blocks	150			2.7	520	1404	13.77
Fiberglass Batt	50			0.9	32	28.8	0.283
Cement Screed	5			0.09	2700	243	2.38
Total	260			4.68		1948	19.1

Table 4.10. Plain Parapet Wall Dead Load

Plain Parapet Walls							
Material	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m^3)**	**Density (kg/m^3)**	**Mass (kg)**	**Weight (kN)**
Granite panel	15	762	6000	0.0686	2700	185	1.82
Air	0			0	0	0	0
Metal stud layer	1.3716			0.00627	7820	49	0.481
Mineral wool	101.6			0.465	50	23.23	0.228
Vapour permeable membrane	0.45			0.002	300	0.62	0.006
Brick	380			1.74	2300	3996.0	39.2
Cement Screed	5			0.023	2100	48.0	0.471
Total	503.4			2.3		4302	42.2

Table 4.11. Stairs Dead Load

Plain Parapet Walls						
Element	**Material**	**Dimension (mm)**	**Volume (m^3)**	**Density (kg/m^3)**	**Mass (kg)**	**Weight (kN)**
Riser height	Concrete	158	2.356	2400	5654	55.5
Thread		263				
Width		1000				
Slab height		345				
Landing height		200				
Waist		150				
Slab width		775				
Landing width		1000				
Length		2950				
Slab length		2400				
Landing length		2185				

4.1.1.2. Calculation of Live Loads

1. Floor Live Load Reduction:

The task of our project is to make a reasonable design of a thirteen-storey hotel building. In this regard, we need to make a reliable assumption for our live load for further calculations. Referring to the ASCE 7-10 code, we will use the figures below.

Based on ASCE 7-16 for Hotels minimum uniformly distributed live loads L_0 for different occupancies are shown below.

1. Private rooms and corridors serving them: $L_0 = 1.92\ kN/m^2$
2. Public rooms and corridors serving them: $L_0 = 4.79\ kN/m^2$

3. Stairs and exit ways: $L_0 = 4.79\ kN/m^2$

With accordance to ASCE 7-10, Floor Live Load Reduction is applicable when Influence Area (A_I) is bigger than 37.2 m^2:

$$A_I = K_{LL} * A_T \geq 37.2\ m^2 (400\ ft^2)$$
$$K_{LL} - live\ load\ element\ factor$$

$A_T - tributary\ area$ in m^2

$$L = L_0 * (0.25 + \frac{4.57}{\sqrt{K_{LL} * A_T}})$$
$$(L \leq L_0)$$
$$L - reduced\ design\ live\ load\ per\ m^2\ of\ area\ supported\ by\ the\ member$$
$$L_0 - unreduced\ design\ live\ load\ per\ m^2\ of\ area\ supported\ by\ the\ member$$
$$K_{LL} - live\ load\ element\ factor$$

$A_T - tributary\ area$ in m^2

Conditions:

1. $L > 0.5 * L_0$ (1 floor)
2. $L > 0.4 * L_0$ (2 or more floors)

Table 4.12. Live Load Element Factor, K_{LL}

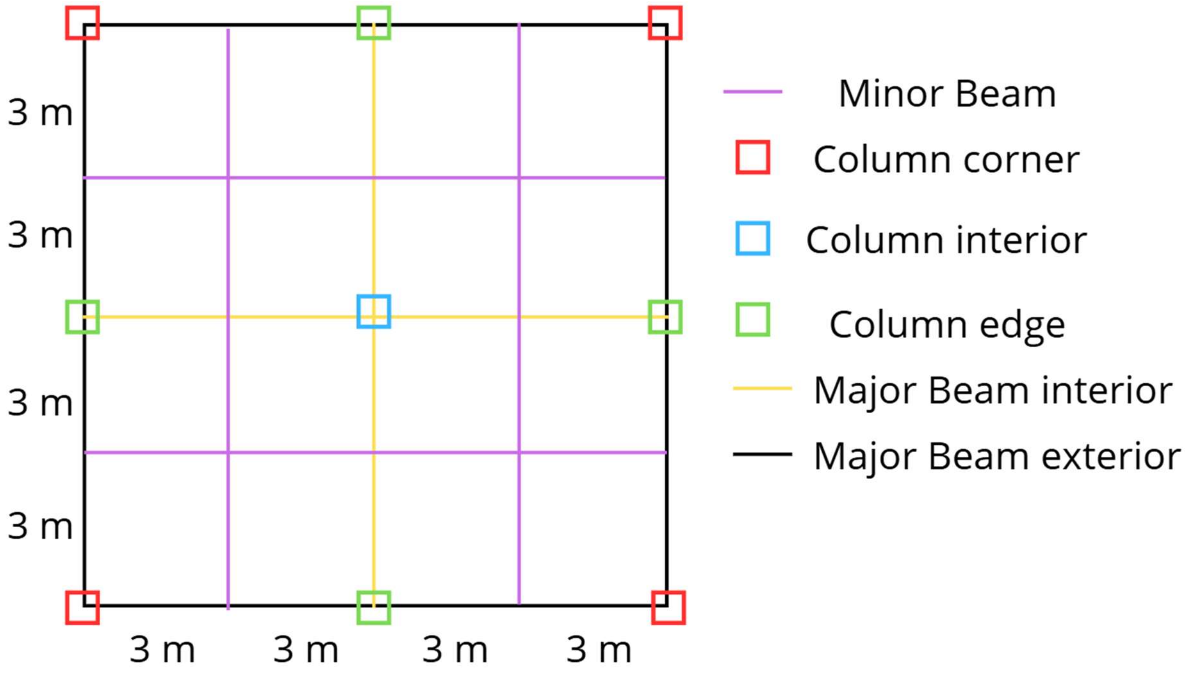
Element	K_{LL}[a]
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
One-way slabs	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

4.1.1.3. Floor Live Load calculations

Two-way Slab with Minor beams:

Width = 6 m

Length = 6 m



Levels 2,3,4 will be occupied for public services, therefore for these floors minimum uniformly distributed live loads $L_0 = 4.79\ kN/m^2$ is used for our calculations.

Table 4.13. Floor Live Load For Level 2-3-4

$L_0 = 4.79\ kN/m^2$	$A_T\ (m^2)$	$K_{LL}(-)$	$L(kN/m^2)$
Column(interior)	36	4	$L = 4.79 * (0.25 + \frac{4.57}{\sqrt{4 * 36}}) = 3.02$
Column(exterior)	18	4	$L = 4.79 * (0.25 + \frac{4.57}{\sqrt{4 * 18}}) = 3.78$
Column(corner)	9	1	$A_I = 9 * 1 = 9\ m^2 \Rightarrow Not\ \ applicable$
Beam(interior)	18	2	$A_I = 18 * 2 = 36\ m^2 \Rightarrow Not\ \ applicable$
Beam(exterior)	9	2	$A_I = 9 * 2 = 18\ m^2 \Rightarrow Not\ \ applicable$

Tributary area calculations:

$$A_T\ (Beam - Interior) = \boldsymbol{6 * 1.5 * 2 = 18}\ m^2$$

$$A_T\ (Beam - Exterior) = \boldsymbol{6 * 1.5 = 9}\ m^2$$

$$A_T\ (Column - Interior) = \boldsymbol{6 * 6 = 36}\ m^2$$

$$A_T\ (Column - Exterior) = \boldsymbol{6 * 3 = 18}\ m^2$$

$$A_T\ (Column - Corner) = \boldsymbol{3 * 3 = 9}\ m^2$$

Levels 5-13 will be occupied for private apartments, therefore for these floors minimum uniformly distributed live loads $L_0 = 1.92\ kN/m^2$ is used for our calculations.

Table 4.14. Floor Live Load For Level 5-13

$L_0 = 1.92\ kN/m^2$	$A_T(m^2)$	$K_{LL}(-)$	$L(kN/m^2)$
Column(interior)	36	4	$L = 1.92 * (0.25 + \frac{4.57}{\sqrt{4 * 36}}) = 1.21$
Column(exterior)	18	4	$L = 1.92 * (0.25 + \frac{4.57}{\sqrt{4 * 18}}) = 1.51$
Column(corner)	9	1	$A_I = 9 * 1 = 9\ m^2 \Rightarrow Not\ \ applicable$
Beam(interior)	18	2	$A_I = 18 * 2 = 36\ m^2 \Rightarrow Not\ \ applicable$
Beam(exterior)	9	2	$A_I = 9 * 2 = 18\ m^2 \Rightarrow Not\ \ applicable$

Table 4.15. Floor Live Load For Stairs and Exit Ways

$L_0 = 4.79\ kN/m^2$	$A_T(m^2)$	$K_{LL}(-)$	$L(kN/m^2)$
Column(interior)	36	4	$L = 4.79 * (0.25 + \frac{4.57}{\sqrt{4 * 36}}) = 3.02$
Column(exterior)	18	4	$L = 4.79 * (0.25 + \frac{4.57}{\sqrt{4 * 18}}) = 3.78$
Beam(interior)	18	2	$A_I = 18 * 2 = 36\ m^2 \Rightarrow Not\ \ applicable$
Beam(exterior)	9	2	$A_I = 9 * 2 = 18\ m^2 \Rightarrow Not\ \ applicable$

4.1.1.4. Roof Live Load reduction

Two-way Slab with Minor beams:

$$L_r = L_0 R_1 R_2$$
$$(\boldsymbol{0.58} \leq L_r \leq \boldsymbol{0.96})$$

□□
$- reduced\ roof\ live\ load\ per\ m^2 of\ \boldsymbol{h}orizontal\ projection\ supported\ by\ t\boldsymbol{he}\ member$
□$_{\boldsymbol{0}}$
$- unreduced\ roof\ live\ load\ per\ m^2 of\ \boldsymbol{h}orizontal\ projection\ supported\ by\ t\boldsymbol{he}\ member$

Conditions:

1. $R_1 = 1\ for\ A_T \leq 18.58\ m^2$
$$R_1 = 1.2 - 0.011 A_T\ for\ 18.58\ m^2 \leq A_T \leq 55.74\ m^2$$
$$R_1 = 0.6\ for\ A_T \geq 55.74\ m^2$$

2. $R_2 = 1\ for\ F \leq 4$
$$R_2 = 1.2 - 0.05F\ for\ 4 < F < 12$$
$$R_2 = 0.6\ for\ F \geq 12$$

4.1.1.5. Roof Live Load calculations

Roofs	
Ordinary flat, pitched, and curved roofs	20 (0.96)[n]
Roofs used for roof gardens	100 (4.79)
Roofs used for assembly purposes	Same as occupancy served
Roofs used for other occupancies	[o]
Awnings and canopies	
Fabric construction supported by a skeleton structure	5 (0.24) nonreducible

Figure 3.14. Minimum Uniformly Distributed Roof Live Loads, L_0

From the table above:

$$Roofs - Ordinary\ flat, pitched, and\ curved\ roofs = 0.96\ kN/m^2$$

Table 4.16. Roof Live Load Summary, L

$L_0 = 0.96\ kN/m^2$	$A_T(m^2)$	$R_1(-)$	$L(kN/m^2)$
Column(interior)	36	$R_1 = 1.2 - 0.011 * 36 = 0.804$	$L_r = \boldsymbol{0.96} * 0.804 = \boldsymbol{0.77}$
Column(edge)	18	$R_1 = 1$	$L_r = \boldsymbol{0.96 * 1 = 0.96}$
Column(corner)	9	$R_1 = 1$	$L_r = \boldsymbol{0.96 * 1 = 0.96}$
Beam(interior)	18	$R_1 = 1$	$L_r = \boldsymbol{0.96 * 1 = 0.96}$
Beam(exterior)	9	$R_1 = 1$	$L_r = \boldsymbol{0.96 * 1 = 0.96}$

The Floor live load for the building on levels 5-13 equals 1.92 kPa, with the exception of interior and exterior columns, which have live loads equals 1.21 and 1.51 kPa, respectively. The roof live load is 0.96 kPa, with the exception of the interior columns, which have a load of 077 kPa. Levels 1-4 are considered separately as they are planned to be used for public services. Floor live load for the building on levels 2-4 equals 4.79 kPa, with the exception of interior and exterior columns, which have live loads equals 3.02 and 3.78 kPa, respectively.

4.1.1.6. Calculation of Snow Loads

According to Figure 7.2-1 in ASCE 7-16, the ground snow load for Seattle city, $P_g = 20$ psf. The surface roughness is classified as B since the building is located in the big city, and the exposure is considered sheltered because of the likelihood of location in the city centre, leading to a Ce value of 1.2. The building falls under risk category II, as no specific purposes or requirements were mentioned, resulting in an Is value of 1. Additionally, no special roof uses were indicated, so Ct is assigned a value of 1.

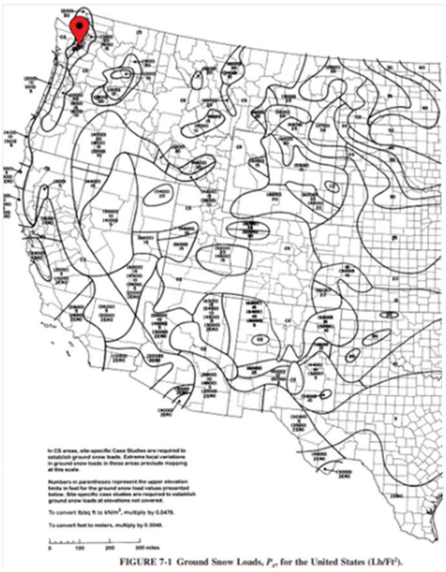


Figure 4.1. Ground Snow Loads for USA

Using the formula $P_f = 0.7 * Ce * Ct * Is * P_g$, we calculated the initial snow load on flat roofs value, which accounted for 16.8 psf. However, we also had to determine the minimum snow load value for low-slope roofs through $P_m = Is * P_g = 1*20 = 20$ psf. Since P_m was higher than the initial P_f, the final value for snow load on flat roofs would be 20 psf or
0.958 KPa.

$\gamma = 0.13 * P_g + 14 = 0.13 * 20 + 14 = 16.6$ psf; $hb = \frac{P_g}{\gamma} = \frac{20}{16.6} = \boldsymbol{1.2}$ ft ;

hc= h- hb =134.5 ft-1.2 ft =133.3 ft;

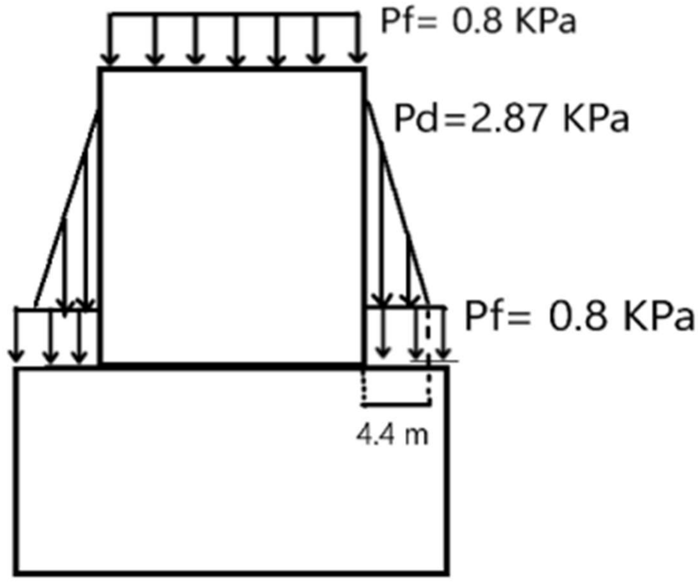


Figure 4.2. Snow drifts for the whole building

Since our building is symmetric from all sides, the value of Lu* will be the same for calculating P_d, the maximum intensity of the drift surcharge load.

$$hd = 0.43 * (Lu)^{\frac{1}{3}} * (P_{g} + 10)^{\frac{1}{4}} - 1.5$$

Leeward (Lu=10 m): hd= 1.71 ft

Windward (Lu=40m) : hd= 3.61 ft (controlled)

Since hd< hc W= 4*hd = 14.44 ft= 4.4 m

$$P_d = hd * \gamma = 3.61 * 16.6 = 59.9 \text{ psf} = 2.87 \text{ kPa}$$

h = vertical separation distance between the edge of a higher roof and the edge of a lower adjacent roof.

hb = height of balanced snow load

hc = clear height from the top of the balanced snow load to the top of the parapet

hd = height of snow drift

Lu = length of the roof upwind of the drift

W = horizontal distance from eave to ridge

γ= snow density

For the roof side we are going to use same procedure:

Leeward (Lu=5 m): hd= 1.23 ft

Windward (Lu=10 m) : hd= 1.29 (controlled)

W= 4*hd= 4* 1.29 = 5.16 ft = 1.57 m

P_d= hd * γ = 1.29 * 16.6 = 21.4 psf= 1.02 KPa

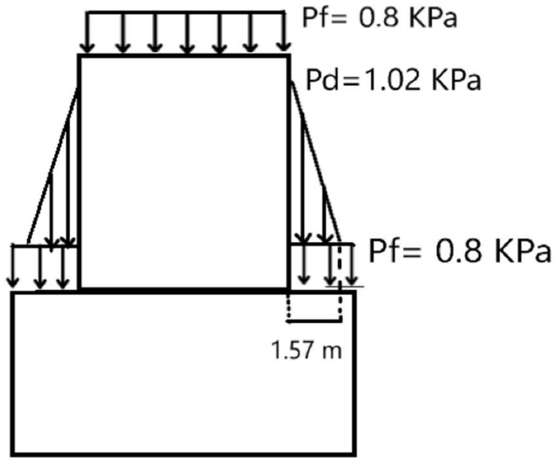


Figure 4.3. Snow drifts for the roof side.

4.2. Dimensions of structural members

4.2.1. Two-Way slab without minor beams

- **For Major Beam:**

Beam spacing = L_{beam}= 6 m

$h_{beam} \approx 10\% * L = 0.1 * 6 = 0.6\,m$

$$h_{beam} - beam\ height$$
$$L_{beam} - beam\ span$$

$$w_{beam} \approx 50\% * h_{beam} = 0.6 * 0.5 = 0.3\,m$$
$$w_{beam} - beam\ width$$
$$h_{beam} - beam\ height$$

- **For slab:**

L_{slab}= b_{slab} =6 m
Materials are assumed to be:

Reinforced concrete Grade C35($f_c = 35\ MPa$) for both major beam and slab.
Reinforcing steel rebar Grade 75($f_y = 75\ ksi = 520\ MPa$).

$$I_b = \frac{0.35}{12} * {h_{beam}}^3 * w_{beam} = \frac{0.35}{12} * (0.6m)^3 * 0.3m = 0.00189\,m^4$$
$$I_b - beam\ moment\ of\ inertia$$
$$w_{beam} - beam\ width$$
$$h_{beam} - beam\ height$$
$$initial\ guess\ h_{slab} = 0.2m$$

$$I_s = \frac{0.25}{12} * {h_{slab}}^3 * L_{yn} = \frac{0.25}{12} * (0.2m)^3 * 6 = 0.001\ m^4$$
$$I_s - slab\ moment\ of\ inertia$$
$$L_{yn}\ - slab\ span(bigger)$$
$$h_{slab} - slab\ height$$

According to ACI Code Modulus of Elasticity of Concrete:

For $1440\ kg/m^3 \leq w_c \leq 2560\ kg/m^3$:
$$E = w_c^{1.5} * 0.043 * \sqrt{f_c'}\ (MPa)$$

For normal-weight concrete:

$$E = 4700\sqrt{f_c'}\ (MPa)$$

Since our concrete is normal weight, let's calculate modulus of elasticity for slab and beam:

$$C35\ concrete:$$
$$E_{cb} = E_{cs} = E = 4700\sqrt{f_c'}\ (MPa) = 4700\sqrt{35} = 27806\ MPa$$

$$\alpha_{f1} = \frac{E_{cb} * I_b}{E_{cs} * I_s} = \frac{\boldsymbol{27806}\frac{kN}{m^2} * 0.00189m^4}{\boldsymbol{27806}\frac{kN}{m^2} * 0.001m^4} = \boldsymbol{1.89}$$
$$E_{cb} = \boldsymbol{27806}MPa(beam\ modulus\ of\ elasticity\ for\ reinforced\ concrete\ C\boldsymbol{35})$$
$$E_{cs} = \boldsymbol{27806}MPa(slab\ modulus\ of\ elasticity\ for\ reinforced\ concrete\ C\boldsymbol{35})$$
$$I_b - beam\ moment\ of\ inertia$$
$$I_s - slab\ moment\ of\ inertia$$

Beam dimensions are same and slab width and length are equal, therefore:
$$\alpha_{f1}=\alpha_{f2}=\alpha_{f3}=\alpha_{f4}$$
$$\alpha_{fm} = \frac{\alpha_{f1} + \alpha_{f2} + \alpha_{f3} + \alpha_{f4}}{\boldsymbol{4}} = \frac{\boldsymbol{4}\alpha_{f1}}{\boldsymbol{4}} = \boldsymbol{1.89}$$

Conditions:

1. $\alpha_f \leq \boldsymbol{0.2} \Rightarrow flat\ slab$

2. $\boldsymbol{0.2} \leq \alpha_f < \boldsymbol{2} \Rightarrow \boldsymbol{h}_{min} = \frac{L_n(0.8+\frac{fy}{200000})}{36+5\beta(\alpha_{fm}-0.2)} \geq \boldsymbol{0.127}\ m$

3. $\alpha_f > \boldsymbol{2} \Rightarrow \boldsymbol{h}_{min} = \frac{L_n(0.8+\frac{fy}{200000})}{36+9\beta} \geq \boldsymbol{0.0889}\ m$

$$\boldsymbol{0.2} < \alpha_{fm} = \boldsymbol{1.89} \leq \boldsymbol{2}$$

$$\textbf{2. } \boldsymbol{h}_{min} = \frac{6*(0.8+\frac{75000}{200000})}{36+5*1(1.89-0.2)} = \boldsymbol{0.1586}\, m \geq \boldsymbol{0.127}\, m$$

$$L_n = max\ of\ [L_{yn}, L_{xn}] = max\ of\ [\boldsymbol{6}, \boldsymbol{6}] = \boldsymbol{6}$$

$$\beta = \frac{L_{yn}}{L_{xn}} = \frac{\boldsymbol{6}}{\boldsymbol{6}} = \boldsymbol{1}$$

$$f_y - reinforcing\ steel\ yield\ strengt\boldsymbol{h}$$

Table 4.17. Final member sizes.

Final member sizes				
Member	$L(m)$	$h(m)$	$b(m)$	n
Major beam	6	0.6	0.3	4
Slab	6	0.159	6	1

$$Volume\ of\ major\ beam = V_{major\ beam} = L * \boldsymbol{h} * b = \boldsymbol{6} * \boldsymbol{0.6} * \boldsymbol{0.3} = \boldsymbol{1.08}\, m^3$$

$$Volume\ of\ slab = V_{slab} = L * \boldsymbol{h} * b = \boldsymbol{6} * \boldsymbol{0.159} * \boldsymbol{6} = \boldsymbol{5.71}\, m^3$$

$$Volume\ of\ concrete = V_{concrete} = \boldsymbol{4} * V_{major\ beam} + V_{slab} = \boldsymbol{4} * \boldsymbol{1.08} + \boldsymbol{5.71} = \boldsymbol{10.03}\, m^3$$

4.2.2. Two-Way slab with minor beams

- **For Major Beam:**

Beam spacing = L= 6 m

$h_{beam} \approx 8\% * L = 0.08 * 6 = 0.48\, m$

$$h_{beam} - beam\ height$$
$$L_{beam} - beam\ span$$

$$w_{beam} \approx 50\% * h_{beam} = 0.48 * 0.5 = 0.24\, m$$
$$w_{beam} - beam\ width$$
$$h_{beam} - beam\ height$$

- **For Minor Beam:**

For simply supported beam: $h_{min} = \frac{L_{beam}}{16}$

$h_{beam} = \frac{L_{beam}}{16} = \frac{6}{16} = 0.375\, m$

$$h_{beam} - beam\ height$$
$$L_{beam} - beam\ span$$

$$w_{beam} \approx 50\% * h_{beam} = 0.375 * 0.5 = 0.1875\ m$$
$$w_{beam} - beam\ width$$
$$h_{beam} - beam\ height$$

- **For slab:**

L_{slab}= b_{slab} =3 m
Materials are assumed to be:

Reinforced concrete Grade C35($f_c = 35\ MPa$) for major, minor beams and slab.
Reinforcing steel rebar Grade 75($f_y = 75\ ksi = 520\ MPa$).

$$I_{b,major} = \frac{0.35}{12} * h_{major\ beam}{}^{3} * w_{major\ beam} = \frac{0.35}{12} * (0.48m)^3 * 0.24m$$
$$= 0.000774\ m^4$$
$$I_{b,major} - major\ beam\ moment\ of\ inertia$$
$$w_{major\ beam} - major\ beam\ width$$
$$h_{major\ beam} - major\ beam\ height$$

$$I_{b,minor} = \frac{0.35}{12} * h_{minor\ beam}{}^{3} * w_{minor\ beam} = \frac{0.35}{12} * (0.375m)^3 * 0.1875m$$
$$= 0.000288\ m^4$$
$$I_{b,minor} - minor\ beam\ moment\ of\ inertia$$
$$w_{minor\ beam} - minor\ beam\ width$$
$$h_{minor\ beam} - minor\ beam\ height$$

$$initial\ guess\ h_{slab} = 0.2m$$
$$I_s = \frac{0.25}{12} * h_{slab}{}^{3} * L_{yn} = \frac{0.25}{12} * (0.2m)^3 * 3 = 0.0005\ m^4$$
$$I_s - slab\ moment\ of\ inertia$$
$$L_{yn} - slab\ span(bigger)$$
$$h_{slab} - slab\ height$$

According to ACI Code Modulus of Elasticity of Concrete:

For $1440\ kg/m^3 \leq w_c \leq 2560\ kg/m^3$:
$$E = w_c^{1.5} * 0.043 * \sqrt{f_c'}\ (MPa)$$

For normal-weight concrete:

$$E = 4700\sqrt{f_c'}\ (MPa)$$

Since our concrete is normal weight, let’s calculate modulus of elasticity for slab and beams:

$$C35\ concrete:$$

$$E_{cb,minor} = E_{cb,major} = E_s = E = 4700\sqrt{f_c'}\ (MPa) = 4700\sqrt{35} = 27806\ MPa$$

$$\alpha_{f1} = \frac{E_{cb,major} * I_{b,major}}{E_{cs} * I_s} = \frac{\mathbf{27806}\frac{kN}{m^2} * 0.000774m^4}{\mathbf{27806}\frac{kN}{m^2} * 0.0005m^4} = \mathbf{1.548}$$

□□□,□□□□□

$$= \mathbf{27806}MPa(major\ beam\ modulus\ of\ elasticity\ for\ reinforced\ concrete\ C\mathbf{35})$$

$$E_{cs} = \mathbf{27806}MPa(slab\ modulus\ of\ elasticity\ for\ reinforced\ concrete\ C\mathbf{35})$$

$$I_{b,major} - major\ beam\ moment\ of\ inertia$$

$$I_s - slab\ moment\ of\ inertia$$

$$\alpha_{f3} = \frac{E_{cb,minor} * I_{b,minor}}{E_{cs} * I_s} = \frac{\mathbf{27806}\frac{kN}{m^2} * 0.000288m^4}{\mathbf{27806}\frac{kN}{m^2} * 0.0005m^4} = \mathbf{0.577}$$

□□□,□□□□□

$$= \mathbf{27806}MPa(minor\ beam\ modulus\ of\ elasticity\ for\ reinforced\ concrete\ C\mathbf{35})$$

$$E_{cs} = \mathbf{27806}MPa(slab\ modulus\ of\ elasticity\ for\ reinforced\ concrete\ C\mathbf{35})$$

$$I_{b,minor} - minor\ beam\ moment\ of\ inertia$$

$$I_s - slab\ moment\ of\ inertia$$

Major beams dimensions are the same and minor beams dimensions are the same, therefore:

$$\alpha_{f1} = \alpha_{f2}\ ;\ \alpha_{f3} = \alpha_{f4}$$

$$\alpha_{fm} = \frac{2 * \alpha_{f1} + 2 * \alpha_{f3}}{4} = \frac{2 * \mathbf{1.548} \quad + 2 * \mathbf{0.577}}{4} = \mathbf{1.063}$$

Conditions:

1. $\alpha_f \le 0.2 \Rightarrow flat\ slab$

2. $0.2 \le \alpha_f < 2 \Rightarrow \boldsymbol{h}_{min} = \frac{L_n(0.8+\frac{fy}{200000})}{36+5\beta(\alpha_{fm}-0.2)} \ge \mathbf{0.127}\ m$

3. $\alpha_f > 2 \Rightarrow \boldsymbol{h}_{min} = \frac{L_n(0.8+\frac{fy}{200000})}{36+9\beta} \ge \mathbf{0.0889}\ m$

$$\mathbf{0.2} < \alpha_{fm} = \mathbf{1.063} \le \mathbf{2}$$

$$\mathbf{2.}\ \boldsymbol{h}_{min} = \frac{3*(0.8+\frac{75000}{200000})}{36+5*1(1.063-0.2)} = \mathbf{0.08744}\ m \le \mathbf{0.127}\ m \Rightarrow \boldsymbol{h}_{min} = \mathbf{0.127}\ m$$

$$L_n = max\ of\ [L_{yn}, L_{xn}] = max\ of\ [3, 3] = \mathbf{3}$$

$$\beta = \frac{L_{yn}}{L_{xn}} = \frac{3}{3} = \mathbf{1}$$

$f_y - reinforcing\ steel\ yield\ strength$

Table 4.18. Final member sizes including minor beams.

Final member sizes				
Member	$L(m)$	$h(m)$	$b(m)$	n
Major beam	6	0.48	0.24	4
Minor beam	6	0.375	0.188	2
Slab	3	0.127	3	4

$$Volume\ of\ major\ beam = V_{major\ beam} = L * \boldsymbol{h} * b = \boldsymbol{6 * 0.48 * 0.24}$$
$$= \boldsymbol{0.691}\ m^3$$
$$Volume\ of\ minor\ beam = V_{minor\ beam} = L * \boldsymbol{h} * b = \boldsymbol{6 * 0.375 * 0.188}$$
$$= \boldsymbol{0.422}\ m^3$$
$$Volume\ of\ slab = V_{slab} = L * \boldsymbol{h} * b = \boldsymbol{3 * 0.127 * 3 = 1.143}\ m^3$$
$$Volume\ of\ concrete = V_{concrete}$$
$$= \boldsymbol{4} * V_{major\ beam} + \boldsymbol{2} * V_{minor\ beam} + \boldsymbol{4} * V_{slab}$$
$$Volume\ of\ concrete = \boldsymbol{4 * 0.691 + 2 * 0.422 + 4 * 1.143 = 8.18}\ m^3$$

4.2.3. One-Way slab with minor beams

- **For Major Beam:**

Beam spacing = L= 6 m

$$h_{beam} \approx 8\% * L = 0.08 * 6 = 0.48\ m$$
$$h_{beam} - beam\ height$$
$$L_{beam} - beam\ span$$

$$w_{beam} \approx 50\% * h_{beam} = 0.48 * 0.5 = 0.24\ m$$
$$w_{beam} - beam\ width$$
$$h_{beam} - beam\ height$$

- **For Minor Beam:**

For simply supported beam: $h_{min} = \frac{L_{beam}}{16}$

$$h_{beam} = \frac{L_{beam}}{16} = \frac{6}{16} = 0.375\ m$$
$$h_{beam} - beam\ height$$
$$L_{beam} - beam\ span$$

$$w_{beam} \approx 50\% * h_{beam} = 0.375 * 0.5 = 0.1875\, m$$
$$w_{beam} - beam\ width$$
$$h_{beam} - beam\ height$$

- **For slab:**

$$L_{slab} = 6\, m$$
$$b_{slab} = 3\, m$$
$$For\ Solid\ One - Way\ slabs \Rightarrow Simply\ supported:$$
$$h_{min} = \frac{L}{20} = \frac{3}{20} = 0.15\, m$$

Table 4.19. Final member sizes with Slab changes.

Final member sizes				
Member	$L(m)$	$h(m)$	$b(m)$	n
Major beam	6	0.48	0.24	4
Minor beam	6	0.375	0.188	1
Slab	6	0.15	3	2

$$Volume\ of\ major\ beam = V_{major\ beam} = L * \boldsymbol{h} * b = \boldsymbol{6 * 0.48 * 0.24} = \boldsymbol{0.691}\, m^3$$
$$Volume\ of\ minor\ beam = V_{minor\ beam} = L * \boldsymbol{h} * b = \boldsymbol{6 * 0.375 * 0.188} = \boldsymbol{0.422}\, m^3$$
$$Volume\ of\ slab = V_{slab} = L * \boldsymbol{h} * b = \boldsymbol{6 * 0.15 * 3 = 2.7}\, m^3$$
$$Volume\ of\ concrete = V_{concrete} = \boldsymbol{4} * V_{major\ beam} + \boldsymbol{1} * V_{minor\ beam} + \boldsymbol{2} * V_{slab}$$
$$Volume\ of\ concrete = \boldsymbol{4 * 0.691 + 1 * 0.422 + 2 * 2.7 = 8.587}\, m^3$$

Table 4.20. Final member sizes for choosing method.

Final member sizes						
Method	Member	$L\ (m)$	$h\ (m)$	$b\ (m)$	n	$V_{concrete}(m^3)$
Two-Way slab without minor beams	Major beam	6	0.6	0.3	4	10.30
	Slab	6	0.159	6	1	

Two-Way slab with minor beams	Major beam	6	0.48	0.24	4	8.181
	Minor beam	6	0.375	0.188	2	
	Slab	3	0.127	3	4	
One-Way slab with minor beams	Major beam	6	0.48	0.24	4	8.587
	Minor beam	6	0.375	0.188	1	
	Slab	6	0.15	3	2	

From the table above we could see that the usage of "**Two-Way slab with minor beams**" is reliable, as it is the most economical one.

Table 4.21. Final member sizes characteristics.

Two-Way slab with minor beams(appropriate dimensions selection)					
Member	$L(m)$	$h(m)$	$b(m)$	n	$V_{concrete}(m^3)$
Major beam	6	0.48	0.24	4	***8.2248***
Minor beam	6	$0.375 \approx 0.4$	$0.188 \approx 0.2$	2	
Slab	3	$0.127 \approx 0.125$	3	4	

$$Volume\ of\ concrete = \boldsymbol{4} * \boldsymbol{6} * \boldsymbol{0.48} * \boldsymbol{0.24} + \boldsymbol{2} * \boldsymbol{6} * \boldsymbol{0.4} * \boldsymbol{0.2} + \boldsymbol{4} * \boldsymbol{3} * \boldsymbol{0.125} * \boldsymbol{3}$$
$$Volume\ of\ concrete = \boldsymbol{8.2248}\ m^3$$

The next step is to calculate the total interior and exterior wall span for further calculations of Dead Load.

For that purpose we will use the Revit integrated function "Schedule", which will separately calculate total Volumes of our materials on each Level. For our Total interior wall span calculations we chose Total volume of Autoclaved aerated concrete blocks, while for Total exterior wall span we chose Total volume of Brick. These materials are only included in interior and exterior walls, therefore there will not be any inaccuracies.

4.2.4. Total Interior Wall Span Calculations

For Level 2:

L2			
Autoclaved aerated concrete block			
Базовая стена: Interior capstone	Autoclaved aerated concrete	306.12 м³	L2

$$H_{Wall} = 6\,m$$
$$t_{Concrete\ blocks} = 0.15\,m$$
$$V_{Concrete\ blocks} = L_{Concrete\ blocks} * H_{Wall} * t_{Concrete\ blocks}$$
$$306.12\,m^3 = L_{Concrete\ blocks} * 6\,m * 0.15\,m$$
$$L_{Concrete\ blocks} = L_{wall} = 340.13\,m$$

For Level 3:

L3			
Autoclaved aerated concrete block			
Базовая стена: Interior capstone	Autoclaved aerated concrete	210.10 м³	L3

$$H_{Wall} = 4\,m$$
$$t_{Concrete\ blocks} = 0.15\,m$$
$$V_{Concrete\ blocks} = L_{Concrete\ blocks} * H_{Wall} * t_{Concrete\ blocks}$$
$$210.1\,m^3 = L_{Concrete\ blocks} * 4\,m * 0.15\,m$$
$$L_{Concrete\ blocks} = L_{wall} = 350.16\,m$$

For Level 4:

L4			
Autoclaved aerated concrete block			
Базовая стена: Interior capstone	Autoclaved aerated concrete	131.98 м³	L4

$$H_{Wall} = 4\,m$$
$$t_{Concrete\ blocks} = 0.15\,m$$
$$V_{Concrete\ blocks} = L_{Concrete\ blocks} * H_{Wall} * t_{Concrete\ blocks}$$
$$131.98\,m^3 = L_{Concrete\ blocks} * 4\,m * 0.15\,m$$
$$L_{Concrete\ blocks} = L_{wall} = 219.96\,m$$

For Level 5-13:

Autoclaved aerated concrete block			
Базовая стена: Interior capstone	Autoclaved aerated concrete	232.81 м³	L5

$$H_{Wall} = 3\,m$$
$$t_{Concrete\ blocks} = 0.15\,m$$
$$V_{Concrete\ blocks} = L_{Concrete\ blocks} * H_{Wall} * t_{Concrete\ blocks}$$
$$232.81\,m^3 = L_{Concrete\ blocks} * 3\,m * 0.15\,m$$
$$L_{Concrete\ blocks} = L_{wall} = 517.35\,m$$

4.2.5. Total Exterior Wall Span Calculations

For Level 2-4:

Basic Wall: Exterior	Glass	L2	487.63 m³

$$H_{Wall} = 6\,m$$
$$t_{Glass} = 0.38\,m$$
$$V_{Glass} = L_{Glass} * H_{Wall} * t_{Glass}$$
$$487.63\,m^3 = L_{Glass} * 6\,m * 0.38\,m$$
$$L_{Glass} = L_{wall} = 213.43\,m$$

Total Wall span for floors 2-4 are the same, therefore $L_{wall\ 2} = L_{wall\ 3} = L_{wall\ 4}$

For Level 5-13:

Brick, Common			
Базовая стена: Exterior wall	Brick, Common	161.88 м³	L5

$$H_{Wall} = 3\ m$$
$$t_{Brick} = 0.38\ m$$
$$V_{Brick} = L_{Brick} * H_{Wall} * t_{Brick}$$
$$161.88\ m^3 = L_{Brick} * 3\ m * 0.38\ m$$
$$L_{Brick} = L_{wall} = 142\ m$$

4.2.6. Dead Load of structural element by category

Next step is to calculate dead load arising from the whole category, for example the exterior wall, on the floor. For that reason we will use previously found total span values of each category

Volume and weight is calculated as follows:

$$Volume = Thickness * Height * Span$$

$$Mass = Volume * Density$$

Applying these formulas to the existing information, our team found total weights of each member category.

$$Weight = \frac{Mass\ *\ 9.81}{1000}$$

Table 4.22. Interior wall in Public Space Floor 2

Interior wall in Public Space Floor 2								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Cement plaster	Cement Screed	5	6000	340130	10.2039	2,700.00	27550.53	270.2706993
Insulation	Fiberglass Batt	50	6000	340130	102.039	32.00	3265.248	32.03208288
Autoclaved aerated concrete block	Autoclaved aerated concrete	150	6000	340130	306.117	520.00	159180.84	1561.56404
Insulation	Fiberglass Batt	50	6000	340130	102.039	32	3265.248	32.03208288
Cement plaster	Cement Screed	5	6000	340130	10.2039	2,700.00	27550.53	270.2706993
	Total	260			530.6028		220812	2166.169605

Table 4.23. Interior wall in Public Space Floor 3

Interior wall in Public Space Floor 3								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Cement plaster	Cement Screed	5	4000	350160	7.0032	2,700.00	18908.64	185.4937584
Insulation	Fiberglass Batt	50	4000	350160	70.032	32.00	2241.024	21.98444544
Autoclaved aerated concrete block	Autoclaved aerated concrete	150	4000	350160	210.096	520.00	109249.92	1071.741715
Insulation	Fiberglass Batt	50	4000	350160	70.032	32	2241.024	21.98444544
Cement plaster	Cement Screed	5	4000	350160	7.0032	2,700.00	18908.64	185.4937584
	Total	260			364.1664		151549	1486.698123

Table 4.24. Interior wall in Public Space Floor 4

Interior wall in Public Space Floor 4								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Cement plaster	Cement Screed	5	4000	219960	4.3992	2,700.00	11877.84	116.5216104
Insulation	Fiberglass Batt	50	4000	219960	43.992	32.00	1407.744	13.80996864
Autoclaved aerated concrete block	Autoclaved aerated concrete	150	4000	219960	131.976	520.00	68627.52	673.2359712
Insulation	Fiberglass Batt	50	4000	219960	43.992	32	1407.744	13.80996864
Cement plaster	Cement Screed	5	4000	219960	4.3992	2,700.00	11877.84	116.5216104
	Total	260			228.7584		95199	933.8991293

Table 4.25. Interior wall between apartments Floor 5-13

Interior wall between apartments Floor 5-13								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Cement plaster	Cement Screed	5	3000	517350	7.76025	2,700.00	20952.675	205.5457418
Insulation	Fiberglass Batt	50	3000	517350	77.6025	32.00	2483.28	24.3609

								768
Autoclaved aerated concrete block	Autoclaved aerated concrete	150	3000	517350	232.807 5	520.00	121059. 9	1187.59 7619
Insulation	Fiberglass Batt	50	3000	517350	77.6025	32	2483.28	24.3609 768
Cement plaster	Cement Screed	5	3000	517350	7.76025	2,700.00	20952.6 75	205.545 7418
	Total	260			403.533		167932	1647.41 1056

Table 4.26. Exterior Wall in Public Space Floor 2

Exterior Wall in Public Space Floor 2								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Exterior finish	Glass	6	6000	213430	7.68348	2500	19208.7	188.4373 47
Thermal Membrane	Thermal Control Layer	12	6000	213430	0	0	0	0
Interior finish	Glass	6	6000	213430	7.68348	2500	19208.7	188.4373 47
	Total	24			15.36696		38417	376.8746 94

Table 4.27. Exterior Wall in Public Space Floors 3-4

Exterior Wall in Public Space Floors 3-4								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Exterior finish	Glass	6	4000	213430	5.12232	2500	12805.8	125.6248 98
Thermal Membrane	Thermal Control Layer	12	4000	213430	0	0	0	0
Interior finish	Glass	6	4000	213430	5.12232	2500	12805.8	125.6248 98
	Total	24			10.24464		25612	251.2497 96

Table 4.28. Exterior Wall in Apartment Floors 5-13

Exterior Wall in Apartment Floors 5-13

Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Mass (kg)	Weight (kN)
Exterior finish	Granite panel	15	3000	142000	6.39	2700	17253	169.25193
Air infiltration barrier	Air	0	3000	142000	0	0.00	0	0
	Metal stud layer	1.3716	3000	142000	0.5843016	7820	4569.238512	44.8242298
Insulation	Mineral wool	101.6	3000	142000	43.2816	50	2164.08	21.2296248
Air infiltration barrier	Vapour permeable membrane	0.45	3000	142000	0.1917	300.00	57.51	0.5641731
Autoclaved aerated concrete block	Brick	380	3000	142000	161.88	2300	372324	3652.49844
cement plaster	Cement Screed	5	3000	142000	2.13	2,100.00	4473	43.88013
	Total	503.4216			214.4576016		400841	3932.248528

Table 4.29. Parapet

Parapet								
Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)	
Exterior finish	Granite panel	15	762	142000	1.62306	2700	4382.262	42.98999022
Air infiltration barrier	Air	0	762	142000	0	0.00	0	0
	Metal stud layer	1.3716	762	142000	0.1484126064	7820	1160.586582	11.38535437
Insulation	Mineral wool	101.6	762	142000	10.9935264	50	549.67632	5.392324699
Air infiltration barrier	Vapour permeable membrane	0.45	762	142000	0.0486918	300.00	14.60754	0.1432999674
Autoclaved aerated concrete block	Brick	380	762	142000	41.11752	2300	94570.296	927.7346038
cement plaster	Cement Screed	5	762	142000	0.54102	2,100.00	1136.142	11.1455302
	Total	503.4216			54.4722 3081		101814	998.791126

Floor area

1-4	
Floor area	3059.290
5-13	
Floor area	1640.493

Dead load per area calculation

Floors 1-4:

$$DL\ from\ \ roof\ parapet\ per\ area = \frac{Total\ mass\ of\ parapet\ *\ g}{\mathbf{1000} * Floor\ Area}$$

$$= \frac{\mathbf{101814 * 9.81}}{\mathbf{1000 * 3059.290}} = \mathbf{0.326}\ kN/m^{\mathbf{2}}$$

$$DL\ from\ \ interior\ walls\ per\ area = \frac{Total\ mass\ of\ interior\ walls\ *\ g}{\mathbf{1000} * Floor\ Area}$$

$$= \frac{\mathbf{220812 * 9.81}}{\mathbf{1000 * 3059.290}} = \mathbf{0.708}\ kN/m^{\mathbf{2}}$$

$$DL\ from\ \ exterior\ walls\ per\ area = \frac{Total\ mass\ of\ exterior\ walls\ *\ g}{\mathbf{1000} * Floor\ Area}$$

$$= \frac{\mathbf{38417 * 9.81}}{\mathbf{1000 * 3059.290}} = \mathbf{0.123}\ kN/m^{\mathbf{2}}$$

$$DL\ from\ \ stairs\ per\ floor = \frac{Total\ mass\ of\ stairs\ *\ g}{\mathbf{1000}} = \frac{\mathbf{5654.12 * 9.81}}{\mathbf{1000}}$$

$$= \mathbf{55.467}\ kN$$

$$DL\ from\ \ stairs\ on\ major\ beams = \frac{Total\ mass\ of\ stairs\ *\ g}{\mathbf{1000 * 18}} = \frac{\mathbf{5654.12 * 9.81}}{\mathbf{1000 * 18}}$$

$$= \mathbf{3.081}\ kN/m^{\mathbf{2}}$$

$$Major\ beam\ self-weight = \rho * V = \rho * L * h * b = 2400 * 6 * 0.48 * 0.24$$

$$= 1658.88kg$$

$$Minor\ beam\ self-weight = \rho * V = \rho * L * h * b = 2400 * 6 * 0.4 * 0.2 = 1152kg$$

$$DL\ from\ \ beams\ on\ interior\ columns = \frac{(M_{major\ beam} * \mathbf{2} + *\ M_{minor\ beam} * \mathbf{4}) * g}{\mathbf{1000}}$$

$$= \frac{(1658.88 * 2 + 1152 * 4) * 9.81}{\mathbf{1000}}$$

$$DL\ from\ \ beams\ on\ interior\ columns = 77.752kN$$

$$DL\ from\ \ beams\ on\ exterior\ columns$$

$$= \frac{(\square_{\square\square\square\square\square\ \square\square\square\square} * \boldsymbol{1.5} + * \ \square_{\square\square\square\square\square\ \square\square\square\square} * \boldsymbol{2}) * \square}{\boldsymbol{1000}}$$

$$= \frac{(1658.88 * 1.5 + 1152 * 2) * 9.81}{\boldsymbol{1000}}$$

$$DL\ from\ \ beams\ on\ exterior\ columns = \boldsymbol{47.013} kN$$

$$DL\ from\ \ beams\ on\ corner\ columns = \frac{M_{major\ beam} * g}{\boldsymbol{1000}} = \frac{1658.88 * 9.81}{\boldsymbol{1000}}$$

$$DL\ from\ \ beams\ on\ corner\ columns = \boldsymbol{16.274} kN$$

Table 4.30. Dead loads for different structural elements 1

Per area	DL from roof parapets	0.326	kN/m2
On interior columns	DL from beams	77.752	kN
On exterior columns	DL from beams	47.013	kN
On corner columns	DL from beams	16.274	kN
On major beams	DL from stairs	3.081	kN/m2
Per area	DL from partition walls	0.708	kN/m2
Per area	DL from external walls	0.123	kN/m2
Per floor	DL from stairs	55.467	kN

Floors 5-13:

$$DL\ from\ \ roof\ parapet\ per\ area = \frac{Total\ mass\ of\ parapet\ * \ g}{\boldsymbol{1000} * Floor\ Area}$$

$$= \frac{\boldsymbol{101814 * 9.81}}{\boldsymbol{1000 * 1640.493}} = \boldsymbol{0.609}\ kN/m^{\boldsymbol{2}}$$

$$DL\ from\ \ interior\ walls\ per\ area = \frac{Total\ mass\ of\ interior\ walls\ * \ g}{\boldsymbol{1000} * Floor\ Area}$$

$$= \frac{\boldsymbol{167932 * 9.81}}{\boldsymbol{1000 * 1640.493}} = \boldsymbol{1.004}\ kN/m^{\boldsymbol{2}}$$

$$DL\ from\ \ exterior\ walls\ per\ area = \frac{Total\ mass\ of\ exterior\ walls\ * \ g}{\boldsymbol{1000} * Floor\ Area}$$

$$= \frac{\boldsymbol{400841 * 9.81}}{\boldsymbol{1000 * 1640.493}} = \boldsymbol{2.397}\ kN/m^{\boldsymbol{2}}$$

$$DL\ from\ \ stairs\ per\ floor = \frac{Total\ mass\ of\ stairs\ * \ g}{\boldsymbol{1000}} = \frac{\boldsymbol{5654.12 * 9.81}}{\boldsymbol{1000}}$$

$$= \boldsymbol{55.467}\ kN$$

$$DL\, from\ \ stairs\ on\, major\, beams = \frac{Total\, mass\, of\ stairs\ *\ g}{\boldsymbol{1000 * 18}} = \frac{\boldsymbol{5654.12 * 9.81}}{\boldsymbol{1000 * 18}}$$

$$= \boldsymbol{3.081}\ kN/m^{2}$$

$$Major\, beam\, self - weight = \rho * V = \rho * L * h * b = 2400 * 6 * 0.48 * 0.24$$

$$= 1658.88kg$$

$$Minor\, beam\, self - weight = \rho * V = \rho * L * h * b = 2400 * 6 * 0.4 * 0.2 = 1152kg$$

$$DL\, from\ \ beams\ on\ interior\ columns = \frac{(M_{major\ beam} * \boldsymbol{2} + *\ M_{minor\ beam} * \boldsymbol{4}) * g}{\boldsymbol{1000}}$$

$$= \frac{(1658.88 * 2 + 1152 * 4) * 9.81}{\boldsymbol{1000}}$$

$$DL\, from\ \ beams\ on\ interior\ columns = 77.7\boldsymbol{52}kN$$

$$DL\, from\ \ beams\ on\ exterior\ columns$$

$$= \frac{(\square_{\square\square\square\square\square\ \square\square\square\square} * \boldsymbol{1.5} + *\ \square_{\square\square\square\square\square\ \square\square\square\square} * \boldsymbol{2}) * \square}{\boldsymbol{1000}}$$

$$= \frac{(1658.88 * 1.5 + 1152 * 2) * 9.81}{\boldsymbol{1000}}$$

$$DL\, from\ \ beams\ on\ exterior\ columns = \boldsymbol{47.013}kN$$

$$DL\, from\ \ beams\ on\ corner\ columns = \frac{M_{major\ beam} * g}{\boldsymbol{1000}} = \frac{1658.88 * 9.81}{\boldsymbol{1000}}$$

$$DL\, from\ \ beams\ on\ corner\ columns = \boldsymbol{16.274}kN$$

Table 4.31. Dead loads for different structural elements 2

per area	**DL from roof parapets per area**	0.609	kN/m2
Interior columns	**DL from beams**	77.752	kN
Exterior columns	**DL from beams**	47.013	kN
Corner columns	**DL from beams**	16.274	kN
Major beams	**Dead load from stairs**	3.081	kN/m2
per area	**DL from partition walls per area**	1.004	kN/m2
per area	**DL from externall walls per area**	2.397	kN/m2
	DL from stairs per floor	55.467	kN

1.$DL\ for\ column\ exterior = \frac{DL_{floor}+DL_{exterior\ beams}}{\boldsymbol{18}} = \frac{\boldsymbol{5.426+47.013}}{\boldsymbol{18}} = \boldsymbol{8.038}kN/m^{2}$

2.$DL\ for\ column\ interior = \frac{DL_{floor}+DL_{interior\ beams}}{\boldsymbol{36}} = \frac{\boldsymbol{5.426+77.752}}{\boldsymbol{36}} = 7.\boldsymbol{586}kN/m^{2}$

$3.DL\ for\ column\ interior = \frac{DL_{floor}+DL_{corner\ beams}}{9} = \frac{5.426+16.274}{9} = \mathbf{7.234}kN/m^{\mathbf{2}}$

$3.DL\ for\ column\ (roof) = DL_{roof} = \mathbf{4.80592}kN/m^{\mathbf{2}}$

$3.DL\ for\ minor\ beam\ (roof) = DL_{roof} = \mathbf{4.80592}kN/m^{\mathbf{2}}$

$3.DL\ for\ major\ beam\ (roof) = DL_{roof} = \mathbf{4.80592}kN/m^{\mathbf{2}}$

$3.DL\ for\ minor\ beam = DL_{floor} + DL_{floor} = \mathbf{4.80592}kN/m^{\mathbf{2}}$

$3.DL\ for\ major\ beam = DL_{roof} = \mathbf{4.80592}kN/m^{\mathbf{2}}$

Table 4.32. Dead loads for different types of columns

Dead Load for Column exterior	8.038
Dead load for column interior	7.586
Dead load for column corner	7.234

From all calculations above results for column sizing are depicted in the table below.

