

**DESIGN OF “SKYLINE PALACE”: A 12-STORY
HOTEL IN SEATTLE, WA, USA
(Capstone Project)**

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



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Declaration

We hereby declare that this report entitled “Design of ‘Skyline Palace’: a 12-story hotel in Seattle, WA, 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.

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Abstract

The proposed project involves the construction of a hotel located at 401 8th Avenue North in the South Lake Union neighborhood of Seattle, Washington. The site offers a prime location in a rapidly growing area known for its technological and biotechnological hubs, vibrant commercial spaces, and proximity to iconic landmarks such as the Space Needle. The hotel design accounts for Seattle's unique geotechnical and climatic conditions, including high seismic activity and moderate corrosion risk, ensuring compliance with International Building Code (IBC) standards.

Architectural plans include provisions for emergency evacuation, natural lighting, accessibility, and adequate parking facilities, all optimized using Revit tools. Construction materials, primarily reinforced concrete, are selected based on their durability, cost-effectiveness, and suitability for seismic resilience. The project addresses environmental and urban connectivity considerations, leveraging nearby public transport options and tourist attractions to maximize accessibility and appeal. This development is expected to contribute significantly to the local economy by catering to corporate clients, tourists, and medical visitors, aligning with the dynamic growth of South Lake Union.

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1. Introduction

1.1 Project description

This project entails the design of a 12-story hotel in Seattle, Washington. Included here are extensive literature reviews of regional and international codes, after which the design procedure in four different areas was started. The distribution of workloads across the areas and the responsible team members are shown in Table 1.1.1.

Table 1.1.1. Workload percentages and responsible members

Area	Workload	Responsible member(s)
Structural, Architectural and Construction Materials	40%	Zhanserik Shadiyar Assem Nurtaza
Geotechnical	35%	Meruyert Kerimbay Aruzhan Serikova
Environmental	15%	Kazybek Bestan
Construction Management	10%	Rolan Rysbek

The hotel will be located in a growing neighborhood near Downtown Seattle, which houses major technological companies and a university hospital. Furthermore, popular tourist attractions are located within walking distance from the hotel. This increases the attractiveness of the hotel for city visitors.

The main objectives for the project are:

- Perform literature review of codes for design;
- Make an architectural design that aligns with code requirements;
- Calculate loads and assign in software;
- Calculate different foundation load capacities and compare against the building load;
- Make a preliminary design of the storm sewer system;
- Create charts and graphs for planning and managing construction activities.

These objectives need to be achieved for successfully completing the design of the hotel in a safe, durable and organized manner.

1.2 Ethics and Professional Considerations

There are serious moral and professional responsibilities associated with the construction of a 12-story hotel in Seattle, Washington, United States of America. Seattle is known for its dedication to community well-being and environmental sustainability, requires that building projects be conducted with a high degree of accountability, Considering regulatory requirements, ensures trust and stable relationships among local citizens, and minimizes harmful effects in the environment and nearby communities depend on ethics and professionalism in the construction industry.

Ethical Considerations

Reducing environmental damage is significantly important and prioritized in cities like Seattle. It is crucial for construction activities to be focused on strategies on waste management, conservation of energy, and prevention of pollution, which means that environmentally responsible measurements such as reuse of materials and energy, efficient appliances, and water conservation will be implemented. It is significantly important to follow guidelines of the Green Building Certification in order to prevent the building construction to interfere the other sustainability goals of the city and that the project boosts environmental initiatives.

The impact of construction activities on the surrounding neighborhood has to be taken into consideration. Large-scale developments may cause disturbance to local businesses and residences from, for instance, noises, vehicle congestion, and dust pollution. Environmental ethics include informing the community about project timelines and addressing project concerns.

Ensuring the safety of construction workers is also one of the ethical concerns during the construction process. This includes informing contractors and subcontractors about the OSHA rules, conducting proper training, and supplying workers with safety equipment.

Professional Considerations

There are several main professional responsibilities, such as building codes, zoning laws, and environmental regulations. In order to receive permission for the land and building construction, there have to be multiple inspections in the construction stages.

Professional workers are also required to deliver high-quality construction and guarantee its safety. It may include using high-quality materials, following ISO standards, and

ensuring proper control over the construction process. Prioritizing the high-quality work from the beginning of the building construction may guarantee the prevention of future issues and ensure the longevity of the hotel.

To sum up, ethical and professional consideration of the construction project is a significantly important aspect for the future success of the building. Ensuring the use of high-quality materials, recruiting professional workers, controlling every stage of the construction, minimizing environmental hazardous impact, receiving legal permission, and transparent communication among workers and citizens in the neighborhood may result in positive results, such as hotel's timely scheduled completion, and development of infrastructure in Seattle.

2. Architectural, Structural and Construction Materials Area

Selected scope for Architectural, Structural and Construction Materials area is a high-rise building in a high seismic zone (short period response acceleration larger than 1.0 g for a return period of 2475 years).

2.1 Architectural Area

2.1.1 Site selection

Seattle is a city in Washington state, washed by Puget Sound, an arm of the Pacific Ocean to the west and Lake Washington to the east. Its breathtaking landscape, with mountains, lakes, and national parks, attracts tourists from all over the world. Additionally, the city's symbol, the Space Needle, Museum of Pop Culture, Pike Place Market are considered popular tourist destinations and must-visit places for international visitors.

The city is also well known for its growing role as the economic hub, being home to big companies like Amazon, Microsoft and Starbucks. Furthermore, the Seattle Convention Center hosts more than 350 national-level conferences and events. The city is also well-connected with neighboring counties. Travel via ferries is possible with Washington State Ferries and land travel is carried out by public buses, commuter rail and light rail transports.

The proposed building will be located on 401 8th Avenue North in the South Lake Union neighborhood. According to the Seattle Land Use Zoning map, this address belongs to zone SM-SLU-85-280, a mixed-use zone that allows commercial and residential development (Seattle GeoData, 2024).