Table 4.33. Column Sizing

Column size							
Dead ext (kN/m^2)	Live ext (kN/m^2)	At ext (m^2)	fc (MPa)	fy (MPa)	ϕ	Roof	Roof dead load
8.038	3.78	18	35	520	0.650	0.960	4.810
Dead int (kN/m2)	Live int (kN/m^2)	At int (m^2)					
7.586	3.022	36					
Dead cor (kN/m2)	Live cor(kN/m^2)	At cor (m^2)					
7.234	4.790	9					

For our columns we chose concrete Grade C35 with $f_c = 5076\,psi = 35\ Mpa$ and reinforcing steel rebar Grade 75 with $f_y(MPa) = 75\ ksi = 520\ MPa.$

1.

$$W_{u,n} = (13 - n) * (1.2 * DL_{exr} + 1.6 * LL_{ext}\ \ ) + 1.2 * Roof\ DL + 0.5 * Roof\ LL$$

2.

Since Floors 2-4 are different heights out of other floors, we need to consider them separately. For the first iteration we make assumption $a = b = 0.5m \Rightarrow A_g = 0.25$

$1)DL_2 = 1.2 * \rho_{concrete} * h_{floor} * g * \frac{A_g}{1000} = 1.2 * 2400 * 6 * 9.81 * \frac{0.203}{1000}$

$$2)DL_3 = 1.2 * \rho_{concrete} * h_{floor} * g * \frac{A_g}{1000} = 1.2 * 2400 * 4 * 9.81 * \frac{0.203}{1000}$$

$$3)DL_4 = 1.2 * \rho_{concrete} * h_{floor} * g * \frac{A_g}{1000} = 1.2 * 2400 * 4 * 9.81 * \frac{0.203}{1000}$$

Other floors are the same, so we could calculate them all together.

$$4)DL_{5-13} = 1.2 * \rho_{concrete} * h_{floor} * g * \frac{\Sigma Area}{1000} = 1.2 * 2400 * 3 * 9.81 * \frac{0.968}{1000}$$

$$5)DL_{walls\ and\ parapet} = 1.2 * DL_{interior\ wall} * A_{t,ext} * (13 - n) + 1.2 * DL_{roof\ parapet} * A_{t,ext} + +1.2 * DL_{exterior\ wall} * A_{t,ext} * (13 - n)$$

$$6)Force\ from\ load\ combination\ = W_{u,n} * A_{t,ext}$$

Finally equations 1)-7) are considered in P_u calculation.

$$P_u = DL_2 + DL_3 + DL_4 + DL_{5-13} + DL_{walls\ and\ parapet} + Force\ from\ load\ combination$$

3.

$$P_n = 0.8\varphi * (\beta * f_c{}'(A_g - A_{st}) + fA_{st})$$
$$A_{st} = 0.01 * A_g$$

4.

$$A_c = \frac{P_u}{P_n}$$

$$a = \sqrt{A_c}$$

Continuing these iterations in the same way demonstrated above the values of column dimensions are obtained for exterior, interior,and corner columns.

Table 4.34. Exterior column for 1-13

Exterior column for 1-13								
Level	Wu (kN/m^2)	Pu (kN)	Pn (kN)	Ac (m^2)	a (m)	a=b (m)	Ag	Sum of area
1	217.503	4299.665	18174.000	0.23658	0.486	0.500	0.250	1.575
2	178.831	3551.289	18174.000	0.19540	0.442	0.450	0.203	1.373
3	163.142	3228.046	18174.000	0.17762	0.421	0.450	0.203	1.170
4	147.453	3410.502	18174.000	0.18766	0.433	0.450	0.203	0.968
5	131.764	3037.469	18174.000	0.16713	0.409	0.450	0.203	0.765
6	116.075	2668.039	18174.000	0.14681	0.383	0.400	0.160	0.605
7	100.386	2298.608	18174.000	0.12648	0.356	0.400	0.160	0.445
8	84.697	1932.356	18174.000	0.10633	0.326	0.350	0.123	0.323
9	69.008	1568.859	18174.000	0.08632	0.294	0.300	0.090	0.233
10	53.319	1205.362	18174.000	0.06632	0.258	0.300	0.090	0.143

11	37.630	844.195	18174.000	0.04645	0.216	0.250	0.063	0.080
12	21.940	484.936	18174.000	0.02668	0.163	0.200	0.040	0.040
13	6.251	125.676	18174.000	0.00692	0.083	0.200	0.040	0.000

Table 4.35. Interior column for 1-13

Interior column 1-13								
Level	Wu (kN/m^2)	Pu (kN)	Pn (kN)	Ac (m^2)	a (m)	a=b (m)	Ag	Sum of area
1	207.455	8091.251	18174.000	0.44521	0.667	0.700	0.490	2.438
2	159.565	6275.627	18174.000	0.34531	0.588	0.600	0.360	2.078
3	145.628	5702.600	18174.000	0.31378	0.560	0.600	0.360	1.718
4	131.690	5251.216	18174.000	0.28894	0.538	0.550	0.303	1.415
5	117.752	4680.440	18174.000	0.25753	0.507	0.550	0.303	1.113
6	103.815	4114.113	18174.000	0.22637	0.476	0.500	0.250	0.863
7	89.877	3551.813	18174.000	0.19543	0.442	0.450	0.203	0.660
8	75.940	2989.513	18174.000	0.16449	0.406	0.450	0.203	0.458
9	62.002	2430.814	18174.000	0.13375	0.366	0.400	0.160	0.298
10	48.064	1875.295	18174.000	0.10319	0.321	0.350	0.123	0.175
11	34.127	1322.529	18174.000	0.07277	0.270	0.300	0.090	0.085
12	20.189	772.095	18174.000	0.04248	0.206	0.250	0.063	0.023
13.000	6.251	225.051	18174.000	0.01238	0.111	0.150	0.023	0.000

Table 4.36. Corner column 1-13

Corner column 1-13								
Level	Wu (kN/m^2)	Pu (kN)	Pn (kN)	Ac (m^2)	a (m)	a=b (m)	Ag	Sum of area
1	202.392	2025.309	18174.000	0.11144	0.334	0.350	0.123	0.888
2	186.047	1848.460	18174.000	0.10171	0.319	0.350	0.123	0.765
3	169.702	1678.534	18174.000	0.09236	0.304	0.350	0.123	0.643
4	153.357	1761.456	18174.000	0.09692	0.311	0.350	0.123	0.520
5	137.012	1569.990	18174.000	0.08639	0.294	0.300	0.090	0.430
6	120.667	1378.523	18174.000	0.07585	0.275	0.300	0.090	0.340
7	104.322	1187.057	18174.000	0.06532	0.256	0.300	0.090	0.250
8	87.976	997.921	18174.000	0.05491	0.234	0.250	0.063	0.188
9	71.631	808.786	18174.000	0.04450	0.211	0.250	0.063	0.125
10	55.286	621.557	18174.000	0.03420	0.185	0.200	0.040	0.085
11	38.941	434.329	18174.000	0.02390	0.155	0.200	0.040	0.045
12	22.596	248.584	18174.000	0.01368	0.117	0.150	0.023	0.023
13	6.251	62.838	18174.000	0.00346	0.059	0.150	0.023	0.000

Table 4.37. Final values of column size estimation

Exterior column for 1-13	Interior column 1-13	Corner column 1-13
a=b (m)	a=b (m)	a=b (m)
0.500	0.700	0.350
0.450	0.600	0.350
0.450	0.600	0.350
0.450	0.550	0.350
0.450	0.550	0.300
0.400	0.500	0.300
0.400	0.450	0.300
0.350	0.450	0.250
0.300	0.400	0.250
0.300	0.350	0.200
0.250	0.300	0.200
0.200	0.250	0.150
0.200	0.150	0.150

4.3. Assigning Loads to SAP 2000

Defining Frame elements:

Since column sizes on each floor are different, it is required to define them separately. While beams and slab sizes are the same from floor to floor and could be defined once.

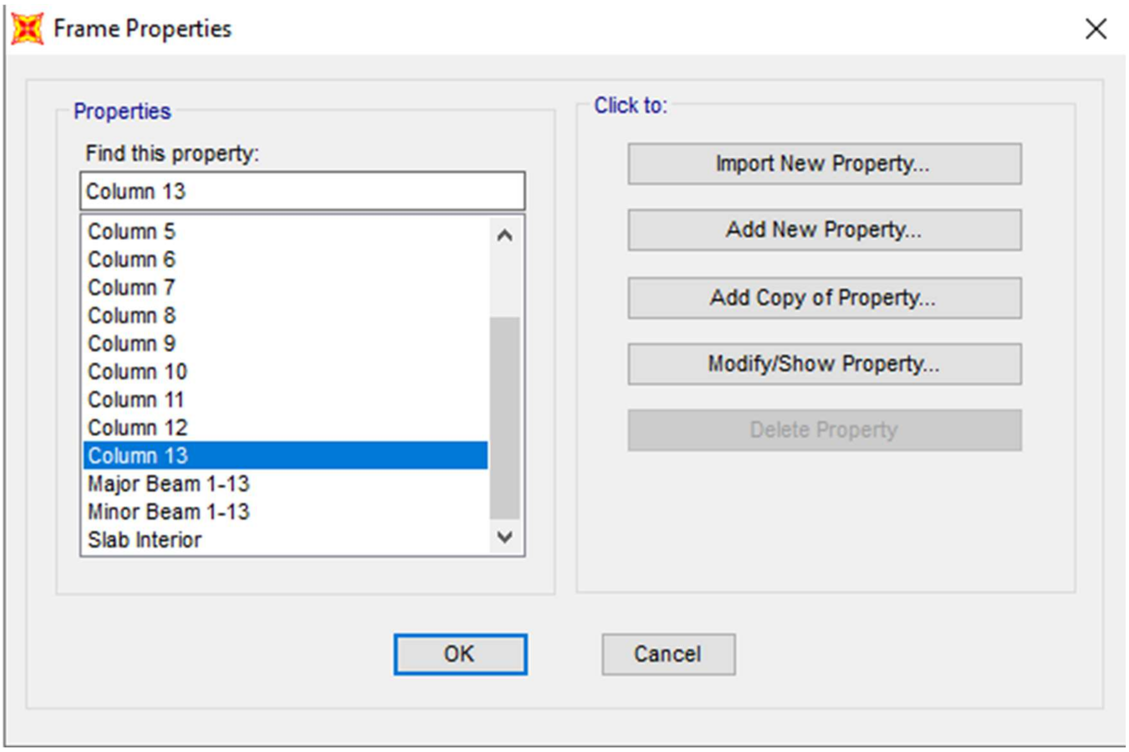


Figure 4.4. SAP2000 Defining Frame elements.

Column properties:

- **Material: Concrete** $E_c = 27805.575\ MPa, f_c' = 35MPa = 5076\ psi, f_y' = 520MPa$

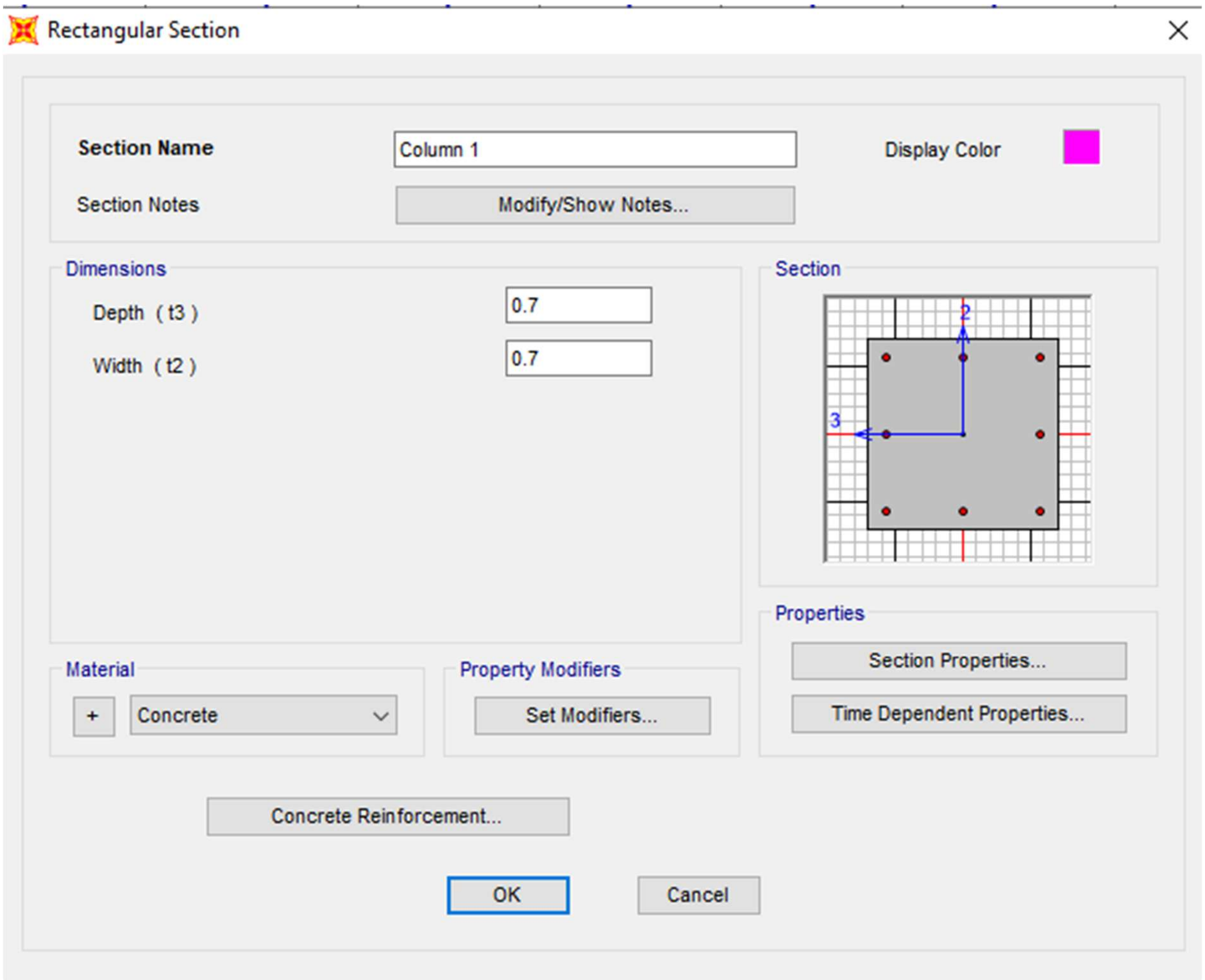


Figure 4.5. SAP2000 Column Properties.

Defining Loads:

- Dead Load
- Live Load
- Roof Live Load
- Wind Load
- Seismic Load
- Snow Load

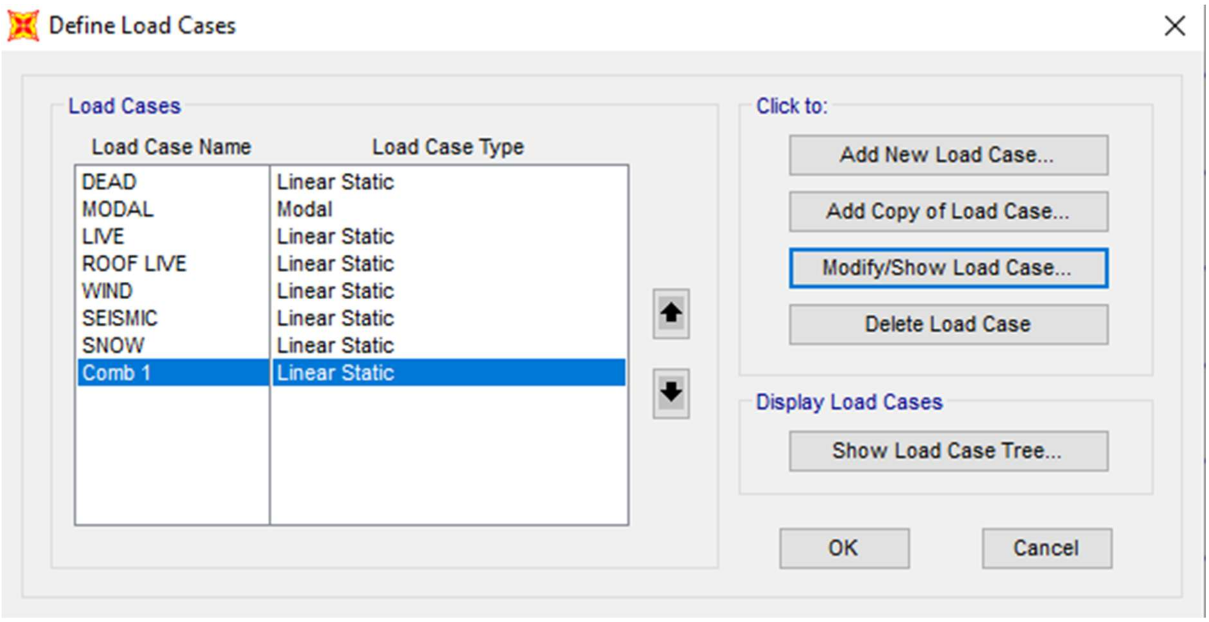


Figure 4.6. SAP2000 Defining Loads.

Load combination:

- Live load + Dead Load + Roof Live Load

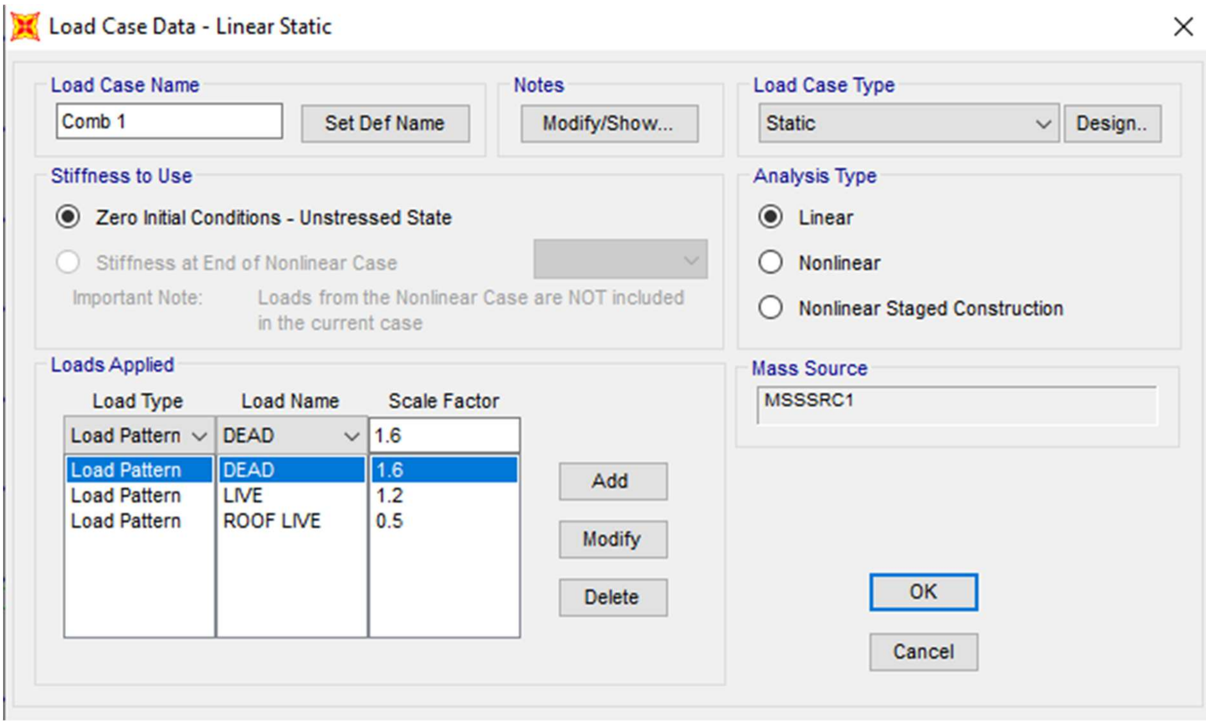


Figure 4.7. SAP2000 Defining Load Combinations.

1)Axial Force

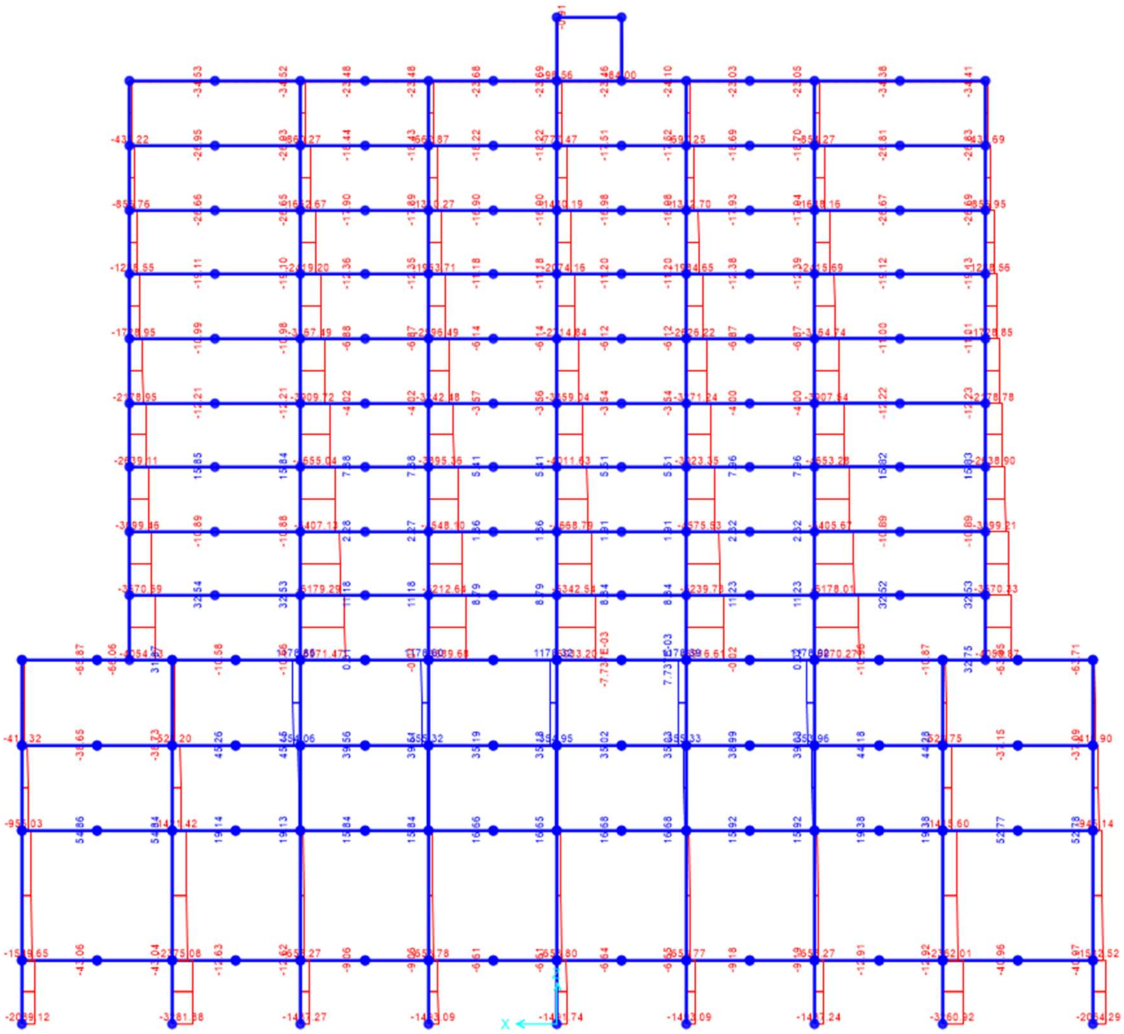


Figure 4.8. SAP2000 2D Dead+Live+Snow combo analysis (Axial Force).

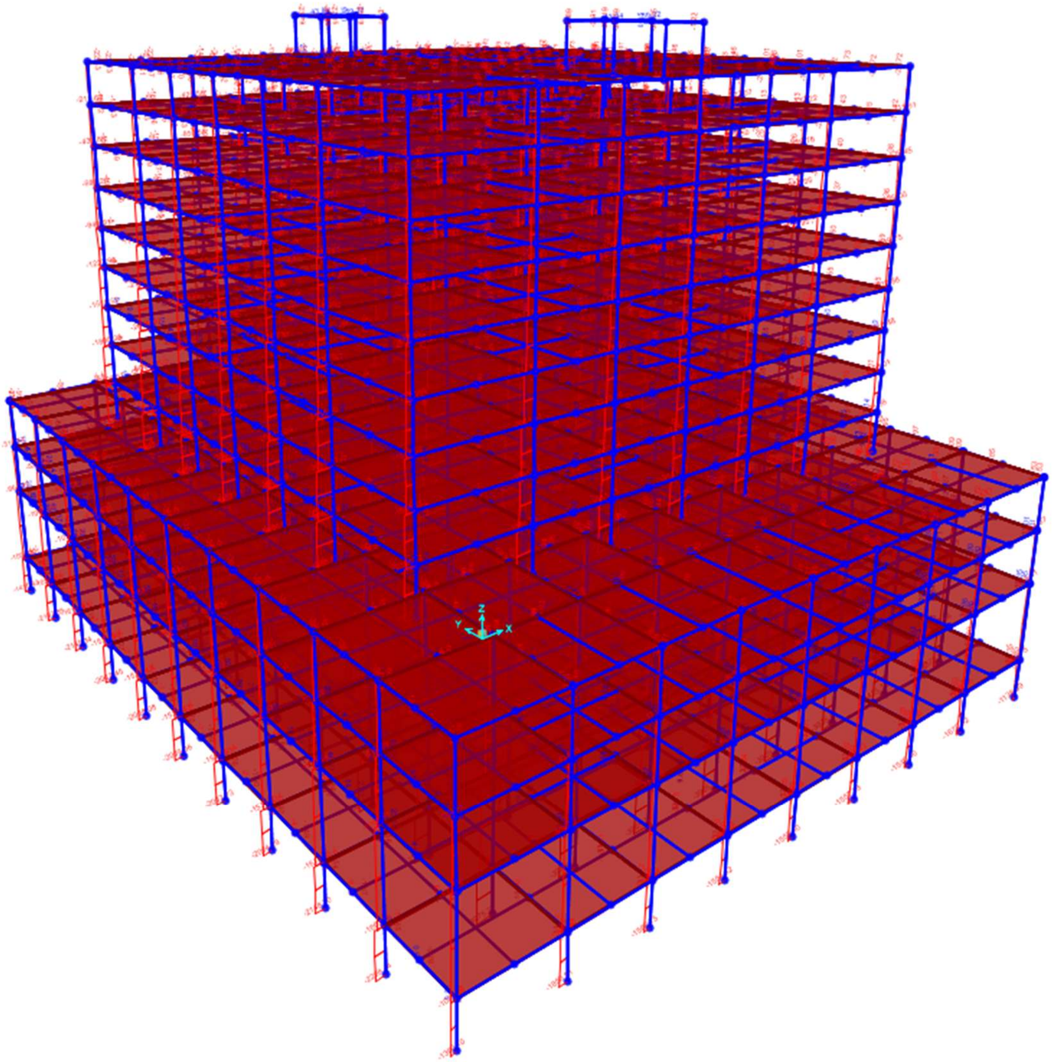


Figure 4.9. SAP2000 3D Dead+Live+Snow combo analysis (Axial Force).

2) Shear 2-2

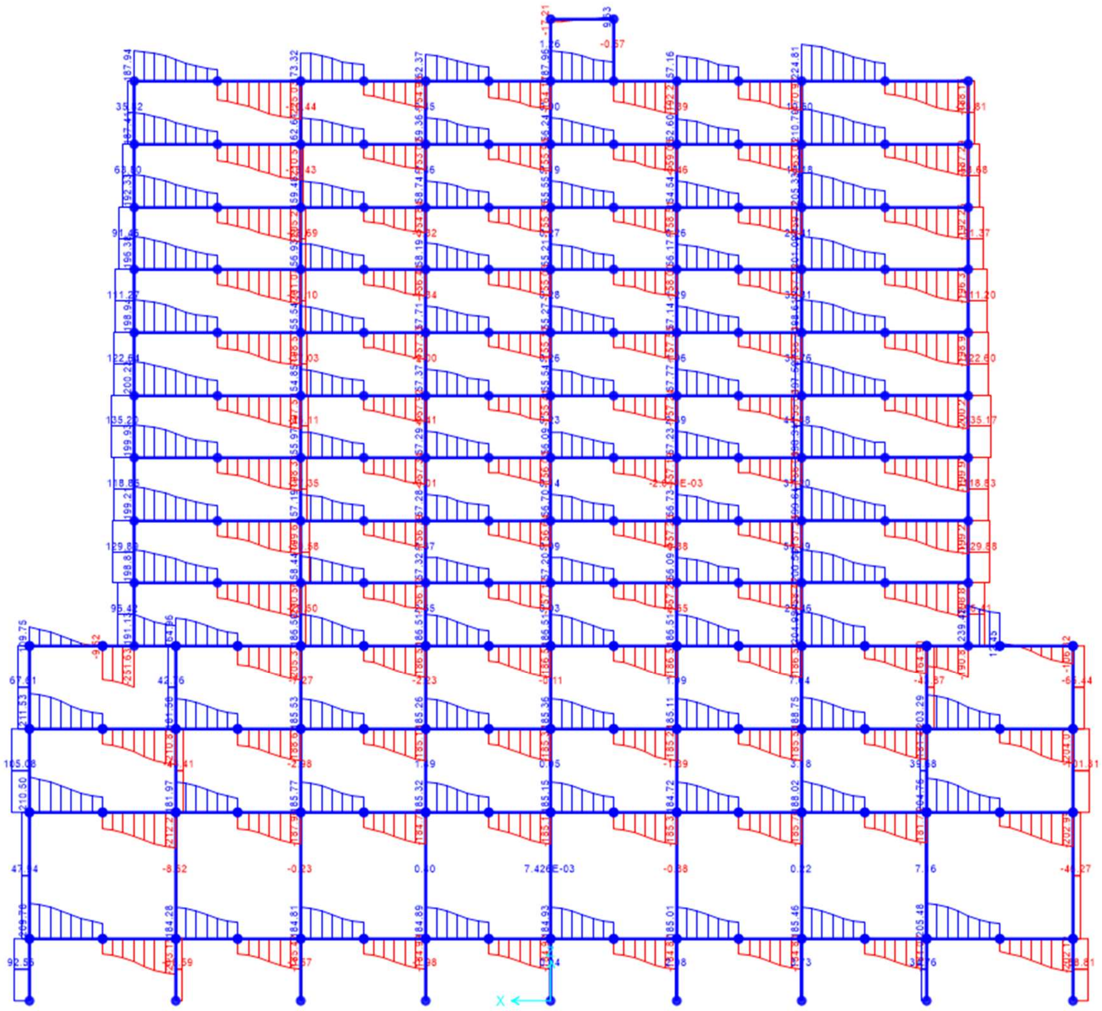


Figure 4.10. SAP2000 2D Dead+Live+Snow combo analysis (Shear 2-2).

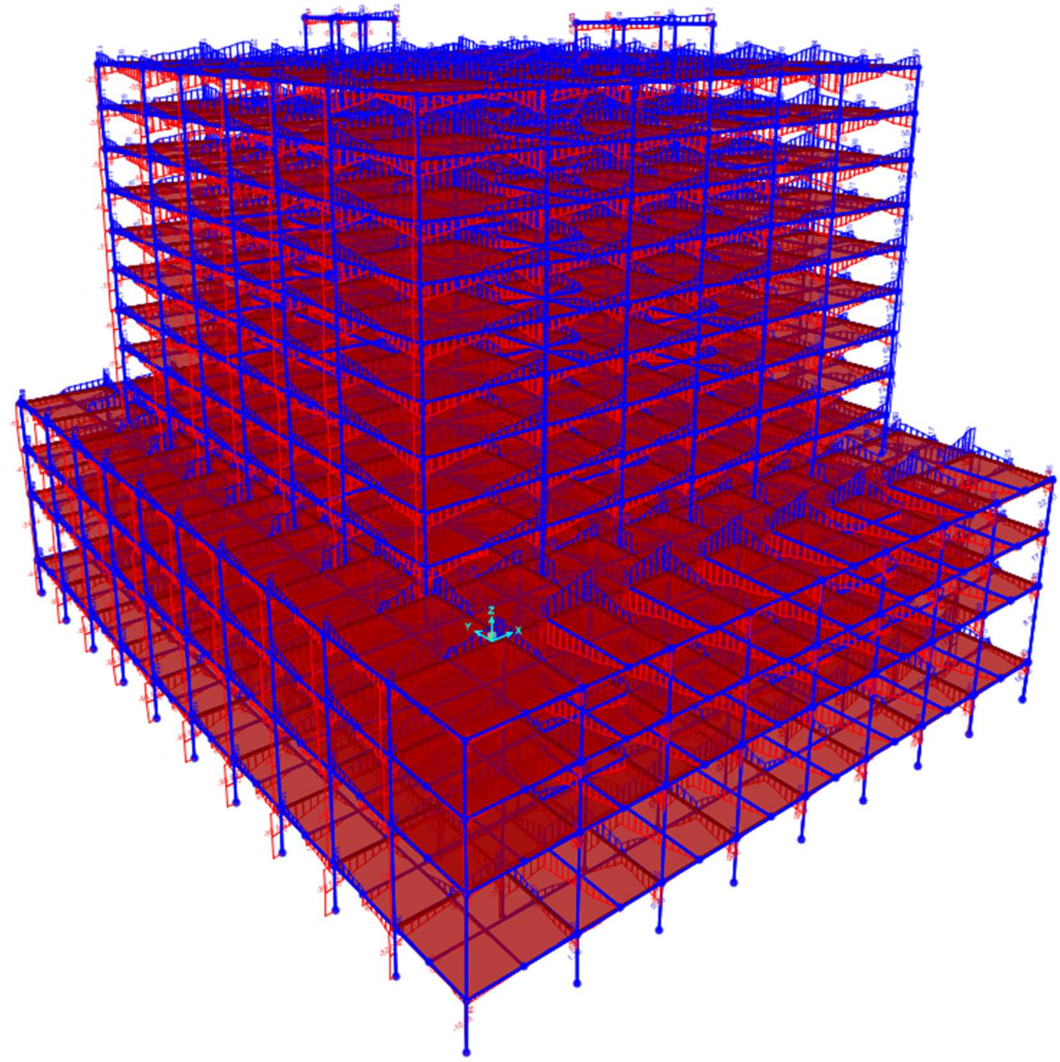


Figure 4.11. SAP2000 3D Dead+Live+Snow combo analysis (Shear 2-2).

3) Moment 3-3

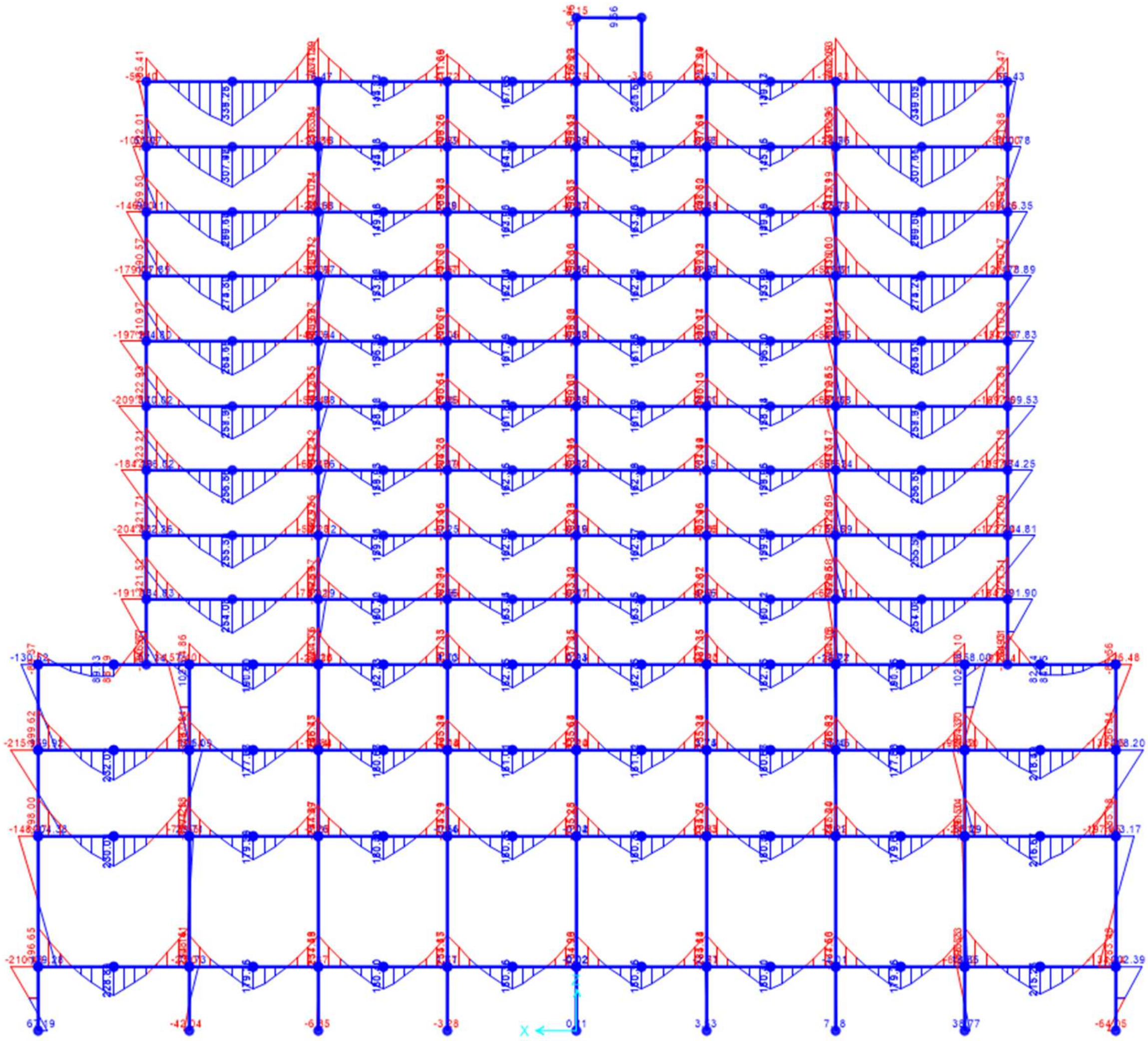


Figure 4.12. SAP2000 2D Dead+Live+Snow combo analysis (Moment 3-3).

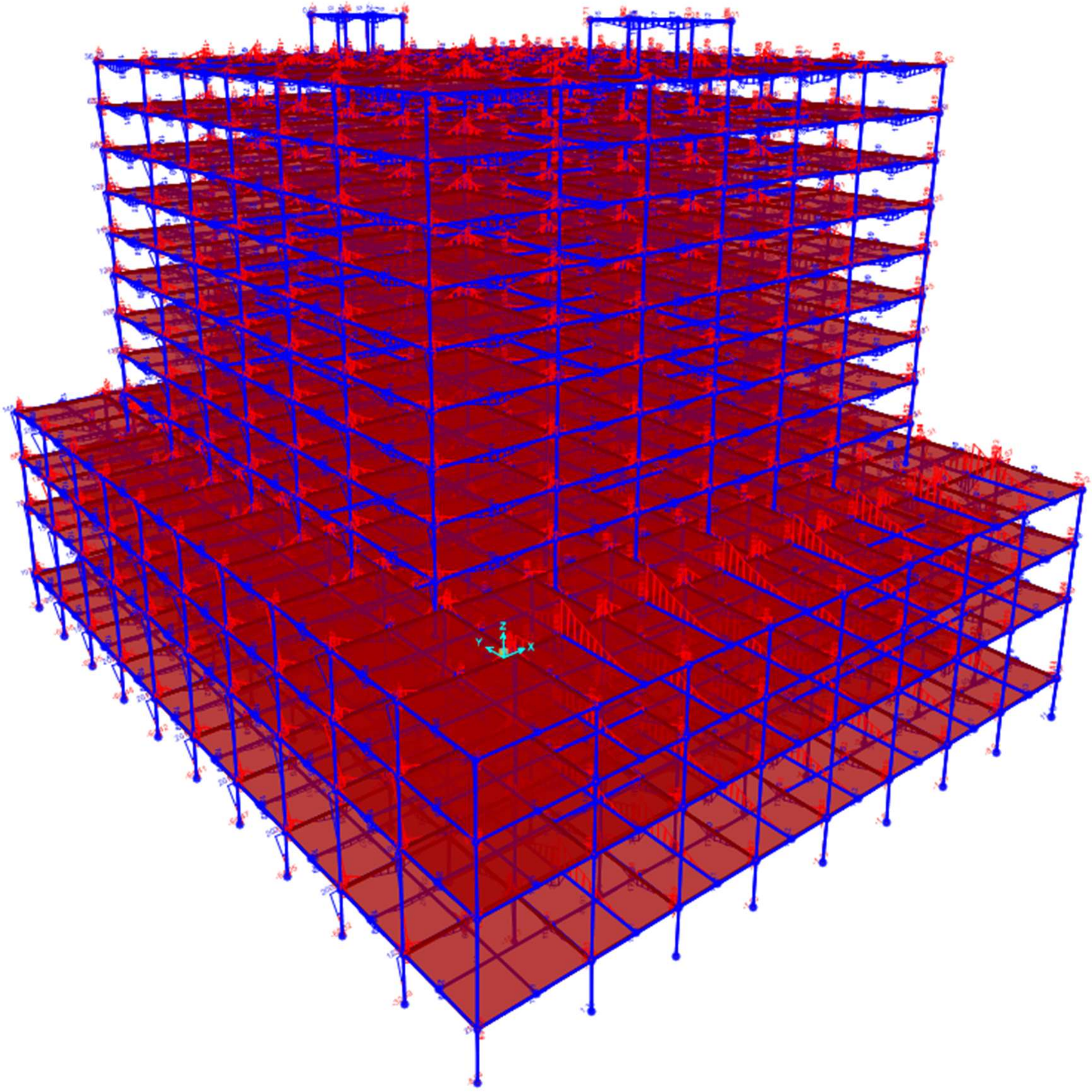


Figure 4.13. SAP2000 3D Dead+Live+Snow combo analysis (Moment 3-3).

4.4. Analysis and Design of Lateral Force Resisting System (LFRS)

4.4.1. Calculation of Seismic Loads Including Torsional Effect

Using the Hazard Tool application we are able to get response acceleration parameters for our location.

REPORT SUMMARY

Site Information

Address:	*211 Fairview Ave N, Seattle, Washington, 98109*
Elevation:	32 m (NAVD 88)
Lat:	47.620233
Long:	-122.3351
Standard:	ASCE/SEI 7-22
Risk Category:	II
Soil Class:	C - Very Dense Soil and Soft Rock

Seismic Data

S_S	1.54
S_1	0.65
S_{MS}	1.72
S_{M1}	0.89
S_{DS}	1.15
S_{D1}	0.59
T_L	6
PGA_M	0.67
V_{S30}	530
Seismic Design Category	D
Note	Where values of the multi-period 5%-damped MCER response spectrum are not available from the USGS Seismic Design Geodatabase, the design response spectrum shall be permitted to be determined in accordance with Section 11.4.5.2

Figure 4.14. Report Summary regarding seismic condition.

Table 4.38. Seismic zone initial conditions.

S_S	1.54
S_1	0.65
Soil Class	C
S_{MS}	1.72
S_{M1}	0.89
S_{DS}	1.147
S_{D1}	0.593
T_L	6
SDC	D

We should choose a seismic force-resisting system for our project. Since we have Seismic Design Category D, we are permitted to use Special reinforced concrete moment frames.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[h]	NP[h]	NP[h]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[i]	NP[i]	NP[i]
5. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP

Figure 4.15. Moment resisting frame systems and their properties.

Referring to the figure above, we have got the values for the response modification coefficient, overstrength factor, and deflection amplification factor. Permitted values are listed in the table below.

Table 4.39. Moment resisting frame systems values for Longitudinal and Transverse.

	Longitudinal	Transverse
LFRS	Special RC	Special RC
Ω_0	3	3
C_d	5.5	5.5
R	8	8
I_e	1	1

In order to find the fundamental period T of the structure we need to find coefficient C_u value.

$$S_{D1} = 0.593 \geq 0.4 \Rightarrow C_u = 1.4$$

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

Figure 4.16. Coefficient C_u values.

The fundamental period of a structure can be determined from the approximation of the fundamental period of a structure using T_a

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

Figure 4.17. Coefficient $C_t$and x values.

For our project we have the concrete moment resisting frame, therefore the values of the C_t and x can be found from the figure above.

$$Concrete\ moment - resisting\ frames \Rightarrow x = 0.9;\ C_t = 0.0466$$

The approximate fundamental period is found applying the formula below.

$$T_a = C_t * h_n^x$$

$$C_t - building\ period\ coefficient$$

$$h_n - height\ above\ the\ base\ to\ highest\ level\ of\ building$$

$$T_a = 0.00466 * 46.883^{0.9} = 1.487$$

Next step is to find original fundamental period via C_u coefficient and the approximate period values.

$$T = C_u * T_a$$

$$T = 1.4 * 1.487 = 2.082$$

The formula for determining the seismic response coefficient is:

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}} = \frac{1.147}{\frac{8}{1}} = 0.143$$

$S_{DS} -$
$the\ design\ spectral\ response\ acceleration\ parameter\ in\ the\ short\ period\ range$

$$R - the\ response\ modification\ factor$$

$$I_e - \ the\ importance\ factor$$

However, we need to satisfy Equations

1. The value of C_s should not exceed:

$$1)\ C_{s,max} = \frac{S_{D1}}{T * \frac{R}{I_e}} = \frac{0.593}{2.082 * \frac{8}{1}} = 0.0356\ for\ T \leq T_L$$

$$2)\ C_{s,max} = \frac{S_{D1} * T_L}{T^2 * \frac{R}{I_e}} = \frac{0.593 * 6}{2.082^2 * \frac{8}{1}} = 0.1026\ \ for\ T > T_L$$

Since $T \geq T_L$:

$$C_{s,max} = 0.0356$$

Moreover, C_s shall not be less than

1) $C_{s,min} = max[0.044 S_{DS} * I_e\ ;\ 0.01]\ for\ S_1 < 0.6g$

2) $C_{s,min} = \frac{0.5 * S_1}{\frac{R}{I_e}}\ for\ S_1 \geq 0.6g$

Since $S_1 \geq 0.6g$:

$$C_{s,min} = \frac{0.5 * S_1}{\frac{R}{I_e}} = \frac{0.5 * 0.65}{\frac{8}{1}} = 0.041$$

$$C_s = max[min[C_s, C_{s,max}]\ ;\ C_{s,min}] = 0.041$$

Where,

$\square_{\square 1}$

$- \ is\ the\ design\ spectral\ response\ acceleration\ parameter\ at\ period\ of\ 1.0\ s$

$$T - \ is\ the\ fundamental\ period\ of\ the\ structure$$

$$T_L - \ is\ the\ long\ period\ transition\ period$$

$\square_1$

$- is\ the\ mapped\ maximum\ considered\ earthquake\ spectral\ response\ acceleration\ paramete$

Next step is to find the total weight of all floors for base shear calculations. For that reason we should find the number of frame elements on each floor. Likely, we have the same floor plans for floors from fifth to thirteenth and also the same plans for floors from second to the fourteenth floor. Table below outlines the quantity of columns and beams for different types.

Table 4.40. Number of frame elements on each floor between 5-13

Number of frame elements on each floor between 5-13					
	Tributary area	# of columns		Tributary area	# of beams
Interior column	36	25	**Major beam**	18	84
Exterior column	18	20	**Minor beam**	9	72
Corner column	9	4			

Table 4.41. Number of frame elements on each floor between 1-4

Number of frame elements on each floor between 1-4					
	Tributary area	# of columns		Tributary area	# of beams

Interior column	36	63	**Major beam**	18	178
Exterior column	18	32	**Minor beam**	9	160
Corner column	9	4			

Firstly, column weight is analysed. Interior columns are taken into consideration as they apply the bigger force. Column areas are taken from the members sizing section.

Table 4.42. Interior columns

Interior column					
Stories	Area of column	Volume (m3)	Density (kg/m3)	Self-weight	Weight (kN)
1	0.490	1.4700	2400	3528.00	34.61
2	0.360	2.1600	2400	5184.00	50.86
3	0.360	1.44	2400	3456	33.90
4	0.303	1.21	2400	2904	28.49
5	0.303	0.9075	2400	2178	21.37
6	0.250	0.75	2400	1800	17.66
7	0.203	0.6075	2400	1458	14.30
8	0.203	0.6075	2400	1458	14.30
9	0.160	0.48	2400	1152	11.30
10	0.123	0.3675	2400	882	8.65
11	0.090	0.27	2400	648	6.36
12	0.063	0.1875	2400	450	4.41
13	0.023	0.0675	2400	162	1.59

After calculating the weight of columns on each floor, we need to take into account other frame elements which apply loads on slabs such as major and minor beams.

1. Total column weight $Total\ column\ weight\ = weight\ of\ column *$ $(\#interior\ col + \#exterior\ col + \#corner\ col)$

For Floor 1-4:

$$Total\ column\ weight\ 1 = 34.61 * (63 + 32 + 4) = 3426.29kN$$

$Total\ column\ weight\ 2 = 50.86 * (63 + 32 + 4) = 5035.14kN$ and so on.

For Floor 5-13:

$$Total\ column\ weight\ 5 = 21.37 * (25 + 20 + 4) = 1047.13kN$$

$Total\ column\ weight\ 6 = 17.66 * (25 + 20 + 4) = 865.34kN$ and so on.

2. Volume of beams

Table 4.43. Member sizing

Member size	h	b	L
Major beam	0.48	0.24	6
Minor beam	0.4	0.2	6
E	27805.575	MPa	

$$V_{major} = h * b * L = 0.48 * 0.24 * 6 = 0.6912m^3$$

$$V_{minor} = h * b * L = 0.4 * 0.2 * 6 = 0.48m^3$$

3. Beam weight

For Floor 1-4:

$$W_{minor} = \rho * V_{minor} * \frac{g}{1000} * \#minor\ beams = 2400 * 0.48 * \frac{9.81}{1000} * 160 = 1808.2$$

$$W_{major} = \rho * V_{major} * \frac{g}{1000} * \#major\ beams = 2400 * 0.6912 * \frac{9.81}{1000} * 178 = 2896.7$$

$$W_{major+mino} = W_{minor} + W_{major} = 1808.2 + 2896.7 = 4704.9kN$$

For Floor 5-13:

$$W_{minor} = \rho * V_{minor} * \frac{g}{1000} * \#minor\ beams = 2400 * 0.48 * \frac{9.81}{1000} * 72 = 813.68$$

$$W_{major} = \rho * V_{major} * \frac{g}{1000} * \#major\ beams = 2400 * 0.6912 * \frac{9.81}{1000} * 84 = 1366.98$$

$$W_{major+mino} = W_{minor} + W_{major} = 813.68 + 1366.98 = 2180.66kN$$

Table 4.44. Total beam weight for every floor.

Floor	Weight of column (kN)	Total column weight (kN)	Vol of major beams (m3)	Vol of minor beams (m3)	Density of concrete	Beam weight (kN)

1	34.61	3426.39	0.6912	0.48	2400	4704.88
2	50.86	5035.14	0.6912	0.48	2400	4704.88
3	33.9	3356.10	0.6912	0.48	2400	4704.88
4	28.49	2820.51	0.6912	0.48	2400	4704.88
5	21.37	1047.13	0.6912	0.48	2400	2180.66
6	17.66	865.34	0.6912	0.48	2400	2180.66
7	14.3	700.70	0.6912	0.48	2400	2180.66
8	14.3	700.70	0.6912	0.48	2400	2180.66
9	11.3	553.70	0.6912	0.48	2400	2180.66
10	8.65	423.85	0.6912	0.48	2400	2180.66
11	6.36	311.64	0.6912	0.48	2400	2180.66
12	4.41	216.09	0.6912	0.48	2400	2180.66
13	1.59	77.91	0.6912	0.48	2400	2180.66
Roof	1.59	12.72	0.6912	0.48	2400	175.39

In order to find the total weight of all members on slab all members listed are taken into account.

- Floor DL
- Columns weight
- Beams weight
- Walls DL
- Stairs DL

Table 4.45. Total floor weight.

Floors	Floor finishing and slab	Column weight (kN)	Beam weight (kN)	Exterior/partition walls, kN/m2)	Stairs (kN)	Floor weight (kN)
1	5.426	3426.39	4704.88	0.831	111	27384.2
2	5.426	5035.14	4704.88	0.831	111	28993.0
3	5.426	3356.10	4704.88	0.831	111	27314.0
4	5.426	2820.51	4704.88	1.157	111	27775.7
5	5.426	1047.13	2180.66	3.401	111	17819.4
6	5.426	865.34	2180.66	3.401	111	17637.6
7	5.426	700.70	2180.66	3.401	111	17473.0
8	5.426	700.70	2180.66	3.401	111	17473.0
9	5.426	553.70	2180.66	3.401	111	17326.0
10	5.426	423.85	2180.66	3.401	111	17196.1
11	5.426	311.64	2180.66	3.401	111	17083.9
12	5.426	216.09	2180.66	3.401	111	16988.4
13	5.426	77.91	2180.66	3.401	111	16850.2
Roof	5.426	12.72	175.39	0.609	0	10088.5
Total						277403

Now the seismic design base shear can be calculated using the following formula:

$$V = C_s * W = 277403 * 0.041 = 11270kN$$

$$V - base\ shear$$

$$C_s - seismic\ response\ coefficient$$

$$W - effective\ seismic\ weight$$

Story Force

$$F_x = C_{vx} * V$$

$$C_{vx} - vertical\ distribution\ factor$$

$$V - total\ design\ lateral\ force\ or\ shear\ at\ base\ of\ structure$$

$$C_{vx} = \frac{W_x * h_x^k}{\sum_{i=1}^{n} \quad W_i * h_i^k}$$

$W_x\ and\ W_i$

$- the\ portion\ of\ the\ total\ seismic\ weight\ of\ structure\ assigned\ to\ Level\ i\ or\ x$

$$h_x\ and\ h_i - the\ height\ from\ base\ to\ Level\ i\ or\ x$$

$$k - an\ exponent\ related\ to\ structure\ period$$

$$For\ period\ of\ 0.5s\ or\ less\ k\ = 1$$

$$For\ period\ of\ 2.5s\ or\ more\ k\ = 2$$

$$0.5s < T = 2.082 < 2.5s \Rightarrow \frac{k-1}{2-1} = \frac{2.018-0.5}{2.5-0.5} \Rightarrow k = 1.759$$

Table 4.46. Calculating seismic load.

					Longitudinal (NS)			Traverse (NS)		
Floors	**hi (m)**	W_x **(kN)**	$W_x * h_x^k$	C_{vx}	F_x**(kN)**	**Fdirect (kN)**	**T (kN- m)**	**Fx (kN)**	**Fdirect (kN)**	**T (kN-m)**
13	46.883	26938.7	8433464	0.221466	2495.82	356.55	5319.51	2495.82	356.55	5054.40
12	43.647	16988.4	4779611	0.125515	1414.49	202.07	3014.80	1414.49	202.07	2864.55
11	40.411	17083.9	4284146	0.112504	1267.86	181.12	2702.28	1267.86	181.12	2567.60
10	37.175	17196.1	3806895	0.099971	1126.62	160.95	2401.25	1126.62	160.95	2281.57
9	33.939	17326.0	3347865	0.087916	990.77	141.54	2111.71	990.77	141.54	2006.46
8	30.703	17473.0	2906989	0.076339	860.30	122.90	1833.62	860.30	122.90	1742.24
7	27.467	17473.0	2461526	0.064641	728.47	104.07	1552.64	728.47	104.07	1475.26
6	24.231	17637.6	2060499	0.054110	609.79	87.11	1299.69	609.79	87.11	1234.91
5	20.995	17819.4	1680538	0.044132	497.34	71.05	1060.02	497.34	71.05	1007.19
4	17.759	27775.7	2040082	0.053573	603.75	86.25	1286.81	603.75	86.25	1222.68
3	13.506	27314.0	1332922	0.035003	394.47	56.35	840.76	394.47	56.35	798.86
2	9.253	28993.0	804299	0.021121	238.03	34.00	507.32	238.03	34.00	482.04
1	3	27384.2	141277	0.003710	41.81	5.97	89.11	41.81	5.97	84.67
Total		250018.9	38080113							

Table 4.47. Torsinal Effect consideration

TORSIONAL EFFECT CONSIDERATION		
Mass center		
x	20.252	m
y	20.36	m
Stiffness center		
x'	20.252	m
y'	20.252	m
Eccentricity		
ex'	0	m
ey'	0.11	m
e_ax	2.02515	m
e_ay	2.02515	m
ex	2.02515	m
ey	2.13	m

Table 4.48. Elevators and stairs consideration.

	x	y	area	x dist	y dist
Elevator shaft	5.35	2.562	27.41	20.252	12.879
Stair #1 shaft	7.464	2	29.856	4.096	21.389
Stair #2 shaft	0	0	0	0	0
Total area of slabs	1640.493009	m2		60	
Area of elevator and stairs	57.27	m2			

Table 4.49. Frame stiffness calculation.

FRAME STIFFNESS CALCULATION per frame								
							Transverse	**Longitudinal**
Floor	**Column size**	**Ic,cr, m4**	**Ib,cr, m4**	**Minor b, Ib,cr**	**D, kN/m**	**h, m**	**Cf, kN/rad**	**Cf, kN/rad**
1	0.7	1.40E-02	7.74E-04	0.0003733333333	1133	6	122388	149586
2	0.6	7.56E-03	7.74E-04	0.0003733333333	2519	4	181350	221649
3	0.6	7.56E-03	7.74E-04	0.0003733333333	2519	4	181350	221649
4	0.55	5.34E-03	7.74E-04	0.0003733333333	4460	3	80281	80281
5	0.55	5.34E-03	7.74E-	0.000373	4460	3	80281	80281

			04	3333333				
6	0.5	3.65E-03	7.74E-04	0.0003733333333	4324	3	77838	77838
7	0.45	2.39E-03	7.74E-04	0.0003733333333	4117	3	74110	74110
8	0.45	2.39E-03	7.74E-04	0.0003733333333	4117	3	74110	74110
9	0.4	1.49E-03	7.74E-04	0.0003733333333	3799	3	68378	68378
10	0.35	8.75E-04	7.74E-04	0.0003733333333	3317	3	59703	59703
11	0.3	4.73E-04	7.74E-04	0.0003733333333	2629	3	47330	47330
12	0.25	2.28E-04	7.74E-04	0.0003733333333	1773	3	31905	31905
13	0.15	2.95E-05	7.74E-04	0.0003733333333	339	3	6103	6103

<u>Seismic load calculations were done for all floors, however, we include here only the first, typical and last floors. The rest of them are available in the Appendix!</u>

First Floor

Table 4.50. First floor seismic load Transverse.

	Transverse								
Floors	**Frame**	**yi, m**	**yi^2, m2**	**Cf, kN/rad**	**yi^2*Cf**	**T, kN-m**	**Ftorsion, kN**	**Fdirect, kN**	**Ftotal, kN**
1	A	-25	625	122388	76492717	84.67	-0.5227	5.97	5.4502
	B	-18	324	122388	39653824	84.67	-0.3763	5.97	5.5965
	C	-12	144	122388	17623922	84.67	-0.2509	5.97	5.7220
	D	-6	36	122388	4405980	84.67	-0.1254	5.97	5.8474
	E	0	0	122388	0	84.67	0.0000	5.97	5.9728
	F	6	36	122388	4405980	84.67	0.1254	5.97	6.0983

	G	12	144	122388	17623922	84.67	0.2509	5.97	6.2237
	H	18	324	122388	39653824	84.67	0.3763	5.97	6.3492
	I	25	625	122388	76492717	84.67	0.5227	5.97	6.4955
				Total	123367454				

Table 4.51. First floor seismic load Longitudinal.

Longitudinal								
Frame	**xi, m**	**xi^2, m2**	**Cf. kN/rad**	**xi^2*Cf**	**T, kN-m**	**Ftorsion, kN**	**Fdirect, kN**	**Ftotal, kN**
1	-30.08	904.8064	149586	135346151	89.11	-0.8090	5.97	5.16
2	-24	576	149586	86161396	89.11	-0.6454	5.97	5.33
3	-18	324	149586	48465785	89.11	-0.4841	5.97	5.49
4	-12	144	149586	21540349	89.11	-0.3227	5.97	5.65
5	-6	36	149586	5385087	89.11	-0.1614	5.97	5.81
6	0	0	149586	0	89.11	0.0000	5.97	5.97
7	6	36	149586	5385087	89.11	0.1614	5.97	6.13
8	12	144	149586	21540349	89.11	0.3227	5.97	6.30
9	18	324	149586	48465785	89.11	0.4841	5.97	6.46
10	24	576	149586	86161396	89.11	0.6454	5.97	6.62
11	30.08	904.8064	149586	135346151	89.11	0.8090	5.97	6.78
			Total	372289990				

Fifth Floor

Table 4.52. Fifth floor seismic load Transverse.

	Transverse								
Floors	**Frame**	**yi, m**	**yi^2, m2**	**Cf, kN/rad**	**yi^2*Cf**	**T, kN-m**	**Ftorsion, kN**	**Fdirect, kN**	**Ftotal, kN**
5	A	-20.252	410.143504	80281	32926558	1007.19	-8.6410	71.05	62.4079
	B	-12	144	80281	11560403	1007.19	-5.1201	71.05	65.9288
	C	-6	36	80281	2890101	1007.19	-2.5600	71.05	68.4888
	D	0	0	80281	0	1007.19	0.0000	71.05	71.0488
	E	6	36	80281	2890101	1007.19	2.5600	71.05	73.6089
	F	12	144	80281	11560403	1007.19	5.1201	71.05	76.1689
	G	20.252	410.143504	80281	32926558	1007.19	8.6410	71.05	79.6898
				Total	94754124				

Table 4.53. Fifth floor seismic load Longitudinal.

Longitudinal								
Frame	**xi, m**	**xi^2, m2**	**Cf. kN/rad**	**xi^2*Cf**	**T, kN-m**	**Ftorsion, kN**	**Fdirect, kN**	**Ftotal, kN**
1	-20.252	410.143504	80281	32926558	1060.02	-9.0942	71.05	61.95
2	-12	144	80281	11560403	1060.02	-5.3886	71.05	65.66
3	-6	36	80281	2890101	1060.02	-2.6943	71.05	68.35
4	0	0	80281	0	1060.02	0.0000	71.05	71.05

5	6	36	80281	2890101	1060.02	2.6943	71.05	73.74
6	12	144	80281	11560403	1060.02	5.3886	71.05	76.44
7	20.252	410.143504	80281	32926558	1060.02	9.0942	71.05	80.14
			Total	94754124				

Thirteenth Floor

Table 4.54. Thirteenth floor seismic load Transverse.

	Transverse								
Floors	**Frame**	**yi, m**	**yi^2, m2**	**Cf, kN/rad**	**yi^2*Cf**	**T, kN-m**	**Ftorsion, kN**	**Fdirect, kN**	**Ftotal, kN**
13	A	-20.252	410.143504	6103	2503275	5054.40	-43.3631	356.55	313.1820
	B	-12	144	6103	878892	5054.40	-0.7469	356.55	355.7982
	C	-6	36	6103	219723	5054.40	-0.3734	356.55	356.1716
	D	0	0	6103	0	5054.40	0.0000	356.55	356.5451
	E	6	36	6103	219723	5054.40	0.3734	356.55	356.9185
	F	12	144	6103	878892	5054.40	0.7469	356.55	357.2919
	G	20.252	410.143504	6103	2503275	5054.40	1.2605	356.55	357.8055
				Total	7203780				

Table 4.55. Thirteenth floor seismic load Longitudinal.

Longitudinal								
Frame	**xi, m**	**xi^2, m2**	**Cf. kN/rad**	**xi^2*Cf**	**T, kN-m**	**Ftorsion, kN**	**Fdirect, kN**	**Ftotal, kN**

1	-20.252	410.1435 04	6103	2503275	5319.51	-45.6375	356.55	310.91
2	-12	144	6103	878892	5319.51	-27.0418	356.55	329.50
3	-6	36	6103	219723	5319.51	-13.5209	356.55	343.02
4	0	0	6103	0	5319.51	0.0000	356.55	356.55
5	6	36	6103	219723	5319.51	13.5209	356.55	370.07
6	12	144	6103	878892	5319.51	27.0418	356.55	383.59
7	20.252	410.1435 04	6103	2503275	5319.51	45.6375	356.55	402.18
			Total	7203780				

4.4.2. Calculation of Wind Loads Including Torsional Effect

According to ASCE 7-16, basic wind speed in the Seattle was accounted for 49 m/s. Moreover, our exposure category is B, since our building will be located in the downtown area of the city. Topographical effect Kzt= 1.0, since the land wa flat and direction effect Kd=0.85, since main wind load force resisting system.

Table 4.56. Constants for exposure categories.

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

We used $n_a = 43.5/h^{0.9}$ in order to find the type of our building for the further wind load calculations. Hence, approximate lower bound natural frequency, na (ft) was 0.5280 which was lower 1 Hz that means we have flexible building. Then using $K_z = 2.01 * (\frac{z}{z_g})^{2/\alpha}(z\ in\ the\ between\ 15ft\ and\ zg)$ we defined elocity pressure exposure coefficient.

Table 4.57. Height effect and Kz values.

Story	h (m)	z (m)	Kz
13	3	44	1.0975
12	3	41	1.0756
11	3	38	1.0525
10	3	35	1.0281
9	3	32	1.0021
8	3	29	0.9743
7	3	26	0.9443
6	3	23	0.9118
5	3	20	0.8761
4	3	17	0.8364
3	4	14	0.7913
2	4	10	0.7187
1	6	6	0.6211

For Gust effect, we considered 2 different cases with 2 possible wind directions : transverse and longitudinal.

Case 1 (1-4 floors): B= 50m and L= 60m for longitudinal, number of frames=11; B= 60m and L= 50m for transverse, number of frames=9.

Case 2 (5-13 floors): B= 40 m and L= 40 m for longitudinal, transverse (square shape), number of frames=7.

We use the following equations for Gust effect. For example, Case 1 (Transverse):

$$\underline{Z} = max(0.6 * h; 9.14) = 26.4\ m$$

$I_z = c(\frac{10}{\underline{Z}})^{1/6}$=$0.3 * (\frac{10}{26.4})^{1/6}$=0.2552

$V_z = \underline{b} * (\frac{Z}{10})^a v = 0.45 * (\frac{26.4}{10})^{0.25} * 49 = 28.107\ m/s$

$$L_z = l * (\frac{Z}{10})^{\epsilon} = 97.54 * (\frac{26.4}{10})^{1/3} = 134.80\ m$$

$N_1 = \frac{n*L_z}{V_z} = \frac{0.528*134.8}{28.107} = 2.532$

$$R_n = \frac{7.47 * N_1}{(1 + 10.3 * N_1)^{5/3}} = \frac{7.47 * 2.532}{(1 + 10.3 * 2.532)^{5/3}} = 0.0774$$

$$\eta_h = \frac{4.6 * n_a * H}{V_z} = \frac{4.6 * 0.528 * 44}{28.107} = 3.802$$

$$\eta_b = \frac{4.6 * n_a * B}{V_z} = \frac{4.6 * 0.528 * 60}{28.107} = 5.185$$

$$\eta_L = \frac{15.4 * n_a * L}{V_z} = \frac{15.4 * 0.528 * 50}{28.107} = 14.464$$

$R_h = \frac{1}{\eta_h} - \frac{1}{2*\eta_h^2} * (1 - e^{-2\eta_h}) = \frac{1}{3.802} - \frac{1}{2*3.802^2} * (1 - e^{-2*3.802}) = 0.228$

$R_B = \frac{1}{\eta_h} - \frac{1}{2*\eta_B^2} * (1 - e^{-2\eta_B}) = \frac{1}{5.185} - \frac{1}{2*5.185^2} * (1 - e^{-2*5.185}) = 0.174$

$R_h = \frac{1}{\eta_L} - \frac{1}{2*\eta_L^2} * (1 - e^{-2\eta_L}) = \frac{1}{14.464} - \frac{1}{2*14.464^2} * (1 - e^{-2*14.464}) = 0.067$

$R = \sqrt{\frac{1}{\beta} * R_h * R_B * R_h * (0.53 + 0.47 * R_L)} = 0.294$

$g_R = \sqrt{2 * ln * (3600 * n_a)} + \frac{0.577}{\sqrt{2*ln*(3600*n_a)}} = 3.905$

$Q = \sqrt{\frac{1}{1 + 0.63 * (\frac{B+h}{L_z})^{0.63}}} = \sqrt{\frac{1}{1 + 0.63 * (\frac{60+44}{134.809})^{0.63}}} = 1.239$

$G_f = 0.925 * (\frac{1 + 1.7 * I_z * \sqrt{g_Q^2 * Q^2 + g_R^2 * R^2}}{1 + 1.7 * g_v * I_z}) = 1.082$ $g_v = g_Q = 3.4$

We applied the same procedure of calculations for the Case 1 transverse and Case 2 both transverse and longitudinal. Results are here:

Table 4.58. Gust effect results for different cases.

	Case 1:1-3 floors (60 to 50 m)		Case 2:4-13 floors (40 to 40 m)	
	Transverse	Longitudinal	Transverse	Longitudinal
Gust Factor	1.082	1.079	1.078	
Structural Type	Reinforced Concrete Moment Frame			

The velocity pressures $q_z = 0.613 * K_z * K_{zt} * K_d * V^2$, wind pressures $P = q_z * G * C_{p,windward} - q_h * G * C_{p,leeward}$, and wind forces acting on each floor $F_{13} = (B/number\ of\ frames) \cdot h_{13}/2 \cdot P_{13}$ and $F_i = (B/number\ of\ frames) \cdot \frac{(P_{i+1}*h_{i+1})}{2} + \frac{(P_i*h_i)}{2}$ were determined:

Table 4.59. Case 1 Transverse wind force calculations on each frame ($C_{pw} =$ 0.8 and $C_{pl} = -0.5$).

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)
13	3	43	1.0903	49	0.85	1	1364.053	1.082	1180.724	1908.702	171.783	85.892
12	3	41	1.0756	49	0.85	1	1345.616	1.082	1164.765	1892.744	170.347	15.486
11	3	38	1.0525	49	0.85	1	1316.717	1.082	1139.751	1867.729	168.096	15.281
10	3	35	1.0281	49	0.85	1	1286.140	1.082	1113.283	1841.261	165.713	15.065
9	3	32	1.0021	49	0.85	1	1253.628	1.082	1085.140	1813.119	163.181	14.835
8	3	29	0.9743	49	0.85	1	1218.860	1.082	1055.045	1783.024	160.472	14.588
7	3	26	0.9443	49	0.85	1	1181.419	1.082	1022.636	1750.615	157.555	14.323
6	3	23	0.9118	49	0.85	1	1140.751	1.082	987.434	1715.413	154.387	14.035
5	3	20	0.8761	49	0.85	1	1096.096	1.082	948.781	1676.759	150.908	13.719
4	3	17	0.8364	49	0.85	1	1046.364	1.082	905.732	1633.711	147.034	13.367
3	4	14	0.7913	49	0.85	1	989.899	1.082	856.857	1584.835	190.180	17.289
2	4	10	0.7187	49	0.85	1	899.167	1.082	778.319	1506.297	180.756	16.432
1	6	6	0.6211	49	0.85	1	777.060	1.082	672.623	1400.602	252.108	22.919

Table 4.60. Case 1 Longitudinal wind force calculations on each frame ($C_{pw} = 0.8$ and $C_{pl} = -0.46$).

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)
13	3	43	1.0903	49	0.85	1	1364.053	1.079	1177.450	1847.190	138.539	69.270
12	3	41	1.0756	49	0.85	1	1345.616	1.079	1161.536	1831.276	137.346	15.261
11	3	38	1.0525	49	0.85	1	1316.717	1.079	1136.590	1806.331	135.475	15.053
10	3	35	1.0281	49	0.85	1	1286.140	1.079	1110.196	1779.936	133.495	14.833
9	3	32	1.0021	49	0.85	1	1253.628	1.079	1082.132	1751.872	131.390	14.599
8	3	29	0.9743	49	0.85	1	1218.860	1.079	1052.120	1721.860	129.140	14.349
7	3	26	0.9443	49	0.85	1	1181.419	1.079	1019.801	1689.541	126.716	14.080
6	3	23	0.9118	49	0.85	1	1140.751	1.079	984.696	1654.437	124.083	13.787
5	3	20	0.8761	49	0.85	1	1096.096	1.079	946.150	1615.890	121.192	13.466
4	3	17	0.8364	49	0.85	1	1046.364	1.079	903.221	1572.961	117.972	13.108
3	4	14	0.7913	49	0.85	1	989.899	1.079	854.481	1524.221	152.422	16.936
2	4	10	0.7187	49	0.85	1	899.167	1.079	776.161	1445.901	144.590	16.066
1	6	6	0.6211	49	0.85	1	777.060	1.079	670.759	1340.499	201.075	22.342

Table 4.61. Case 2 Longitudinal and Transverse wind force calculations on each frame

$(C_{pw} = 0.8 \; and \; C_{pl} = -0.5)$.

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)
13	3	43	1.0903	49	0.85	1	1364.053	1.078	1176.359	1904.337	114.260	57.130
12	3	41	1.0756	49	0.85	1	1345.616	1.078	1160.459	1888.437	113.306	16.187
11	3	38	1.0525	49	0.85	1	1316.717	1.078	1135.537	1863.515	111.811	15.973
10	3	35	1.0281	49	0.85	1	1286.140	1.078	1109.167	1837.145	110.229	15.747
9	3	32	1.0021	49	0.85	1	1253.628	1.078	1081.129	1809.107	108.546	15.507
8	3	29	0.9743	49	0.85	1	1218.860	1.078	1051.145	1779.123	106.747	15.250
7	3	26	0.9443	49	0.85	1	1181.419	1.078	1018.856	1746.834	104.810	14.973
6	3	23	0.9118	49	0.85	1	1140.751	1.078	983.784	1711.762	102.706	14.672
5	3	20	0.8761	49	0.85	1	1096.096	1.078	945.273	1673.251	100.395	14.342
4	3	17	0.8364	49	0.85	1	1046.364	1.078	902.384	1630.362	97.822	13.975
3	4	14	0.7913	49	0.85	1	989.899	1.078	853.689	1581.667	126.533	18.076
2	4	10	0.7187	49	0.85	1	899.167	1.078	775.441	1503.419	120.274	17.182
1	6	6	0.6211	49	0.85	1	777.060	1.078	670.137	1398.115	167.774	23.968

As for the calculating wind loads including torsional effect, using the figure X from ASCE-7 (2016) we found torsion values and torsional force acting on each frame at each floor.

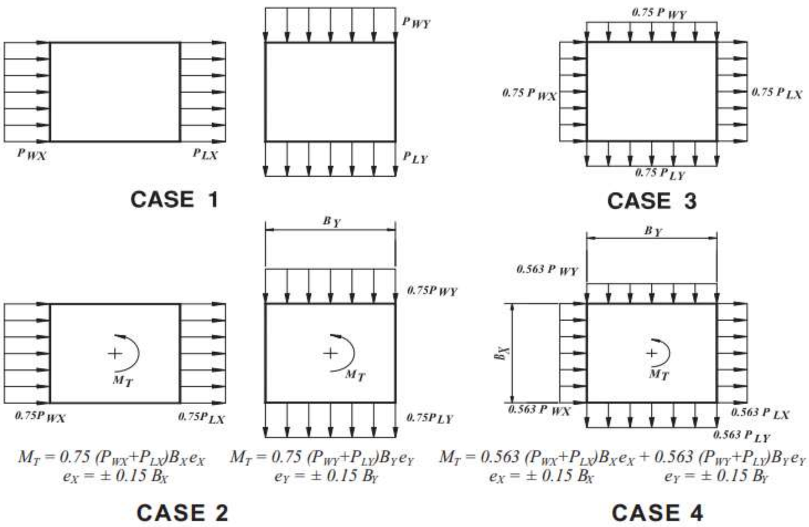


Figure 4.18. Wind load cases description.

Generally, for Case 1 we have different e_x and e_y values, however those values are similar during the calculation of Case 2, where we have square shape of the building.

Table 4.62. Direct force component Longitudinal Case 1.

ex	9			
Story	Pw, Pl (Pa)	MT (kN-m)	0.75Fi (kN)	Fdirect (kN)
13	1180.724	97.9164297	128.8374075	64.41870375
12	1164.765	766.5612606	127.7602101	11.61456455
11	1139.751	756.4302513	126.0717085	11.46106441
10	1113.283	745.7106745	124.2851124	11.29864658
9	1085.140	734.3131261	122.385521	11.12595646
8	1055.045	722.1245847	120.3540974	10.94128159
7	1022.636	708.9989426	118.1664904	10.74240822
6	987.434	694.7421258	115.7903543	10.52639585
5	948.781	679.0875311	113.1812552	10.28920502
4	905.732	661.6528951	110.2754825	10.02504387
3	856.857	641.8582557	142.6351679	12.96683345
2	778.319	610.0502639	135.5667253	12.32424776
1	672.623	567.2437614	189.0812538	17.18920489

Table 4.63. Direct force component Transverse Case 1.

ey	7.5			
Story	Pw, Pl (Pa)	MT (kN-m)	0.75Fi (kN)	Fdirect (kN)
13	1177.450	78.9673868	103.9044563	51.95222816
12	1161.536	515.0464037	103.0092807	11.44547564
11	1136.590	508.0304872	101.6060974	11.28956638
10	1110.196	500.6069767	100.1213953	11.12459948
9	1082.132	492.7139578	98.54279157	10.94919906
8	1052.120	484.2731614	96.85463227	10.76162581
7	1019.801	475.1834049	95.03668098	10.55963122
6	984.696	465.3102884	93.06205768	10.34022863
5	946.150	454.4691842	90.89383684	10.0993152
4	903.221	442.3953675	88.47907351	9.831008168
3	854.481	428.6872037	114.3165876	12.70184307
2	776.161	406.6595651	108.4425507	12.0491723
1	670.759	377.0152488	150.8060995	16.75623328

Table 4.64. Direct force component Transverse & Longitudinal Case 2.

ex=ey	6			
Story	Pw, Pl (Pa)	MT (kN-m)	0.75Fi (kN)	Fdirect (kN)
13	1176.359	466.7530881	85.69516319	42.8475816
12	1160.459	462.8561247	84.97968643	12.1399552
11	1135.537	456.7476539	83.85818016	11.97974002
10	1109.167	450.2843076	82.67151926	11.81021704
9	1081.129	443.4121796	81.40980667	11.62997238
8	1051.145	436.0631242	80.06053088	11.4372187
7	1018.856	428.1490458	78.60751772	11.22964539
6	983.784	419.5529281	77.02928313	11.0041833
5	945.273	410.1140231	75.29631403	10.75661629
4	902.384	399.6018459	73.36629374	10.48089911

Table 4.65. Case 3 force estimation from both sides (1-3 floors).

Story	0.75F1 (kN)	F1 per frame (kN)	0.75F2 (kN)	F2 per frame (kN)
13	128.8374075	64.41870375	103.9044563	51.95222816
12	127.7602101	11.61456455	103.0092807	11.44547564
11	126.0717085	11.46106441	101.6060974	11.28956638
10	124.2851124	11.29864658	100.1213953	11.12459948
9	122.385521	11.12595646	98.54279157	10.94919906
8	120.3540974	10.94128159	96.85463227	10.76162581
7	118.1664904	10.74240822	95.03668098	10.55963122
6	115.7903543	10.52639585	93.06205768	10.34022863
5	113.1812552	10.28920502	90.89383684	10.0993152
4	110.2754825	10.02504387	88.47907351	9.831008168
3	142.6351679	12.96683345	114.3165876	12.70184307
2	135.5667253	12.32424776	108.4425507	12.0491723
1	189.0812538	17.18920489	150.8060995	16.75623328

Table 4.66. Case 3 force estimation from both sides (4-13 floors).

Story	0.75F1 (kN)	F1 per frame (kN)	0.75F2 (kN)	F2 per frame (kN)
13	85.69516319	42.8475816	85.69516319	42.8475816
12	84.97968643	12.1399552	84.97968643	12.1399552
11	83.85818016	11.97974002	83.85818016	11.97974002
10	82.67151926	11.81021704	82.67151926	11.81021704
9	81.40980667	11.62997238	81.40980667	11.62997238
8	80.06053088	11.4372187	80.06053088	11.4372187
7	78.60751772	11.22964539	78.60751772	11.22964539
6	77.02928313	11.0041833	77.02928313	11.0041833
5	75.29631403	10.75661629	75.29631403	10.75661629
4	73.36629374	10.48089911	73.36629374	10.48089911

Table 4.67. Torsion and direct force component estimations (1-3 floors).

ex	9						
ey	7.5						
Story	Pw, Pl (Pa) -	Pw, Pl (Pa) - y	MT (kN-m)	0.563F1 (kN)	0.563F2 (kN)	F1 direct (kN)	F2 direct (kN)
13	1180.724	1177.450	463.4906593	96.71394723	77.99761188	48.35697361	38.99880594
12	1164.765	1161.536	962.0601533	95.90533105	77.3256334	8.718666459	8.591737045
11	1139.751	1136.590	949.1885277	94.63782922	76.27231048	8.60343902	8.474701165
10	1113.283	1110.196	935.5691168	93.29669105	75.15779409	8.481517368	8.350866011
9	1085.140	1082.132	921.088331	91.8707311	73.97278887	8.351884646	8.219198763
8	1055.045	1052.120	905.6025747	90.34580915	72.70554396	8.213255377	8.078393773
7	1022.636	1019.801	888.9262155	88.70364548	71.34086852	8.063967771	7.926763169
6	987.434	984.696	870.812679	86.9199593	69.85858463	7.901814482	7.762064959
5	948.781	946.150	850.923241	84.96139556	68.23097352	7.723763233	7.58121928
4	905.732	903.221	828.7722292	82.78012888	66.41829118	7.525466262	7.379810131
3	856.857	854.481	803.6227915	107.0714661	85.81365179	9.733769641	9.534850199
2	778.319	776.161	763.2101783	101.7654218	81.40420805	9.251401982	9.044912006
1	672.623	670.759	708.8237636	141.9369945	113.2051121	12.90336314	12.57834578

Table 4.68. Torsion and direct force component estimations (4-13 floors).

ex	6.0						
ey	6.0						
Story	Pw, Pl (Pa	Pw, Pl (Pa)	MT (kN-m	0.563F1 (kN	0.563F2 (kN)	F1 direct (kN)	F2 direct (kN)
13	1176.359	1176.359	370.53217	64.3285025	64.3285025	32.16425125	32.16425125
12	1160.459	1160.459	325.33623	63.7914179	63.79141794	9.113059706	9.113059706
11	1135.537	1135.537	321.04265	62.9495405	62.94954057	8.99279151	8.99279151
10	1109.167	1109.167	316.49964	62.0587537	62.05875379	8.865536256	8.865536256
9	1081.129	1081.129	311.6693	61.1162821	61.11162821	8.730232601	8.730232601
8	1051.145	1051.145	306.50373	60.0987718	60.09877185	8.585538836	8.585538836
7	1018.856	1018.856	300.94102	59.0080433	59.0080433	8.429720472	8.429720472
6	983.784	983.784	294.8989	57.8233152	57.8233152	8.2604736	8.2604736
5	945.273	945.273	288.2644	56.5224330	56.52243307	8.074633295	8.074633295
4	902.384	902.384	280.87551	55.0736311	55.07363117	7.867661595	7.867661595

Table 4.69. Case 4 Calculation of wind force on each frame for the first floor including the torsional effect.

	Longitudinal								
Floors	Frame	yi	yi^2	Cf (kN/rad	yi^2*Cf	Torsion (kN-m)	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-30.08	904.8064	149556	135319226	567.2437614	-2.93321	17.18920489	14.25599
	B	-24	576	149556	86144256	567.2437614	-2.34033	17.18920489	14.84888
	C	-18	324	149556	48456144	567.2437614	-1.75525	17.18920489	15.43396
	D	-12	144	149556	21536064	567.2437614	-1.17016	17.18920489	16.01904
	E	-6	36	149556	5384016	567.2437614	-0.58508	17.18920489	16.60412
	F	0	0	149556	0	567.2437614	0.00000	17.18920489	17.18920
	G	6	36	149556	5384016	567.2437614	0.58508	17.18920489	17.77429
	H	12	144	149556	21536064	567.2437614	1.17016	17.18920489	18.35937
	I	18	324	149556	48456144	567.2437614	1.75525	17.18920489	18.94445
	J	24	576	149556	86144256	567.2437614	2.34033	17.18920489	19.52953
1	K	30.08	904.8064	149556	135319226	567.2437614	2.93321	17.18920489	20.12242
				Total	593679411.9				

Trasnsverse								
Frame	xi	xi^2	Cf (kN/rad)	xi^2*Cf	Torsion (kN-m)	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	-25	625	122364	76477500	377.0152488	-1.32570	16.75623328	15.43054
2	-18	324	122364	39645936	377.0152488	-0.95450	16.75623328	15.80173
3	-12	144	122364	17620416	377.0152488	-0.63634	16.75623328	16.11990
4	-6	36	122364	4405104	377.0152488	-0.31817	16.75623328	16.43807
5	0	0	122364	0	377.0152488	0.00000	16.75623328	16.75623
6	6	36	122364	4405104	377.0152488	0.31817	16.75623328	17.07440
7	12	144	122364	17620416	377.0152488	0.63634	16.75623328	17.39257
8	18	324	122364	39645936	377.0152488	0.95450	16.75623328	17.71074
9	25	625	122364	76477500	377.0152488	1.32570	16.75623328	18.08193
			Total	276297912				

Table 4.70. Case 4 Calculation of wind force on each frame for the forth floor including the torsional effect

.	Longitudinal								
Floors	Frame	yi	yi^2	Cf (kN/rad)	yi^2*Cf	Torsion (kN-m)	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-20.252	410.143504	187320	76828081.17	399.6018459	-3.428291817	10.48089911	7.052607289
	B	-12	144	187320	26974080	399.6018459	-2.031379706	10.48089911	8.4495194
	C	-6	36	187320	6743520	399.6018459	-1.015689853	10.48089911	9.465209253
	D	0	0	187320	0	399.6018459	0	10.48089911	10.48089911
	E	6	36	187320	6743520	399.6018459	1.015689853	10.48089911	11.49658896
	F	12	144	187320	26974080	399.6018459	2.031379706	10.48089911	12.51227881
	G	20.252	410.143504	187320	76828081.17	399.6018459	3.428291817	10.48089911	13.90919092
				Total	221091362.3				
4									

Trasnsverse								
Frame	xi	xi^2	Cf (kN/rad)	xi^2*Cf	Torsion (kN-m)	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	-20.252	410.143504	187320	76828081.17	399.6018459	-3.428291817	10.48089911	7.052607289
B	-12	144	187320	26974080	399.6018459	-2.031379706	10.48089911	8.4495194
C	-6	36	187320	6743520	399.6018459	-1.015689853	10.48089911	9.465209253
D	0	0	187320	0	399.6018459	0	10.48089911	10.48089911
E	6	36	187320	6743520	399.6018459	1.015689853	10.48089911	11.49658896
F	12	144	187320	26974080	399.6018459	2.031379706	10.48089911	12.51227881
G	20.252	410.143504	187320	76828081.17	399.6018459	3.428291817	10.48089911	13.90919092
			Total	221091362.3				

4.4.3. Lateral drift analysis

Wind drift was estimated for Frame A and Frame 3 accordingly.

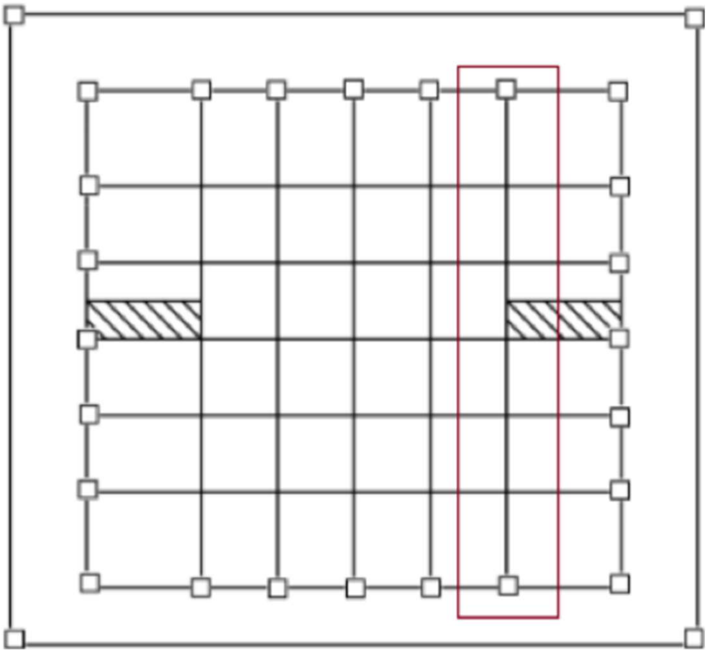


Figure 4.19. Frame layout used in the analysis

Wind Drift:

Table 4.71. Longitudinal Shear Drift of Frame A

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	abs (mm)
12	3000	3000	15.26	15.26	1.53	1.53	2.28E+08	7.74E+08	2.28E+08	0.30	0.51	0.81	0.87	39.57
11	3000	3000	15.05	30.31	3.03	2.28	4.73E+08	7.74E+08	3.51E+08	0.45	0.49	0.94	1.05	38.70
10	3000	3000	14.83	45.15	4.51	3.77	8.75E+08	7.74E+08	6.74E+08	0.74	0.42	1.16	1.26	37.65
9	3000	3000	14.60	59.75	5.97	5.24	1.49E+09	7.74E+08	1.18E+09	1.03	0.34	1.36	1.47	36.39
8	3000	3000	14.35	74.09	7.41	6.69	2.39E+09	7.74E+08	1.94E+09	1.31	0.26	1.57	1.71	34.92
7	3000	3000	14.08	88.17	8.82	8.11	2.39E+09	7.74E+08	2.39E+09	1.59	0.26	1.84	1.97	33.22
6	3000	3000	13.79	101.96	10.20	9.51	3.65E+09	7.74E+08	3.02E+09	1.86	0.24	2.10	2.20	31.25
5	3000	3000	13.47	115.43	11.54	10.87	5.34E+09	7.74E+08	4.50E+09	2.13	0.18	2.31	2.43	29.04
4	3000	3000	13.11	128.53	12.85	12.20	5.34E+09	7.74E+08	5.34E+09	2.39	0.17	2.56	3.85	26.61
3	4000	3500	16.94	145.47	14.55	13.70	7.56E+09	7.74E+08	6.45E+09	4.76	0.38	5.14	5.42	22.76

2	4000	4000	16.07	161.54	16.15	15.35	7.56E+09	7.74E+08	7.56E+09	5.34	0.36	5.70	10.09	17.33
1	6000	5000	22.34	183.88	18.39	17.27	1.40E+10	7.74E+08	1.08E+10	13.51	0.97	14.48	7.24	7.24
0		3000			0.00	9.19								

Table 4.72. Longitudinal Flexural Drift of Frame A

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
12	4.58E+08	0.25	62500	1.52E-09	4.56E-06	4.42E-04	1.33	12.68
11	1.37E+09	0.3	90000	3.15E-09	9.46E-06	4.37E-04	1.31	11.35
10	2.72E+09	0.35	122500	4.61E-09	1.38E-05	4.28E-04	1.28	10.04
9	4.51E+09	0.4	160000	5.86E-09	1.76E-05	4.14E-04	1.24	8.76
8	6.74E+09	0.45	202500	6.91E-09	2.07E-05	3.97E-04	1.19	7.52
7	9.38E+09	0.45	202500	9.62E-09	2.89E-05	3.76E-04	1.13	6.33
6	1.24E+10	0.5	250000	1.03E-08	3.10E-05	3.47E-04	1.04	5.20
5	1.59E+10	0.55	302500	1.09E-08	3.28E-05	3.16E-04	0.95	4.16
4	1.98E+10	0.55	302500	1.36E-08	4.07E-05	2.83E-04	0.85	3.21
3	2.56E+10	0.6	360000	1.48E-08	5.90E-05	2.42E-04	0.97	2.36
2	3.20E+10	0.6	360000	1.85E-08	7.39E-05	1.83E-04	0.73	1.39
1	4.31E+10	0.7	490000	1.83E-08	1.10E-04	1.10E-04	0.66	0.66
0								

Table 4.73. Transverse Shear Drift of Frame 3

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	abs (mm)
12	3000	3000	15.49	15.49	5.16	5.16	2.28E+08	7.74E+08	2.28E+08	1.01	1.71	2.72	2.95	134.17
11	3000	3000	15.28	30.77	10.26	7.71	4.73E+08	7.74E+08	3.51E+08	1.51	1.66	3.17	3.55	131.22
10	3000	3000	15.06	45.83	15.28	12.77	8.75E+08	7.74E+08	6.74E+08	2.50	1.43	3.93	4.27	127.67
9	3000	3000	14.83	60.67	20.22	17.75	1.49E+09	7.74E+08	1.18E+09	3.47	1.14	4.61	4.96	123.40

8	3000	3000	14.59	75.26	25.09	22.65	2.39E+09	7.74E+08	1.94E+09	4.43	0.88	5.31	5.78	118.44
7	3000	3000	14.32	89.58	29.86	27.47	2.39E+09	7.74E+08	2.39E+09	5.37	0.87	6.24	6.67	112.66
6	3000	3000	14.04	103.61	34.54	32.20	3.65E+09	7.74E+08	3.02E+09	6.30	0.81	7.10	7.46	105.99
5	3000	3000	13.72	117.33	39.11	36.82	5.34E+09	7.74E+08	4.50E+09	7.20	0.62	7.82	8.25	98.53
4	3000	3000	13.37	130.70	43.57	41.34	5.34E+09	7.74E+08	5.34E+09	8.09	0.59	8.67	13.06	90.28
3	4000	3500	17.29	147.99	49.33	46.45	7.56E+09	7.74E+08	6.45E+09	16.15	1.29	17.44	18.39	77.22
2	4000	4000	16.43	164.42	54.81	52.07	7.56E+09	7.74E+08	7.56E+09	18.10	1.24	19.34	34.25	58.83
1	6000	5000	22.92	187.34	62.45	58.63	1.40E+10	7.74E+08	1.08E+10	45.87	3.29	49.16	24.58	24.58
0		3000			0.00	31.22								

Table 4.74. Transverse Flexural Drift of Frame 3

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
12	4.65E+08	0.25	62500	1.54E-09	4.63E-06	4.49E-04	1.35	12.89
11	1.39E+09	0.3	90000	3.20E-09	9.61E-06	4.45E-04	1.33	11.54
10	2.76E+09	0.35	122500	4.68E-09	1.40E-05	4.35E-04	1.30	10.21
9	4.58E+09	0.4	160000	5.95E-09	1.78E-05	4.21E-04	1.26	8.90
8	6.84E+09	0.45	202500	7.01E-09	2.10E-05	4.03E-04	1.21	7.64
7	9.53E+09	0.45	202500	9.77E-09	2.93E-05	3.82E-04	1.15	6.43
6	1.26E+10	0.5	250000	1.05E-08	3.15E-05	3.53E-04	1.06	5.29
5	1.62E+10	0.55	302500	1.11E-08	3.33E-05	3.21E-04	0.96	4.23
4	2.01E+10	0.55	302500	1.38E-08	4.13E-05	2.88E-04	0.86	3.26
3	2.60E+10	0.6	360000	1.50E-08	6.00E-05	2.47E-04	0.99	2.40
2	3.26E+10	0.6	360000	1.88E-08	7.52E-05	1.87E-04	0.75	1.41
1	4.38E+10	0.7	490000	1.86E-08	1.11E-04	1.11E-04	0.67	0.67
0								

Table 4.75. Total Longitudinal Drifts of Frame A

	Shear	Flexural	Total	Vu	Pu	theta	Beta	Amplified	Allowable

Story	Interstory	Absolute	Interstory	Absolute	Interstory	Absolute					Interstory	Absolute	Interstory	Absolute
12	2.95	134.17	1.35	12.889	4.30	147.06	61.0	7300.6	0.0311	1.03E+00	4.433	177.393	60	820
11	3.55	131.22	1.33	11.542	4.89	142.77	121.3	18826.4	0.0460	1.05E+00	5.121	172.960	60	760
10	4.27	127.67	1.30	10.208	5.57	137.88	180.6	30352.3	0.0568	1.06E+00	5.910	167.839	60	700
9	4.96	123.40	1.26	8.904	6.22	132.31	239.0	41878.2	0.0661	1.07E+00	6.664	161.929	60	640
8	5.78	118.44	1.21	7.641	6.99	126.08	296.4	53404.1	0.0763	1.08E+00	7.565	155.265	60	580
7	6.67	112.66	1.15	6.432	7.82	119.10	352.7	64930.0	0.0873	1.10E+00	8.567	147.700	60	520
6	7.46	105.99	1.06	5.286	8.52	111.28	407.8	76455.8	0.0968	1.11E+00	9.435	139.132	60	460
5	8.25	98.53	0.96	4.228	9.21	102.75	461.7	87981.7	0.1064	1.12E+00	10.307	129.697	60	400
4	13.06	90.28	0.86	3.265	13.92	93.54	514.1	99507.6	0.1633	1.20E+00	16.637	119.390	60	340
3	18.39	77.22	0.99	2.401	19.38	79.62	581.9	111033.5	0.1681	1.20E+00	23.293	102.753	80	280
2	34.25	58.83	0.75	1.415	35.00	60.25	646.1	122559.4	0.3017	1.43E+00	50.119	79.460	80	200
1	24.58	24.58	0.67	0.668	25.25	25.25	735.5	134085.2	0.1395	1.16E+00	29.341	29.341	120	120
0														

Table 4.76. Total Transverse Drifts of Frame 3

	Shear		Flexural		Total		Vu	Pu	theta	Beta	Amplified		Allowable	
Story	Interstory	Absolute	Interstory	Absolute	Interstory	Absolute					Interstory	Absolute	Interstory	Absolute
12	2.95	134.17	1.35	12.889	4.30	147.06	61.9	7300.6	0.0307	1.03E+00	4.431	176.718	60	820
11	3.55	131.22	1.33	11.542	4.89	142.77	123.1	18826.4	0.0453	1.05E+00	5.117	172.287	60	760
10	4.27	127.67	1.30	10.208	5.57	137.88	183.3	30352.3	0.0559	1.06E+00	5.904	167.170	60	700
9	4.96	123.40	1.26	8.904	6.22	132.31	242.7	41878.2	0.0651	1.07E+00	6.657	161.266	60	640
8	5.78	118.44	1.21	7.641	6.99	126.08	301.0	53404.1	0.0751	1.08E+00	7.556	154.608	60	580

7	6.67	112.66	1.15	6.432	7.82	119.10	358.3	64930.0	0.0859	1.09E+00	8.555	147.053	60	520
6	7.46	105.99	1.06	5.286	8.52	111.28	414.5	76458.8	0.0953	1.11E+00	9.419	138.498	60	460
5	8.25	98.53	0.96	4.228	9.21	102.75	469.3	87981.7	0.1046	1.12E+00	10.287	129.079	60	400
4	13.06	90.28	0.86	3.265	13.92	93.54	522.8	99507.6	0.1606	1.19E+00	16.584	118.792	60	340
3	18.39	77.22	0.99	2.401	19.38	79.62	592.0	111033.5	0.1652	1.20E+00	23.213	102.208	80	280
2	34.25	58.83	0.75	1.415	35.00	60.25	657.7	122559.4	0.2964	1.42E+00	49.742	78.995	80	200
1	24.58	24.58	0.67	0.668	25.25	25.25	749.4	134085.2	0.1369	1.16E+00	29.253	29.253	120	120
0														

Seismic Drift:

Table 4.77. Longitudinal Shear Drift of Frame A

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	Absolute (mm)
12	3000	3000	202.07	202.07	20.21	20.21	2.28E+08	7.74E+08	1.14E+08	3.95	13.42	17.38	14.71	349.51
11	3000	3000	181.12	383.19	38.32	29.26	4.73E+08	7.74E+08	3.50E+08	5.72	6.33	12.05	13.16	334.80
10	3000	3000	160.95	544.14	54.41	46.37	8.75E+08	7.74E+08	6.74E+08	9.07	5.21	14.27	15.11	321.64
9	3000	3000	141.54	685.68	68.57	61.49	1.49E+09	7.74E+08	1.18E+09	12.02	3.93	15.95	16.74	306.53
8	3000	3000	122.90	808.58	80.86	74.71	2.39E+09	7.74E+08	1.94E+09	14.61	2.91	17.52	18.54	289.79
7	3000	3000	104.07	912.64	91.26	86.06	2.39E+09	7.74E+08	2.39E+09	16.83	2.72	19.55	20.32	271.25
6	3000	3000	87.11	999.76	99.98	95.62	3.65E+09	7.74E+08	3.02E+09	18.70	2.40	21.10	21.54	250.93
5	3000	3000	71.05	1070.81	107.08	103.53	5.34E+09	7.74E+08	4.49E+09	20.25	1.74	21.99	22.68	229.38
4	3000	3000	86.25	1157.05	115.71	111.39	5.34E+09	7.74E+08	5.34E+09	21.78	1.58	23.36	33.93	206.71
3	4000	3500	56.35	1213.41	121.34	118.52	7.56E+09	7.74E+08	6.45E+09	41.20	3.30	44.50	45.10	172.78
2	4000	4000	34.00	1247.41	124.74	123.04	7.56E+09	7.74E+08	7.56E+09	42.77	2.92	45.70	75.26	127.68
1	6000	5000	5.97	1253.38	125.34	125.04	1.40E+10	7.74E+08	1.08E+10	97.81	7.02	104.83	52.41	52.41
0		3000				62.67					0.00	0.00		0.00

Table 4.78. Longitudinal Flexural Drift of Frame A

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
12	6.06E+08	0.25	62500	2.01E-09	6.04E-06	4.47E-04	1.34	12.28
11	1.76E+09	0.3	90000	4.05E-09	1.22E-05	4.41E-04	1.32	10.94
10	3.39E+09	0.35	122500	5.74E-09	1.72E-05	4.29E-04	1.29	9.62
9	5.45E+09	0.4	160000	7.07E-09	2.12E-05	4.12E-04	1.24	8.33
8	7.87E+09	0.45	202500	8.07E-09	2.42E-05	3.90E-04	1.17	7.09
7	1.06E+10	0.45	202500	1.09E-08	3.26E-05	3.66E-04	1.10	5.92
6	1.36E+10	0.5	250000	1.13E-08	3.39E-05	3.34E-04	1.00	4.82
5	1.68E+10	0.55	302500	1.15E-08	3.46E-05	3.00E-04	0.90	3.82
4	2.03E+10	0.55	302500	1.39E-08	4.18E-05	2.65E-04	0.80	2.92
3	2.51E+10	0.6	360000	1.45E-08	5.80E-05	2.23E-04	0.89	2.13
2	3.01E+10	0.6	360000	1.74E-08	6.95E-05	1.65E-04	0.66	1.24
1	3.77E+10	0.7	490000	1.60E-08	9.57E-05	9.57E-05	0.57	0.57
0			0				0.00E+00	

Table 4.79. Transverse Shear Drift of Frame 3

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	Absolute (mm)
12	3000	3000	202.07	202.07	67.36	67.36	2.28E+08	7.74E+08	2.28E+08	13.17	22.37	35.55	37.85	1153.86
11	3000	3000	181.12	383.19	127.73	97.54	4.73E+08	7.74E+08	3.50E+08	19.07	21.08	40.16	43.87	1116.01
10	3000	3000	160.95	544.14	181.38	154.56	8.75E+08	7.74E+08	6.74E+08	30.22	17.36	47.58	50.38	1072.14
9	3000	3000	141.54	685.68	228.56	204.97	1.49E+09	7.74E+08	1.18E+09	40.08	13.10	53.18	55.79	1021.75
8	3000	3000	122.90	808.58	269.53	249.04	2.39E+09	7.74E+08	1.94E+09	48.70	9.70	58.40	61.79	965.96
7	3000	3000	104.07	912.64	304.21	286.87	2.39E+09	7.74E+08	2.39E+09	56.10	9.08	65.18	67.75	904.17
6	3000	3000	87.11	999.76	333.25	318.73	3.65E+09	7.74E+08	3.02E+09	62.33	7.99	70.32	71.81	836.42
5	3000	3000	71.05	1070.81	356.94	345.09	5.34E+09	7.74E+08	4.49E+09	67.48	5.82	73.30	75.59	764.61

4	3000	3000	86.25	1157.05	385.68	371.31	5.34E+09	7.74E+08	5.34E+09	72.61	5.27	77.88	113.11	689.03
3	4000	3500	56.35	1213.41	404.47	395.08	7.56E+09	7.74E+08	6.45E+09	137.35	10.99	148.34	150.33	575.92
2	4000	4000	34.00	1247.41	415.80	410.14	7.56E+09	7.74E+08	7.56E+09	142.58	9.73	152.32	250.87	425.59
1	6000	5000	5.97	1253.38	417.79	416.80	1.40E+10	7.74E+08	1.08E+10	326.02	23.41	349.43	174.72	174.72
0		3000				208.90					0.00	0.00		0.00

Table 4.80. Transverse Flexural Drift of Frame 3

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute
12	6.06E+08	0.25	62500	2.01E-09	6.04E-06	4.47E-04	1.34	12.28
11	1.76E+09	0.3	90000	4.05E-09	1.22E-05	4.41E-04	1.32	10.94
10	3.39E+09	0.35	122500	5.74E-09	1.72E-05	4.29E-04	1.29	9.62
9	5.45E+09	0.4	160000	7.07E-09	2.12E-05	4.12E-04	1.24	8.33
8	7.87E+09	0.45	202500	8.07E-09	2.42E-05	3.90E-04	1.17	7.09
7	1.06E+10	0.45	202500	1.09E-08	3.26E-05	3.66E-04	1.10	5.92
6	1.36E+10	0.5	250000	1.13E-08	3.39E-05	3.34E-04	1.00	4.82
5	1.68E+10	0.55	302500	1.15E-08	3.46E-05	3.00E-04	0.90	3.82
4	2.03E+10	0.55	302500	1.39E-08	4.18E-05	2.65E-04	0.80	2.92
3	2.51E+10	0.6	360000	1.45E-08	5.80E-05	2.23E-04	0.89	2.13
2	3.01E+10	0.6	360000	1.74E-08	6.95E-05	1.65E-04	0.66	1.24
1	3.77E+10	0.7	490000	1.60E-08	9.57E-05	9.57E-05	0.57	0.57
0			0				0.00E+00	

Table 4.81. Total Longitudinal Drifts of Frame A

	Shear		Flexural		Total		Vu	Pu	theta	Beta	Amplified		Allowable	
Story	Interstory	Absolute	Interstory	Absolute	Interstory	Absolute					Interstory	Absolute	Interstory	Absolute
12	14.71	349.51	1.34	12.281	16.05	361.80	808.3	99507.6	0.1198	1.14E+00	18.238	397.420	60	820
11	13.16	334.80	1.32	10.940	14.48	345.74	1532.8	111033.5	0.0636	1.07E+00	15.468	379.182	60	760
10	15.11	321.64	1.29	9.616	16.40	331.26	2176.6	122559.4	0.0560	1.06E+00	17.374	363.714	60	700
9	16.74	306.53	1.24	8.330	17.97	314.86	2742.7	134085.2	0.0533	1.06E+00	18.984	346.340	60	640
8	18.54	289.79	1.17	7.095	19.71	296.88	3234.3	145611.1	0.0538	1.06E+00	20.829	327.356	60	580
7	20.32	271.25	1.10	5.923	21.42	277.17	3650.6	157137.0	0.0559	1.06E+00	22.691	306.527	60	520
6	21.54	250.93	1.00	4.824	22.54	255.75	3999.0	168662.9	0.0576	1.06E+00	23.922	283.836	60	460

5	22.68	229.38	0.90	3.823	23.58	233.21	4283.2	180188.8	0.0601	1.06E+00	25.083	259.913	60	400
4	33.93	206.71	0.80	2.924	34.73	209.63	4628.2	191714.6	0.0872	1.10E+00	38.044	234.830	60	340
3	45.10	172.78	0.89	2.129	45.99	174.90	4853.6	203240.5	0.0875	1.10E+00	50.404	196.786	80	280
2	75.26	127.68	0.66	1.236	75.92	128.91	4989.6	214766.4	0.1485	1.17E+00	89.169	146.382	80	200
1	52.41	52.41	0.57	0.574	52.99	52.99	5013.5	230522.2	0.0738	1.08E+00	57.213	57.213	120	120
0		0.00	0.00E+00		0	0.00	0			1.00E+00	0.000		0	

Table 4.82. Total Transverse Drifts of Frame 3

	Shear		Flexural		Total						Amplified		Allowable	
Story	Interstory	Absolute	Interstory	Absolute	Interstory	Absolute	Vu	Pu	Q	Beta	Interstory	Absolute	Interstory	Absolute
12	37.85	1153.86	1.34	12.281	39.19	1166.14	2222.8	99507.6	0.1063	1.12E+00	43.858	1304.527	60	820
11	43.87	1116.01	1.32	10.940	45.19	1126.95	4215.1	111033.5	0.0722	1.08E+00	48.708	1260.669	60	760
10	50.38	1072.14	1.29	9.616	51.67	1081.75	5985.5	122559.4	0.0641	1.07E+00	55.209	1211.960	60	700
9	55.79	1021.75	1.24	8.330	57.03	1030.08	7542.4	134085.2	0.0614	1.07E+00	60.762	1156.751	60	640
8	61.79	965.96	1.17	7.095	62.96	973.06	8894.3	145611.1	0.0625	1.07E+00	67.157	1095.990	60	580
7	67.75	904.17	1.10	5.923	68.85	910.09	10039.1	157137.0	0.0653	1.07E+00	73.658	1028.833	60	520
6	71.81	836.42	1.00	4.824	72.81	841.25	10997.3	168662.9	0.0677	1.07E+00	78.096	955.175	60	460
5	75.59	764.61	0.90	3.823	76.49	768.44	11778.9	180188.8	0.0709	1.08E+00	82.325	877.079	60	400
4	113.11	689.03	0.80	2.924	113.90	691.95	12727.6	191714.6	0.1040	1.12E+00	127.121	794.755	60	340
3	150.33	575.92	0.89	2.129	151.22	578.05	13347.5	203240.5	0.1047	1.12E+00	168.899	667.633	80	280
2	250.87	425.59	0.66	1.236	251.53	426.82	13721.5	214766.4	0.1790	1.22E+00	306.359	498.734	80	200
1	174.72	174.72	0.57	0.574	175.29	175.29	13787.2	230522.2	0.0888	1.10E+00	192.375	192.375	120	120
0		0.00	0.00E+00		0	0.00				1.00E+00	0.000		0	

4.5. Assigning Forces to SAP 2000 including LFRS

4.5.1. Seismic Analysis

1. **Seismic load assigning**

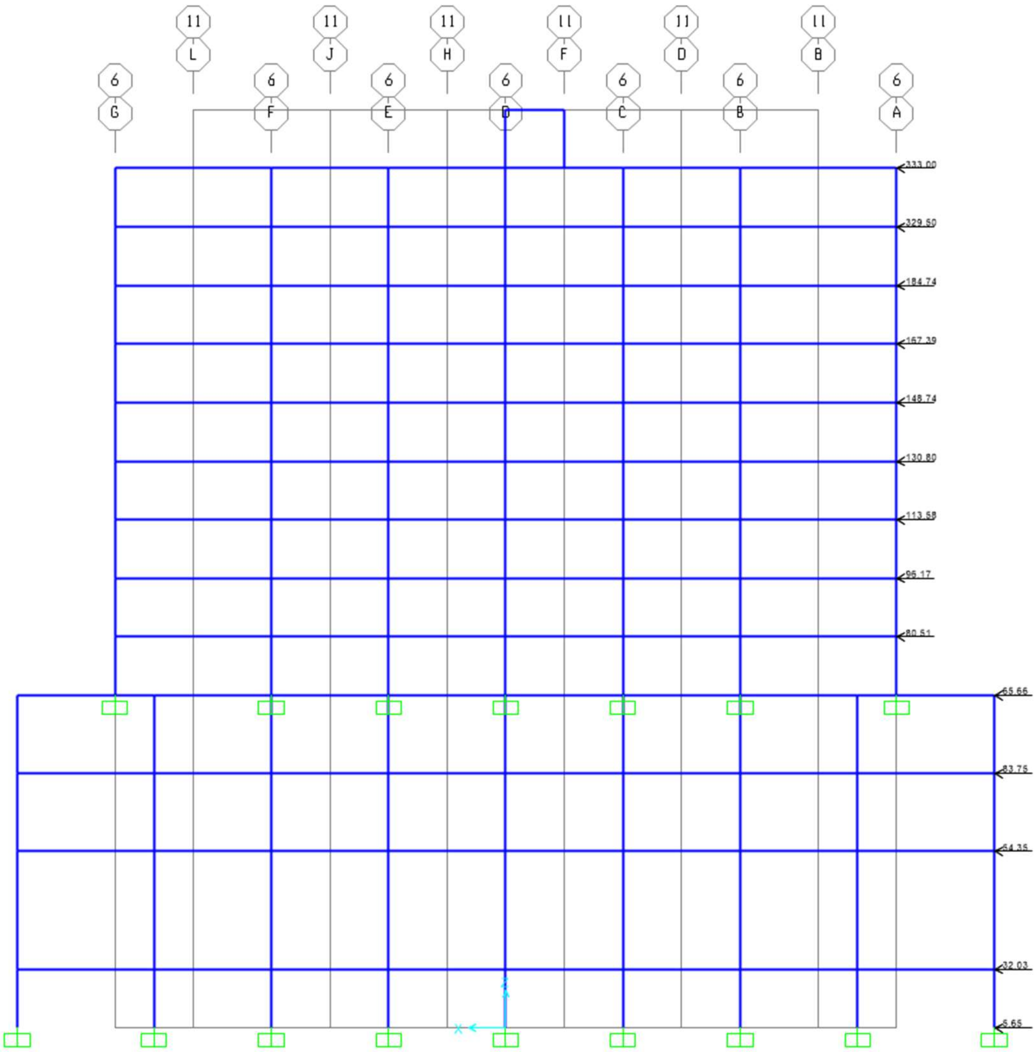


Figure 4.20. SAP2000 2D Seismic analysis Load assigning.

2. **Seismic load run**

1) Axial Force

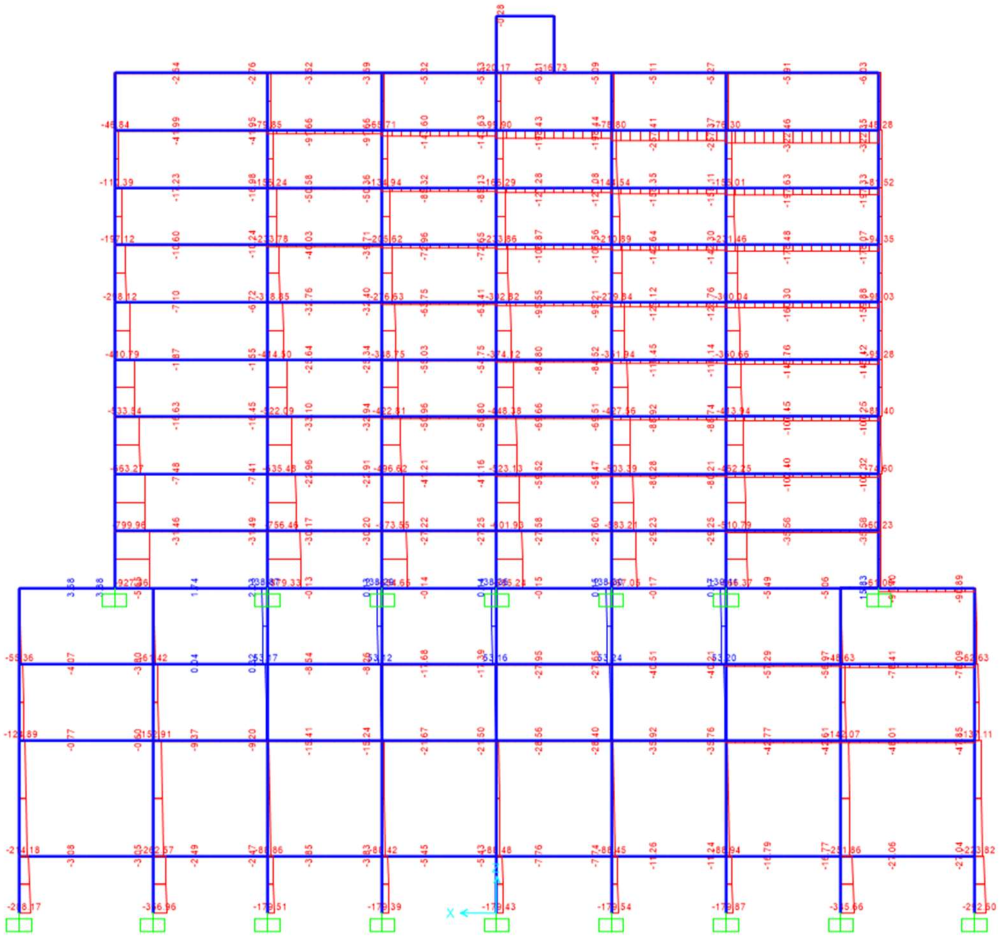


Figure 4.21. SAP2000 2D Seismic analysis (Axial Force).

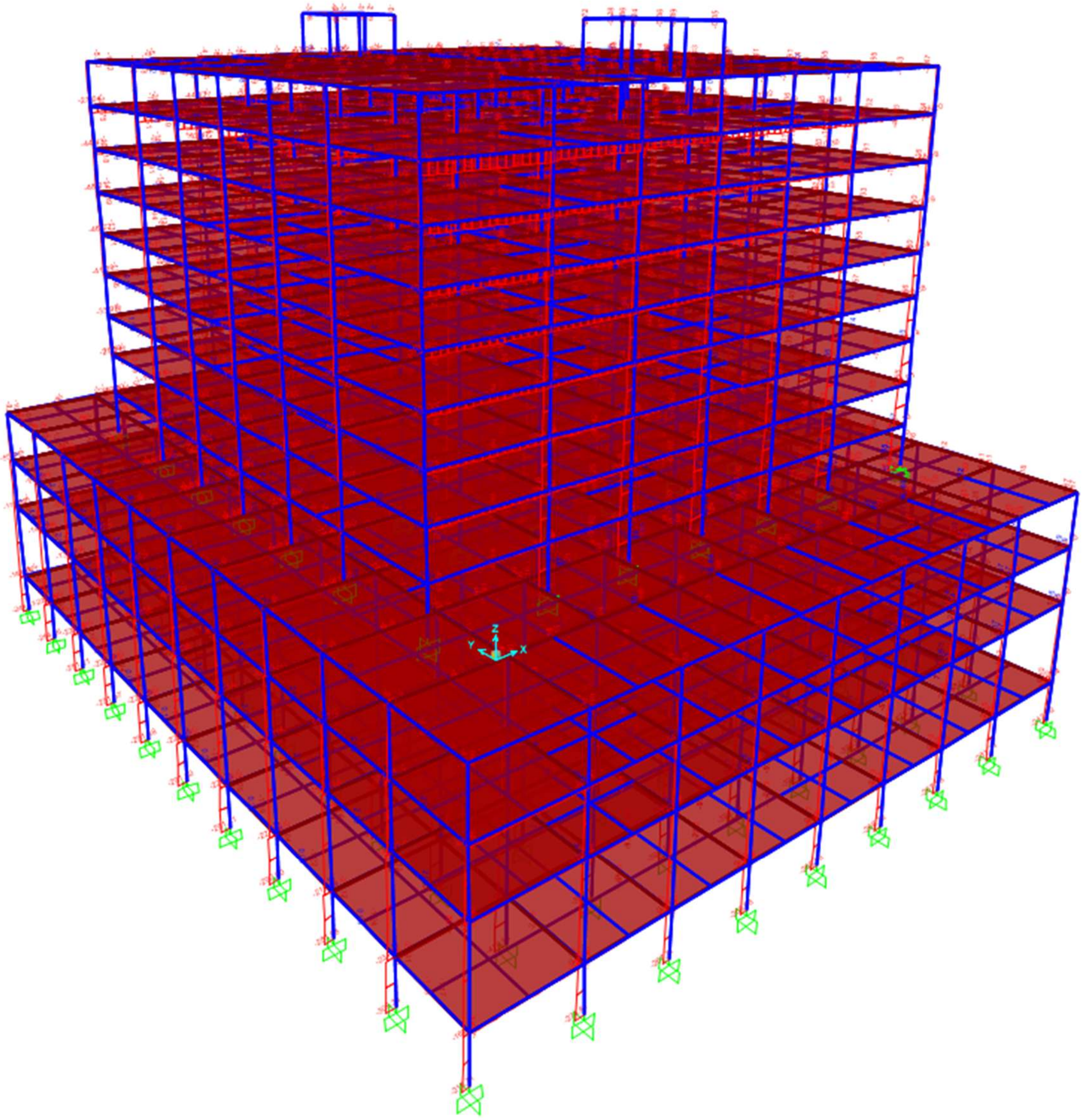


Figure 4.22. SAP2000 3D Seismic analysis (Axial Force).

2) Shear 2-2

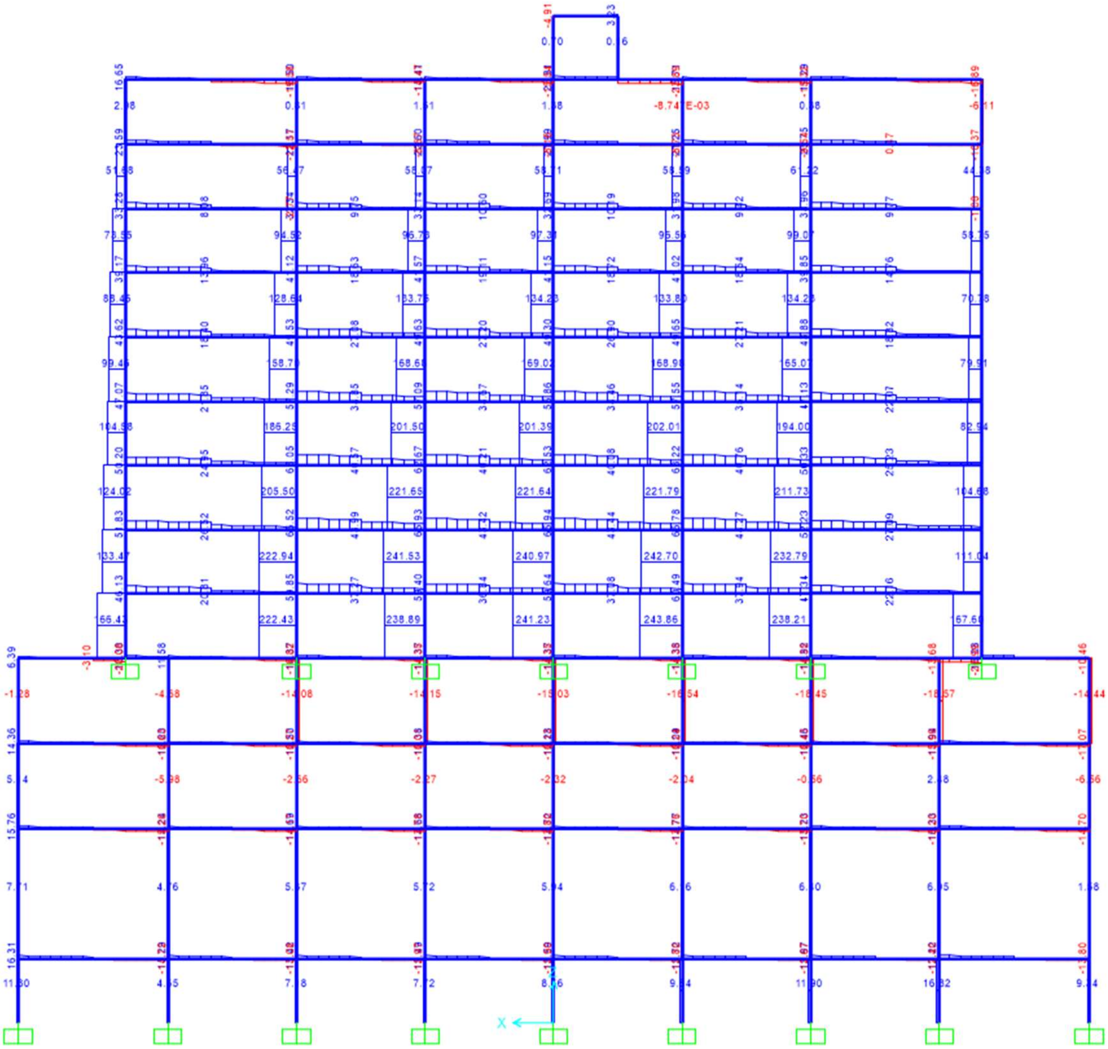


Figure 4.23. SAP2000 2D Seismic analysis (Shear 2-2).

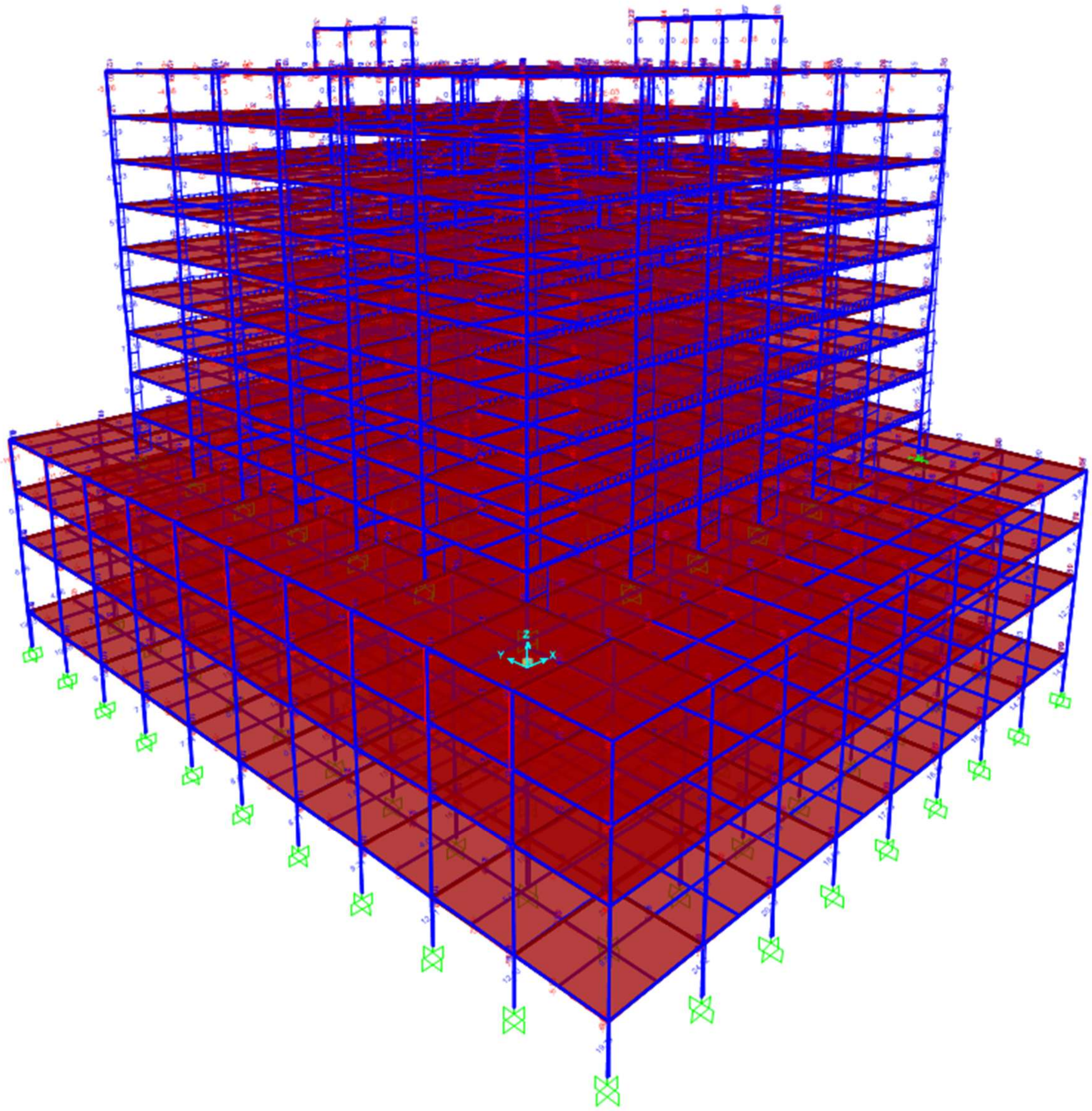


Figure 4.24. SAP2000 3D Seismic analysis (Shear 2-2).

3) Moment 3-3

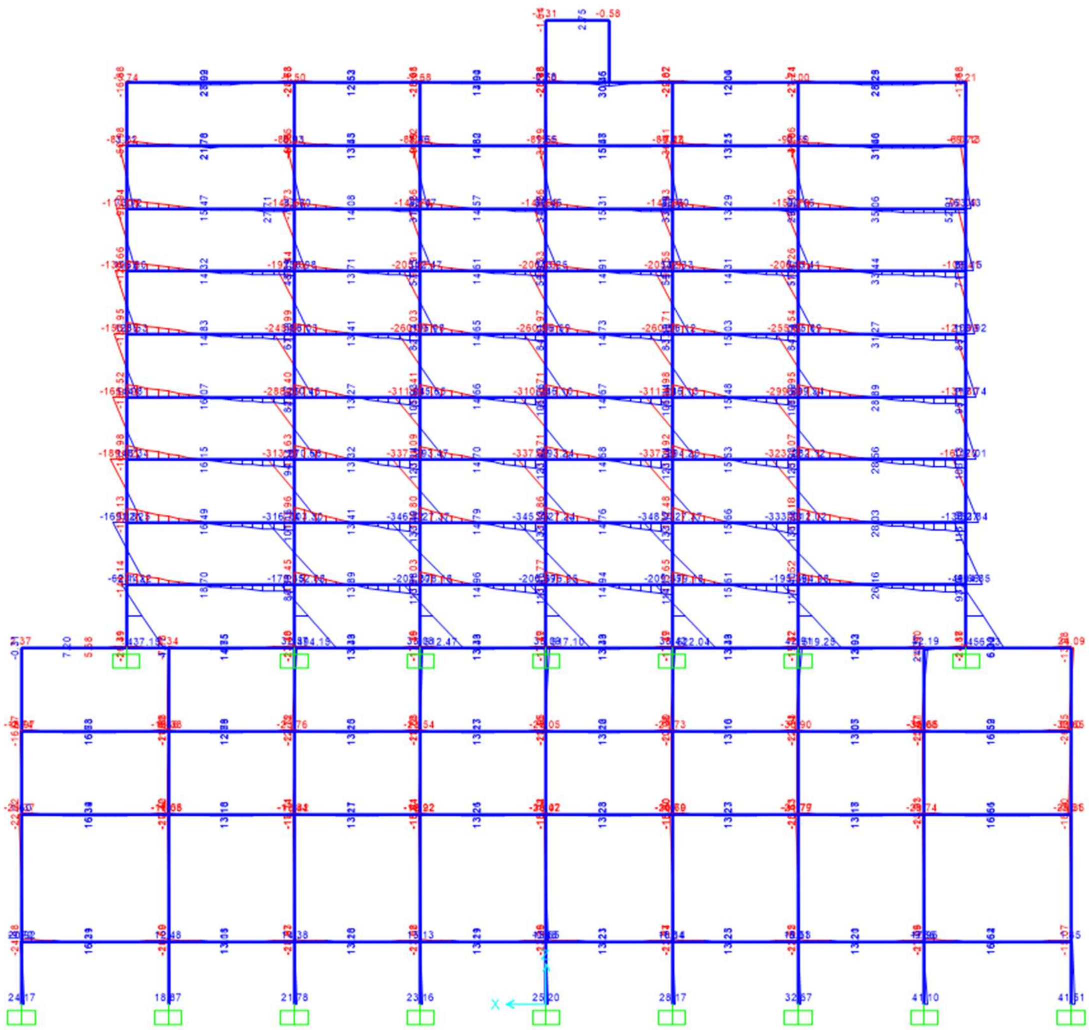


Figure 4.25. SAP2000 2D Seismic analysis (Moment 3-3).

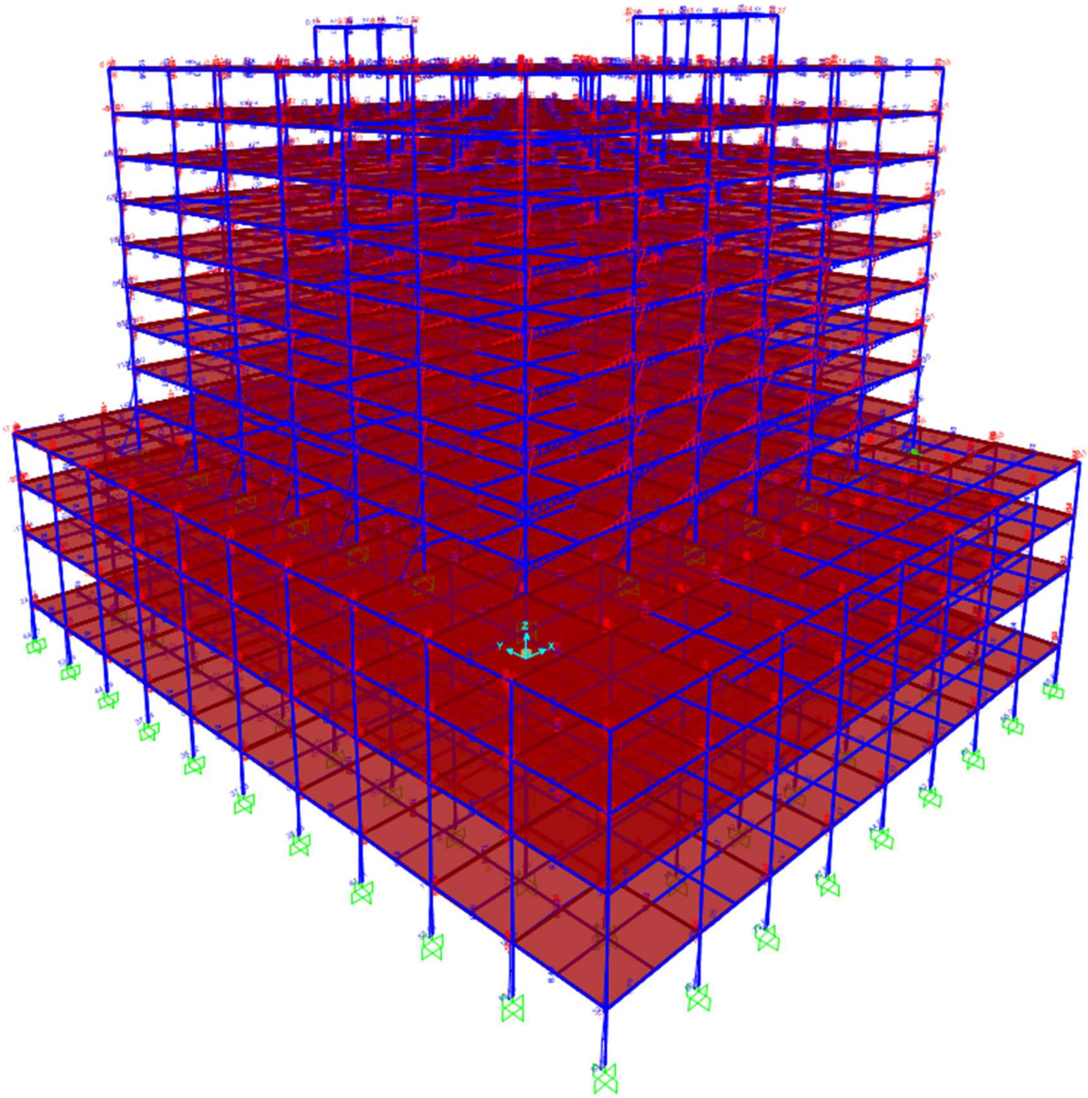


Figure 4.26. SAP2000 3D Seismic analysis (Moment 3-3).

4.5.2. Wind Analysis

1. **Wind load assigning**

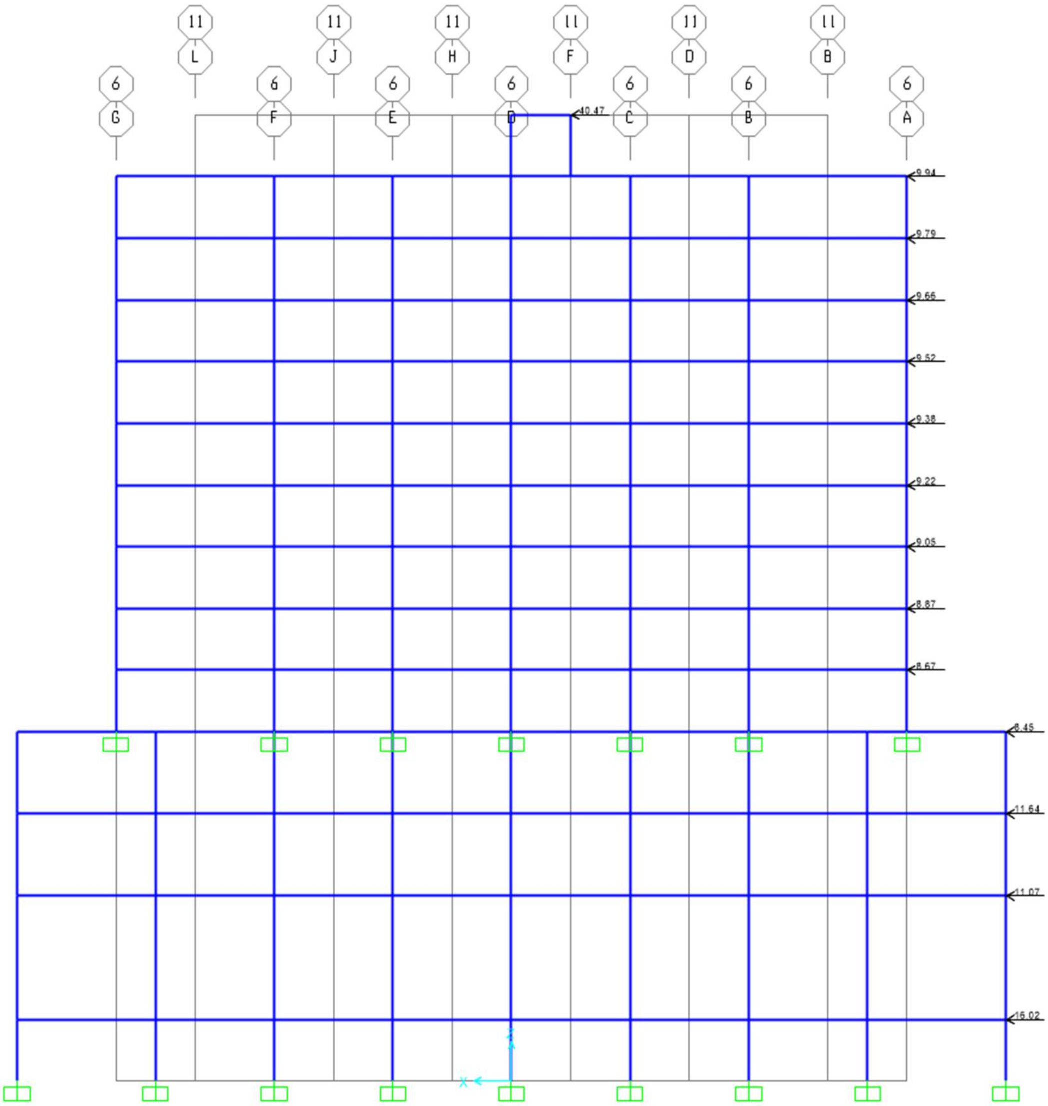


Figure 4.27. SAP2000 2D Wind analysis Load assigning.

2. **Wind load run**

1) Axial Force

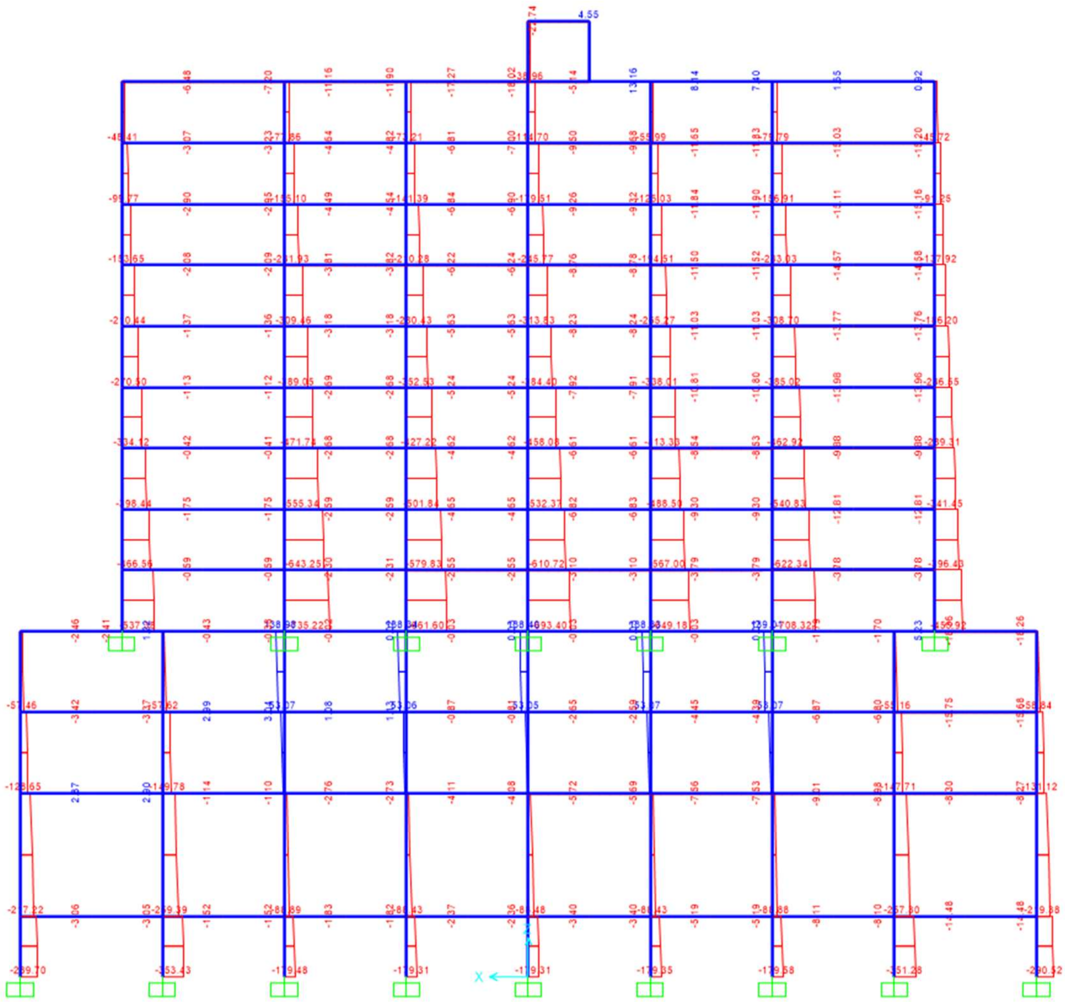


Figure 4.28. SAP2000 2D Wind analysis(Axial Force).

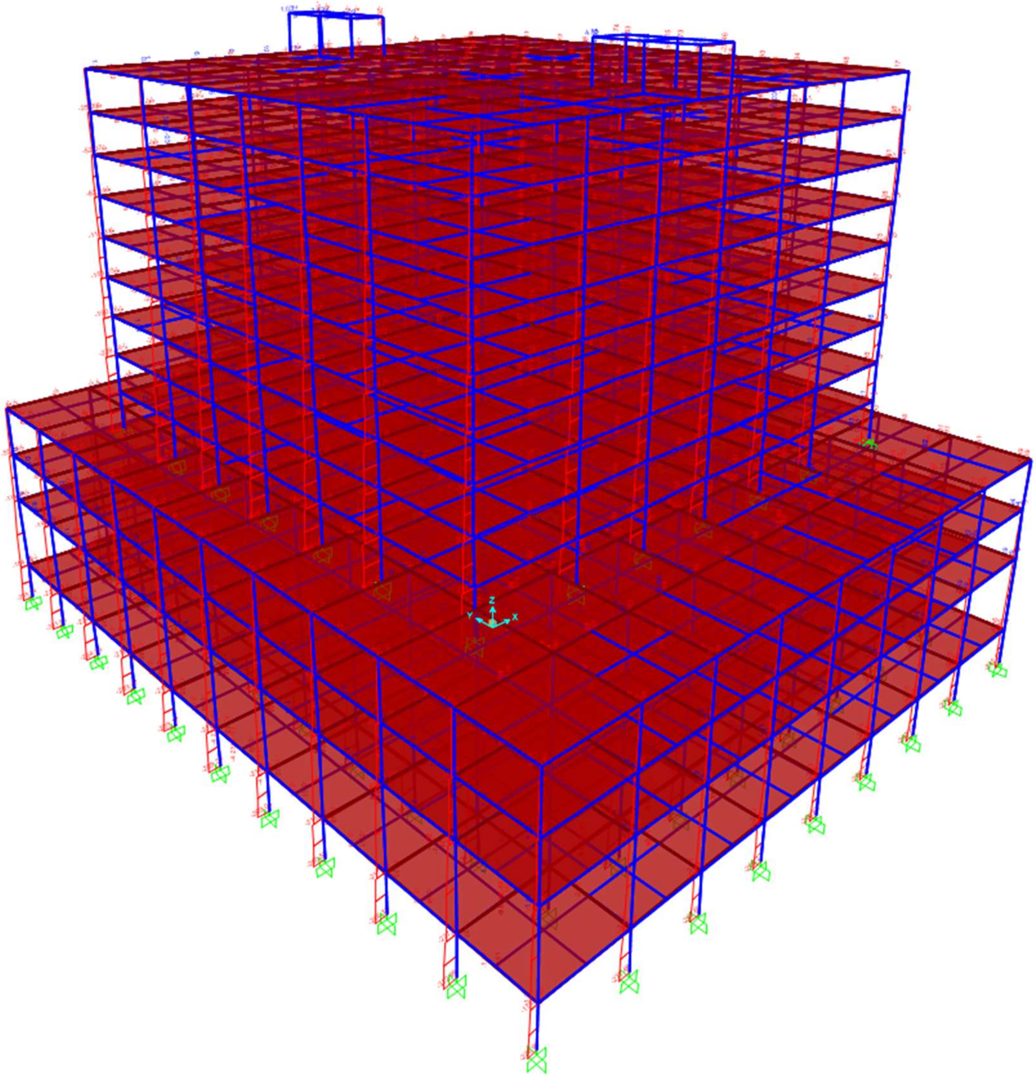


Figure 4.29. SAP2000 3D Wind analysis(Axial Force).

2) Shear 2-2

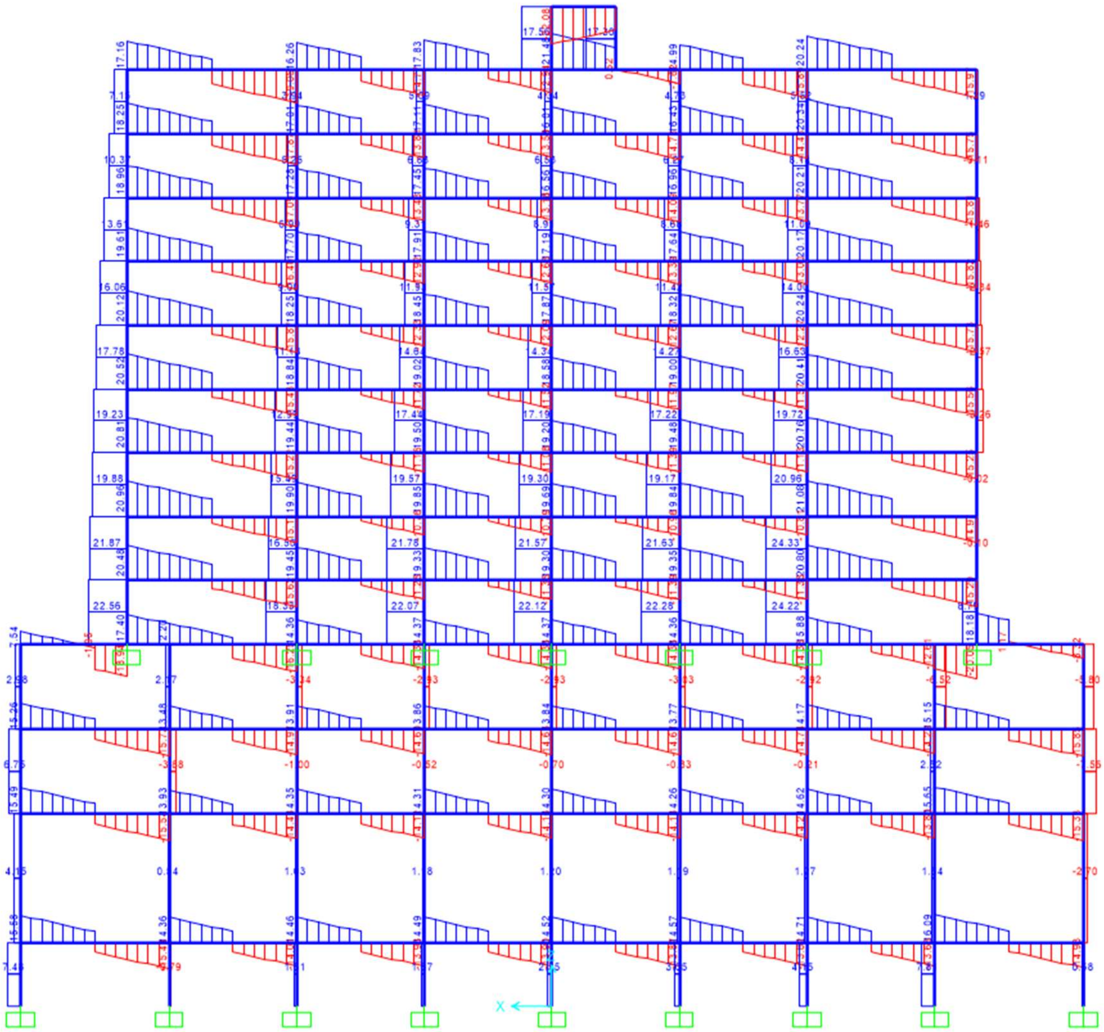


Figure 4.30. SAP2000 2D Wind analysis(Shear 2-2).

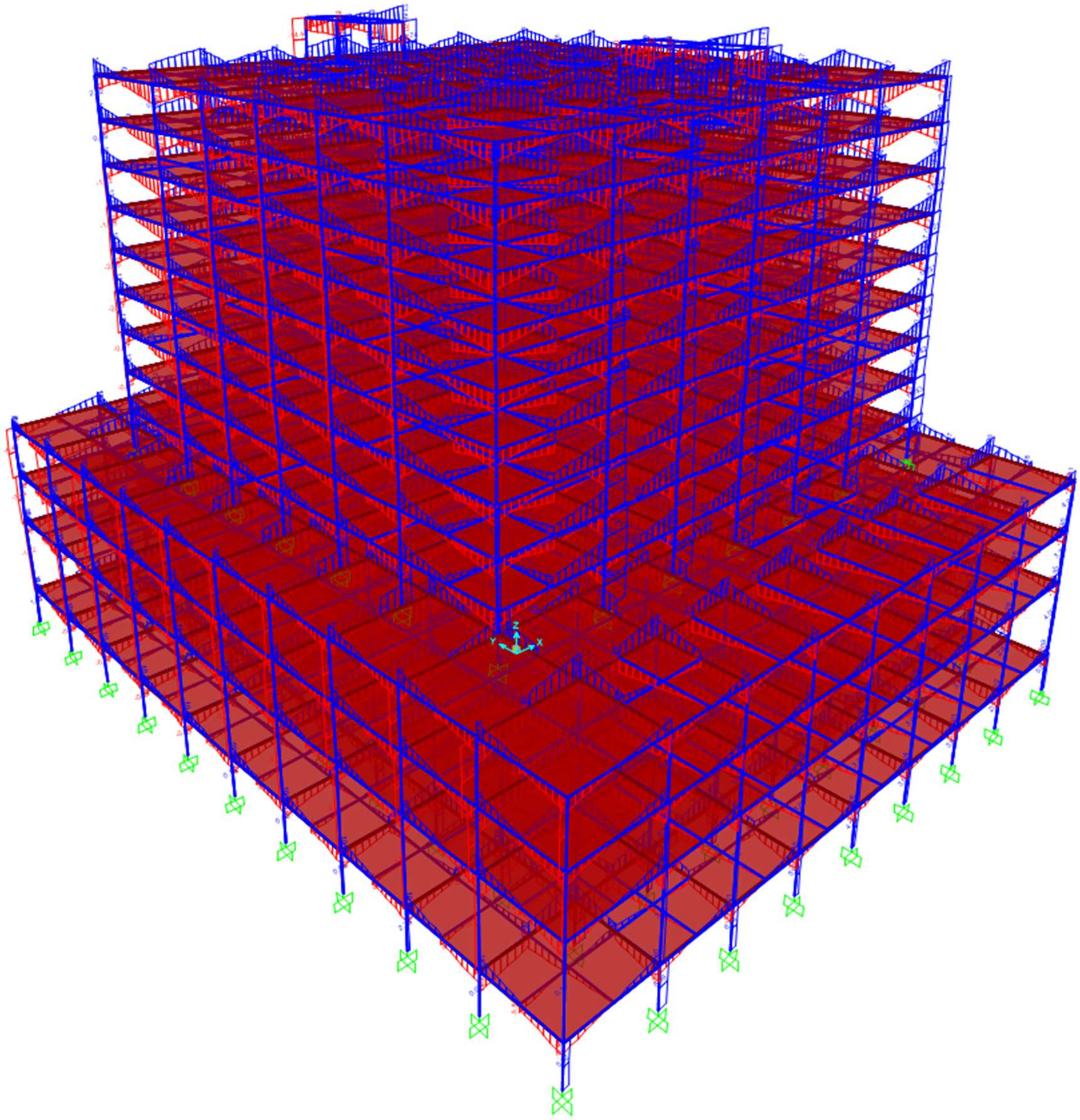


Figure 4.31. SAP2000 3D Wind analysis(Shear 2-2).

3) Moment 3-3

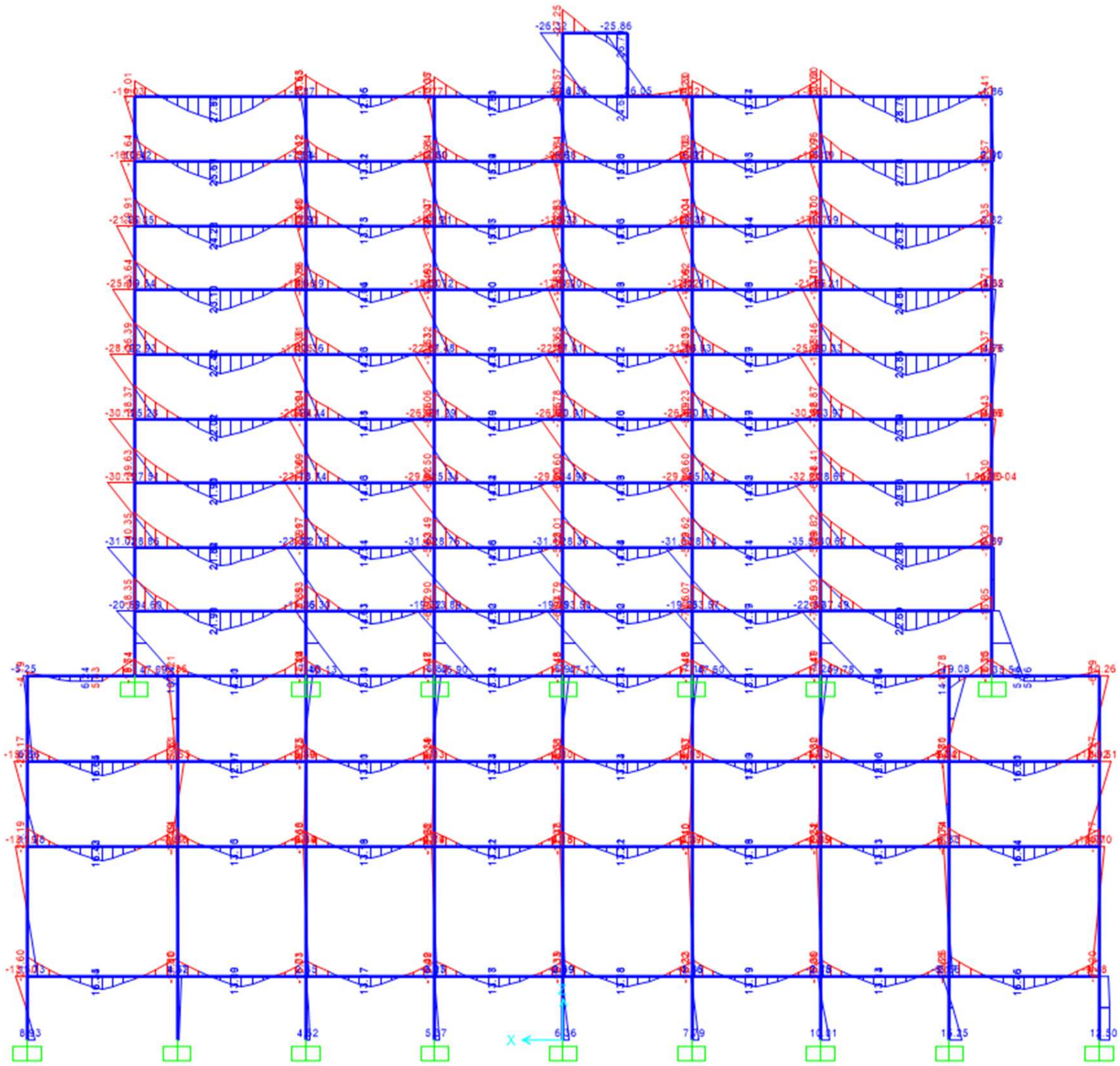


Figure 4.32. SAP2000 2D Wind analysis(Moment 3-3).

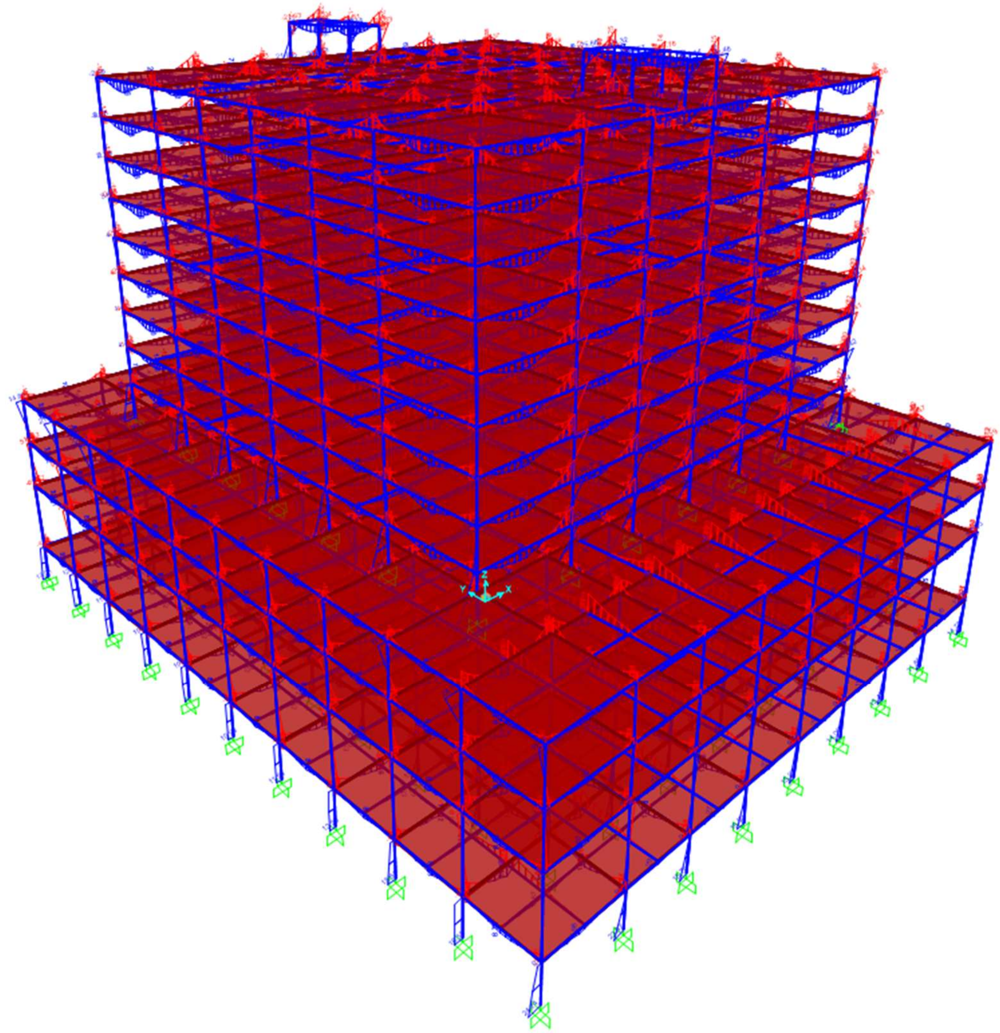


Figure 4.33. SAP2000 3D Wind analysis(Moment 3-3).

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

We used the values of lateral drifts assigned on the above Frame 3 in order to compare the results of hand calculations with 2D and 3D models. Moreover, the interstory drift for the wind drift was taken to be h/600 as well as every wind interstory deflection passed the limit inspection.

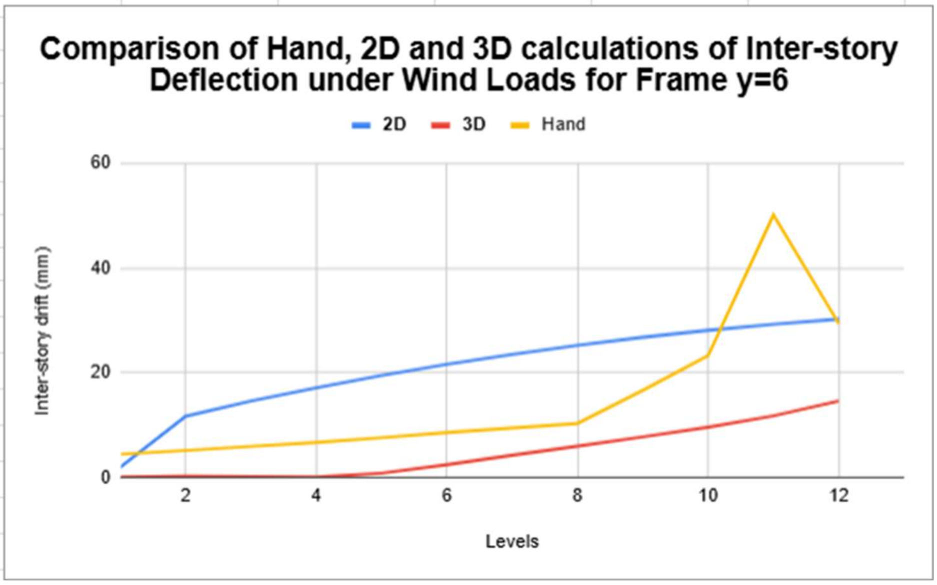


Figure 4.34. Comparison of Hand, 2D, 3D calculations of Inter-story Deflection under Wind Loads

From Figure 4.34 , the 2D solution (blue line) provides the maximum deflection values, increasing steadily with building height. The 3D solution (red line) provides lower deflections than the 2D solution but the same trend. The hand calculations (yellow line) initially have very little deflection but then pick up sharply higher up, peaking about level 11 before dropping off.

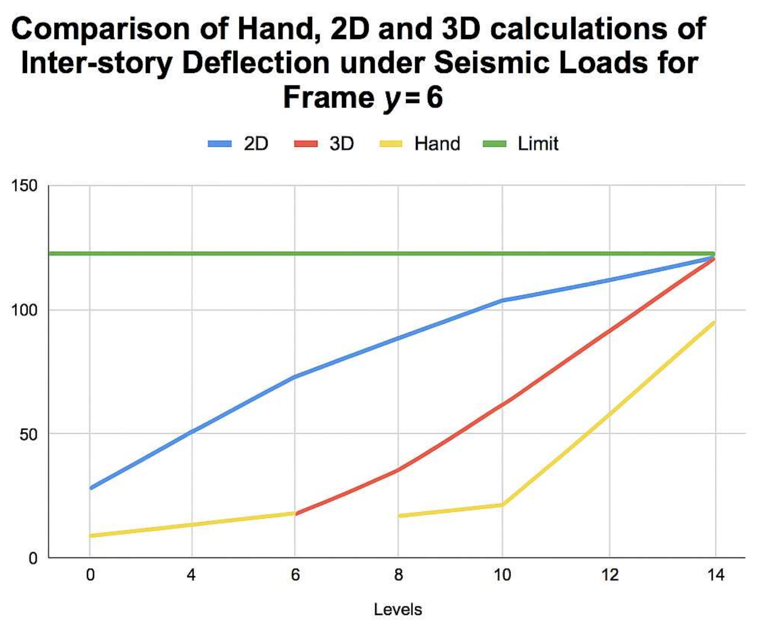


Figure 4.35. Comparison of Hand, 2D, 3D calculations of Inter-story Deflection under Seismic Loads

The 2D analysis (blue line) shows a steady increase in deflection to the highest of the methods. The 3D analysis (red line) starts low but rises steeply at higher levels, crossing the 2D values at the end. The hand calculations (yellow line) show lower deflections at the base and then a steep rise at higher levels, while the green line is a deflection limit that is almost reached but not crossed.

4.5.4. Internal Forces calculations.

4.5.4.1. Axial Force

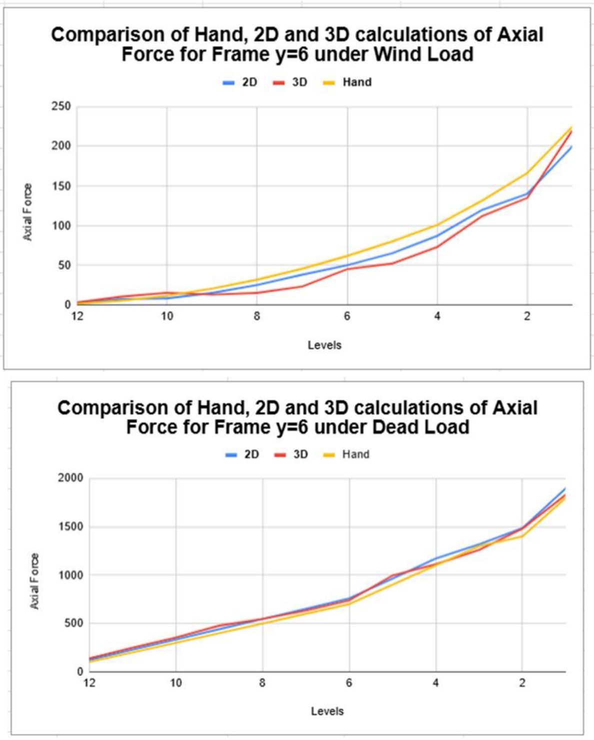
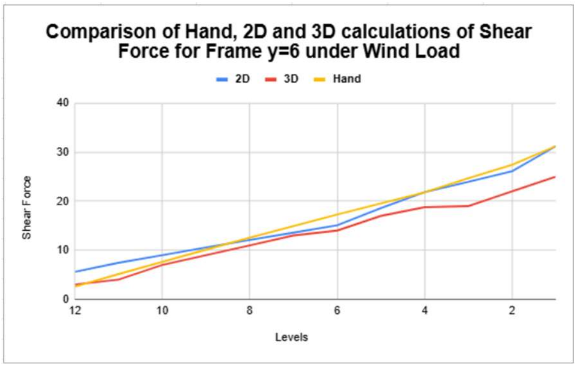


Figure 4.36. Axial Force results for Wind and Dead Loads

Under wind load (left graph), the axial force increases steadily with height, hand calculations (yellow) slightly overestimating values compared to the 2D (blue) and 3D (red) approaches. The differences between methods are minimal, though hand calculations show slightly higher forces at the higher levels. Under dead load (right graph), axial forces are much higher, with values approximately 2000 units, compared to a peak of around 250 in wind load. In this instance, the three methods (hand, 2D, and 3D) are quite close to one another with insignificant difference between them. Dead load results in general have more linear and constant force distribution, and wind load forces vary, particularly towards upper levels.

4.5.4.2. Shear Force



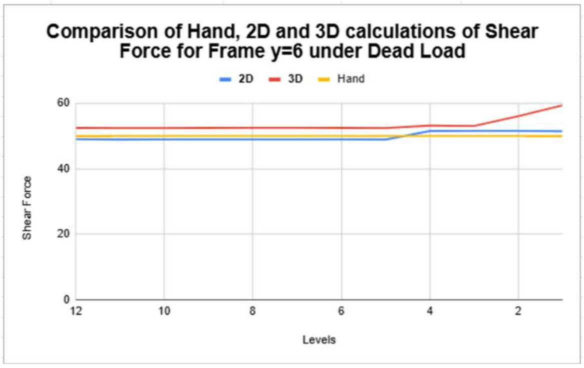
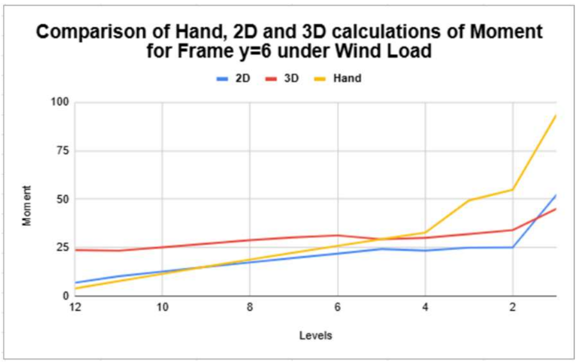


Figure 4.37. Shear Force results for Wind and Dead Loads

Under wind loading (left graph), the shear force increases linearly with height, with the hand calculations (yellow) predicting slightly higher values compared to the 2D (blue) and 3D (red) methods. The 3D analysis is predicting a lower shear force in the upper levels, while the 2D and hand methods have the same tendency. Under dead load (right graph), the shear forces are distributed more evenly across levels, and their magnitude is relatively constant in the range 50–60 units. 2D (blue) and hand (yellow) calculations are near one another, while 3D (red) analysis predicts slightly larger shear forces for the lower levels. Unlike for the wind load case, when the forces continually increase progressively, the dead load results show hardly any variation, which indicates more evenly distributed load effect.

4.5.4.3. Moment



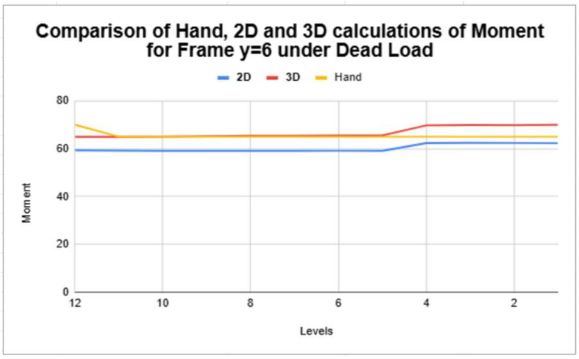


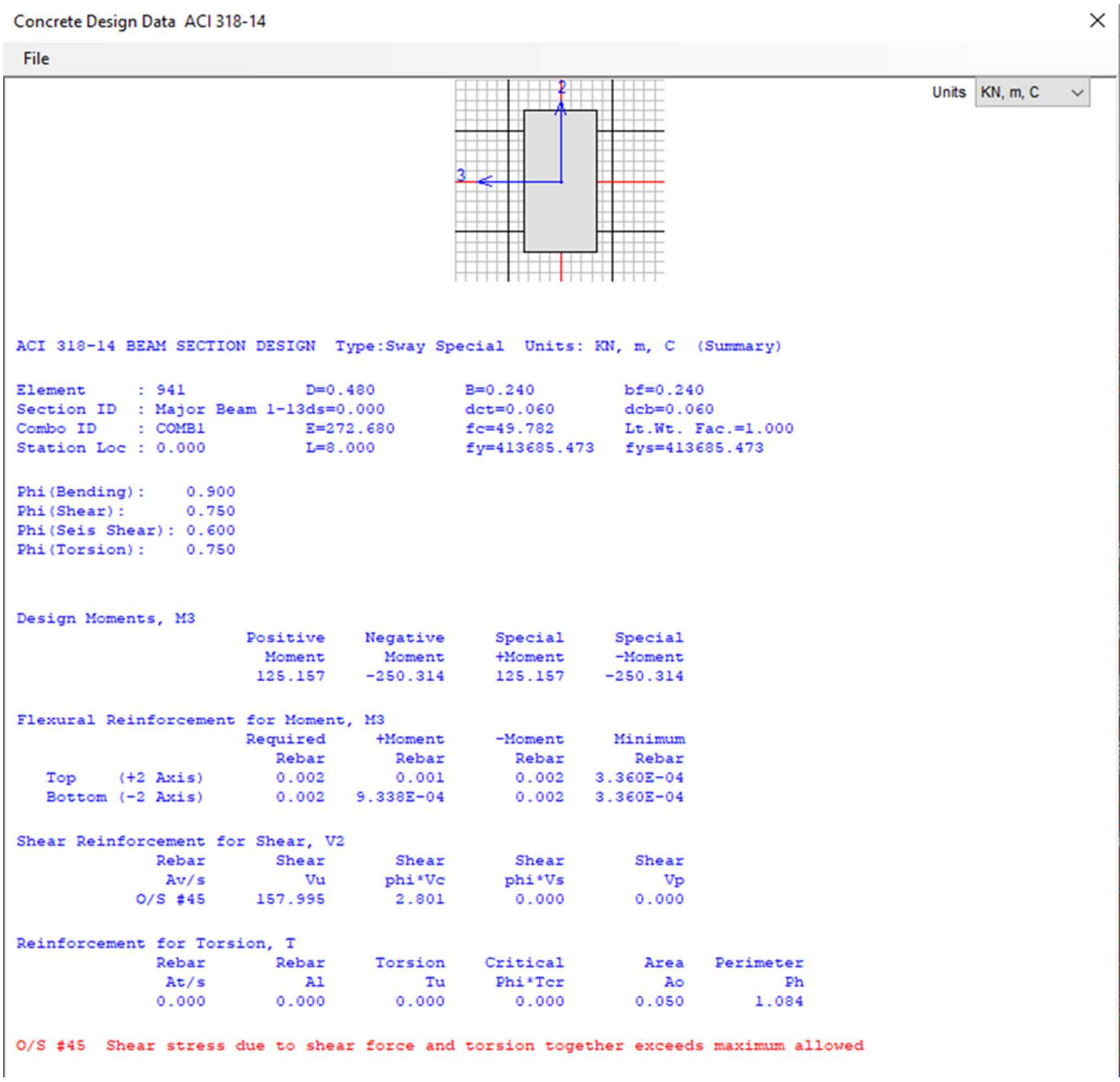
Figure 4.38. Moment results for Wind and Dead Loads.

Under wind loading (left figure), moments are increasing with elevation, with hand calculations (yellow) predicting very large moments at the upper floors compared to the 2D (blue) and 3D (red) methods. The 3D method predicts the lowest values of moments, and the 2D method follows an intermediate trend below the hand calculations. Under dead load (right graph), the moment remains quite uniform across all levels at around 60–80 units. The hand, 3D, and 2D calculations are very close to one another with minimum deviation. Contrary to what happens under the wind load condition, except a sudden spike toward the higher levels, the results of the dead load show the same force application across the complete structure.

4.6. Structural member design

4.6.1. Reinforcement Design

1. **Major Beam**



Concrete Design Data ACI 318-14

File

Units KN, m, C

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

Element      : 941             D=0.480         B=0.240         bf=0.240
Section ID   : Major Beam 1-13ds=0.000         dct=0.060       dcb=0.060
Combo ID     : COMB1           E=272.680       fc=49.782       Lt.Wt. Fac.=1.000
Station Loc  : 0.000           L=8.000         fy=413685.473   fys=413685.473

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


Design Moments, M3
                        Positive     Negative     Special      Special
                          Moment       Moment     +Moment      -Moment
                         125.157     -250.314     125.157     -250.314

Flexural Reinforcement for Moment, M3
                        Required     +Moment      -Moment      Minimum
                           Rebar       Rebar        Rebar        Rebar
   Top    (+2 Axis)        0.002       0.001        0.002    3.360E-04
   Bottom (-2 Axis)        0.002   9.338E-04        0.002    3.360E-04

Shear Reinforcement for Shear, V2
                   Rebar       Shear       Shear       Shear       Shear
                    Av/s          Vu     phi*Vc      phi*Vs          Vp
                 O/S #45     157.995       2.801       0.000       0.000

Reinforcement for Torsion, T
                   Rebar       Rebar     Torsion    Critical        Area   Perimeter
                    At/s          Al          Tu     Phi*Tcr          Ao          Ph
                   0.000       0.000       0.000       0.000       0.050       1.084

O/S #45  Shear stress due to shear force and torsion together exceeds maximum allowed
```

- ***Negative moment calculations***

$$M_u = 250.314\ kN - m$$

$$b = 400\ mm, h = 600\ mm, d = 600 - 2.5 * 25.4\ mm = 536.5\ mm$$

$$f'_c = 35\ MPa;\ f_y = 520\ MPa$$

$$R_n = \frac{M_u}{\Phi b d^2} = 2174.13\ kN/m^2$$

$$\rho = \frac{0.85 f'_c}{f_y}[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}] = 0.00435$$

$$A_s = \rho b d = 0.00435 * 400 * 536.5 = 932.67\ mm^2$$

$$A_s f_y = 0.85f'_c ab$$

$$a = \frac{A_s f_y}{0.85f'_c b} = 40.76\,mm$$

$$\beta_1 = 0.85 - 0.05(\frac{5076 - 4000}{1000}) = 0.796$$

$$c = \frac{a}{\beta_1} = 51.21\,mm$$

$$\varepsilon_t = 0.003(\frac{d - c}{c}) = 0.003(\frac{536.5 - 51.21}{51.21}) = 0.028 > 0.005$$

$$\varepsilon_t > 0.005 \rightarrow tension\ controlled\ and\ ductile$$

Hand calculated A_s is different from the value of SAP2000, that is why the error should be also calculated. The A_s value of SAP2000 is $0.002\ m^2$ which is equal to $2000\ mm^2$.

$$Error\ = \frac{2000 - 1865.873}{2000} * 100\% = 6.7\%$$

At first it can be seen as a huge error, but in fact it is not. The difference can be explained by the concrete cover assumptions and also the software calculates both compression and tension reinforcements. Next we can calculate the minimum steel area.

Minimum steel area

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 835.62\,mm^2 \geq 731.42\,mm^2$$

$$A_s = 2000\,mm^2 > 835.62\,mm^2 \rightarrow sufficient\ reinforcement$$

Based on above calculations, we can use the SAP2000 area values for all beams since the error is not too big and all areas more than minimum steel area. At the negative moment locations, 5#5 bars ($A_s = 1.53\ in^2$) will be used.

- ***Positive moment calculations***

$$M_u = 125.157\,kN - m$$

$$b = 400\,mm, h = 600\,mm, d = 600 - 2.5 * 25.4\,mm = 536.5\,mm$$

$$f'_c = 35\,MPa;\ f_y = 520\,MPa$$

$$R_n = \frac{M_u}{\Phi bd^2} = 1087.07\,kN/m^2$$

$$\rho = \frac{0.85f'_c}{f_y}[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}] = 0.00213$$

$$A_s = \rho bd = 0.00213 * 400 * 536.5 = 457.13\, mm^2$$

$$A_s f_y = 0.85 f'_c ab$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = 19.98\, mm$$

$$\beta_1 = 0.85 - 0.05(\frac{5076 - 4000}{1000}) = 0.796$$

$$c = \frac{a}{\beta_1} = 25.1\, mm$$

$$\varepsilon_t = 0.003(\frac{d - c}{c}) = 0.003(\frac{536.5 - 51.21}{51.21}) = 0.061 > 0.005$$

$$\varepsilon_t > 0.005 \rightarrow tension\ controlled\ and\ ductile$$

Hand calculated A_s is different from the value of SAP2000, that is why the error should be also calculated. The A_s value of SAP2000 is $500\, mm^2$.

$$Error\ = \frac{500 - 457{,}13}{1000} * 100\% = 4.27\%$$

The same explanation as in negative moments can be applied to this case also.

Next we can calculate the minimum steel area.

Minimum steel area

$$A_{s,min} = 3\sqrt{f'_c} bd/f_y \geq 200bd/f_y = 608.18\, mm^2 \geq 569.09\, mm^2$$

$$\rightarrow sufficient\ reinforcement$$

Based on above calculations, we can use the SAP2000 area values for all beams since the error is not too big and all areas more than minimum steel area. At the negative moment locations, 3#5 bars ($A_s = 0.94\, in^2$) will be used.

Shear analysis

$$V_u = 157.995\, kN$$

$$\lambda = 1\ \ for\ normal\ weight\ concrete$$

$$\Phi V_c = \Phi * 2\lambda \sqrt{f'_c} b_w d = 0.75 * 2 * 1 * \sqrt{35000} * 400 * 536.5 = 60.22\, kN$$

$$V_u > V_c \rightarrow shear\ reinforcement\ is\ needed$$

$$V_{c1} = 4\sqrt{f'_c} b_w d = 160.59\, kN$$

$$V_{c2} = 8\sqrt{f'_c} b_w d = 321.18\, kN$$

$$V_s = \frac{V_u - \Phi V_c}{\Phi} = 130.36 < V_{c1}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = 0.000467\ m^2/m$$

Torsional analysis

Since this design is also subject to seismic forces, the torsion should be also checked. From the same figure, $T_u = 0.766\ kN - m$.

$$A_{cp} = 0.350 * 0.700 = 0.245\ m^2$$

$$P_{cp} = 2(0.35 + 0.7) = 2.1\ m$$

$$T_n = \Phi\lambda\sqrt{f'_c}\left(\frac{{A_{cp}}^2}{P_{cp}}\right) = 4.2875\ kN - m$$

Since $T_u < T_n \rightarrow torsional\ reinforcement\ is\ not\ required$

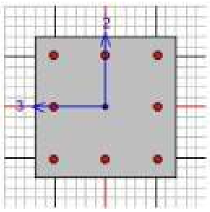
Stirrups spacing: $S_1 = \frac{A_v f_{yt}}{V_s} = 0.764\ m\ \rightarrow \#3\ stirrups$

$$S_2 = \frac{d}{2} \leq 24\ in.\ if\ V_s \leq V_{c1} = (4\sqrt{f_c'})b_w d$$

$$S_3 = \frac{A_v f_{yt}}{50 b_w} \geq \frac{A_v f_{yt}}{0.75\sqrt{f_c'} b_w}$$

$$S = S_2 = 0.3\ m \rightarrow \#3\ stirrups\ at\ 300\ mm\ spacing$$

2. Column




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

Element     : 215              B=500,        D=500,        dc=67,026
Section ID  : Column 19-21     E=30,442      fc=0,04       Lt.Wt. Fac.=1,
Combo ID    : 1.2D+1L-1Ex      L=3000,       Fy=0,414      fys=0,414
Station Loc : 3000,            RLLF=1,

Phi(Compression-Spiral):  0,75       Overstrength Factor:  1,25
Phi(Compression-Tied):      0,65
Phi(Tension Controlled):  0,9
Phi(Shear):                          0,75
Phi(Seismic Shear):            0,6
Phi(Joint Shear):                0,85


AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                     Rebar      Design      Design      Design     Minimum     Minimum
                      Area          Pu          M2          M3          M2          M3
                     2500,     499,458  111300,733  180803,068   15103,604   15103,604

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                                    Cm   Delta_ns    Delta_s           K           L
                                Factor     Factor     Factor      Factor      Length
   Major Bending(M3)               0,4         1,         1,          1,       3000,
   Minor Bending(M2)               0,4         1,         1,          1,       3000,

SHEAR DESIGN FOR V2,V3
                                 Rebar      Shear      Shear       Shear       Shear
                                  Av/s         Vu    phi*Vc      phi*Vs          Vp
   Major Shear(V2)               0,476    109,055    156,194      51,16          0,
   Minor Shear(V3)                  0,     75,286    156,194         0,          0,
```

Column design using ACI 318-19

P_u =499 kN $\qquad$ M_u =111.3 kN-m

$A_{g,req} = \frac{P_u}{0.5*f'c}$ =0.02079m^2 $\qquad$ $A_g = 0.7*0.7 = 0.49m^2$

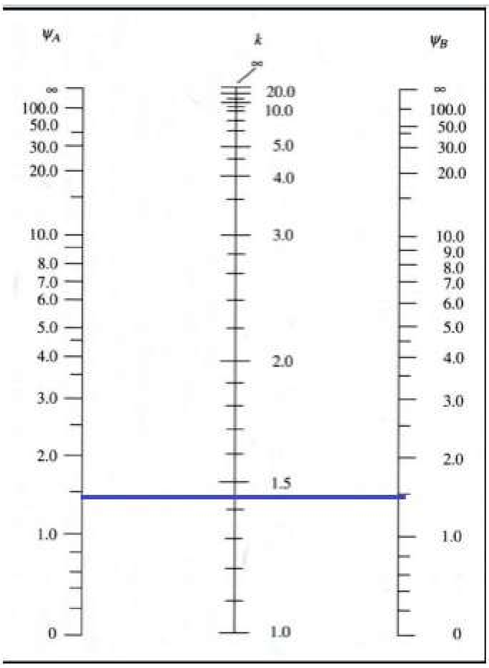
$e = \frac{M_u}{P_u} = 0.226m$ $\qquad$ $P_n = \frac{P_u}{\phi} = 767.69\ kN$

$K_n = \frac{P_n}{A_g * f'c} = 0.039$ $\qquad$ $R_n = \frac{P_n * e}{f'c * A_g * h} = 0.022$

From diagram p_g =0.042

$A_g = 0.042 * 0.49 = 0.02058m^2$(required area of steel reinforcement)

Slenderness ratio calculations



Interior Column 0.5x0.5

P_u =499 kN I_{column} = *0,00362 mm*^{4} I_{beam} = *0.005 mm*^{4}

$\Psi = \frac{\frac{\sum EI}{I_{column}}}{\frac{\sum EI}{I_{beam}}}$ =0.73 K=1.22 $\frac{Kl_u}{r}$ =18.81<22 short column

Corner Column

$\Psi = \frac{\frac{\sum EI}{I_{column}}}{\frac{\sum EI}{I_{beam}}}$ =1.44 K= 1.38 $\frac{Kl_u}{r}$ =20.8<22 short column

Exterior Column

$\Psi = \frac{\frac{\sum EI}{I_{column}}}{\frac{\sum EI}{I_{beam}}}$ = ***0.91*** K=1.2 $\frac{Kl_u}{r}$ =18.7<22 short column

The obtained results demonstrate that the slenderness ratio stays below maximum allowable thresholds. It means column needs to be designed as short with both axial and bending.

moments.

Axial and moment analysis

P_u =499 kN $\quad P_n = \frac{P_u}{0.65}$=767.69 kN

M_u =111.3 kN-m $\quad M_n = \frac{M_u}{0.65} = \mathbf{171.23}\ kN - m$

$e = \frac{M_n}{P_n} = 0.223\ m \quad K_n = \frac{P_n}{A_g * f'c} = \mathbf{0.0767}$

$R_n = \frac{K_n * e}{h} = \mathbf{0.0342}$ from diagram p_g =0.01

$A_s = 0.01 * 0.25 = 0.0025\ m^2$

Selected Bar : 8#8 bars ($A_s = 0.004052\ m^2$)

Shear strength check

V_{u1} =109.055 kN $\quad V_{u2} = 75.286\ kN$
$V_c = 423.5\ kN \quad V_{u1} > \frac{\varphi * V_c}{2}$ $\rightarrow$#4 ties at 190 mm

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}b_w d$$

Shear analysis

$$V_c = 0.17 * (1 + \frac{P_u}{14 * A_g}) * \lambda * f'_s{}^{\frac{1}{3}} * b_w * d = \mathbf{1025.26.}\ kN$$

0.65* ***1025.26***= 666.42 kN

$\frac{0.65*1025.26}{2} = \mathbf{333.21}\ kN > V_u = \mathbf{109.055}$ minimum reinforcement is required!
$48 * d_t = 0.4572\ m \quad 16 * d_t = 0.3556\ m$

We need use 0.3* 0.3 as the minimum column dimensions.

Biaxial bending check

$M_{ux} = 180.08\ kN - m \quad M_{nx} = \frac{M_{ux}}{0.65} = \mathbf{277.05}\ kN - m$
M_{uy} =111.30 $kN - m \quad M_{ny} = \frac{M_{uy}}{0.65} = \mathbf{171.23}\ kN - m$

$\gamma_x = \gamma_y = 0.85$; $P_{nx} = 20402\ kN$; e_x =0.462 $\quad e_y$ =0.286

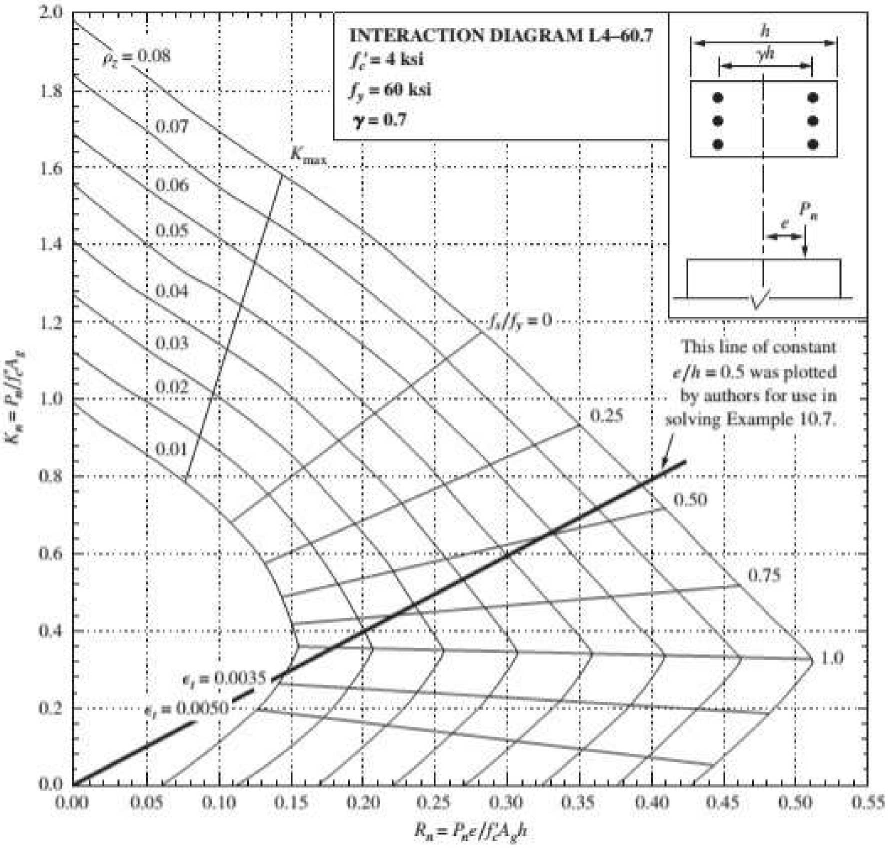


Figure 4.39. Interaction diagram for rectangular column

$P_{nx} = \frac{K_{nx} * f_s * A_g}{\Phi} = 19856\ kN$; $\qquad P_{ny} = \frac{K_{ny} * f_s * A_g}{\Phi} = \boldsymbol{10528}\ kN$

$P_{n0} = 0.85 * f'_c * A_g + A_{st}(f_y - \boldsymbol{0.85}\, f'_c) = \boldsymbol{8357}\ kN$

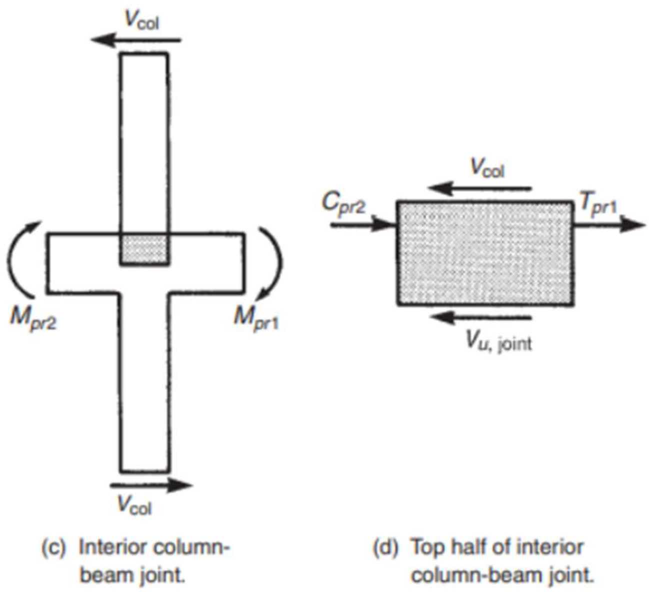
$$\frac{\boldsymbol{1}}{P_n} = \frac{\boldsymbol{1}}{P_{nx}} + \frac{\boldsymbol{1}}{P_{ny}} - \frac{\boldsymbol{1}}{P_{n\boldsymbol{0}}}$$

$P_n = \boldsymbol{38930}\ kN > \boldsymbol{0.1} P_{no}$ it is ok! 0.65* 38930= 25305 kN > P_u

The column is safe against biaxial bending.

Joint design

The joint design was completed in compliance with ACI 318-19 Chapter 15. The comprehensive computation process adhered to the methods described in Wight and MacGregor's design book (Wight & MacGregor, 2009). Figure shows the internal beam-column joint's free body diagram.



Beam-column joint free body diagram.

An interior beam-column joint shear was calculated using the following equation:

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

$$T_{pr1} = \alpha A_s f_y = 1.25 * 1935.48 * 0.42 = 1016.127\ kN$$

$$C_{pr2} = \alpha A_s f_y = 1.25 * 851.6 * 0.42 = 447\ kN$$

$$V_{col} = 417\ kN \rightarrow From\ SAP2000$$

$$V_{u,joint} = 1016.127 + 447 - 417 = 1046\ kN$$

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

$$V_n = \gamma \sqrt{f'_c} A_j = 1.7 * \sqrt{40} * 4125 = 4435.1\ kN$$

$$\Phi = 0.85 \rightarrow strain\ hardening\ of\ the\ reinforcement$$

$$\Phi V_n \geq V_u \rightarrow 0.85 * 4435.1 = 3769.8 \geq 1046\ kN$$

As we can see, the joint is safe against shear forces. To check for other directions, the same procedure can be repeated. According to ACI 15.4.2, the spacing requirement can be checked.

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

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

To meet this requirements, #3 ties at 300 mm spacing will be provided.

Reinforcement detailing

Bar selection and spacing

The final bars and spacing selected for major beams and columns are presented in Tables 4.83. – 4.85.

Table 4.83. Final selected bars for major beams

Floor	Top reinforcement	Bottom reinforcement	Stirrups
1	5#7	3#6	#3 @ 300 mm
2	5#7	2#8	#3 @ 300 mm
3	5#7	2#8	#3 @ 300 mm
4	5#7	2#8	#3 @ 300 mm
5	3#9	5#5	#3 @ 300 mm
6	3#9	5#5	#3 @ 300 mm
7	3#9	3#6	#3 @ 300 mm
8	3#9	3#6	#3 @ 300 mm
9	3#9	3#6	#3 @ 300 mm
10	3#7	3#7	#3 @ 300 mm
11	5#5	2#7	#3 @ 300 mm
12	2#8	2#7	#3 @ 300 mm

Figure 4.84. Interior slab panel reinforcement bars.

Interior	
Bar selection	3#6
Spacing (mm)	800

Table 4.85. Exterior slab panel reinforcement bars.

Exterior

Bar selection	3#6	4#4	6#4	4#4	4#4	4#4
Spacing (mm)	500	400	400	400	400	400

Table 4.86. Final selected bars for columns.

Floor	$a = b\ (m)$	Reinforcement
0-2	0.75	12#8
3-4	0.65	6#10
5-6	0.55	8#7
7-8	0.50	8#7
9-10	0.40	8#6
11-12	0.30	8#4

Development length

ACI 318-19 provides the method to determine reinforcing bar development lengths. The equation designed to determine tension bar development length is:

$$\frac{l_d}{d_b} = \frac{f_y \Psi_t \Psi_e \Psi_g}{20\lambda\sqrt{f_c'}} \quad (for\ bars\ larger\ than\ \#7)$$

Taking into consideration a certain conditions the development length of required bars in tension was calculated:

$$1)\ Clear\ cover = 2.5 - \frac{0.875}{2} = 2.0625\ in > d_b$$

$$2)\ Clear\ spacing\ of\ bars = 2.05\ in > d_b$$

$$3)\ minimum\ \#3\ stirrups\ were\ arranged$$

$$l_d = 875\ mm > 750\ mm\ (column\ width) \rightarrow Hooked\ bars$$

Development length for hooked bars can be found via next equation:

$$l_{dh} = \left(\frac{f_y \Psi_e \Psi_r \Psi_o \Psi_c}{55\lambda\sqrt{f_c'}}\right) {d_b}^{1.5}\ (modification\ factor)$$

$$l_{dh} = 205\ mm > 8d_b = 177.8\ mm.$$

Subsequently, the 90° hook's parameters had to be determined:

$$D = 6d_b = 133.35\ mm, r = D/2 = 66.675\ mm$$

$$The\ distance\ of\ the\ hook: 12d_b = 266.7\ mm$$

In addition, stirrups will be placed along the development length at intervals of $\leq 3d_b$.

To calculate the development length for bars under compression, the following equation is applied:

$$l_{dc} = (\frac{f_y \Psi_r}{50\lambda\sqrt{f_c'}})d_b \geq 0.0003 f_y \Psi_r d_b$$

Subsequently, the development length for bars under compression is:

$$l_{dc} = 250\ mm < 286\ mm$$

Lap slices

For the beams in tension:

$$l_{st} = l_{dh} = 205\ mm$$

For beams in compression:

$$l_{sc} = 0.0005 f_y d_b = 476\ mm$$

For columns:

As an example, column with $d_b = 0.875\ m$

$$l_{sc} = 0.0005 f_y d_b = 667\ mm$$

Table. 4.87.

Floor	Reinforcement	d_b, in	Lap splices, mm
0-2	12#8	1	762
3-4	6#10	1.27	967.74
5-6	8#7	0.875	667
7-8	8#7	0.875	667
9-10	8#6	0.75	571.5
11-12	8#4	0.5	381

Special seismic provisions for reinforcement detailing

Beams

Since the building is in a high seismic risk area, special seismic detailing was included in the design. The beam's clear span and its width-to-depth ratio meet the requirements of Section 18.6 in ACI 318-19. To ensure strength and stability, at least two continuous longitudinal bars were placed at both the top and bottom of the beam, as required by Section 18.6.3.1. The beam was also designed so that the positive moment capacity at the joint face is more than half of the negative moment capacity. For added shear strength and ductility, hoops were installed over a length equal to twice the beam depth from the face of the support. The spacing of these hoops followed code limits—not exceeding d/4, 6 times the bar diameter (db), or 150 mm.

Columns

Seismic requirements for the columns were addressed following Section 18.7 of ACI 318-19. The reinforcement area in the columns was kept within the required range of 1-6 % of the cross-sectional area. Transverse reinforcement was placed on both sides of the joint, extending a length l_0, which was taken as one-sixth of the column's clear span. All other relevant provisions from Section 18.7 were also considered during the design process to ensure the columns meet seismic performance standards.

Joints

The longitudinal reinforcement in the beams was extended all the way to the far face of the column at each joint. In joints where the reinforcement continued through the column, the column depth hhh was designed to be greater than 20 times the bar diameter ($20d_b$) to ensure proper anchorage. Additionally, the development lengths of the bars were checked and confirmed to meet the required criteria:

$$l_{dh} \geq \begin{cases} \dfrac{f_y d_b}{65\lambda\sqrt{f'_c}} \\ 8d_b \\ 6 \text{ in.} \end{cases}$$

Structural serviceability design

Deflection

Building structures need to be checked for deflection because it affects their serviceability. Different members must follow specific tabulated minimum thickness demands outlined in building codes. Deflection checks become redundant when the specified minimum thickness requirements for structural members are achieved or surpassed (Hassoun & Manaseer, 2020). The necessary minimum beam thickness for $f_y = 60000\,psi\ (413.7\,MPa)$ can be calculated through the following equation.

$$h_b = L/21$$

$$where,$$

$$L - span\ length\ (m)$$

For our case, the minimum thickness of the beam will be:

$$h_b = \frac{7000}{21} = 333\,mm < 350\,mm\ (actual\ depth\ of\ the\ beam)$$

As it can be seen, the selected beam member surpasses minimum requirements therefore the need for deflection calculations becomes unnecessary.

Crack width

The design of reinforced concrete structures depends fundamentally on crack control measures because these measures guarantee fundamental structural safety and durability together with reliability. The quantity of flexural cracking creates durability problems and visual concerns along with the deterioration of reinforcement materials. It is necessary to test that structural cracks stay within the boundaries defined by design standards. This empirical formula enables the assessment of maximum flexural crack width:

$$w = 0.076\beta_h f_s \sqrt[3]{d_c A}$$

$$where,$$

$$w - estimated\ crack\ width\ (in)$$

$$\beta_h - ratio\ of\ the\ depth\ to\ the\ neutral\ axis\ and\ the\ cover\ to\ reinforcement\ (1.2)$$

$$f_s - stress\ in\ the\ steel\ (ksi)$$

$$d_c - cover\ (in)$$

$$A - effective\ tension\ area\ (in^2)$$

$$w = 0.076 \times 1.20 \times 0.6 \times 60 \times \sqrt[3]{2.5 \times \frac{2.5 \times (350/25.4)}{3}} = 0.01\ in\ < 0.016\ in$$

This building meets the allowable crack width standard of 0.016 inches since estimated crack dimensions stay below this threshold. The observed crack width meets the requirements.

The formula below determines the maximum allowable spacing of longitudinal reinforcement in the design.

$$s\ (in) = [15(\frac{40}{f_s}) - 2.5C_c]$$

$$s\ (in) = [15(\frac{40}{0.6 \times 60}) - 2.5 \times 2.5] = 10.42\ in = 264.67\ mm$$

The maximum distance between longitudinal reinforcement bars should be set at 264.67 mm.

Lap splices

Beam:

$$l_{sc} = (0.0005 f_y d_b) \rightarrow l_{sc} = 476\ mm\ (compression)\ and\ l_{sc} = 205\ mm\ (tension)$$

Column:

$$l_{sc} = (0.0005 f_y d_b) \rightarrow l_{sc} = 667\ mm$$

3. Two-way Slab

Table 4.88. Summary of the exterior beam-supported panel design.

Exterior						
	Column Strip				Middle Strip	
M	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
%	96.98	75.00	75.00	3.02	25.00	25.00
new Mu (kN-m)	222.96	92.85	172.43	6.94	30.95	57.48
Mu slab (kN-m)	33.44	13.93	25.86	1.04	4.64	8.62
b	1.20	1.20	1.20	1.20	1.20	1.20
Rn (kPa)	3225.69	1343.24	2494.58	100.42	447.75	831.53
Ro	0.0066	0.0026	0.0050	0.0002	0.0009	0.0016
As (m^2)	0.00077	0.00031	0.00059	0.00002	0.00010	0.00019
As (in^2)	1.19950	0.48190	0.91437	0.03525	0.15812	0.29562
Asmin (in^2)	0.41850	0.41850	0.41850	0.41850	0.41850	0.41850
As final (in^2)	1.530	1.530	1.530	1.530	1.530	1.530
Bar Selected	5#5	5#5	5#5	5#5	5#5	5#5
Spacing (m)	0.5	0.5	0.5	0.5	0.5	0.5

Table 4.89. Summary of the interior beam-supported panel design.

Interior						
	Column Strip				Middle Strip	
M	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
%	75.00	75.00	75.00	25.00	25.00	25.00
new Mu (kN-m)	172.43	92.85	172.43	57.48	30.95	57.48
Mu slab (kN-m)	25.86	13.93	25.86	8.62	4.64	8.62
b	2.20	2.20	2.20	2.20	2.20	2.20
Rn (kPa)	1360.68	732.67	1360.68	453.56	244.22	453.56
Ro	0.0027	0.0014	0.0027	0.0009	0.00047	0.00088
As (m^2)	0.00058	0.00031	0.00058	0.00019	0.00010	0.00019
As (in^2)	0.89524	0.47671	0.89524	0.29368	0.15757	0.29368
Asmin (in^2)	0.76725	0.76725	0.76725	0.76725	0.76725	0.76725
As final (in^2)	0.920	0.920	0.920	0.920	0.920	0.920
Bar Selected	3#5	3#5	3#5	3#5	3#5	3#5
Spacing (m)	0.5	0.5	0.5	0.5	0.5	0.5

4.7. Structural serviceability design

4.7.1. Deflection

$$h_b = \frac{L}{21} = \frac{6000}{21} = 286\,mm < 300\,mm\ (minimum\ thickness\ of\ the\ beam)$$

So, the member dimensions selected exceed the minimum beam requirements because of which deflection calculations can be disregarded.

4.7.2. Crack Width

$d_c = 2.5$ $\quad A = \frac{5*20}{6} = 24.6\,in^2$

$$w = 0.076 * \beta_h * f_s * (d_c * A)^{\frac{1}{3}} = 0.013 < 0.016$$

The acceptable crack width for this building reaches 0.016 inches since estimates show actual crack widths remain below this amount.

5. Geotechnical Part

5.1 Site Location

Site where the building will be located is 211 Fairview Ave N, Seattle, WA 98109, USA. The geotechnical report of the site was done for the area of 320 Terry Ave, Seattle, WA 98109. The total number of borings in a geotechnical investigation is 8.

Investigation includes SPT and CPT tests for each of the borings and data of tests is written in accordance with Unified Soil Classification System (USCS). The soil profile is provided for a depth of 21 m and will be written in detail in further pages.

5.1.1 Site seismicity

The site is seismically active in accordance with ASCE. In order to prove the seismicity of the site, shear-wave velocity (V_{30}) was calculated and its graph was plotted.

The shear wave velocity formula for the chosen area is presented below:

$$V_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} t_i} \tag{5.1}$$

The shear wave velocity graph is presented below:

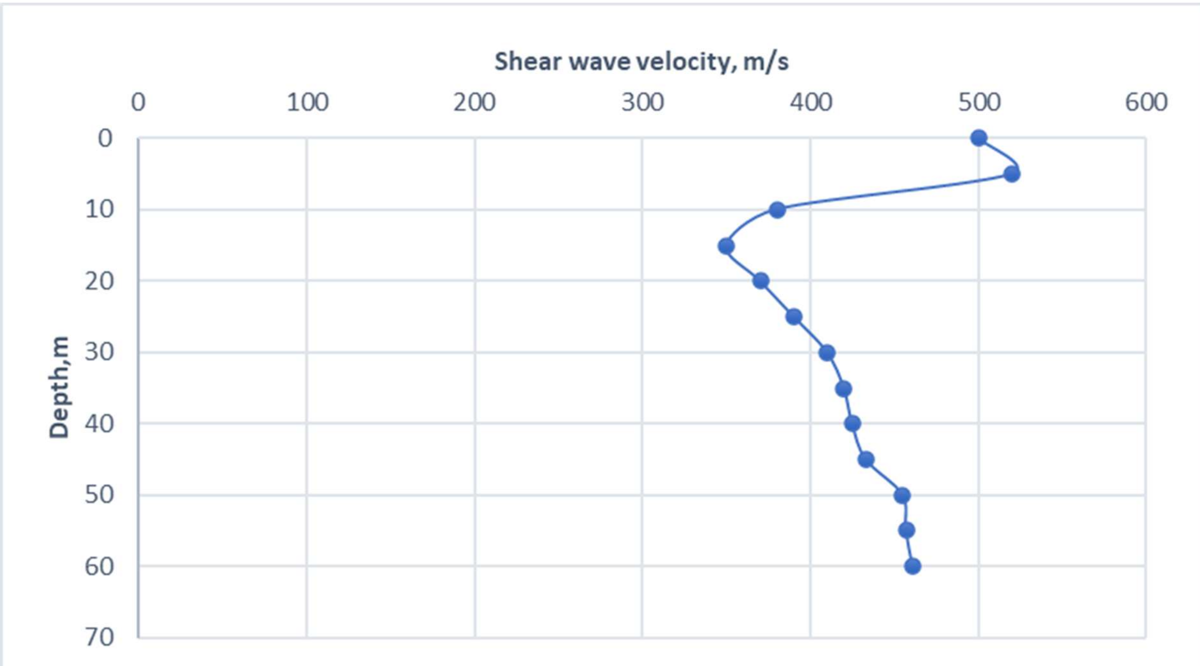


Figure 5.1. Shear-wave velocity

As it can be seen from the graph, the V_{30} for the chosen location is 373.27 m/s, so the site seismicity can be concluded as Site Class C and Seismic Design category D and Risk Category II, according the ASCE-7, Section 20.6

5.1.2 Liquefaction

The liquefaction potential at the location was calculated based on shear wave velocity-based methods, i.e., following the Andrus and Stokoe (2004) method. Groundwater level at the time of investigation at the site was 9.144 m below the ground surface but was not reached during boring. Loose to medium-dense sand exists above the groundwater level, and dense to very dense sand and gravel exist beneath it. Due to the fact that the lower soil layers are very dense, it was assumed liquefaction did not occur. This was checked with the calculation of cyclic stress ratio (CSR), stress-corrected shear wave velocity, and cyclic resistance ratio (CRR). The average velocity of the shear wave of the site was computed to be 373.27 m/s, and the site was categorized as Site Class C (according to ASCE-7) and Seismic Design Category D with Risk Category II. The ultimate liquefaction analysis indicated that the factor of safety against liquefaction was greater than 1 in every layer of soil, and liquefaction at the site is not expected.

Table 5.1. Liquefaction results

Z (m)	ΔH (m)	USCS	σ_0' (kN/m^2)	V_{s1} (m/s)	CSR	CRR	r_d	FS
$0-2.44$	2.591	SM (Fill)	51.82	439.95	0.0362	0.4776	0.9802	13.20
$2.44-5.79$	3.35	SM	85.425	388.26	0.0566	0.361	0.9545	6.383
$5.79-6.4$	0.61	ML	14.335	606.63	0.3707	0.941	0.9497	2.539
$6.4-7.92$	1.52	SM	38.57	473.65	0.1707	0.5604	0.938	3.283
$7.92-10.05$	2.13	SM	60.705	422.88	0.1375	0.438	0.901	3.185
$10.05-14.01$	3.96	SM	94.05	379.04	0.107	0.341	0.792	3.184
$14.01-18.58$	4.57	SW-SM	107.395	366.67	0.104	0.315	0.669	3.029
$18.58-20.71$	2.13	SP	50.055	443.77	0.225	0.4865	0.6084	2.159

5.2 Soil Profile

Using the 8 borehole information and the geotechnical report of the selected location soil profile was created. The following table shows the soil profile. Overall, the location has mostly sandy soil with little variations in characteristics.

Table 5.2. Soil profile

Depth,m	Soil Type	USCS	γ, kN/m^3	c', kN/m^2	φ, °	E, kPa	μ_s
$0 - 2.44$	Silty SAND	SM (Fill)	20	0	48.48	23784.74	0.276
$2.44 - 5.79$	Silty and Gravelly SAND	SM	25.5	0	34.31	33348.56	0.366
$5.79 - 6.4$	Sandy SILT with Gravel	ML	23.5	0	47.22	27500.00	0.465
$6.4 - 7.92$	Silty SAND	SM	25.4	4.64	47.34	26372.10	0.440
$7.92 - 10.05$	Silty SAND	SM	28.5	4.64	47.00	27062.36	0.456
$10.05 - 14.01$	Silty SAND	SM	23.8	4.64	42.39	12320.84	0.320
$14.01 - 18.58$	Well-graded SAND with Silt and Gravel	SW-SM	23.5	0	38.27	7673.19	0.299
$18.58 - 20.71$	Poorly-graded SAND with Gravel	SP	23.5	0	32.11	6003.56	0.276

5.3 Site response analysis

The location of the site is in the region with high seismic activity. Site response analysis was done in order to understand the behaviour of soil at the site under seismic loads. The strong motion data for the Nisqually earthquake was recorded by USGS National Strong-Motion Network and was used in the analysis, which was done in Plaxis 2D. The Nisqually earthquake is considered as one of the strongest earthquakes that happened in Seattle with a magnitude of 6.2 and depth of approximately 56 km. The input data for Plaxis 2D is shown below:

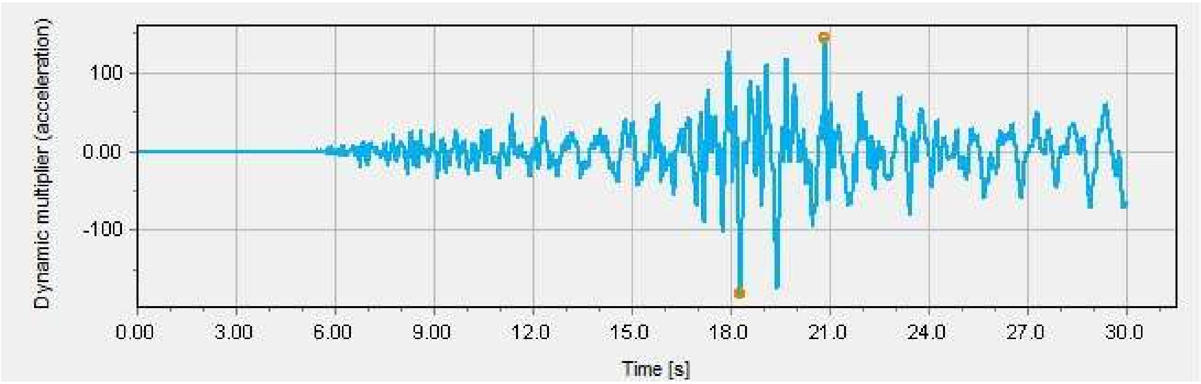


Figure 5.2. The strong motion data for Nisqually Earthquake

The model in Plaxis 2D consists of 2 phases, one of them is the initial phase and the other is the "Phase 1". The dynamic displacement, that is applied as the linear displacement along the x-axis is activated only at Phase 1. Dynamic time interval was chosen to be 60 second, as the length of the earthquake duration.

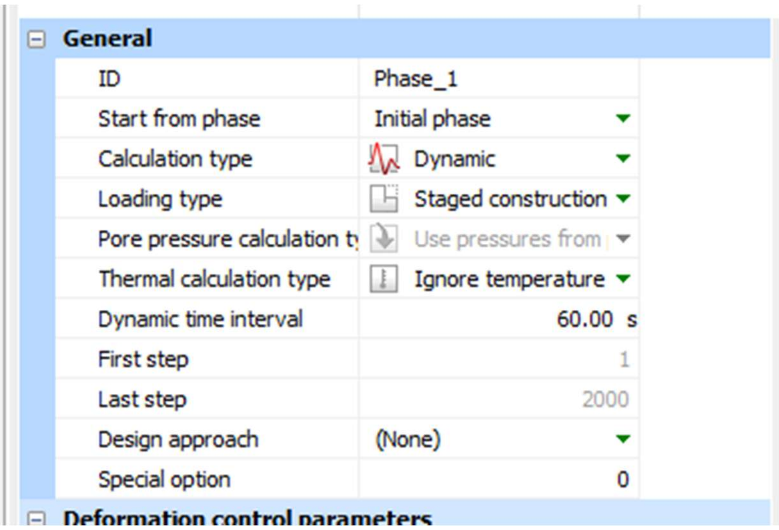


Figure 5.3. The chosen parameters for the 'Phase 1'

The results from the site response analysis are shown below. The maximum deformation of the soil is 24.94 mm.

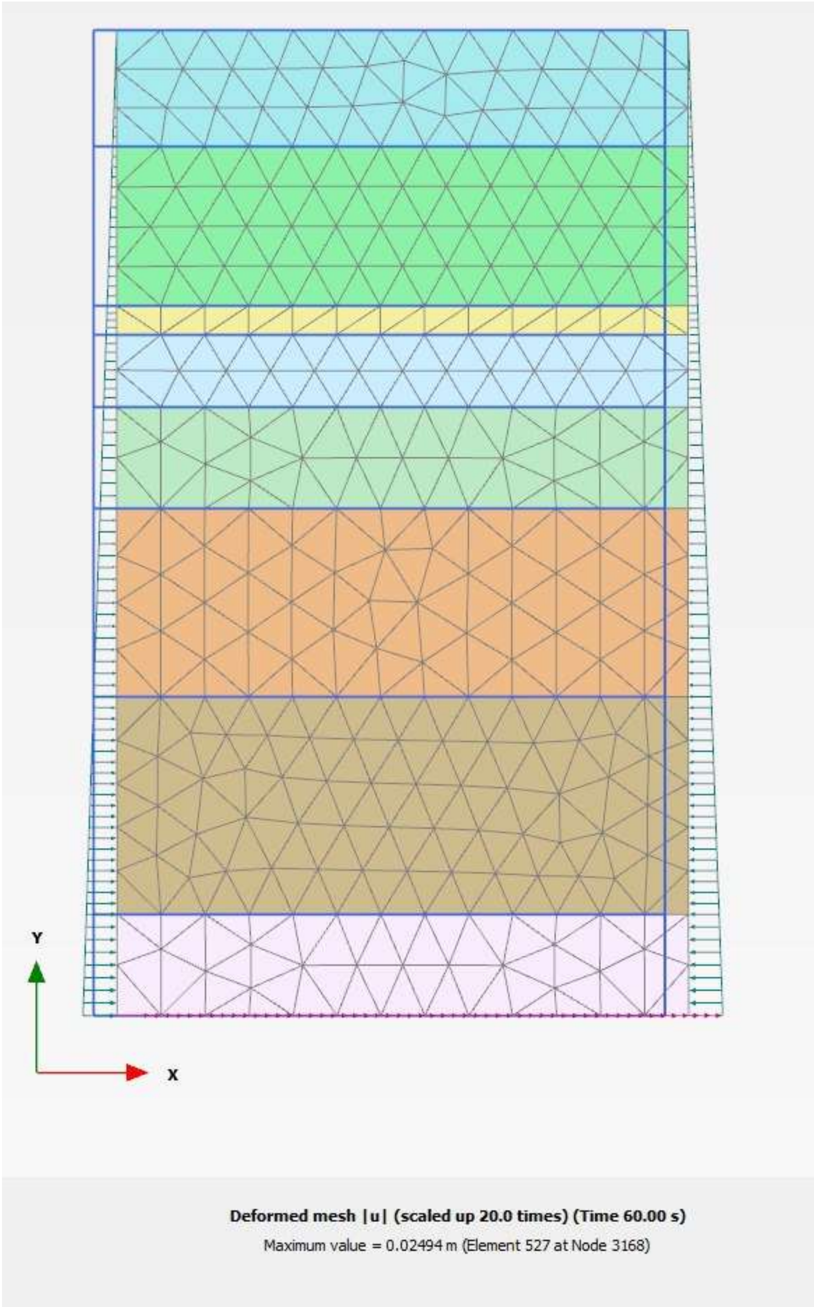


Figure 5.4. The deformation of the soil on the site

In addition, as shown in the figure below, the acceleration vs. time graph was plotted in order to get the strong motion data for this earthquake.

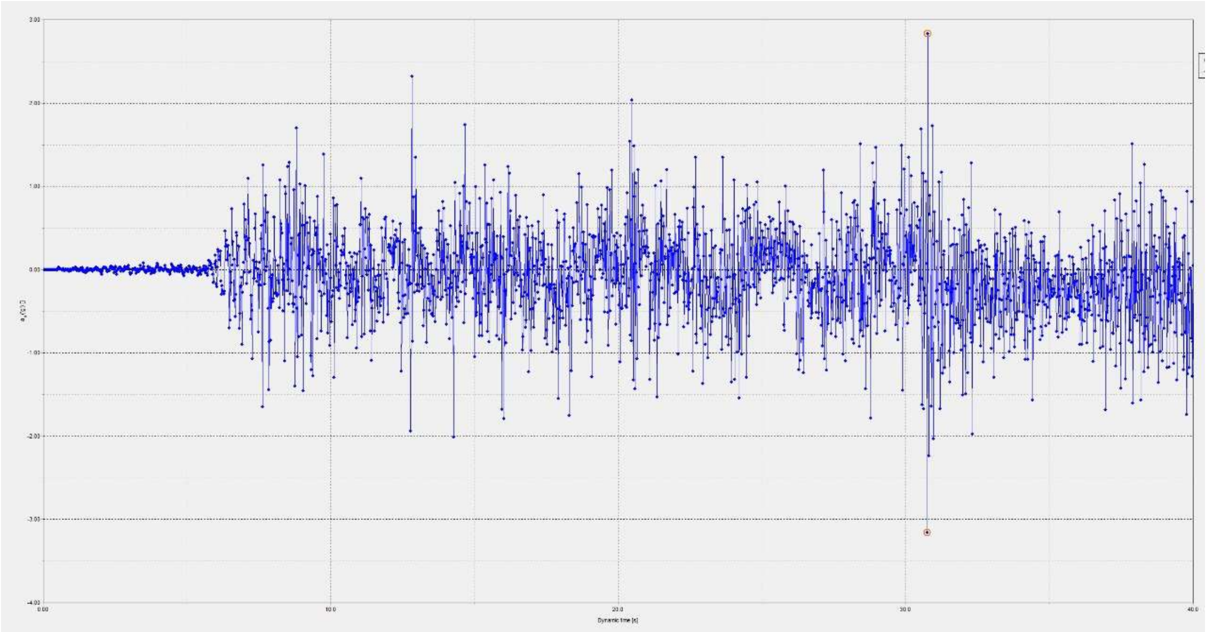


Figure 5.5. The accelerogram for Nisqually earthquake

Then, the plot of PSA vs period was also illustrated to demonstrate the peak acceleration values for different periods of time.

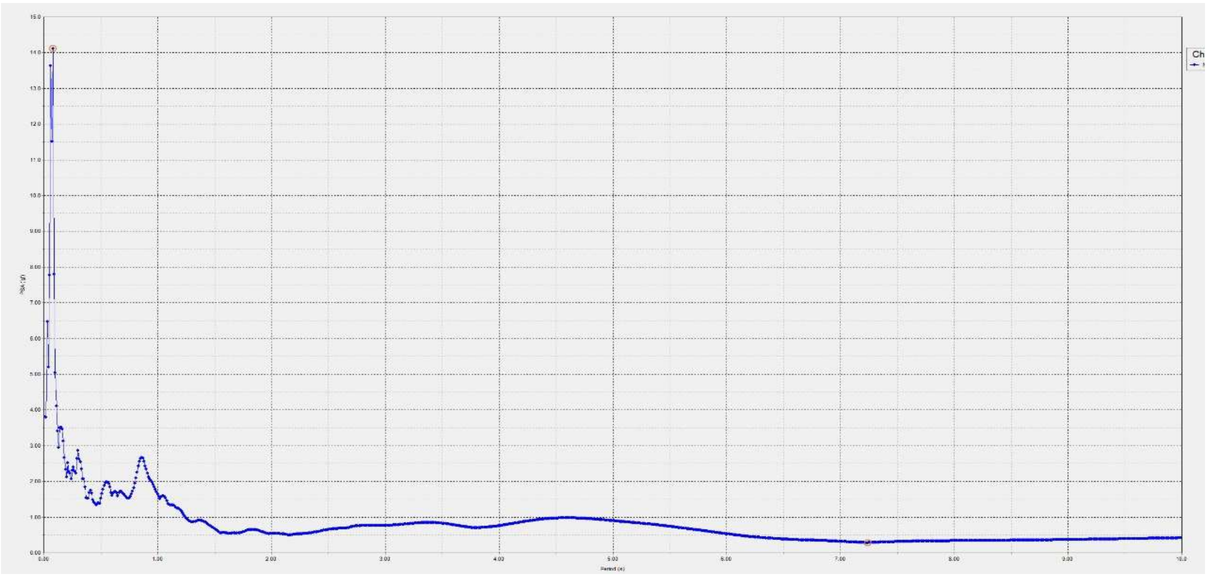


Figure 5.6. PSA vs. period graph

Finally, using the Fast Fourier transformation to change the acceleration vs. time plot, the Power density graph is also plotted, in order to understand which frequencies are more impactful in ground motion.

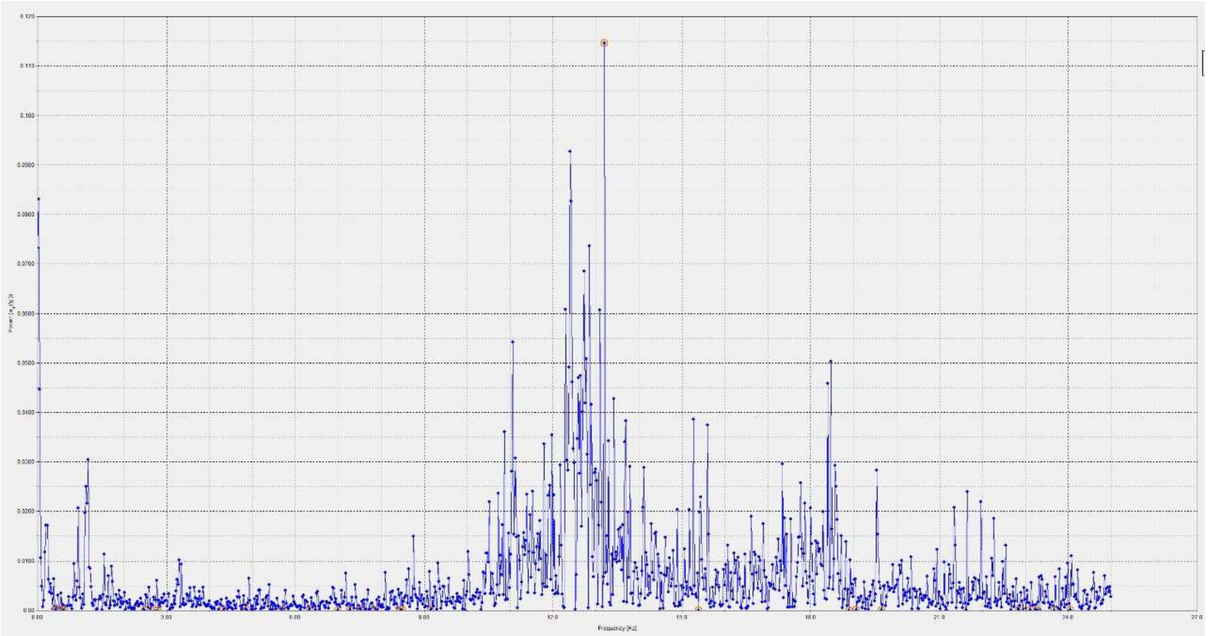


Figure 5.7. Power vs. Frequency plot

Overall, these plots are very helpful in helping to understand the ground motion at the surface of our site. Since our site is located in a high-seismic region, it is important to know to what extent acceleration and frequency values will increase and use that output data in designing our building. Further, when we design the pile layout and evaluate the deformations of pile groups, such as corner, exterior and interior, we will also assign the values of acceleration time history as the dynamic boundary condition in Plaxis 3D.

5.4 Foundation design under axial loading

5.4.1 Foundation selection

Foundation selection should be related to the performance characteristics of a proper foundation for various types of soil, since the structure should endure loads acting vertically and laterally, due to seismic actions and wind forces, in the structure's service life. In view of the described conditions, different foundation alternatives will be presented within this chapter, and the selection that was done, together with justification for this kind of final solution.

Three types of foundations were considered in light of the geotechnical and structural aspects. First to be considered was a pad foundation since it is simple and economical. Calculations showed that the dimensions of the pad foundation depended on the column load type, which had dimensions ranging from 2.027 m to 2.222 m. However, pad foundations proved inappropriate for seismic consideration because they cannot provide sufficient stability for lateral forces. Additionally, pad foundations would not distribute loads effectively for the structure of this scale and thus would lead to potential differential settlement.

Mat foundations are considered in support of pad foundations. Mat foundations offer better distribution of loads, and they are commonly used for high-rise buildings. These analyses were based on an embedded depth of 1.5 m and an overall basement depth of 3 m placing the mat foundation in the silty sand layer. However, this type of mat foundation was discarded due to bearing capacity and settlement reasons. Although the bearing capacity is adequate, settlement analysis has shown an excessive deformation which may lead to structural integrity loss. In addition, the high seismicity in the area made the lateral forces a relevant issue that a mat foundation would not have been able to effectively resist.

Given the inability of shallow foundations, the pile foundations were checked for being able to transfer the structural loads from the superstructure to deeper, more stable soil layers. In this respect, single piles and group pile systems were analysed. The calculations for single piles included both point-bearing capacity and skin friction evaluations. However, the bearing capacity of individual piles calculated was inadequate for column loads, while excessive settlement was realized in elastic settlement calculations. On the other hand, efficiency in distributing structural loads was assessed for a group pile system. Calculations showed that the capacity of bearing and settlement would be satisfied in the group pile system and, consequently, act as a single block for optimum load transfer from piles. From all the calculations, the group pile foundation was the best option that could give factual service to this project. The main reasons for choosing it are because of the increase in its carrying capacity, since the structural loads have been transmitted to deeper, more stable layers of soil with much effectiveness; seismic resistance through the provision of lateral stability against earthquake-induced forces; and thirdly, that the calculation of settlement showed a group pile foundation to be within acceptable limits, for it reduces structural deformation over a period. The interconnected nature of the pile system enhances stability and longevity, making it the most reliable choice.

Geotechnical investigation and structural considerations led further to the selection of a group pile foundation. Shallow foundations, including pad and mat foundations, were considered but excluded in the initial stages due to the inability to take seismic forces and other settlement issues. This makes the pile group system develop as an adequate solution in offering stability and safety for a structure during its lifespan.

5.4.2 Foundation design

5.4.2.1 Bearing capacity hand calculations

The efficiency of the group of piles was calculated to find if the group of piles acts as block or single pile. Its formula is as follows:

$$\eta = [\frac{2(n_1+n_2-2)d+4D}{pn_1n_2}] \tag{4.41}$$

Where,

d − minimum spacing between piles, m

D − diameter of piles, m

$n_{1,}n_2$ − number of piles in a group

p − perimeter of pile, m

Minimum spacing between piles is calculated as 2.5D. If the efficiency value is smaller than 1, the group acts as a block which means their bearing capacity is equal to ΣQ_u. If efficiency value is higher than one, then group acts as a single pile and their bearing capacity is equal to $n\Sigma Q_u$.

$$Q_{g(all)} = \frac{Q_{g(u)}}{FS} \tag{4.42}$$

Where,

$Q_{g(all)}$ − allowable bearing capacity of the group pile, kN

Q_u − ultimate bearing capacity of each pile, kN

$Q_{g(u)}$ − ultimate bearing capacity of the group pile, kN

FS − factor of safety

The dimensions of the pile cap is determined by the following formulas:

$$L_g = (n_1 - 1)d + 2(D/2) \tag{4.43}$$

$$B_g = (n_2 - 1)d + 2(D/2) \tag{4.44}$$

The dimensions of piles for exterior, interior and corner columns were calculated by these formulas, and they are shown in the table below.

Table 5.3. Dimensions of piles for each column**.**

	Exterior	Interior	Corner
L, m	12	12	12
D, m	0.5	0.6	0.5
d, m	1.25	1.5	1.25
n_1	2	3	2
n_2	2	2	2
L_g, m	1.75	2.1	1.75
B_g, m	1.75	2.1	1.75
P, m	1.571	1.885	1.571
A, m^2	0.196	0.283	0.196

The layout of the group of piles under the building is presented in the figure below.

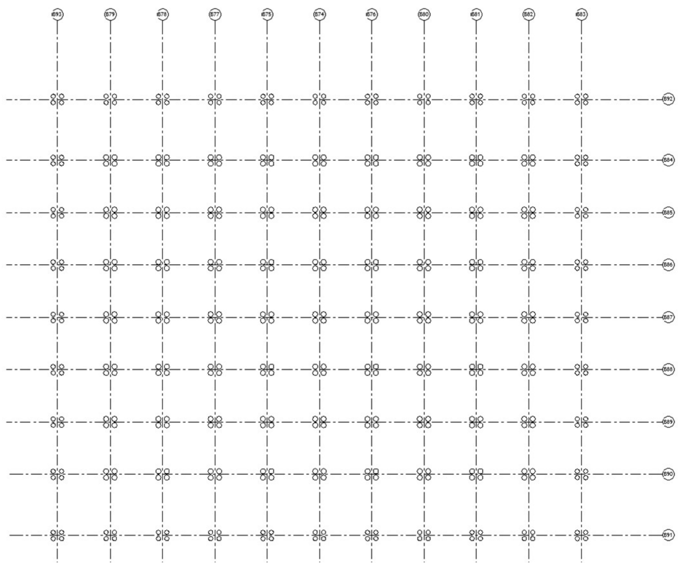
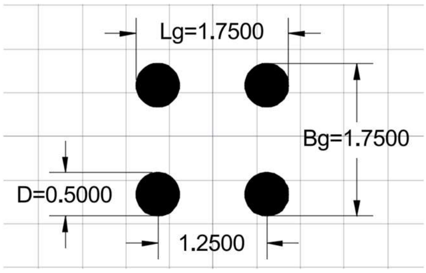
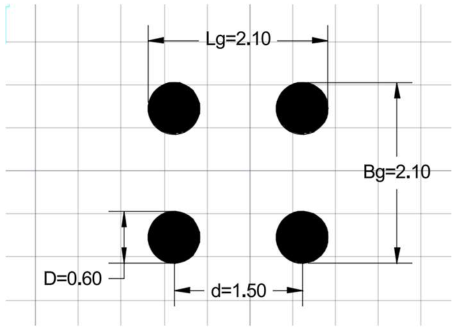


Figure 5.8. Distribution of pile on the building layout

The dimensions of the group of piles for different types of columns are presented in the figures below.



a) Group of piles dimensions for exterior and corner columns



b) Group of piles dimensions for interior columns

Figure 5.9. Group of piles under each type of column

The efficiency calculations for an exterior column were done first. The point-bearing capacity and frictional resistance were calculated in the same way as in the previous parts. Their values are 6707.93 and 4885.26, respectively. The efficiency is calculated as follows:

$$\eta = \frac{2*(2+2-2)*1.25+4*0.5}{1.571*2*2} = 0.63$$

Since the efficiency number is higher than 1.01, the bearing capacity is:

$$Q_{u(g)} = n\Sigma Q_u = \Sigma(Q_u + Q_s) = 2*2*(6707.93+4885.26) = 46372.75$$

$$Q_{all(g)} = \frac{Q_{u(g)}}{FS} = \frac{46372.75}{3} = 15457.58\ kN > Q_{design} = 8440\ kN$$

Efficiency number for interior and corner columns were calculated as follows:

$$\eta_{interior} = \frac{2*(3+2-2)*1.5+4*0.6}{1.885*3*2} = 1.01$$

$$\eta_{corner} = \frac{2*(2+2-2)*1.25+4*0.5}{1.571*2*2} = 1.11$$

Bearing capacity calculations are summarised in the table below.

Table 5.4. Summary of bearing capacity calculations for a group of piles

	η	Q_p, kN	Q_s, kN	Q_u, kN	$\Sigma Q_u, kN$	$Q_{all(g)}, kN$	Q_{design}, kN

Exterior	1.11	6707.93	4885.26	11593.19	46372.75	15457.58	8440.52
Interior	1.11	9659.42	5092.41	14751.83	59007.34	19669.11	7774.52
Corner	1.11	6707.93	4885.26	11593.19	46372.75	15457.58	6435.66

5.4.2.2 Settlement hand calculations

According to Meyerhoff (1976) for the sands and gravel, the empirical formula of settlement for group of piles is expressed as:

$$\boldsymbol{s_{g(e)}} = \frac{\mathbf{0.96}\boldsymbol{q}\sqrt{\boldsymbol{B_g}}\boldsymbol{I}}{\boldsymbol{N_{60}}} \tag{3.45}$$

Where,

$$\boldsymbol{q = Q_g/(L_g B_g)} \tag{3.46}$$

$$\boldsymbol{I = 1 - L/8B_g \geq 0.5} \tag{3.47}$$

$L_g\ and\ B_g$ = the length and width of the group pile section respectively.

The calculation outputs are shown in the table below.

Table 5.5. Settlement for each type of column

	L, m	Q_g, kN	L_g, m	B_g, m	$q, kN/m^2$	$\boldsymbol{N_{60}}$	$\boldsymbol{I}$	$\boldsymbol{S_g, mm}$
Exterior	12	15457.58	1.75	1.75	5047.373	75	0.5	42.733
Interior	12	19669.11	2.1	2.1	4460.116	75	0.5	41.365
Corner	12	15457.58	1.75	1.75	5047.373	75	0.5	42.733

5.4.2.3 Software analysis

Using Plaxis 3D, the deformation for the different pile groups were determined. When constructing a soil profile, a Mohr-Coulomb model with Drained condition was selected and all the necessary soil parameters, such as Young's Modulus, Poisson's ratio and friction angle were added. Overall, the deformation for corner, exterior and interior pile groups can be found in the figures below.

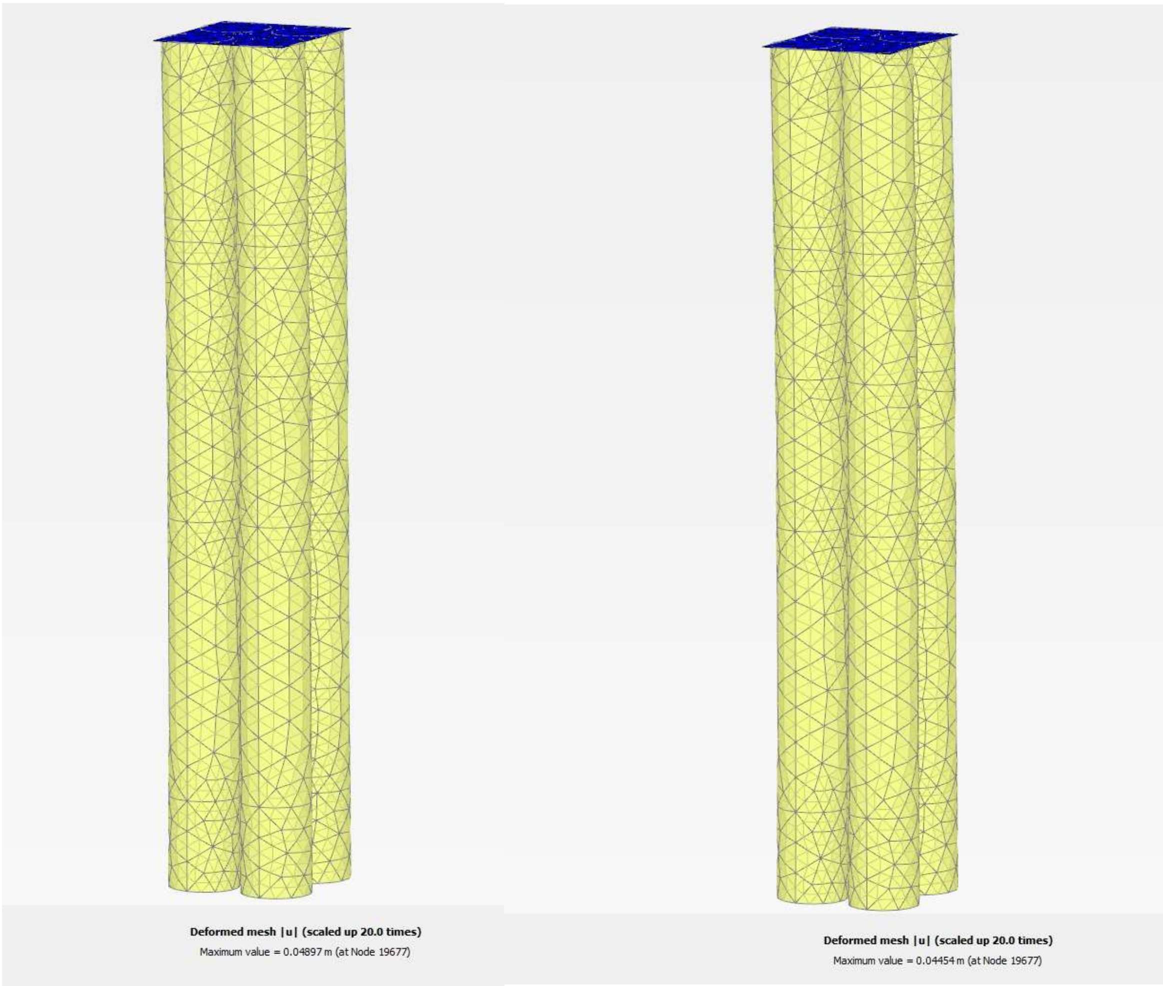


Figure 5.10. Deformation for pendixexterior (at the left) and corner (at the right) group of piles

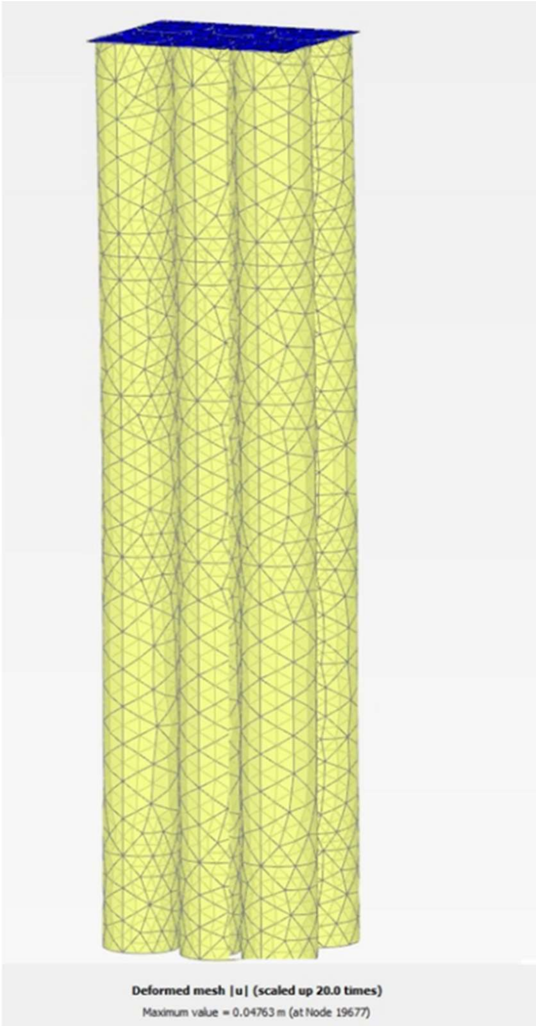


Figure 5.11. Deformation of the interior pile group.

In addition, deformations for different pile groups were done in Geo5 software and later compared with hand calculations and Plaxis 3D output. Below, the figures that show the output from Geo5 software is also illustrated.

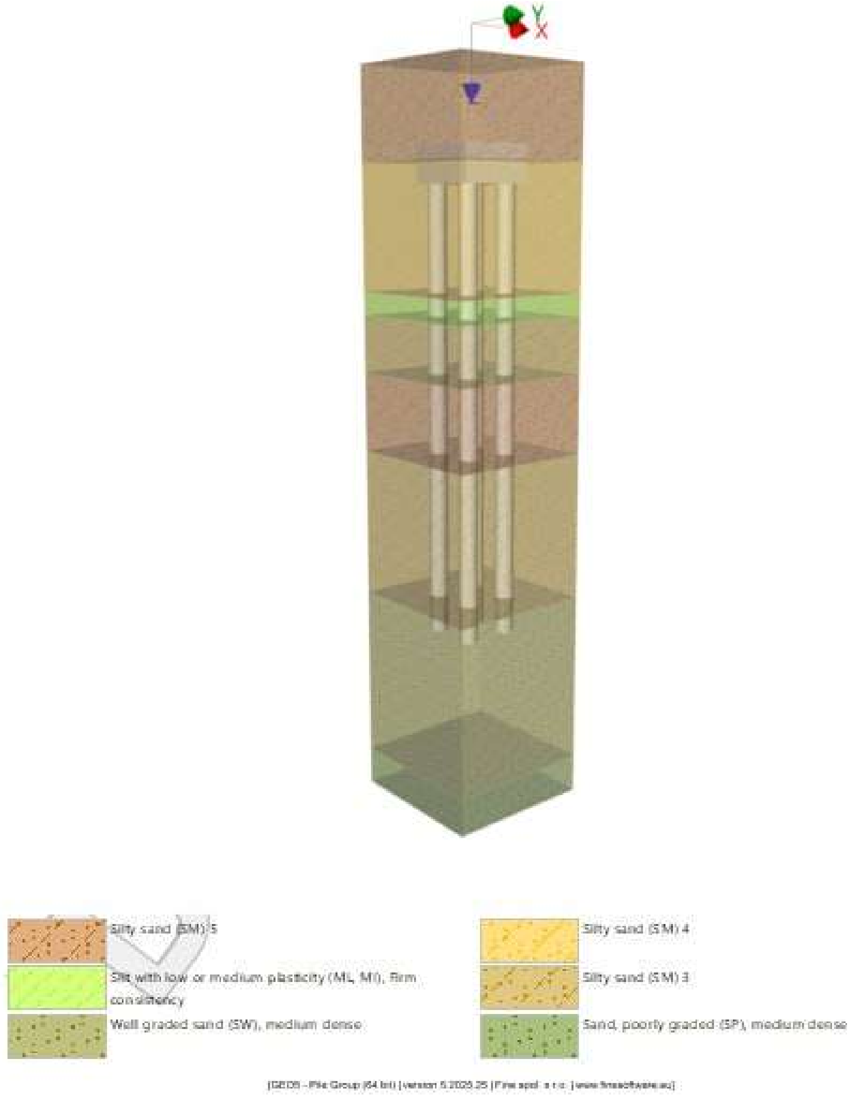


Figure 5.12. The pile group in Geo5

Table 5.6. The comparisons of settlement values for pile groups.

Group of Piles	Meyerhof's method S_g, mm	Plaxis 3D, S_g, mm	Geo5, S_g, mm
Exterior	42.733	48.97	29.6
Interior	41.365	47.63	28.4
Corner	42.733	44.54	29.2

As it can be seen, hand calculation values along with Plaxis 3D values are very similar, while values for Geo5 are different from them. The difference in values can be explained by the fact that since we used a demo version of Geo5 software, there were limitations when it came to making a soil profile. Also, the method that Geo5 used to calculate the maximum settlement could be different from the one that was used in hand calculations leading to the significant difference in the values.

5.5 Foundation design under lateral loading

5.5.1 Hand calculation of lateral bearing capacity

The critical lateral loading combinations were taken in order to calculate the lateral load of each column. The results are given below:

Table 5.7. Critical lateral loading cases

Column type	Combination	F_x, kN	F_y, kN	Number of piles	Load
Exterior	1.2D+1.6L+0.5S	-6.505	190.665	4	46.04
Interior		-0.17	120.989	6	20.1365
Corner		-0.183	254.111	4	63.482

In order to calculate the lateral bearing capacity constant modulus of subgrade reaction and subgrade modulus are needed, their values are provided in the following table.

Table 5.8. Constant modulus of subgrade reaction and subgrade modulus values

Layer depth, m	Soil	η_h	k_z
2.44	Silty SAND	16500	40260
3.35	Silty and Gravelly SAND	16500	55725
0.61	Sandy SILT with	6250	3813

	Gravel		
1.52	Silty SAND	16500	25080
2.13	Silty SAND	16500	35145
3.96	Silty SAND	16500	65340

Before proceeding to calculation of bearing capacity, the classification of pile whether it is long or short needs to be determined. The elastic method is used to do so. Characteristics length of pile is calculated using the following formula:

$$T = \sqrt[5]{\frac{E_p I_p}{\eta_h}}$$

Where,

$E_p -$ Elastic modulus of pile, MPa

$I_p -$ Moment of inertia, m^4

$n_h -$ horizontal modulus of subgrade reaction

Elastic modulus of pile is determined by the following formula:

$$E_p = 4700\sqrt{f_c'} = 4700 * \sqrt{35000000} = 27805.58 * 10^3\ Pa$$

$f_c' -$ compressive strength of concrete, 35 MPa

Moment of inertia is calculated using the equation below:

$$I_p = \frac{\pi D^4}{64} = \frac{\pi * 0.6^4}{64} = 0.00636\ m^4$$

Constant modulus of subgrade for dry and dense sands is 16500

$$T = \sqrt[5]{\frac{27805.58 * 10^3 * 0.00636}{16500}} = 1.607\ m$$

The length of the pile is 12 m which is higher than 5T. This means the pile is considered as long.

Proceeding to the lateral bearing capacity, the Rankine passive earth pressure coefficient is calculated.

$$K_p = tan^2(45 + \frac{\phi'}{2})$$

Next, section modulus of pile is determined:

$$S = \frac{\pi D^3}{32} = \frac{\pi * 0.3^3}{32} = 0.0212$$

Yield moment of pile is calculated by the following equation:

$$M_y = SF_y = 0.0212 * 35000 = 741.825\ kN \text{ for interior column}$$

Where,

$F_y -$ yield stress of pile material, MPa

After determining all the values, the ultimate bearing capacity is determined by the following chart.

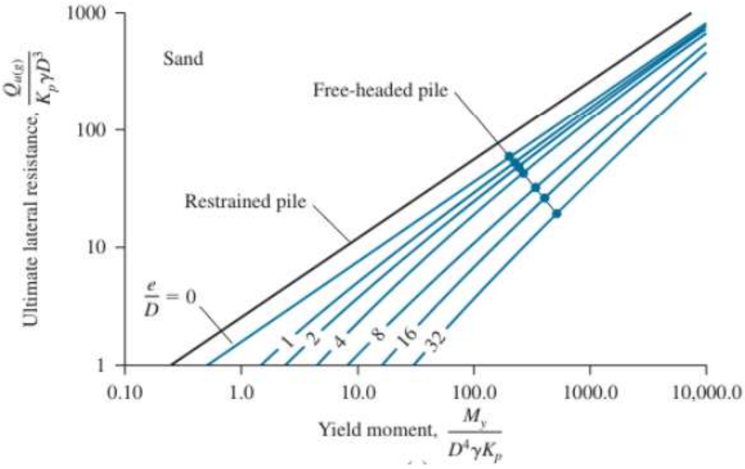


Figure 5.13. Ultimate lateral resistance graph for long piles in sandy soils (Das & Sivakugan, 2019)

Using the ultimate bearing capacity, the allowable bearing capacity was calculated by the formula below. Factor of safety is assumed to be 3.

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

Values of bearing capacity for each type of column is given in the following table:

Table 5.9. Bearing capacity for each type of column compared with lateral load

Column type	Q_u, kN	Q_{all}, kN	$Q_{lateral}$, kN
Exterior	426.18	142.06	46.04
Interior	512.87	170.96	20.1365
Corner	368.34	122.78	63.482

Table shows that the allowable bearing capacity of each column is higher than the lateral load that will affect them, therefore the design can be said satisfactory.

5.5.2 Hand and software calculation of deflection

The lateral deflection is calculated using an elastic method (Das & Sivakugan, 2019). Deflection of pile head can be calculated by the equation below:

$$\eta = \sqrt[5]{\frac{\eta_h}{E_p I_p}}$$

Pile head deflection can also be determined by Brom's method. The dimensionless length is calculated by following formula:

$$Dimensionless\ length = \eta * L$$

The value of deflection is taken from the following graph:

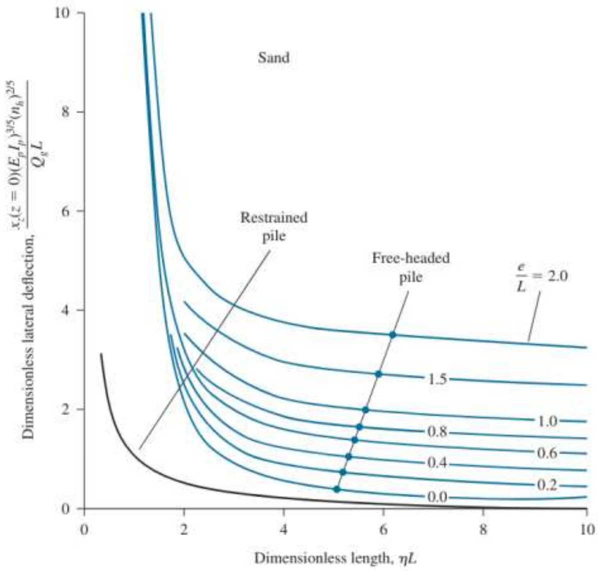


Figure 5.14. Deflection of the pile head for long piles in sand

In regard to pile deflection, the equation used is below:

$$x_z(z) = A_x \frac{Q_g T^3}{E_p I_p} + B_x \frac{M_g T^2}{E_p I_p}$$

The results of the deflection calculation are given in Table 4.18.

Table 5.10. Pile deflection for each type of column

Layer number	Layer depth, m	A_x	B_x	$x_{exterior}$, mm	$x_{interior}$, mm	x_{corner}, mm
0	2.44	2.324	1.623	26.47	23.46	27.01
1	3.35	1.012	0.523	8.91	8.32	8.76
2	0.61	0.152	-0.034	0.56	0.34	0.41
3	1.52	-0.045	-0.076	-0.89	-1.05	-1.01
4	2.13	-0.042	-0.031	-0.55	-0.39	-0.48
5	3.96	-	0	-0.032	-0.05	-0.01

5.5.3 Design of Group Piles

In order to spread the load from the building to the soil the group of piles is structured with a pile cap. The concrete piles are also structured with reinforcement in order to resist the load, since concrete piles are not able to resist it by themselves. The following procedure was done to determine the size of both piles and pile cap reinforcements.

5.5.3.1. Pile cap design

Pile cap size is determined by the following equation. The group of piles is square-shaped, so:

$$L = B = d(n_{1,2} - 1) + 2 * 1.5D = 1.5 * (4 - 1) + 2 * 1.5 * 0.6 = 6.3\ m$$

$$h_{min} = 1.5D = 1.5 * 0.6 = 0.9\ m$$

The following table shows the results of analysis of acting loads. The following unit weight values were used in the calculations.

$$\gamma_{concrete} = 24\ kN/m^3$$

$$\gamma_{backfill} = 25.5\ kN/m^3$$

$$W_{pc} = h * \gamma_{concrete} = 0.9 * 24 = 21.6\ kN/m^2$$

$$W_{bf} = h_{bf} * \gamma_{soil} = 0.5 * 25.5 = 12.75\ kN/m^2$$

Table 5.11. Load analysis

DL, kN/m^2	LL, kN/m^2	$W_{pc}, kN/m^2$	$W_{bf}, kN/m^2$	$W_{sc}, kN/m^2$
150.12	104.39	21.6	12.75	254.51

Load combinations for designing are as below:

$$LC_1 = 1.2D + 1.6L$$

$$LC_2 = 1.2D + 1L + 1W$$

$$LC_3 = 1.2D + 1L + 1E_x$$

$$W = W_{pc} + W_{bf} + W_{load} = 21.6 + 12.75 + 254.51 = 288.86\ kN/m^2$$

Load factors applied: $1.2W = 346.632\ kN/m^2$ and $1.4W = 404.404\ kN/m^2$

For $1.2W$: $M'_{11} = M'_{22} = \frac{4.5*346.632*1.5^2}{2} = 1754.83\ kN - m$

For $1.4W$: $M'_{11} = M'_{22} = \frac{4.5*404.404*1.5^2}{2} = 2047.30\ kN - m$

The reaction forces (Q_1, Q_2, Q_3, Q_4) are calculated using:

$$Q = \frac{P}{R} \pm \frac{M_{xx}y}{I_{xx}} \pm \frac{M_{yy}x}{I_{yy}}$$

Where,

$P = (1.2\ or\ 1.4)N + W_{pc} + W_{bf}$ $-$ vertical load on group of piles

$M_{xx} = M_x + Ne_y + H_y h + M_x *$ $-$ moment along the x-axis

$M_{yy} = M_y + Ne_x + H_x h + M_y *$ $-$ moment along the y-axis

$I_{xx} = \sum y^2\ about\ x - axis$ for interior, y = 1.5 m

$I_{yy} = \sum x^2\ about\ y - axis$ for interior, x = 1.5 m

The bending moments in the pile cap due to reaction forces:

$$M"_{11} = 1.4(Q_3 + Q_4) = 1.4 * (1310.34 + 1350.23) = 3724.80\ kN * m$$

$$M''_{22} = 0.5 * (Q_1 + Q_2 + Q_3) = 1902.31\ kN * m$$

The combined bending moments:

$$M_{11} = M'_{11} + M''_{11} = 2047.3 + 3724.8 = 5772.1\ kN * m$$

$$M_{22} = M'_{22} + M''_{22} = 2047.3 + 1902.31 = 3949.61\ kN * m$$

The reinforcing bars' diameters are assumed to be 32 mm and 20 mm, in x and y direction, respectively. The compressive strength of concrete and yielding strength of steel rebars are 35 N/mm^2, and 460 N/mm^2. According to ACI-318-19, the minimum requirement for cover is 75 mm. The area of reinforcement is determined as in the below:

$$d_x = h_{pile\ cap} - cover - 0.5d_{bar} = 740 - 80 - 0.5 * 32 = 644\ mm$$

$$K = \frac{M_{11}}{f_{cu}bd^2} = \frac{5772.1 * 10^6}{35 * 3000 * 644^2} = 0.132$$

$$z = d * (0.5 + \sqrt{0.25 - \frac{K}{0.9}}) = 529.02\ mm \ \leq\ 0.95d = 611.8$$

$$A_{st} = \frac{M_{11}}{0.87f_y * z} = \frac{5772.1}{0.87 * 460 * 529} = 27264.72\ mm^2$$

The number of bars and spacing between them is calculated as follows:

$$\#\ of\ bars = \frac{A_{st}}{A_{bar}} = \frac{27264.72}{802.8} = 34$$

$$spacing = \frac{4500}{34} = 132.35\ mm$$

Use of 34 #32 mm at 132.35 mm

Design of pile cap for y direction has the same steps:

$$d_y = h_{pile\ cap} - cover - 0.5d_{bar} = 740 - 80 - 0.5 * 20 - 32 = 618\ mm$$

$$K = \frac{M_{11}}{f_{cu}bd^2} = \frac{5772.1}{35 * 4500 * 618^2} = 0.096$$

$$z = d * (0.5 + \sqrt{0.25 - \frac{K}{0.9}}) = 618 * (0.5 + \sqrt{0.25 - \frac{0.096}{0.9}}) = 556.2\ mm \ \leq\ 0.95d$$
$$= 587.1\ mm$$

$$A_{st} = \frac{M_{11}}{0.87f_yz} = \frac{5772.1}{0.87 * 460 * 556.2} = 25931.39\ mm^2$$

$$\#\ of\ bars = \frac{A_{st}}{A_{bar}} = \frac{25931.39}{314} = 47$$

$$spacing = \frac{4500}{47} = 95.74\ mm$$

Use 47 #20 mm at 95.74 mm

The design of reinforcement was calculated for each type of column, and it is presented below.

Table 5.12. The pile cap reinforcement design for each type of column

Column type	h, mm	d_x, mm	d_y, mm	Design in x	Design in y
Exterior	600	493	473.2	38 #32 mm @ 92 mm	42 #20 mm @ 90 mm
Interior	750	644	618	34 #32 mm @ 132 mm	47 #20 mm @ 95.74 mm
Corner	675	584	550	40 #32 mm @ 80 mm	24 #20 mm @ 120 mm

5.5.3.2. Single Pile Reinforcement

Now, we need to design the reinforcement for each pile. Before proceeding to designing, the slenderness of the pile needs to be checked. If the pile is slender, the moment is magnified for the design. $\beta = 1.2$ for piles fixed with pile caps.

$$l_e = \beta l_o = 1.2 * 12 = 14.4\ m$$

Where,

$l_o -$ unsupported length of pile, m

$l_e -$ effective length of pile, m

$$\frac{l_e}{h} = \frac{14.4}{0.5} = 28.8\ >\ 10\text{, so pile is slender}$$

$$\alpha = \frac{l_e^2}{2000h} K = \frac{14.4^2}{2000 * 0.5} * 1 = 0.21\ m$$

$$M_{add} = Q_{min} * \alpha = 1010.12 * 0.21 = 212.1252\ kN * m$$

$$M_{mag} = M + M_{add} = 103.3 + 212.1252 = 315.43\ kN * m$$

$$e = \frac{M_{mag}}{Q_{min}} = \frac{315.43}{1010.12} = 0.312\ m$$

$$\frac{e}{R} = \frac{0.312}{0.3} = 1.04$$

$$\frac{M}{h^3} = \frac{315.43}{0.5^3} = 2.523\ kN/m^2$$

$$k = \frac{h_s}{h} = 0.6$$

The minimum reinforcement is 1.6%, so:

$$A_{st} = \frac{A_c * 1.3}{100} = \frac{\pi * 0.3^2 * 1.3}{100} = 3673.8\ mm^2$$

Use 8 #25 at 150 mm spacing

The shear check is done to determine a need for shear reinforcement:

$\frac{M_{max}}{Q_{max}} = \frac{315.43}{2237.09} = 0.141 < 0.6h = 0.3$, so no shear reinforcement is needed

The values of determination, the reinforcement and its design for each type of column are presented in the table below.

Table 5.13. The reinforcement design for each type of column

Column type	D, m	l_e/h	M_{add}	M_{mag}	M/h^3	p, %	A_{st}, mm^2	Design
Exterior	0.5	21	150.95	170.23	2.34	2.7	3549.2	10 #25 mm
Interior	0.6	28.8	212.13	315.43	2.523	1.3	3673.8	8 #25 mm
Corner	0.5	20.2	231.23	260.11	2.349	3.1	4732.2	10 #25 mm

Using the calculated values, the reinforcement of the single pile in each of the corner, exterior and interior pile groups was analysed in Geo5 software. The figures below show the results from the software.

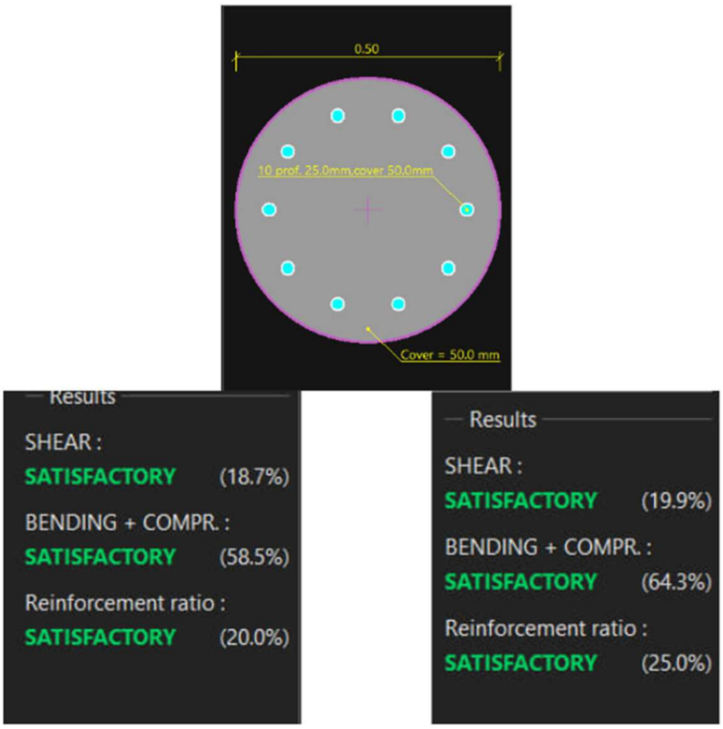


Figure 5.15. The single pile reinforcement for corner and interior pile groups.

Figure 5.16. The single pile reinforcement for the exterior pile group.

The results from Geo5 software shows that calculated values for the reinforcement of the single pile from each of the pile groups are within the acceptable range, as it satisfies shear, bending and compression and reinforcement ratios.

Software analysis of pile groups

Layouts of pile groups were inserted into the PLAXIS 3D program using our soil profile and load conditions. As it can be seen in the figure below, the pile layout of exterior, interior and column has loads that each group of piles are subjected to. The load is considered to be subjected at the exact middle of the pile group. Pile group layout also includes pile and basement height. Piles were constructed as embedded beams. It also includes sheet piles.

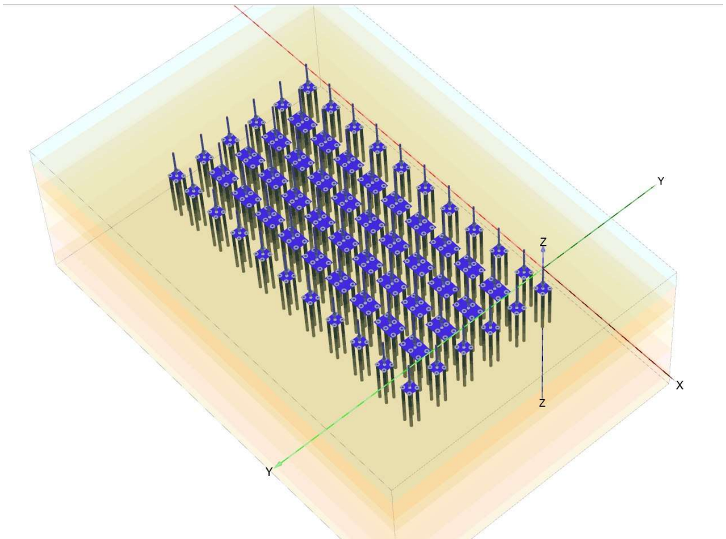


Figure 5.17. Layout of pile groups under the building

After inserting the layout of piles, calculation of deflection and simulation of real-life scenarios was done. Output of the calculation is given below in the figure below. The maximum displacement value is 17.77 mm, which is within the accepted range.

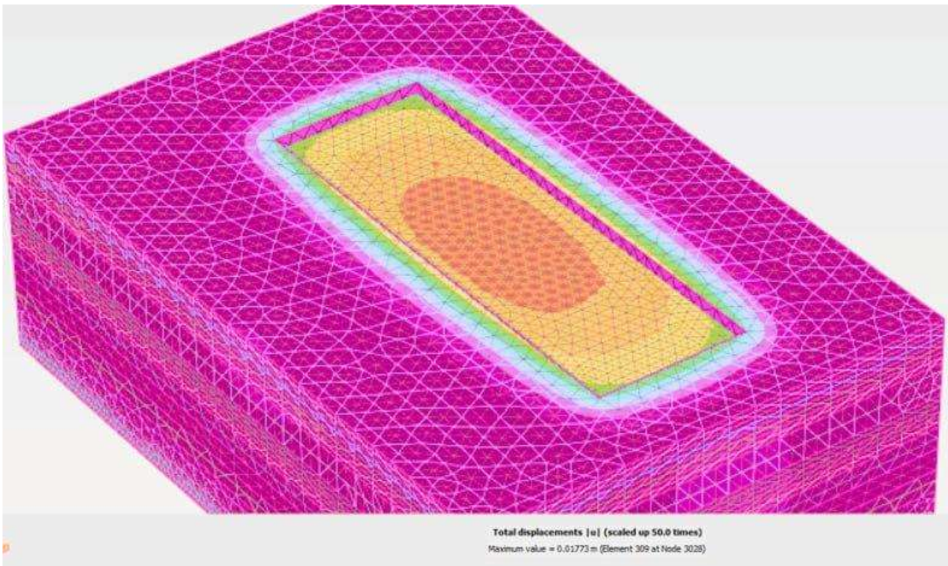


Figure 5.18. Deformed mesh

5.6. Sheet pile design.

To withstand lateral earth pressures and support the walls of excavations, sheet piles were constructed. Sheet piles were chosen due to several reasons. They are easy to install using the vibratory hammers or other press-in methods, and compared to diaphragm walls, when constructing sheet pile, preparation of slurry or time for curing are not needed. It is also relatively cheap in comparison with other walls, such as secant piles and it is thinner, so it takes less space than other walls, such as gravity retaining walls. Finally, they are easier to remove or modify in the future than compared to anchored retaining walls. There are several types of sheet piles, however steel sheet piles were chosen because of its significant benefits. First, compared to the other materials such as concrete, which are considered to be brittle, steel is ductile thus it can deform without breaking and the risk of cracking is less in seismic regions. Secondly, steel piles are easier to install and to drive into soil, as it is thin and strong compared to other materials. Thirdly, steel sheet piles have many types, which covers many requirements of load.

5.6.1. Hand calculations for sheet pile design.

For the calculations below, the height of the sheet pile is assumed to be 3 m, which is also the height of the basement. First, the Rankine earth pressure coefficients were calculated as:

$$K_a = tan^2(45 - \frac{\varphi'}{2})$$

$$K_p = tan^2(45 + \frac{\varphi'}{2})$$

For the soil layer, where sheet pile is located, the corresponding critical angle,φ' is $34.31°$, while the unit weight, $\gamma = 25.5\ kN/m^3$. Using this information, the calculations are shown below:

$$K_a = tan^2(45 - \frac{34.31}{2}) = 0.279$$

$$K_p = tan^2(45 + \frac{34.31}{2}) = 3.584$$

In order to ensure the safety, the value of K_p was corrected using the factor of safety, which was assumed to be 2. (Das,2019)

$$K_{p(design)} = \frac{K_p}{FS}$$

$$K_{p(design)} = \frac{3.584}{2} = 1.792$$

Then the active pressure is calculated using the formula below:

$$\sigma'_2 = \gamma L K_a$$

For the $L = 3\ m$, the value of active pressure is:

$$\sigma'_2 = 25.5 * 3 * 0.279 = 21.344\ kN/m^2$$

To obtain the value of the depth, at which the value of net pressure, $\sigma' = 0$, the following formula was used:

$$L_3 = \frac{LK_a}{K_{p(design)} - K_a}$$

So, the value of depth is:

$$L_3 = \frac{3 * 0.279}{1.792 - 0.279} = 0.553\ m$$

Then, the value of σ'_5 is calculated using the formula below:

$$\sigma'_5 = \gamma L K_{p(design)} + \gamma L_3 (K_{p(design)} - K_a)$$

$$\sigma'_5 = 25.5 * 3 * 1.792 + 25.5 * 0.553 * (1.792 - 0.279) = 158.424\ kN/m^2$$

Using the area of the pressure diagram, P was calculated as:

$$P = 0.5 * \sigma'_2 * L + 0.5 * \sigma'_2 * L_3$$

$$P = 0.5 * 21.344 * 3 + 0.5 * 21.344 * 0.553 = 37.918\ kN/m$$

The centre of pressure is determined from the formula below:

$$\underline{z} = \frac{L(2K_a - K_{p(design)})}{3(K_{p(design)} - K_a)}$$

$$\underline{z} = \frac{3 * (2 * 0.279 - 1.792)}{3(1.792 - 0.279)} = 0.816\ m$$

Next, the values of A'_1, A'_2, A'_3, A'_4 were also calculated:

$$A'_1 = \frac{\sigma'_5}{\gamma(K_{p(design)} - K_a)}$$

$$A'_1 = \frac{158.424}{25.5 * (1.792 - 0.279)} = 4.106$$

$$A'_2 = \frac{8 * P}{\gamma(K_{p(design)} - K_a)}$$

$$A'_2 = \frac{8 * 37.918}{25.5 * (1{,}792 - 0.279)} = 7.862$$

$$A'_3 = \frac{6P[2\underline{z}\gamma(K_{p(design)} - K_a) + \sigma'_5]}{\gamma^2(K_{p(design)} - K_a)^2}$$

$$A'_3 = \frac{6 * 37.918\,[2 * 0.816 * 25.5 * (1.792 - 0.279) + 158.424]}{25.5^2(1.792 - 0.279)^2} = 33.837$$

$$A'_4 = \frac{P[6\underline{z}\sigma'_5 + 4P]}{\gamma^2(K_{p(design)} - K_a)^2}$$

$$A'_4 = \frac{37.918 * [6 * 0.816 * 158.424 + 4 * 37.918]}{25.5^2 * (1.792 - 0.279)^2} = 23.62$$

Using the obtained values, the equation for L_4 was expressed below:

$$L_4{}^4 + A'_1 L^4{}_4 - A'_2 L^2{}_4 - A'_3 L_4 - A'_4 = 0$$

$$L_4 = 2.3\ m$$

Further, the theoretical value of the depth, D_{theory} is calculated:

$$D_{theory} = L_3 + L_4$$

$$D_{theory} = 0.553 + 2.3 = 2.853\ m$$

To obtain the actual length of the sheet pile, the height and the theoretical value of the depth, which was increased by 30% were added to get the final answer. So, the total length of the pile, $L_{total} = 3 + 2.853 + 2.853 * 0.3 = 6.71\ m$.

Also, the values of σ'_3, σ'_4 and L_5 were obtained using the formulas below:

$$\sigma'_3 = \gamma L_4(K_{p(design)} - K_a)$$

$$\sigma'_3 = 25.5 * 2.3 * (1.792 - 0.279) = 88.737\ kN/m^2$$

$$\sigma'_4 = \sigma'_5 + \gamma L_4(K_{p(design)} - K_a)$$

$$\sigma'_4 = 158.424 + 25.5 * 2.3 * (1.792 - 0.279) = 247.16\ kN/m^2$$

$$L_5 = \frac{\sigma'_3 L_4 - 2P}{\sigma'_3 + \sigma'_4}$$

$$L_5 = \frac{88.737 * 2.3 - 2 * 37.918}{88.737 + 247.16} = 0.382\ m$$

The value of the point of zero shear force is obtained below:

$$z' = \sqrt{\frac{2P}{\gamma(K_{p(design)} - K_a)}}$$

$$z' = \sqrt{\frac{2*37.918}{25.5(1.792-0.279)}} = 1.402\ m$$

The formula for maximum bending moment is expressed as:

$$M_{max} = P(\underline{z} + z') - [\frac{1}{2} * \gamma * z'^2 (K_{p(design)} - K_a)](\frac{1}{3})z'$$

$$M_{max} = 37.918 * (0.816 + 1.402) - [0.5 * 25.5 * 1.402^2 (1.792 - 0.279)] * (\frac{1}{3}) * 1.403$$

$$M_{max} = 66.369\ kN \cdot m/m$$

According to the code ASTM A-572, the allowable stress of the sheet pile is 225 MN/m^2, so the section modulus is calculated as:

$$S = \frac{M_{max}}{\sigma_{all}}$$

$$S = \frac{66.369}{225000} = 3.904 * 10^{-4} m^3/m\ of\ the\ wall$$

Based on the section modulus, the chosen section for the steel pile is PZ-22. According to Das (2019), PZ-22 section has a width of 558.8 mm, thickness of the flange is 9.525 mm, while of the web is 59.974 mm and height of 228.6 mm. Below the cross-section view of the sheet pile can be found.

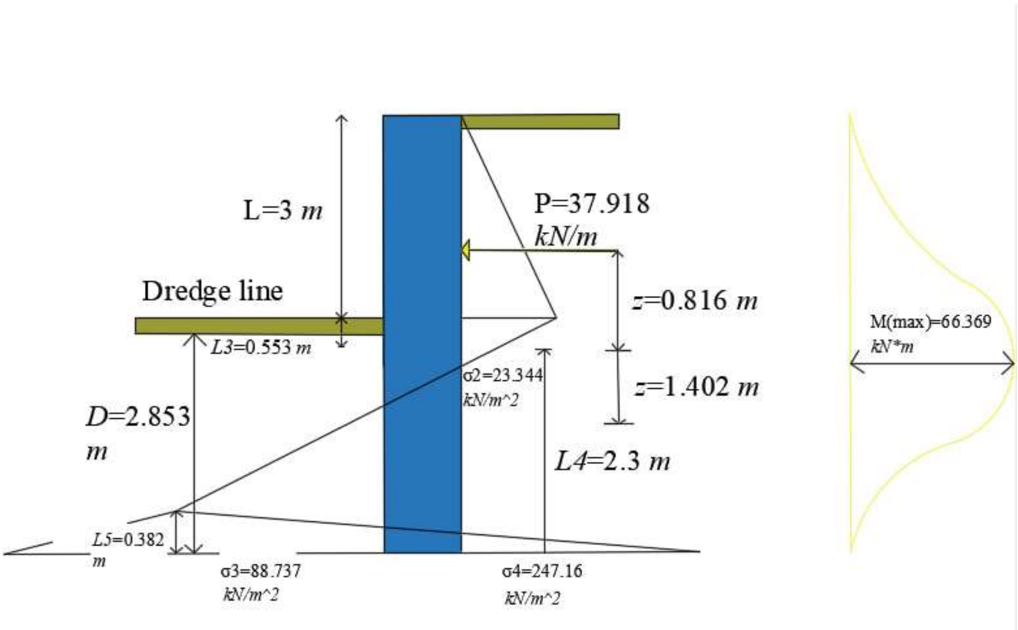


Figure 5.19. The Sheet pile design in AutoCAD

5.6.2. Software analysis of the sheet pile.

Using Geo5 software, the values for the design of sheet piles were double-checked. In the software, the soil profile was added, and the sheet pile was designed with excavation of 3 m being taken into account. From the figure below, it can be seen that for the given sheet pile, the maximum displacement is 16.1 mm, and maximum pressure acting on the structure is 38.73 kPa.

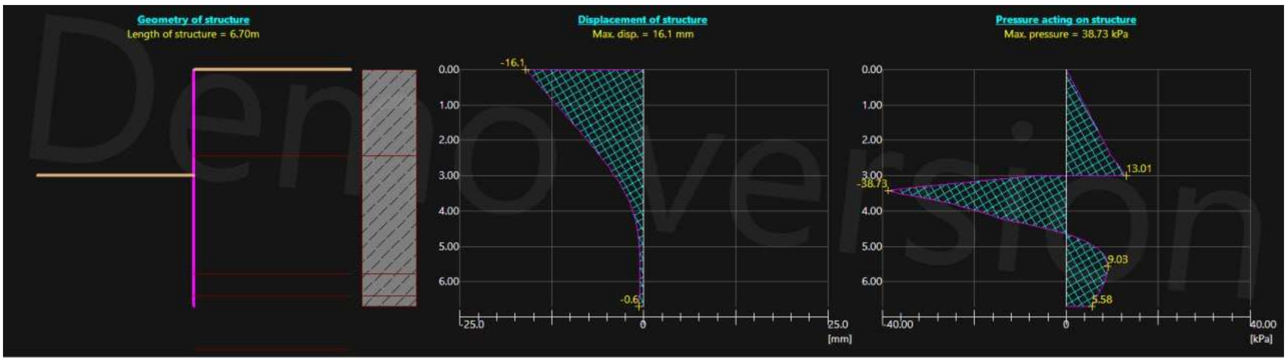


Figure 5.20. Displacement and maximum pressure values for sheet pile

Further, the graphs for bending moments and shear force for sheet pile shows that maximum bending moment is equal to 58.92 kNm/m, while maximum shear force is 92.19 kNm/m.

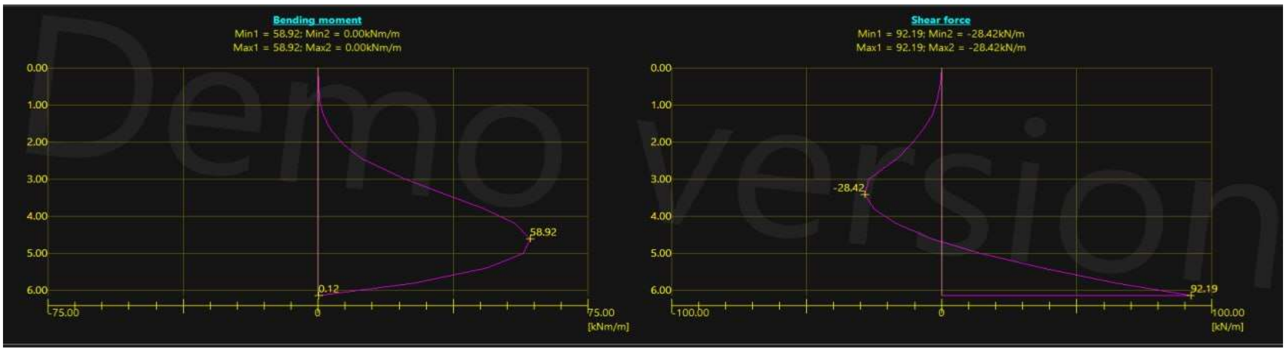


Figure 5.21. Bending moment and shear force graphs for sheet pile

In Geo5, the numerical analysis for bending and shear was found to be satisfactory, as shown in the figure below.

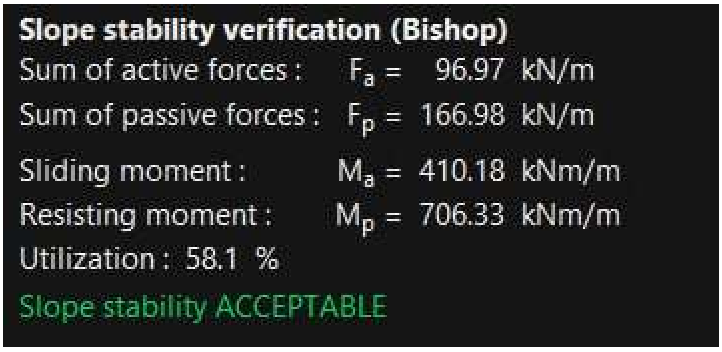
Results

BENDING : SATISFACTORY (41.8%)

SHEAR : SATISFACTORY (24.2%)

Figure 5.22. Numerical analysis for bending and shear in Geo5

Finally, the slope stability for sheet pile was also checked using the Bishop method, and it was also found to be acceptable.



Slope stability verification (Bishop)

Sum of active forces : F_a = 96.97 kN/m

Sum of passive forces : F_p = 166.98 kN/m

Sliding moment : M_a = 410.18 kNm/m

Resisting moment : M_p = 706.33 kNm/m

Utilization : 58.1 %

Slope stability ACCEPTABLE

Figure 5.23. Slope stability analysis in Geo5

5.7. Detailed Construction Procedure

Getting the site ready is one of the best things in a construction job, especially in massive ones like our skyscraper hotel. It makes all the work procedurally. The timeline for site preparation is dependent on the size of the site and the condition of the site. This normally happens over a period of about 1 to 2 months for such a task.

Adhering to the above schedule is very vital as any delay will increase the project cost and also affect the projects' completion time. Site preparation helps in avoiding reworks, a safe working environment, and helps in completing structural works on time. There may be changes in the steps undertaken according to the project, but a general process is written below.

Site Clearing

The first step is to clear vegetation, trash, old structures, or other obstructions from the construction site. This creates a clean and safe working space for construction equipment and personnel. Clearing for a high-rise building such as this also involves opening equipment and material access ways and storage areas.

Site Surveying

Once the land has been cleared, a boundary and topographic survey is conducted to determine the exact property lines and elevation points. Surveying ensures all elements of the building are built according to design coordinates and within the lawful plot limits. It also provides elevation data important in drainage planning and foundation levels.

Soil Test

Geotechnical testing is the process of evaluating the soil so that its bearing capacity, composition, water content, and other engineering properties may be assigned; the result being that this information aids engineers in the selection of the right type of foundation. In our case, it was required in the design of pile foundation and the evaluation of the possible settlement or seismic hazards.

Site Plan Design

Before actual construction, a detailed site layout plan is made. It lays out access road locations, storage facilities, locations of utility lines, safety zones, and equipment locations. The plan arranges the site in a way that minimizes delays and maximizes workflow during construction.

Site Investigation

This is achieved by correlating data from surveys, soil tests, and borehole logs to create a composite image of subsurface conditions. The observations confirm soil strata, ground levels, and any obstructions or weak areas that may affect excavation and piling. The investigation has direct influence on the type, depth, and structure of foundations.

Sheet Pile Installation

To protect the site of excavation, sheet piles are driven along the site perimeter. We used PZ-22 steel sheet piles in this project due to their strength and capacity to resist earth and water pressure. The piles act as temporary retaining walls, holding soil around them and preventing collapse during excavations.

Excavation

With sheet piles in place, excavation is now commenced to the specified depth within normal constraints—typically to the pile cap. Excavation must be properly done in layers for safety and to avoid over-excavation. In the case of deep foundations like ours, dewatering can also be required if water is encountered.

Pile Installation Method and Sequence

After excavation, bored cast-in-place piles are cast. The process includes:

1. Drilling boreholes to the desired depth
2. Inserting reinforcement cages inside
3. Casting concrete by the tremie method to avoid segregation

A lot of planning goes into the order of installation—starting with interior piles, then corner and edge piles. This reduces soil movement and avoids irregular settlement. With all the piles cast and cured, pile caps are constructed to join them and transfer loads appropriately.

6. Environmental part

On the topography map below the neighbourhood area of the site is shown. The map provides the information about the elevation rate, so the natural stormwater movement could be predicted. The overall tendency shows that the elevation rate is higher on the eastern part of the city. Our site is located at the moderate elevation level, and it has lowlands on the west side. Stormwater will move from east to west due to gravitational force.

Figure 6.1. Local Topography and Infrastructure

The figure shows the existing sewer system in the city. The main line of the system goes through Fairview Avenue, which is located to the east of the site. In the figure it is illustrated by 2 orange lines. The main sewage pipelines are owned and maintained by the City of Seattle. Seattle Public Utilities is the responsible governing organisation for operating water, sewer and drainage systems in the city.

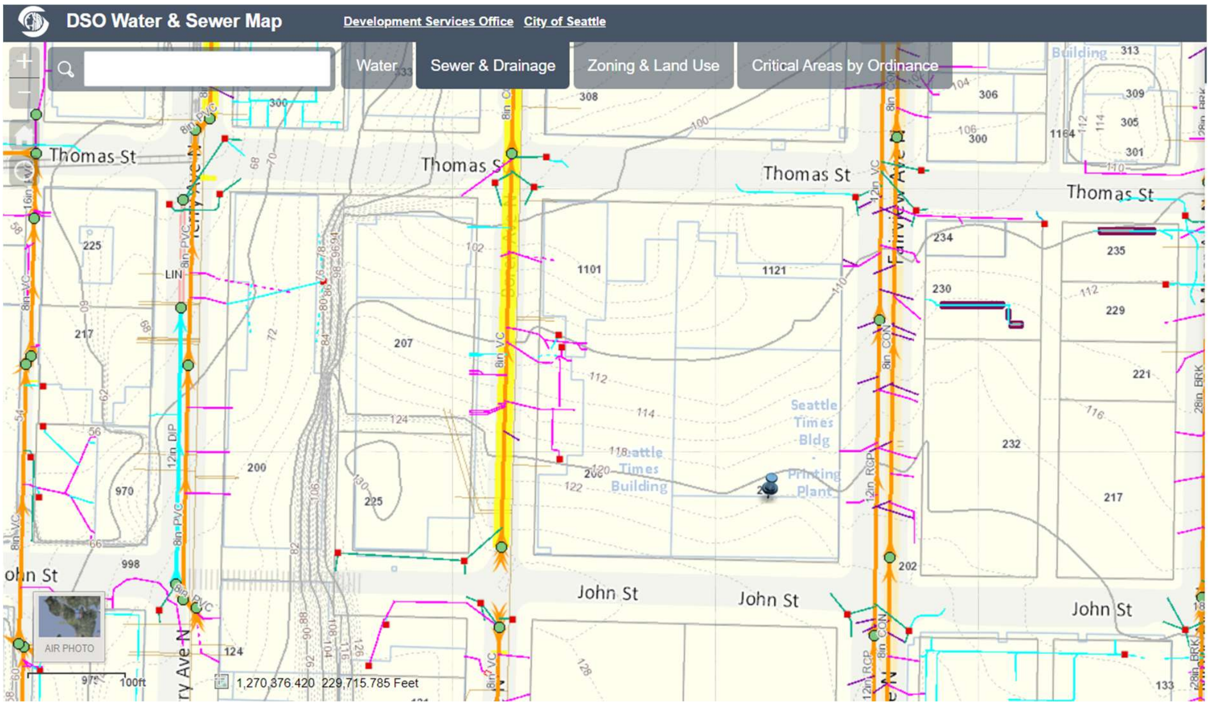


Figure 6.2 Existing Sewer System

The graph below was measured using 10 equally distanced points on the site. The measurements were taken starting from north-west point to the south-east point diagonally. It shows elevation differences throughout the site. From the graph we determined that the elevation is from 31 to 42 metres. It can be clearly seen that the elevation decreases from east to west creating slope, so the stormwater will mostly be accumulated on the north-western part of the site.

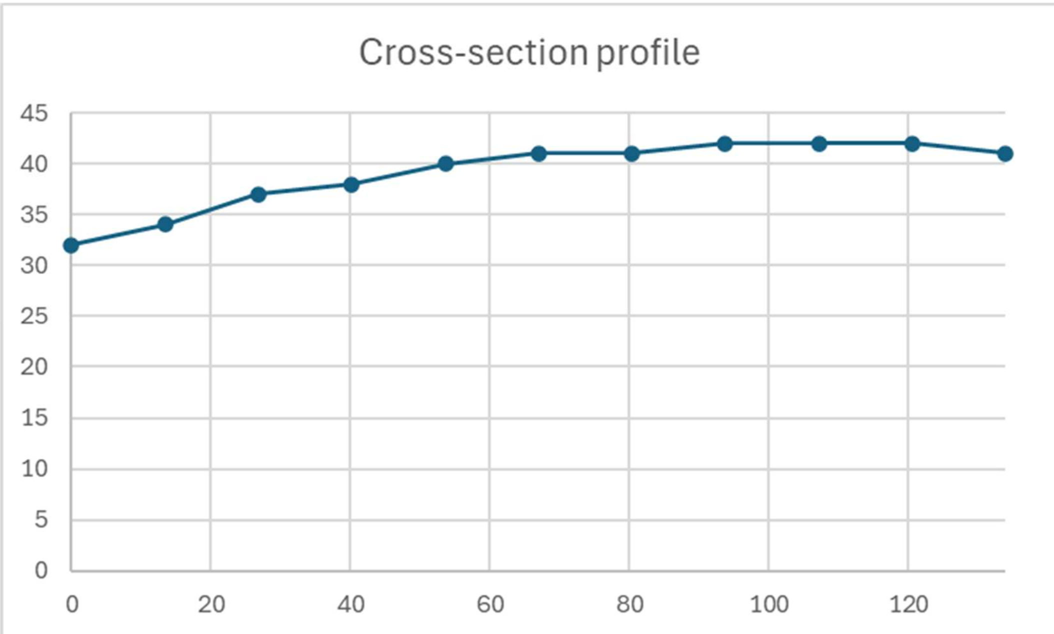


Figure 6.3. Cross-sectional profile

It is a storm sewer map with the directions of movement of stormwater on the site and roofs of the building. Blue rectangle on the left shows where most of the water will be accumulated during rains, because of natural gravitational force created by the slope. 3 red circles approximate the locations of catch basins. Blue arrows show the movement of stormwater. Orange lines are the pipes that will carry the water to the main sewer line. Light blue lines on the roofs of the building show the management of the water from the roofs to the ground. Our building has 2 separate levels of the roof, and the rainwater from them will be collected separately. For now, we decided to use a syphon drainage system for the roofs due to a lot of heavy rains in Seattle. Water from roofs will be collected in different catch basins and then connected to the main pipeline.

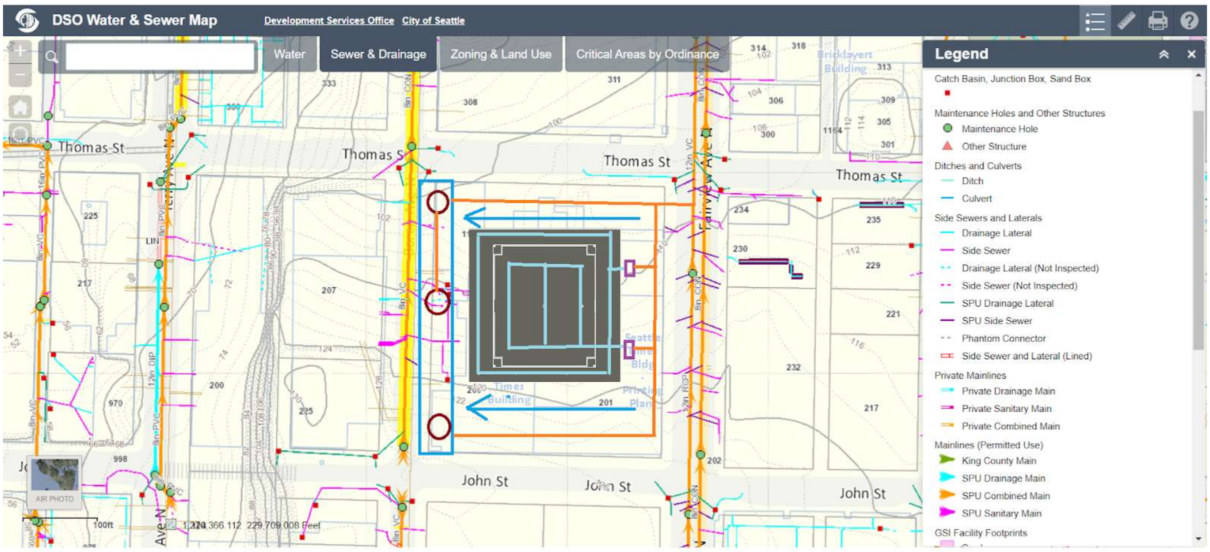


Figure 6.4. Preliminary sketch of the water movement on the site

As shown in Figure 4.5. Topography of the site is favourable for the natural runoff of the water from the Southern part to the North-Western part of the site. The site's contour map shows that the elevation steadily decreases from south to north direction. The preliminary drainage system was designed considering the natural inclinations. However, the final grading and drainage plan implies that all of the stormwaters will be collected on the north-eastern part of the building site. The problem will be solved using trenching and adding fill operations.

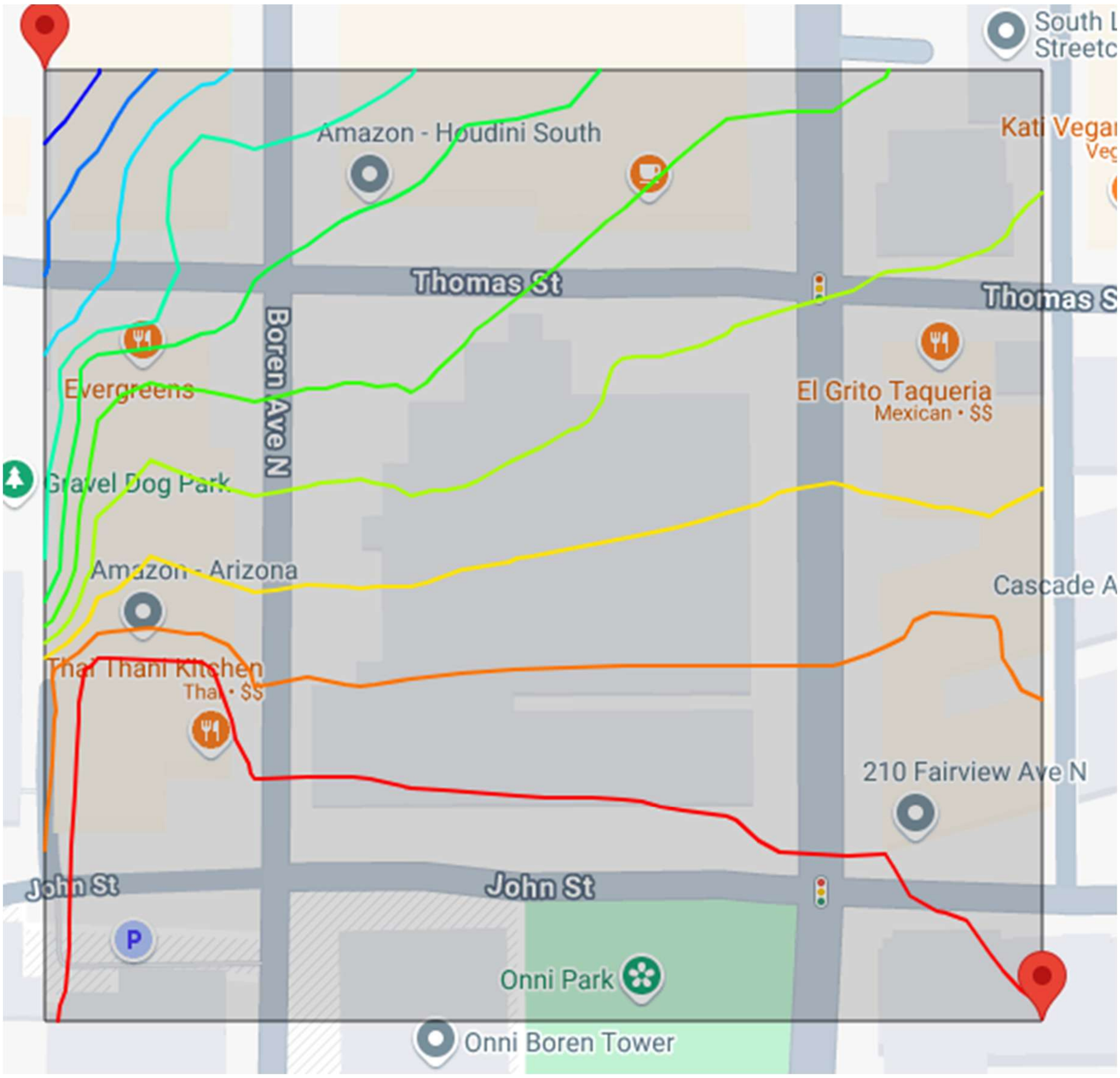


Figure 6.5. Contour map of the site

Grading of the site is important for the proper management of the stormwater and for designing an effective drainage system throughout the site. The topographical and drainage system design for a site situated at the intersection of Thomas Street, Fairview Avenue, Boren Avenue, and John Street is described in the grading and drainage plan that is provided. In order to guarantee appropriate site management of stormwater runoff, the diagram shows topographic contours, necessary drainage components, and the desired grading. It is different from the preliminary sketch. In the preliminary sketch of the site stormwater management only natural elevation lines were considered. Taking everything into account, the natural elevation of the site is favourable for planning grading and drainage systems, because it means minimum earthwork operations.

Adding fill is required for the North-Western part of the site to increase the elevation from 30 m to 34 m. It will create a slope towards the eastern part of the site. The building itself is located at the elevation point of 35.2 m, it prevents the water accumulation near the building and diverts runoff to nearby drainage features. Parking lots are designed in such a way that the stormwater directly flows to the trench drains. Parking area is flat with a slight inclination to the north-east part of the site. The highest point is 39.2 m at the south-west corner, and the lowest point is 32 m at the north-east point. The sidewalk around the building is permeable and will be able to handle the stormwater using natural slope, so it will not have gradings. This is the only green area throughout the building site. As the slope arrows indicate the stormwater from the site will be collected at the North-East corner, and then by implementation of collector pipe will be connected to the main transmission line at the Fairview Ave N Street.

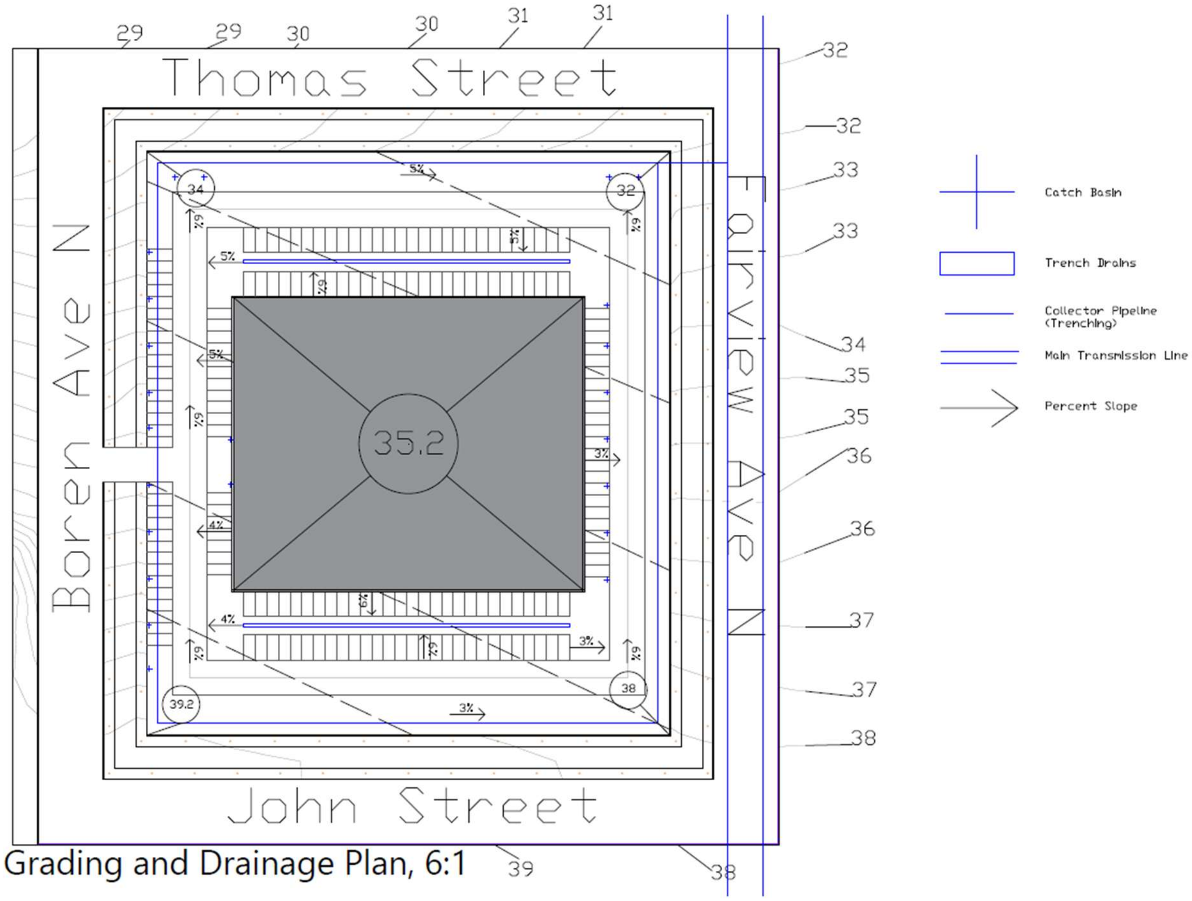


Figure 6.6. Site Grading

The peak discharge flow, Q can be calculated by using a rational *method*. The Rational Method assumes that rainfall intensity remains uniform across the entire tributary watershed for a given duration. This method uses a formula that connects the peak discharge at the site to the product of rainfall intensity and a runoff coefficient:

$$Q = C * I * A \tag{1}$$

Where:

$Q -$ $Peak\ runoff\ rate, feet^3/s;$

$C -$ $Runoff\ coefficient;$

$I -$ $Rainfall\ intensity, in/hr;$

$A - Drainage\ area;$

Figure 6.7. Seattle's active rain gauge network stations

Our structure located near Metro_KC Denny Regulating station, whose ID number is 45-S011

Since our project area includes three primary sources of stormwater runoff, the building's roof, parking lot, and green space, we must calculate the peak discharge runoff and time of concentration individually for each area.

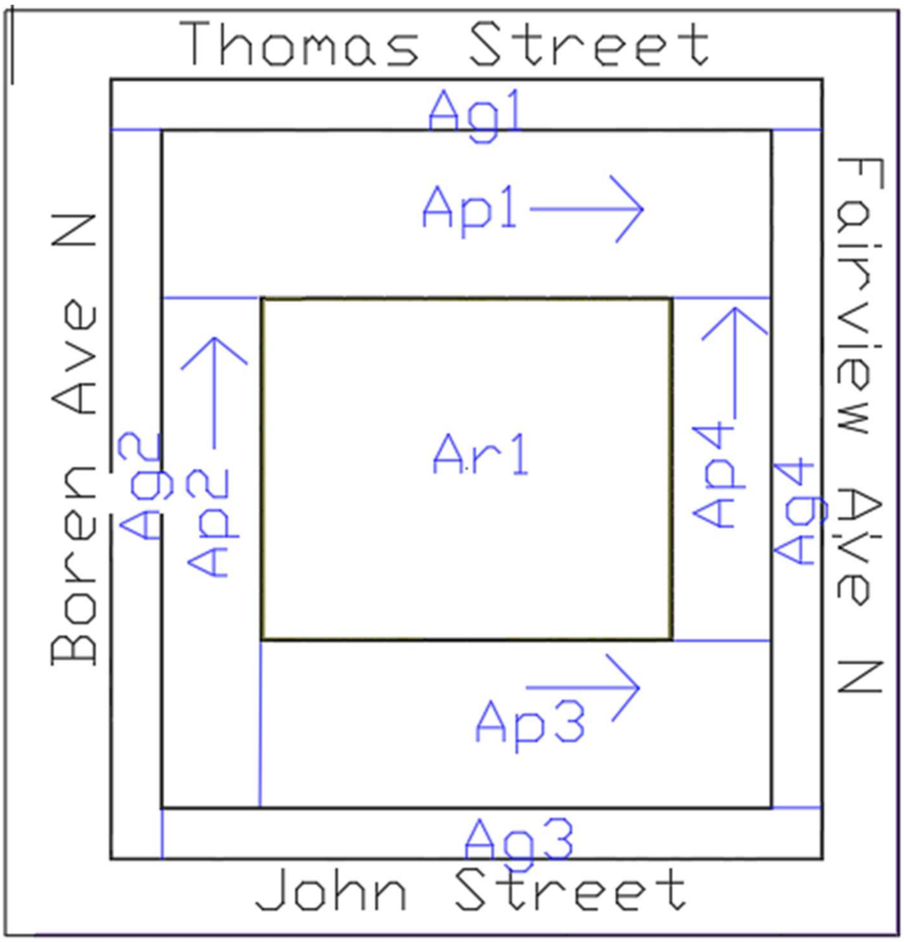


Figure 6.8. Areas and the direction of flow

The area of the roof:

$$A_{r1} = 167{,}257ft * 200{,}5ft = 621.36ft^2$$

The area of the parking lot:

$$A_{p1} = 23701.92\ ft^2$$

$$A_{p2} = 11750.51\ ft^2$$

$$A_{p3} = 19947.87ft^2$$

$$A_{p4} = 7901.12\ ft^2$$

$$A_{total} = 63301.42\ ft^2$$

The area of the green area:

$$A_{g1} = 8204.15\ ft^2$$

$$A_{g2} = 8414.78\ ft^2$$

$$A_{g3} = 7630.55\ ft^2$$

$$A_{g4} = 7841.59\ ft^2$$

$$A_{total} = 32091.07\ ft^2$$

Next, the runoff coefficients for the building roof, parking lot, and green area have been taken from the table below.

Land Cover	Runoff Coefficient (C)
Dense Forest	0.10
Light Forest	0.15
Pasture	0.20
Lawns	0.25
Gravel Areas	0.80
Pavement and Roofs	0.90
Open Water (Ponds Lakes and Wetlands)	1.00

Figure 6.9. Runoff coefficients for different land cover

$$C_{roof} = 0{,}9;$$

$$C_{parking\ lot} = 0{,}8;$$

$$C_{green\ area} = 0{,}25$$

We determined the rainfall intensity using IDF (intensity-duration-frequency) curves. IDF curves help estimate the average design rainfall intensity based on how often a storm of a certain size is expected to occur over different time periods. The rainfall data in *figure* # was analysed using measurements from 17 local gauges in Seattle and the national NOAA cooperative gauge network.

Duration (hr)	Precipitation (in)										
	Recurrence Interval (years)										
		0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr		25-Yr	50-Yr	100-Yr	500-yr
6		0.78	0.92	1.07	1.27	1.42		1.64	1.80	1.98	2.39
12		1.10	1.32	1.55	1.86	2.08		2.42	2.67	2.93	3.55
24		1.46	1.77	2.10	2.55	2.87		3.36	3.73	4.12	5.04
48		1.76	2.16	2.57	3.14	3.55		4.16	4.64	5.12	6.29
72		2.16	2.63	3.11	3.75	4.21		4.88	5.39	5.90	7.09
168		3.09	3.76	4.42	5.27	5.86		6.70	7.30	7.88	9.18

Figure 6.10. Precipitation magnitude frequency estimates for SPU Gauge 11

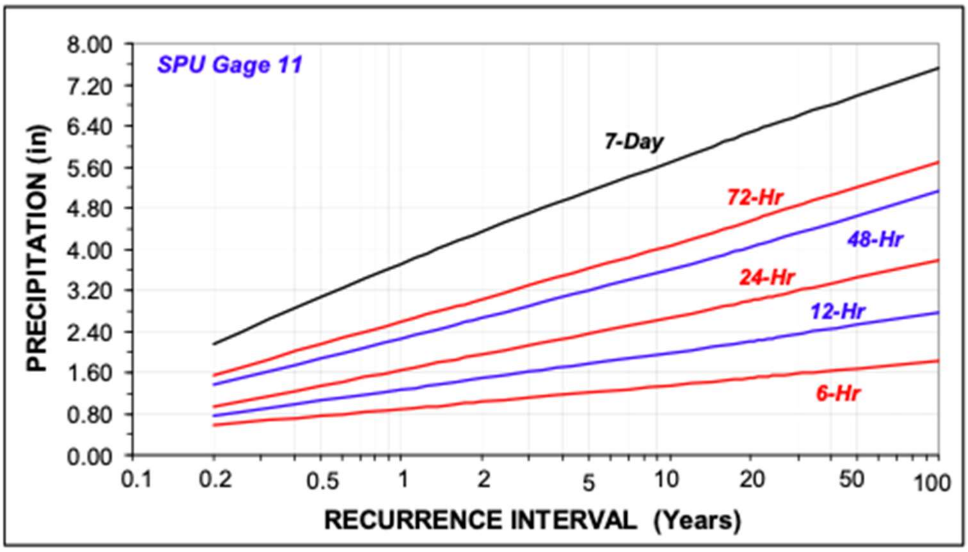


Figure 6.11. Precipitation magnitude frequency estimates for SPU Gauge 11

$$I = 1{,}46 in/hr = (\frac{1{,}46}{12 * 3600})ft/s = 3{,}38 * 10^{-5} ft/s$$

The runoff volume of the roof:

$$Q_{roof} = C_{roof} * I * A_{roof} = 0{,}9 * 3{,}38 * 10^{-5} * 33567{,}62 ft^2 = 1{,}021127 ft^3/s$$

The runoff volume of the parking lot:

$$Q_{p1} = 0{,}8 * 3{,}38 * 10^{-5} * 23701.92\, ft^2 = 0.641\, ft^3/s$$

$$Q_{p2} = 0{,}8 * 3{,}38 * 10^{-5} * 11750.51\, ft^2 = 0.318\, ft^3/s$$

$$Q_{p3} = 0{,}8 * 3{,}38 * 10^{-5} * 19947.87\, ft^2 = 0.539\, ft^3/s$$

$$Q_{p4} = 0{,}8 * 3{,}38 * 10^{-5} * 7901.12\, ft^2 = 0.214\, ft^3/s$$

$$Q_{total} = 0.641 + 0.318 + 0.539 + 0.214 = 1.712\, ft^3/s$$

The runoff volume of the green area:

$$Q_{g1} = 0{,}25 * 3{,}38 * 10^{-5} * 8204.15\, ft^2 = 0.069\, ft^3/s$$

$$Q_{g2} = 0{,}25 * 3{,}38 * 10^{-5} * 8414.78\, ft^2 = 0.071\, ft^3/s$$

$$Q_{g3} = 0{,}25 * 3{,}38 * 10^{-5} * 7630.55\, ft^2 = 0.064\, ft^3/s$$

$$Q_{g4} = 0{,}25 * 3{,}38 * 10^{-5} * 7841.59\, ft^2 = 0.066\, ft^3/s$$

$$Q_{total} = 0.069 + 0.071 + 0.064 + 0.066 = 0.27\, ft^3/s$$

The total runoff volume of the project area:

$$Q_{total} = Q_{roof} + Q_{parking\ lot} + Q_{green\ area}$$

$$Q_{total} = (1{,}021 + 1{,}712 + 0{,}270) ft^3 = 3.003\, ft^3$$

The time of concentration is the duration required for water to travel from the hydraulically most distant point to the outlet. Water is considered to travel through a watershed in the form of sheet flow, shallow concentrated flow, open channel flow, or a combination of these movement types. Thus, time of concentration is calculated by summing travel times for various consecutive flow segments:

$$T_c = \sum T^{sheet}$$

$$T_c - time\ of\ concentration;$$

T^{sha} $\quad - shallow\ concentrated\ flow\ time;$

Since stormwater from our structure initially flows over the smooth surface of the building's roof and parking lot, we determine the time of concentration for **sheet flow**. The most common equation for estimating sheet flow time is the **Manning's kinematic equation**:

$$T^{sheet} = \frac{0.42(n_s * L)^{0.8}}{((P_{24})^{0.5} * (S_0)^{0.4})}$$

$$T^{she} \quad - sheet\ flow\ travel\ time\ in\ minutes;$$

$$n_s - roughness\ coefficient\ ;$$

$$L - flow\ length\ in\ feet\ ;$$

$$S_0 - surface\ slope\ in\ foot/foot;$$

$$P_{24} - 2\ year, 24\ hour\ rainfall, inches;$$

Table 6.1. Roughness coefficients for overland flow

Surface Description		Roughness Coefficient, n
Pavement	Smooth asphalt	0.011
	Smooth concrete	0.012
Grass	Short grass prairie	0.15
	Dense grasses	0.24

Duration (hr)	Precipitation (in)										
	Recurrence Interval (years)										
		0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr		25-Yr	50-Yr	100-Yr	500-yr
6		0.78	0.92	1.07	1.27	1.42		1.64	1.80	1.98	2.39
12		1.10	1.32	1.55	1.86	2.08		2.42	2.67	2.93	3.55
24		1.46	1.77	2.10	2.55	2.87		3.36	3.73	4.12	5.04
48		1.76	2.16	2.57	3.14	3.55		4.16	4.64	5.12	6.29
72		2.16	2.63	3.11	3.75	4.21		4.88	5.39	5.90	7.09
168		3.09	3.76	4.42	5.27	5.86		6.70	7.30	7.88	9.18

Figure 6.12. Precipitation magnitude frequency estimates for SPU Gauge 11

Time of Concentration for sheet flow:

Roof:

$$T_{roof}^{sheet} = \frac{0.42(0.012*200.5)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 2.79\ min$$

Parking lot:

$$T_{p1}^{sheet} = \frac{0.42(0.011*294.27\ )^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 3.55\ min$$

$$T_{p2}^{sheet} = \frac{0.42(0.011*246.342)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 3.08\ min$$

$$T_{p3}^{sheet} = \frac{0.42(0.011*247.663)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 3.09\ min$$

$$T_{p4}^{sheet} = \frac{0.42(0.011*165.6)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 2.24\ min$$

$$T_c = \sum T^{sheet} = 11.96\ \ min$$

Green area:

$$T_{g1}^{sheet} = \frac{0.42(0.15*343.27)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 32.44\ min$$

$$T_{g2}^{she} = \frac{0.42(0.15*352.083)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 33.11\ min$$

$$T_{g3}^{sheet} = \frac{0.42(0.15*319.27)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 30.61\ min$$

$$T_{g4}^{she} = \frac{0.42(0.15*328.1)^{0.8}}{((2.1)^{0.5}*(0.02)^{0.4})} = 31.29\ min$$

$$T_c = \sum T^{sheet} = 127.45\ min$$

6.1. Pipe sizing:

Manning roughness coefficient (for concrete pipes) = 0.013

The Manning Equation:

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{0.5}$$

$$D = (\frac{Q*n*4^{5/3}}{1.486*\pi*S^{1/2}})^{3/8}$$

$$A = \frac{\pi D^2}{4}$$

$$P = \pi D$$

$$R = \frac{D}{4}$$

$$D = (\frac{Q * n * 4^{5/3}}{1.486 * \pi * S^{1/2}})^{3/8}$$

Slope = 2%

Table 6.2. Parking Lot

Subarea	Peak Runoff (Q), ft^3/s	Diameter, in	Standard Pipe size, in
P1	0.641	7.56	8
P2	0.318	5.81	6
P3	0.539	7.08	8
P4	0.214	5.01	6

$$Q_{8-inch\ pipe} = 0.87\ ft^3/s\ >\ Q_{p1.p3}$$
$$Q_{6-inch\ pipe} = 0.38\ ft^3/s\ >\ Q_{p2.p4}$$

Table 6.3. Green Area

Subarea	Peak Runoff (Q), ft^3/s	Diameter, in	Standard Pipe size, in
G1	0.069	3.27	4
G2	0.071	3.31	4
G3	0.064	3.18	4
G4	0.066	3.22	4

$$Q_{4-inch\ pipe} = 0.12\ ft^3/s\ >\ Q_{g1.g2,g3,g4}$$

1. Roof

$$D = \left(\frac{Q * n * 4^{\frac{5}{3}}}{1.486 * \pi * S^{\frac{1}{2}}}\right)^{\frac{3}{8}} = \left(\frac{1.021 * 0.013 * 4^{\frac{5}{3}}}{1.486 * \pi * 0.02^{\frac{1}{2}}}\right)^{\frac{3}{8}} = 8.3\ in\ (Use\ 10 - inch\ pipe)$$

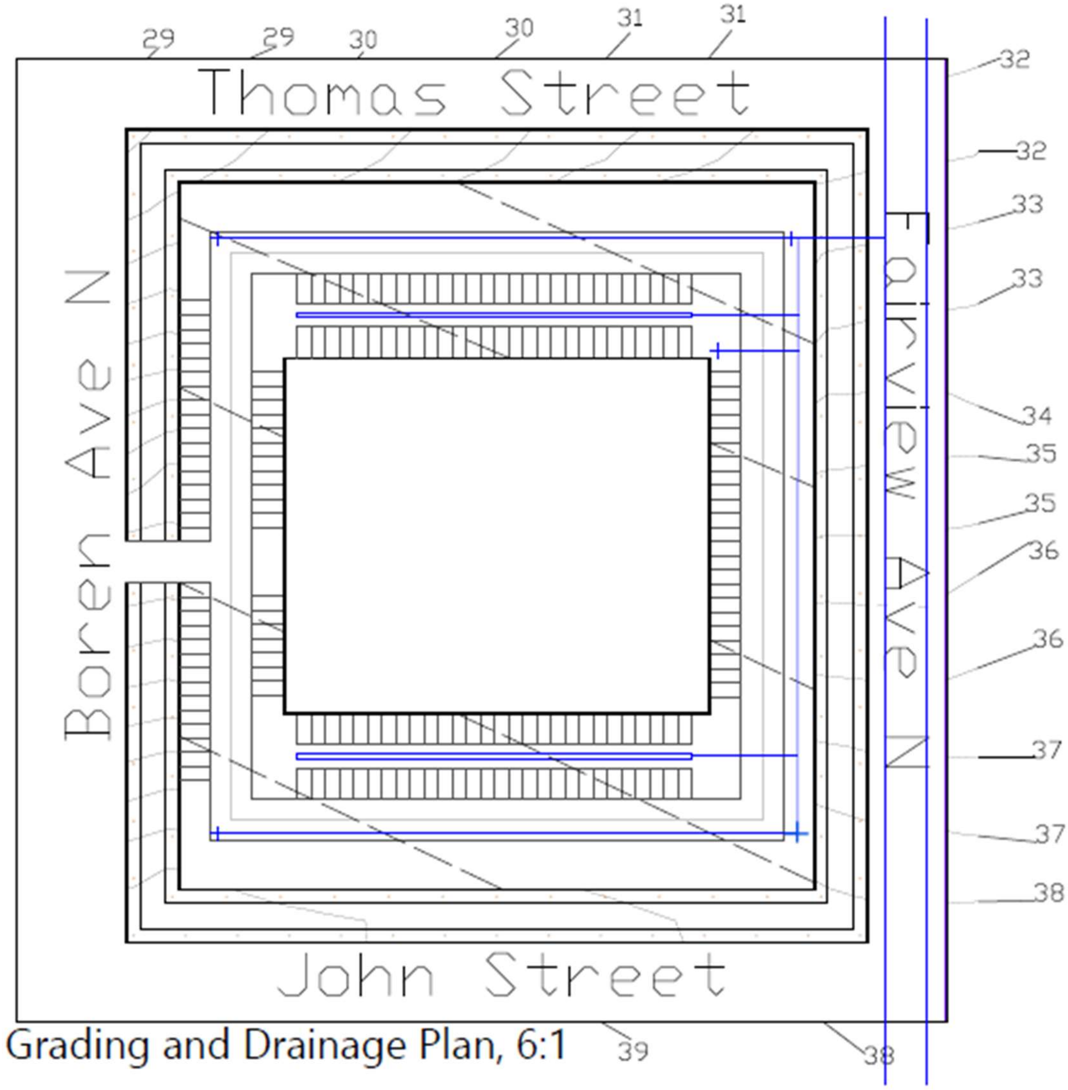


Figure 6.13. Final Drainage Plan

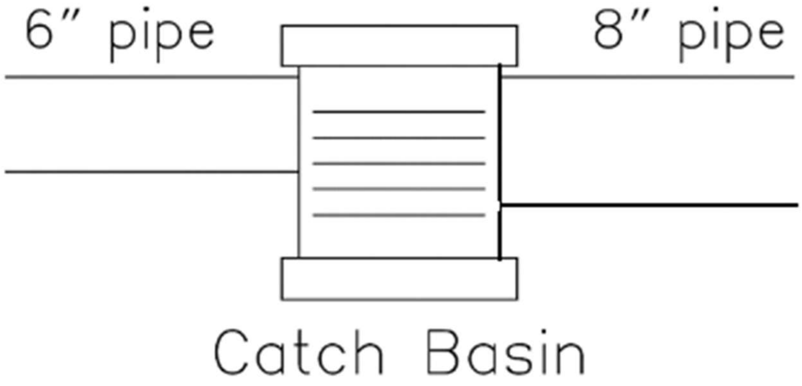


Figure 6.14. Pipe Connection

7. Construction Management

7.1. Project Charter

The Project Charter shows an overall description of the construction project. Including all the main dates and features of the project. In addition, it contains overall information about the purpose of construction and the estimated cost.

Table 7.1. Project Charter

<table>
<tr><th colspan="4">Project Overview</th></tr>
<tr><td>Title</td><td colspan="3">Design of the High-Rise Hotel Building in a High Seismic Zone in Seattle, Washington USA</td></tr>
<tr><td>Description</td><td colspan="3">The project is a high-rise hotel building in a high seismic zone. The name of the hotel is “Pacific Skyline Lodge”, and it consists of 12 floors. The location of this building is 211 Fairview Ave N, Seattle, WA 98109, USA. Restaurants, shops, city attractions, banks, etc. surround the chosen location. Which makes this location convenient for tourists and city guests.</td></tr>
<tr><td>Start Date</td><td>August 2024</td><td>Finish Date</td><td>July 2027</td></tr>
<tr><td>Purpose</td><td colspan="3">The main goal of this project is to design and construct a 12-storey building in a high seismic zone. Additionally, consider geotechnical, structural, architectural, environmental, and construction management standards.</td></tr>
<tr><td>Key Features</td><td colspan="3">• Number of floors: 12
• 1 storey basement
• Number of rooms per floor: 20
• Capacity of people: 40 (per floor); 9 floors = 360
• Fitness centre, SPA, and restaurant included
• Parking</td></tr>
</table>

Assumptions	• No unexpected shifts in regulations or the natural environment, any structural design will be ready to meet high seismic zone requirements as per the City of Seattle • All construction activities shall be performed in conformance with prevailing safety and labour standards for the State of Washington • Seasonal weather conditions for Seattle are not expected to significantly influence any delays. Weather-dependent activities will be scheduled whenever the conditions are most appropriate. • There will always be skilled labour available at all points in time of the project cycle without any major shortage that can have a deterrent effect on productivity.		
Estimated Cost	$ 76,363,066.94		
Key Dates	**Stage**	**Start Date**	**End Date**
	Planning	23.08.2024	09.11.2024
	Site Preparation	13.11.2024	06.02.2025
	Structural	07.02.2025	12.01.2027
	Finishing	13.01.2027	07.09.2027
	Project Finalization	08.09.2027	14.07.2027
Team Members	Arnur Amangeldy Nurgul Amangaliyeva Nursaule Kabizhan Amirzhan Bitimov Rassul Kabdrashitov Veronika Ten Zhanna Kussainova		

7.2. Feasibility Study

The analysis was conducted during the construction project's beginning phase. A feasibility study was also carried out to determine the project's viability.

1. Site Analysis.

211 Fairview Ave N, Seattle, WA 98109, USA was chosen as the location for the hotel. This location is advantageous and convenient for hotel guests as it is close to restaurants, attractions, shopping and business centres. It is anticipated that this location would be convenient for tourists. As well as for business visitors, due to its proximity to important places in the city. With that, the revenue from the hotel and rent is expected to be reasonable to pay back the construction of this project. In addition to that, public transit options could provide convenience and accessibility for guests without personal vehicles.

2. Environment Analysis.

Seattle is located in a high seismic risk area. Considering that is necessary to make a resilient and adaptable structure for the safety of the building. Environmental causes can potentially create risks. These risks were also considered during the project design stage. Environmental regulations will require all necessary permits and inspections. The regulations are made in accordance with the National Environmental Policy Act (NEPA). In addition to NEPA, and local environmental regulations policies, Seattle in this case, will be required to build the construction project.

3. Economic Analysis.

The construction and operation of the hotel building will require staff and personnel. This will generate employment opportunities in Seattle. Numerous skilled labours is required in a construction site. For stable work at the Pacific Lodge Hotel, work staff in hospitality, maintenance, and management are required. Consequently, it will open new vacant job places with long-term positions.

Financing may be given in the form of private equity investment, bank loans or government incentives for those people who construct in an “environmentally friendly” manner. Create a detailed financing plan that will include contingency budgeting for unforeseen seismic-related adjustments. Operating Expenses will include staffing, maintenance, utilities, etc., but will emphasize sustainability by minimizing costs.

Energy-efficient systems are likely to cut utility costs by some 15-20% over the life cycle of the building.
The high-traffic location and competitive amenities of the hotel project promise favourable occupancy rates, with forecasted average room rates above the Seattle market average. This project is supposed to realize its ROI within the first 5-7 years, with an occupancy of 70-80% per year, contingent upon operational cost management.

4. Risk Assessment

The structural reinforcements will include dampers and base isolators for seismic risk mitigation. While this adds to the upfront cost, it is essential for this highly risky area to ensure compliance and safety.
Occupancies may be influenced by economic downturns and/or decreased tourism. A conservative financial model will be prepared with a contingency for market fluctuations to offset any shortfalls in revenues.
Changes in building codes or environmental regulations may, therefore, affect project schedules and costs. As a matter of fact, regular consultation with regulatory authorities will be quite necessary as a part of proactive design adjustments to avoid delays or additional expenses.
Construction Risks The delays in the constructions might be due to bad weather conditions, problems in material supply chains, or shortage of labours. A complete project management plan would be involved that would consider risk management strategies along with buffer timelines and contract provisions that will guarantee timely completion of projects.

7.3. Cost Estimation

7.3.1. Preliminary Cost Estimation Using RS Means

The preliminary costs were calculated using RS Means software.

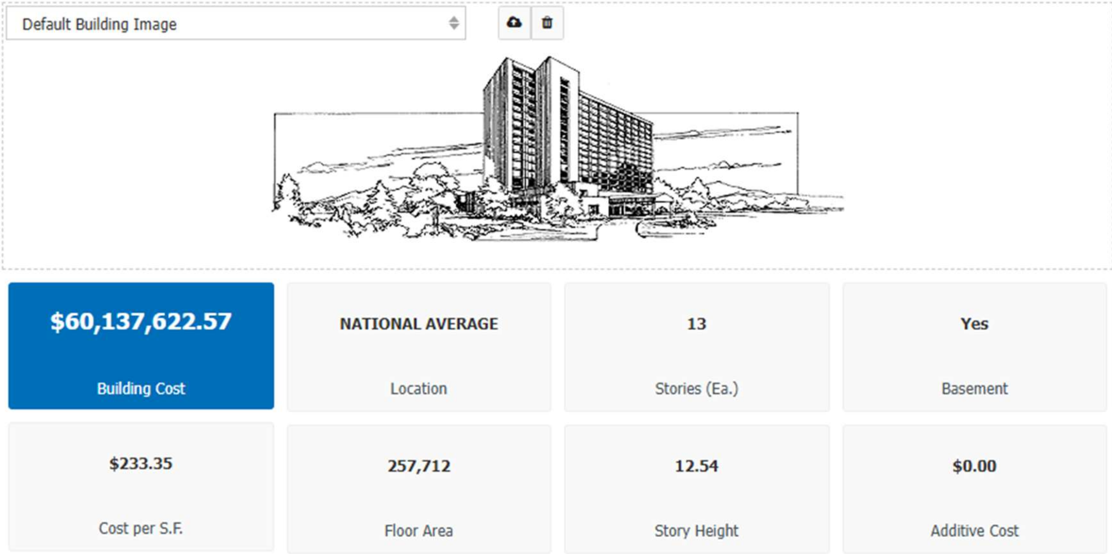


Figure 7.1 Building Cost via RSMeans

Table 7.2. Building Cost Estimate via RSMeans

		Quantity	**% of Total**	**Cost per S.F.**	**Cost**
A	Substructure		5.93%	$10.06	$2,592,735.23
A1010	Standard Foundations			$0.44	$112,119.48
	Pile caps, 6 piles, 8' - 6" x 5' - 6" x 40", 80-ton capacity, 19" column size, 936 K column	18.32		$0.15	$38,026.99
	Pile caps, 8 piles, 8' - 6" x 7' - 9" x 44", 80-ton capacity, 22" column size, 1243 K column	25.77		$0.29	$74,092.49
A1020	Special Foundations			$8.01	$2,065,043.61

	Steel H piles, 100' long, 800K load, end bearing, 5 pile cluster	18.32		$2.29	$590,563.66
	Steel H piles, 100' long, 1200K load, end bearing, 8 pile cluster	25.77		$5.15	$1,327,221.95
	Grade beam, 30' span, 52" deep, 14" wide, 12 KLF load	729		$0.57	$147,258.00
A1030	Slab on Grade			$0.45	$114,979.65
	Slab on grade, 4" thick, non-industrial, reinforced	19824.07		$0.45	$114,979.65
A2010	Basement Excavation			$0.29	$75,331.49
	Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on site storage	19824.07		$0.29	$75,331.49
A2020	Basement Walls			$0.87	$225,261.00
	Foundation wall, CIP, 12' wall height, pumped, .591 CY/LF, 28.79 PLF, 16" thick	729		$0.87	$225,261.00
B	Shell		32.78%	$55.63	$14,337,585.51
B1010	Floor Construction			$35.81	$9,229,130.51

	Cast-in-place concrete column, 26" square, tied, 1000K load, 12' story height, 667 lbs/LF, 4000PSI	1999.93		$1.65	$423,985.27
	Cast-in-place concrete column, 12", square, tied, minimum reinforcing, 150K load, 10'-14' story height, 135 lbs/LF, 4000PSI	954.99		$0.26	$66,467.30
	Cast-in-place concrete column, 16", square, tied, minimum reinforcing, 300K load, 10'-14' story height, 240 lbs/LF, 4000PSI	954.99		$0.35	$91,201.55
	Cast-in-place concrete column, 20", square, tied, minimum reinforcing, 500K load, 10'-14' story height, 375 lbs/LF, 4000PSI	954.99		$0.51	$130,356.14
	Concrete I beam, precast, 18" x 36", 790 PLF, 25' span, 6.44 KLF superimposed load	12305.52		$15.58	$4,014,675.90
	Flat slab, concrete, with drop panels, 6"	19824.07		$1.14	$294,387.54

	slab/2.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 PSF total load				
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 30 PSF superimposed load, 120 PSF total load	237888.92		$15.02	$3,870,452.78
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 75 PSF superimposed load, 165 PSF total load	19824.07		$1.31	$337,604.03
B2010	Exterior Walls			$15.76	$4,060,704.96
	Brick wall, composite double wythe, standard face/CMU back-up, 8" thick, perlite core fill, 3" XPS	98560.8		$15.76	$4,060,704.96
B2020	Exterior Windows			$3.14	$808,841.35
	Windows, aluminum, awning, insulated glass, 4'-5" x 5'-3"	1071.31		$3.14	$808,841.35
B2030	Exterior Doors			$0.32	$81,551.85

	Door, aluminum & glass, without transom, narrow stile, with panic hardware, 3'-0"x 7'-0"opening	5.72		$0.08	$21,762.43
	Door, aluminum & glass, without transom, narrow stile, double door, hardware, 6'-0" x 7'-0" opening	6.87		$0.17	$43,639.40
	Door, steel 18-gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening	5.72		$0.06	$16,150.02
B3010	Roof Coverings			$0.58	$150,348.84
	Roofing, single ply membrane, EPDM, 60 mils, loosely laid, stone ballast	19824.07		$0.13	$33,304.45
	Insulation, rigid, roof deck, extruded polystyrene, 40 PSI compressive strength, 4" thick, R20	19824.07		$0.32	$83,459.36
	Roof edges, aluminum, duranodic, .050" thick, 6" face	729		$0.08	$20,557.80

	Flashing, aluminum, no backing sides, .019"	729		$0.02	$4,096.98
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick	729		$0.03	$8,930.25
B3020	Roof Openings			$0.03	$7,008.00
	Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs	6		$0.03	$7,008.00
C	Interiors		21.44%	$36.38	$9,375,265.37
C1010	Partitions			$6.27	$1,615,029.36
	Concrete block (CMU) partition, light weight, hollow, 6" thick, no finish	22907.82		$0.92	$238,241.35
	Metal partition, 5/8"fire rated gypsum board face, no base,3 -5/8" @ 24" OC framing, same opposite face, sound attenuation insulation	206170.4		$4.70	$1,210,220.25
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"	98560.8		$0.39	$100,532.02

	Add for the following: taping and finishing	98560.8		$0.26	$66,035.74
C1020	Interior Doors			$13.58	$3,499,169.84
	Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"	2863.47		$13.58	$3,499,169.84
C2010	Stair Construction			$2.18	$561,012.57
	Stairs, steel, pan tread for conc in-fill, picket rail, 16 risers w/ landing	35.50		$2.18	$561,012.57
C3010	Wall Finishes			$4.49	$1,157,972.57
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	366525.15		$1.27	$326,207.39
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	98560.8		$0.34	$87,719.11
	Ceramic tile, thin set, 4-1/4" x 4-1/4"	91631.28		$2.89	$744,046.07
C3020	Floor Finishes			$5.67	$1,462,263.56

	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 35 oz	206170.4		$4.17	$1,074,147.78
	Vinyl, composition tile, maximum	25771.3		$0.31	$78,860.18
	Tile, ceramic natural clay	25771.3		$1.20	$309,255.60
C3030	Ceiling Finishes			$4.19	$1,079,817.47
	Gypsum board ceilings, 5/8" fire rated gypsum board, painted and textured finish,1-5/8" metal stud furring, 24" OC support	257713		$4.19	$1,079,817.47
D	Services		39.85%	$67.64	$17,430,866.67
D1010	Elevators and Lifts			$6.91	$1,780,796.83
	Traction geared freight, 4000 lb., 15 floors, 10' story height, 200FPM	0.57		$1.30	$334,740.55
	Traction, geared passenger, 3500 lb,15 floors, 10' story height, 2 car group, 350 FPM	2.86		$5.61	$1,446,056.28
D2010	Plumbing Fixtures			$19.47	$5,016,437.96

	Water closet, vitreous china, bowl only with flush valve, wall hung	644.28		$8.50	$2,190,560.50
	Urinal, vitreous china, wall hung	14.31		$0.08	$21,189.74
	Lavatory w/trim, vanity top, PE on CI, 20" x 18"	644.28		$3.48	$895,552.68
	Kitchen sink w/trim, countertop, stainless steel, 33" x 22" double bowl	4.58		$0.04	$10,720.86
	Service sink w/trim, PE on CI,wall hung w/rim guard, 22" x 18"	17.18		$0.25	$64,857.77
	Bathtub, recessed, PE on CI, mat bottom, 5' long	644.28		$6.89	$1,774,998.29
	Shower, stall, baked enamel, terrazzo receptor, 36" square	14.31		$0.16	$41,377.25
	Water cooler, electric, wall hung, wheelchair type, 7.5 GPH	8.59		$0.07	$17,180.87
D2020	Domestic Water Distribution			$0.21	$54,277.22

	Gas fired water heater, commercial, 100< F rise, 500 MBH input, 480 GPH	1.94		$0.21	$54,277.22
D2040	Rainwater Drainage			$0.24	$61,736.58
	Roof drain, CI, soil,single hub, 5" diam, 10' high	4.58		$0.05	$13,630.15
	Roof drain, CI, soil,single hub, 5" diam, for each additional foot add	641.41		$0.19	$48,106.43
D3010	Energy Supply			$2.84	$731,389.49
	Commercial building heating system, fin tube radiation, forced hot water, 1mil SF, 10 mil CF, total 5 floors	283484.3		$2.84	$731,389.49
D3030	Cooling Generating Systems			$14.00	$3,607,982.00
	Packaged chiller, water cooled, with fan coil unit, medical centers, 60,000 SF, 140.00 ton	257713		$14.00	$3,607,982.00
D4010	Sprinklers			$4.14	$1,067,408.59
	Wet pipe sprinkler systems, steel, light	180399.1		$2.05	$528,569.36

	hazard, 1 floor, 50,000 SF				
	Wet pipe sprinkler systems, steel, light hazard, each additional floor, 50,000 SF	239673.09		$2.04	$524,884.07
	Standard High Rise Accessory Package 16 story	0.54		$0.05	$13,955.16
D4020	Standpipes			$4.33	$1,114,703.37
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor	1.71		$0.11	$27,575.29
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, additional floors	240.53		$4.08	$1,052,328.08
	Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM	1		$0.12	$31,000.00
	Fire pump, electric, for jockey pump system, add	1		$0.01	$3,800.00
D5010	Electrical Service/Distribution			$1.65	$426,340.00

	Underground service installation, includes excavation, backfill, and compaction, 100' length, 4' depth, 3 phase, 4 wire, 277/480 volts, 2000 A	2		$0.40	$103,800.00
	Feeder installation 600 V, including RGS conduit and XHHW wire, 60 A	100		$0.01	$1,990.00
	Feeder installation 600 V, including RGS conduit and XHHW wire, 200 A	100		$0.02	$4,750.00
	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A	400		$0.76	$196,400.00
	Switchgear installation, incl switchboard, panels & circuit breaker, 277/480 V, 2000 A	2		$0.46	$119,400.00
D5020	Lighting and Branch Wiring			$9.05	$2,332,632.48
	Receptacles incl plate, box, conduit, wire, 10	262867.26		$3.89	$1,001,524.26

	per 1000 SF, 1.2 W per SF, with transformer				
	Wall switches, 5.0 per 1000 SF	257713		$1.30	$335,026.90
	Miscellaneous power, to .5 watts	257713		$0.15	$38,656.95
	Central air conditioning power, 4 watts	314409.86		$0.77	$198,078.21
	Motor installation, three phase, 460 V, 15 HP motor size	10		$0.10	$26,200.00
	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		$0.02	$5,685.00
	Motor connections, three phase, 200/230/460/575 V, up to 5 HP	1			$132.50
	Motor connections, three phase, 200/230/460/575 V, up to 100 HP	1			$578.00
	Fluorescent fixtures recess mounted in ceiling, 0.8 watt per SF,	257713		$2.82	$726,750.66

	20 FC, 5 fixtures @32 watt per 1000 SF				
D5030	Communications and Security			$4.47	$1,151,089.44
	Communication and alarm systems, fire detection, addressable, 100 detectors, includes outlets, boxes, conduit and wire	6.58		$2.00	$516,342.31
	Fire alarm command center, addressable with voice, excl. wire & conduit	1.71		$0.08	$21,347.23
	Communication and alarm systems, includes outlets, boxes, conduit and wire, intercom systems, 100 stations	2.02		$1.06	$273,691.21
	Communication and alarm systems, includes outlets, boxes, conduit and wire, master TV antenna systems,100 outlets	1.52		$0.83	$214,073.60
	Internet wiring, 2 data/voice outlets per 1000 S.F.	193.28		$0.49	$125,635.09

D5090	Other Electrical Systems			$0.33	$86,072.71
	Generator sets, w/battery, charger, muffler and transfer switch, diesel engine with fuel tank, 500 kW	387.71		$0.33	$86,072.71
E	Equipment & Furnishings		0%		
E1090	Other Equipment				
F	Special Construction		0%		
G	Building Sitework		0%		
	Subtotal		100%	$169.71	$43,736,452.78
	Contractor Fees (GC, Overhead, Profit)		25.0%	$42.43	$10,934,113.20
	Architectural Fees		10.0%	$21.21	$5,467,056.60
	User Fees		0.0%	$0.00	$0.00
	Total Building Cost			$233.35	$60,137,622.57

Table 7.3. Cost Estimation

Cost Estimation	
Land Cost	$ 6,064,980.81
Materials	$ 60,137,622.57

Labour	$ 13,047,226.09
Services	$ 1,758,611.12
Taxes	$ 8,384,373.60
Total Cost	$ 89,392,814.19

The cost of land was calculated using the nearest land on sale. It is located in ~2 km from our chosen location. The address of the closest location on sale is 2211 16th Ave E, Seattle, WA 98112. It costs $599,950, which is $56 per square foot. Our chosen location is 108303.23 sq. ft. which makes our cost of land equal to $6,064,980.81. Average labour cost in Seattle has a range of $240-550 per sq. ft. The highest amount was used in calculations to show the highest possible cost for this category. In addition, the tax rate in Seattle was calculated. The tax rate for Seattle, WA is 10.35%. It is estimated a minimum combined sales tax rate for this state in 2024.

Cost of materials was calculated using RSMeans software. Due to the fact that RSMeans uses prices from 2018, the realistic total cost should be calculated with the average inflation rate. The inflation rate from 2018 to 2024 is around 30-40%. For this project, 35% of inflation rate was taken.

After the final cost was calculated, the total was increased by 15%, in order to make a reserve of funds for unforeseen circumstances.

Table 7.4. Finalisation of Cost

Totals	
Cost + Inflation Rate	$ 120,680,299.16
Final Cost	$ 138,782,344.03

Table 7.5. Payback period estimation

Year	Cost room/night (USD)	Annual Room Revenue (USD)	Total Annual Revenue (USD)	Cumulative Revenue (USD)	Net Present Value (USD)
1	273.00	12286228.50	12386228.50	12386228.50	-126396115.53
2	300.30	13514851.35	13614851.35	26001079.85	-112781264.18
3	330.33	14866336.49	14966336.49	40967416.34	-97814927.70
4	363.36	16352970.13	16452970.13	57420386.47	-81361957.56
5	399.70	17988267.15	18088267.15	75508653.62	-63273690.41
6	439.67	19787093.86	19887093.86	95395747.48	-43386596.55
7	483.64	21765803.25	21865803.25	117261550.72	-21520793.31
8	532.00	23942383.57	24042383.57	141303934.30	2521590.27
9	585.20	26336621.93	26436621.93	167740556.23	28958212.20
Construction Estimated		138782344.03			

The payback period was calculated as can be seen from Figure 7.5. There are 180 rooms in the hotel, 20 rooms per story. In one room, 2 people can be located. Overall, full capacity 360 guests. Assuming 273 USD per room per night. It is an average price in the hotels near our chosen location. And the average occupancy rate is 68.5% in Seattle in 2024. Assuming the increase in price per room of 10% every year from second year. With rent of space on the 1st/2nd floors at around $ 200,000. The average payback period is estimated to be around 8 years.

7.4. Schedule

7.4.1. Work Breakdown Structure

For this project, the Work Breakdown Structure (WBS) was created. The WBS for this project is shown in Figure 6.2. It is divided into 5 parts: Planning, Site Preparation, Structural Work, Finishing, and Project Finalisation. Every part has subparts that show the main work that needs to be done in the flow of the project.

Figure 7.2. Work Breakdown Structure

7.4.2. Scheduling

The preliminary schedule was created using Primavera P6 software.

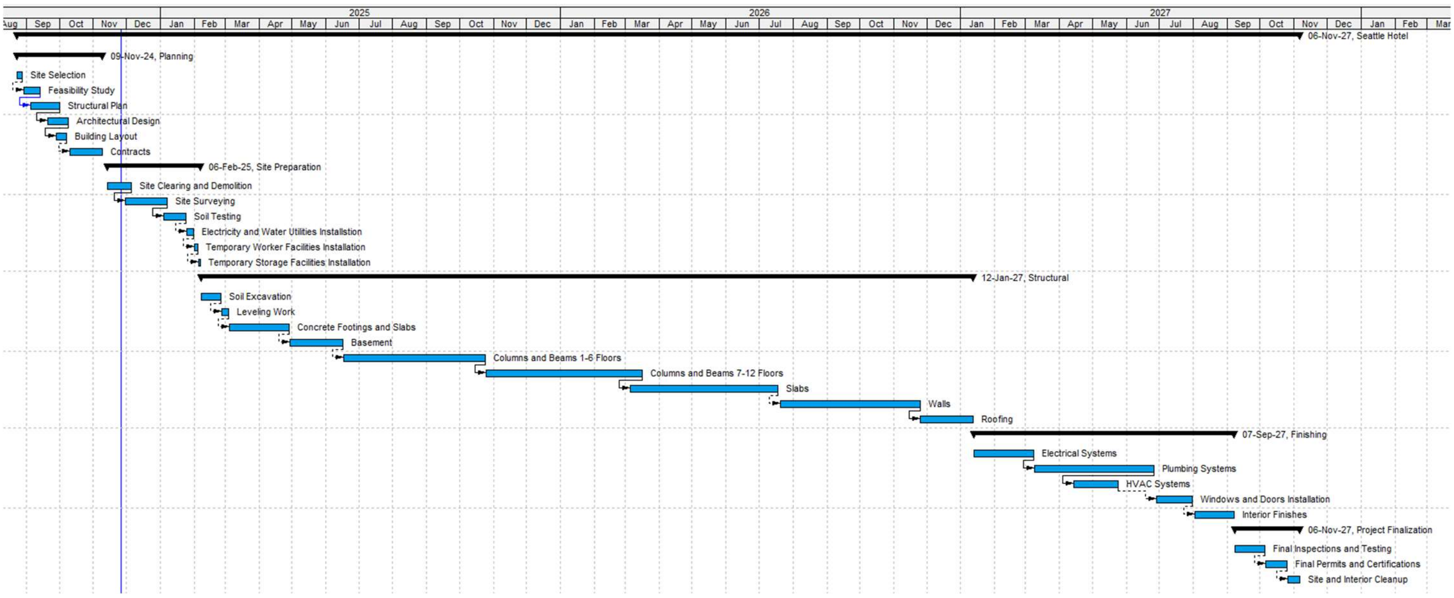


Figure 7.3. Schedule in Primavera P6

Activities

Layout: Classic Schedule Layout | Filter: All Activities

Activity ID	Activity Name	Original Duration	Start	Finish	Resources
Seattle Hotel		848	23-Aug-24	06-Nov-27	
Planning		58	23-Aug-24	09-Nov-24	
1.01	Site Selection	4	23-Aug-24	28-Aug-24	Client/Project Owner
1.02	Feasibility Study	10	29-Aug-24	13-Sep-24	Project Engineer
1.03	Structural Plan	20	04-Sep-24	01-Oct-24	Structural Engineer
1.04	Architectural Design	15	20-Sep-24	09-Oct-24	Architect
1.05	Building Layout	7	28-Sep-24	07-Oct-24	Architect, Structural Engineer
1.06	Contracts	23	10-Oct-24	09-Nov-24	Project Manager
Site Preparation		59	13-Nov-24	06-Feb-25	
2.01	Site Clearing and Demolition	14	13-Nov-24	05-Dec-24	Site Subcontractor – Clearing & Demolition
2.02	Site Surveying	25	30-Nov-24	07-Jan-25	Surveying Subcontractor
2.03	Soil Testing	14	04-Jan-25	24-Jan-25	Soil Testing Subcontractor
2.04	Electricity and Water Utilities Installstion	5	25-Jan-25	31-Jan-25	MEP Engineers
2.05	Temporary Worker Facilities Installation	2	01-Feb-25	04-Feb-25	General Contractor
2.06	Temporary Storage Facilities Installation	2	05-Feb-25	06-Feb-25	General Contractor
Structural		508	07-Feb-25	12-Jan-27	
3.01	Soil Excavation	12	07-Feb-25	25-Feb-25	Excavation Subcontractor
3.02	Leveling Work	5	26-Feb-25	04-Mar-25	Site Subcontractor – Clearing & Demolition
3.03	Concrete Footings and Slabs	40	05-Mar-25	28-Apr-25	Concrete Subcontractor
3.04	Basement	35	29-Apr-25	16-Jun-25	Concrete Subcontractor
3.05	Columns and Beams 1-6 Floors	97	17-Jun-25	24-Oct-25	Structural Framing Subcontractor
3.06	Columns and Beams 7-12 Floors	98	25-Oct-25	16-Mar-26	Structural Framing Subcontractor
3.07	Slabs	99	05-Mar-26	18-Jul-26	Concrete Subcontractor
3.08	Walls	95	20-Jul-26	24-Nov-26	Structural Framing Subcontractor
3.09	Roofing	32	25-Nov-26	12-Jan-27	Roofing Subcontractor
Finishing		175	13-Jan-27	07-Sep-27	
4.01	Electrical Systems	40	13-Jan-27	08-Mar-27	MEP Subcontractor (Electrical)
4.02	Plumbing Systems	82	09-Mar-27	26-Jun-27	MEP Subcontractor (Plumbing)
4.03	HVAC Systems	30	14-Apr-27	24-May-27	MEP Subcontractor (HVAC)
4.04	Windows and Doors Installation	25	28-Jun-27	31-Jul-27	Windows & Doors Subcontractor
4.05	Interior Finishes	27	02-Aug-27	07-Sep-27	Finishing Subcontractor
Project Finalization		45	08-Sep-27	06-Nov-27	
5.01	Final Inspections and Testing	20	08-Sep-27	05-Oct-27	Authorities (Inspection, Permits, Final Approval)
5.02	Final Permits and Certifications	14	06-Oct-27	25-Oct-27	Authorities (Inspection, Permits, Final Approval)
5.03	Site and Interior Cleanup	10	26-Oct-27	06-Nov-27	Cleaning Subcontractor

Figure 7.4. Project Activities in Primavera P6

7.5. Risk Management

For the construction of a 12-storey hotel building in Seattle, the Modified Risk Matrix (MRM) assessment method by Guo et al. (2021) was used. There is a formula for identifying risk level, that is used in this particular method:

$Rik=L_{ik}S_{ik}$

Where,

R - risk level;

i - unsafe behaviour;

k - accident type;

L - level of probability of unsafe behaviour;

S - severity level of the consequence.

The Risk Assessment Table is shown below:

Table 7.6. Risk Assessment

Category		Risk	Likelihood (1-5)	Severity (1-5)	Risk Level	Mitigation Strategy
Site and Environmental	E1	Soil Instability / Liquefaction	3	5	15	Conduct comprehensive geotechnical studies and execute soil stabilization measures including piling and grading.
	E2	Weather Delays	4	3	12	Schedule weather-sensitive activities in favourable seasons; use rain covers, water-resistant materials.
	E3	Flooding and Stormwater	3	4	12	Design a sound stormwater management system; install drainage systems, including permeable paving.
	E4	Seismic Activity	5	5	25	Provide seismic-resistant design features including, among others, base isolators, and bracing systems.
Legal and Regulatory	L1	Permit Delays	3	4	12	Permit application process can be initiated

						early; keep city regulators regularly informed.
	L2	Changes in Building Codes/ Regulations	3	3	9	Periodically check updates to regulatory requirements; allow for design flexibility, if possible, to accommodate changes in the code.
	L3	Environmental Compliance Penalties	2	3	6	Adhere strictly to environmental regulations; all compliance audits of the company are to be conducted periodically.
Financial	F1	Cost Overruns	3	5	15	Negotiate fixed-price contracts with suppliers. Establish a contingency fund to cover price fluctuations.
	F2	Funding Shortfalls	2	5	10	Obtain a diversified funding base; prepare contingency plans as necessary for alternative financing.
	F3	Changes in labor and material prices	2	3	6	Research for alternative/

						unconventional materials.
Project Management	M1	Labour Shortages	3	4	12	Develop effective strategies for Staff Recruitment; make sure wages are competitive; work with reliable subcontractors.
	M2	Scheduling Delays	3	4	12	Monitor progress with project management software; proactively change timelines and communicate with your teams.
	M3	Contractor Performance Issues	1	4	4	Point out all the details in the contract, then regularly check the quality of work done during the regular checkups throughout the whole construction process.
Safety and Security	S1	Worker Injuries/ On-Site Accidents	3	5	15	Put strict safety protocols in place, including regular training and maintenance of safety equipment.
	S2	Fire Hazard	2	3	6	Use fire-resistant materials; install

						temporary fire suppression systems during construction.
Design and Quality	D1	Design Flaws / Revisions	3	3	9	Regularly review the design with stakeholders and allow time for revisions.
	D2	Structural Integrity Issues	2	5	10	Conform to seismic and structural design codes with rigors for quality controls.
	D3	Poor Quality Materials	2	3	6	Source materials from validated suppliers; inspect regularly upon delivery.
Supply Chain	SC1	Material Delivery Delays	3	3	9	Diversify suppliers, place orders well in advance, and make provisions for holding inventory buffers of critical materials.
	SC2	Shortage of Specialized Equipment	2	3	6	Rent specialized equipment early; establish relationships with multiple equipment rental agencies.
Operational	O1	Unexpected Utility Disruptions	2	3	6	Coordinate utility companies; design the temporary utility plan,

						including backup generators.
	O2	Community Complains	3	2	6	Plan noisy works during the hours that noise is acceptable; where practicable, use methods of sound dampening.

After calculating the risk level for the different types of potential risks, the risk matrix is designed.

L_{ik}	S_{ik}				
	1 (Negligible)	2 (Minor)	3 (Moderate)	4 (Serious)	5 (Critical)
1 (Remote)	I	I	I	I	II
2 (Unlikely)	I	I	II	II	III
3 (Likely)	I	II	II	III	III
4 (Highly likely)	I	II	III	III	IV
5 (Near certainty)	II	III	III	IV	IV

Figure 7.5. Modified Risk Matrix (MRM) (Guo et al., 2021)

The Risk Matrix for the hotel building project is shown below:

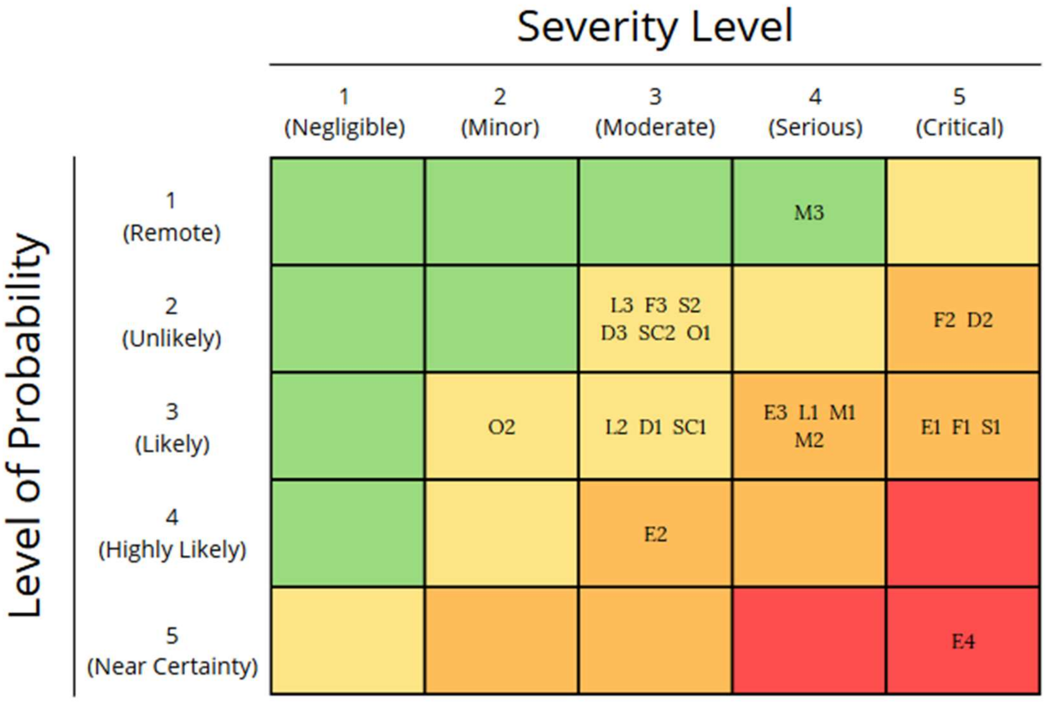


Figure 7.6. Modified Risk Matrix (MRM)

For the hotel building with 12 stories in Seattle, several risks categories are calculated. The highest risk is E4, that can be seen from Figure 6.6. This risk is associated with Seismic Activity, which requires the most attention, especially in a high seismic zone. This risk is a high priority, and the mitigation strategy should be strictly followed. Lower risk level does not show that it is not requires attention. The requirements and rules should be followed to prevent the occurrence of the risks, to make a construction project with quality, and without any delays.

7.6. Quality Management

The structured approach should be followed in every phase of the development of the 12-story hotel in Seattle. The Quality Management Plan shall define measures, checklists, and metrics that are in conformance with industry standards and regulatory requirements. The quality management will seek to maintain high-quality construction with minimal defects that could affect safety. This plan follows ISO 9001:2015, ASTM, ANSI, and OSHA standards, and it also adheres to the SBC regulations.

7.6.1. Quality Control Measures

1. Inspection and Testing of Materials

The prime aim of inspection and testing is compliance of project specification with the general industry standards on all materials used during the actual construction process. On delivery at the site, the materials to be used must meet the relevant standards laid out by ASTM (American Society for Testing and Materials) along with the minimum specifications from ANSI (American National Standards Institute). The following step is the laboratory testing of structural materials, such as steel, concrete, wood, etc. It ensures that the strength, durability, and moisture content of the materials are in accordance with requirements. Further, materials to be accepted on-site require the process of revision of quality certificates from suppliers. Substandard materials are rejected or replaced in order to prevent construction defects. This process ensured the durability and safety of the finished construction project.

2. Workmanship Assessment

A workmanship review is necessary to ensure that the construction activities are within the specification of the design and in line with the industry practices. The workmanship specifications are per SBC and IBC standards. All non-conformities against the specification are reported for immediate follow-up action. Third-party consultants may also be engaged in reviewing critical stages such as foundation works and seismic reinforcement.

3. Non-Conformance Reporting (NCR)

The NCR or Non-Conformance Report is written and issued for failed quality standards. Those corrective actions will have to be done within a specified timeframe and also have to include follow-up inspection. Continuous Improvement: Follow established continuous improvement practices, including documenting and using lessons learned from NCRs.

4. Subcontractor Quality Management

This section is essential to ensure that subcontractors comply with the quality standards of the specific project. The process here is that all subcontractors have to provide QCPs prior to starting the work. Subsequently, their performance is monitored through weekly and monthly reviews. Non-compliant subcontractors are given corrective action plans or replaced if need be.

5. Document Control

The important part of all projects, including construction industry, is to maintain accurate and updated documentation. A digital document management system will be used to store material test reports, inspection checklists, supplier certifications, and NCR reports. The ISO 9001:2015 framework will be applied to ensure that all records are properly maintained and easily accessible for reference during audits or regulatory inspections (ISO, 2023).

7.6.2. Quality Metrics

Figure 7.7. Quality Metrics

Table 7.7. Quality Metrics Description

Metric Category	Description	Target
Material Quality Compliance	Ensures that all materials meet specifications through inspections and laboratory testing	100% compliance

Rework Percentage	Measures errors requiring rework, reducing waste and delays	≤ 5% of total work
Structural Integrity & Safety Compliance	Ensures concrete strength, steel quality, and overall stability through testing	100% compliance
Task Completion Rate	Tracks if work is completed as per schedule	≥ 90% of tasks on time
Cost Variance	Monitors budget deviations to control cost overruns	≤ 10% variance from budget
Waste Management Efficiency	Measures material recycling and disposal	≥ 50% of materials recycled

7.6.3. Quality Control Checklist

Using quality control metrics, the checklist was developed, to ensure that the construction is going without any issues. The checklist can be seen on Figure 6.8. The checklist is divided into 4 phases. All phases are representing the different parts of the construction process.

Hotel Construction Quality Control Checklist

Project Information:

Project Name: "Pacific Skyline Lodge" Hotel

Date of Inspection: 23.03.2025

Location: 211 Fairview Ave N, Seattle, WA 98109, USA

Project Manager: Veronika Ten

Inspection Team Members:

- Nursaule Kabizhan
- Arnur Amangeldy
- Rassul Kabdrashitov

Phase 1: Pre-Construction Checklist

Site Preparation:

- Verify site clearing and removal of debris and vegetation
- Confirm grading and leveling according to site plans
- Check erosion and sediment control measures in place
- Ensure access routes for construction vehicles are established

Foundation Works:

- Confirm excavation depths and alignments are according to the drawings
- Verify soil conditions match geotechnical recommendations
- Inspect formwork and reinforcement placement
- Check that the concrete mix and curing procedures meet specifications

Phase 2: Structural Construction

Framing and Structural Elements

- Verify that structural steel and concrete installations follow design plans
- Inspect welding and bolting connections for compliance
- Confirm vertical and horizontal alignments

Exterior and Roofing:

- Check the installation of exterior walls, cladding, and waterproofing
- Inspect roofing materials and drainage system installation

Phase 3: Interior and Finishing

Interior Installations:

- Inspect framing, drywall, and partitions for accuracy
- Verify mechanical, electrical, and plumbing (MEP) rough-ins
- Confirm fire-stopping and sound-proofing installations

Finishing Works:

- Check painting, flooring, ceiling, and fixtures for quality standards
- Inspect doors, windows, and hardware installations

Phase 4: Final Inspections

- Verify system testing (fire alarm, HVAC, plumbing, electrical)
- Confirm all punch list items are completed
- Ensure the building is cleaned and ready for handover
- Obtain all necessary permits and certifications for occupancy

Figure 7.8. Quality Control Checklist

7.7. Construction Safety

To ensure the safety on the construction site, the safety risk register is developed. The analysis of all the main risks that could potentially occur are shown in the table below:

Table 7.8. Safety Risk Register

Hazard Type	Likelihood (1-5)	Severity (1-5)	Risk Level	Control Measures

Working at Heights	4	5	20	Worker training, use of a harness, secured scaffolding,
Slips, Trips, and Falls	3	3	9	Anti-slip mats, proper signage, regular cleaning
Heavy Material Handling	3	4	12	Mechanical lifting aids, worker training on safe lifting
Fire and Flammable Material Risks	2	5	10	Fire extinguishers, no smoking policy, proper storage
Foundation or Structural Failure	2	5	10	Soil testing, reinforcement checks
Inadequate Safety Gear (PPE)	3	4	12	Mandatory PPE use, routine safety audits
Extreme Weather Conditions	3	3	9	Weather monitoring, hydration stations, breaks in the shade
Electrical Accidents	3	4	12	Insulation tools, scheduled electrical inspections
High Noise Levels	2	3	6	Ear protection, noise-reduction barriers
Entrapment Hazards	4	4	16	Emergency stop buttons
Exposure to Hazardous Substances	3	4	12	Ventilation, protective masks, material safety training

Prolonged Vibration Exposure	3	3	9	Anti-vibration gloves, limited exposure times

All the risks are rated with two criteria that are likelihood and severity. These categories are calculated from 1 to 5, and the total number shows the main hazards on construction site. From this table, it can be seen that the main safety risks on site are working at heights and entrapment hazards, with 20 and 16 risk levels respectively.

7.8. Procurement planning

7.8.1. Stakeholders Analysis

The stakeholder analysis is made to show the interaction and the hierarchy of all the parties involved during the construction process. It is made to make a continuous work on the project, without any issues and delays. The stakeholder's hierarchy is made using the Primavera P6 software. All the parties, from the owner of a project to the subcontractors are shown on the figure below:

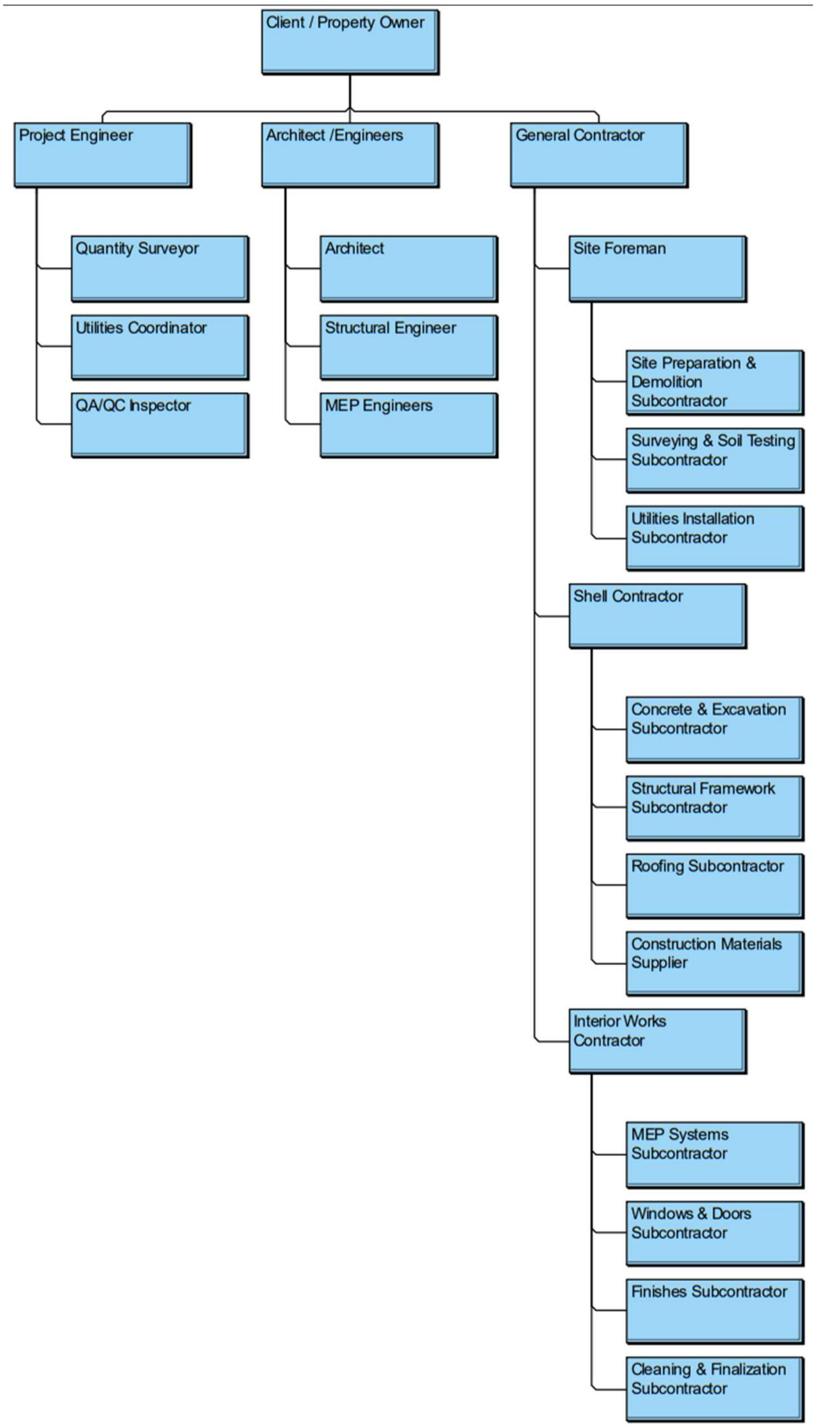


Figure 7.9. Stakeholders Hierarchy

The main party of the project is a client/property owner. They ensure that all the work is in progress, makes the important decisions, and is responsible for funding the project. The next tier of the project hierarchy are engineers, architect and general contractor. They are responsible for overall construction works and oversee the delegation of work with all the parties that are in the 3rd tier. Using this hierarchy, the stakeholder's analysis table is made, with the descriptions of each role, interest and influence criteria. The interest is the criteria that shows the involvement of the parties in the finished project outcome. The influence is the part that describes the importance of these stakeholders in the outcome and results. The higher interest and influence are, the more responsibilities the stakeholder has in the project. The stakeholder's analysis table is shown below:

Table 7.9. Stakeholders Analysis

Position of Stakeholder	Role in the Project	Interest	Influence
Client / Property Owner	Project Initiation, funding, decision-making	High	High
Architect & Engineers	Design of systems and structure	High	Moderate
Project Manager	Manages communication, scope, and execution	High	High
Regulatory Authorities	Permits granting, codes enforcement, safety compliance	Moderate	High
Design Consultants	Assistance with specialist aspects (interior, facade, etc.)	Moderate	Moderate
Main Contractor	Responsible for overall construction works	High	High
Subcontractors	Performance of specific trades (plumbing, electrical, etc.)	Moderate	Moderate
Suppliers	Materials and equipment supply	Moderate	Low
Utility Providers	Delivers electricity, water, gas, etc.	Low	Moderate
Environmental Consultants	Ensures that environmental regulations are followed	Moderate	Low
Site Workers / Technicians	Execution of physical construction tasks	Moderate	Low

Legal Advisors	Contracts and legal compliance management	Moderate	Low
Financial Advisors	Budget monitoring and cost control	Moderate	Moderate

7.9. Construction Site Planning

The construction site has entrances from Fairview Ave N (East Side) and Thomas St (North Side). The entrance gates are 6 meters wide, and the road on site is 5 meters wide. There are three stockpiles on site, aggregates, cement and drywall, and reinforcement. It also has equipment and motor park space, and several worker facilities that are located on the south side of the site. Two construction cranes will be installed on site, with a working radius of 50 m. The waste storage zone is located on the north-east corner of the construction site. It is located farther from all stockpiles and facilities to ensure the safety of workers on site.

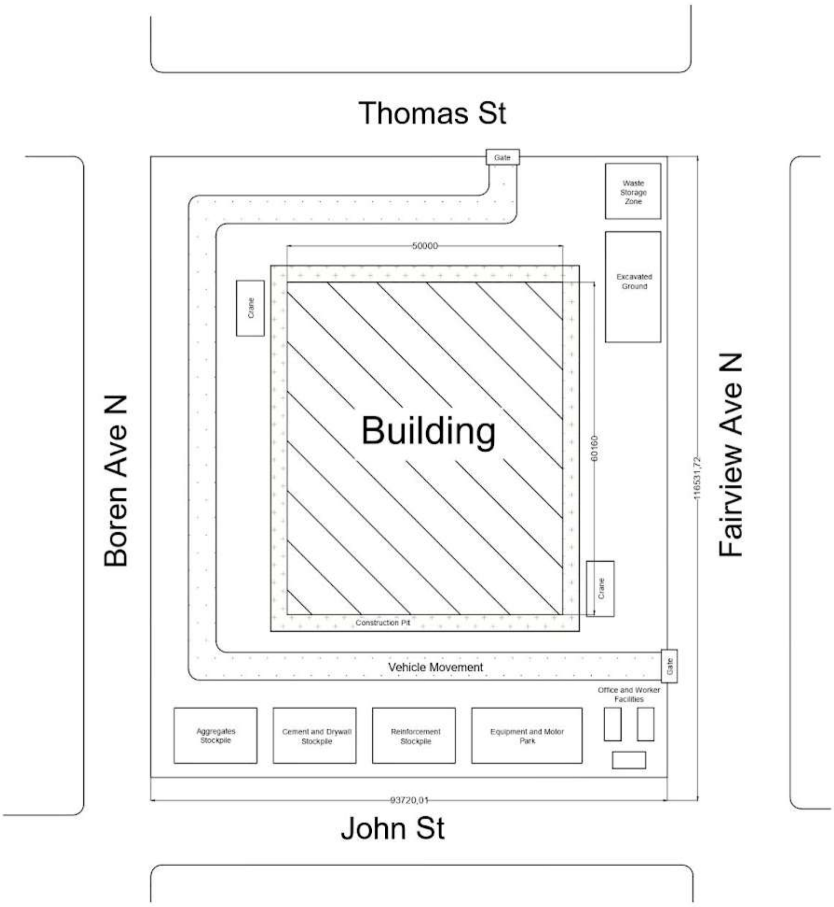


Figure 7.10. Site plan

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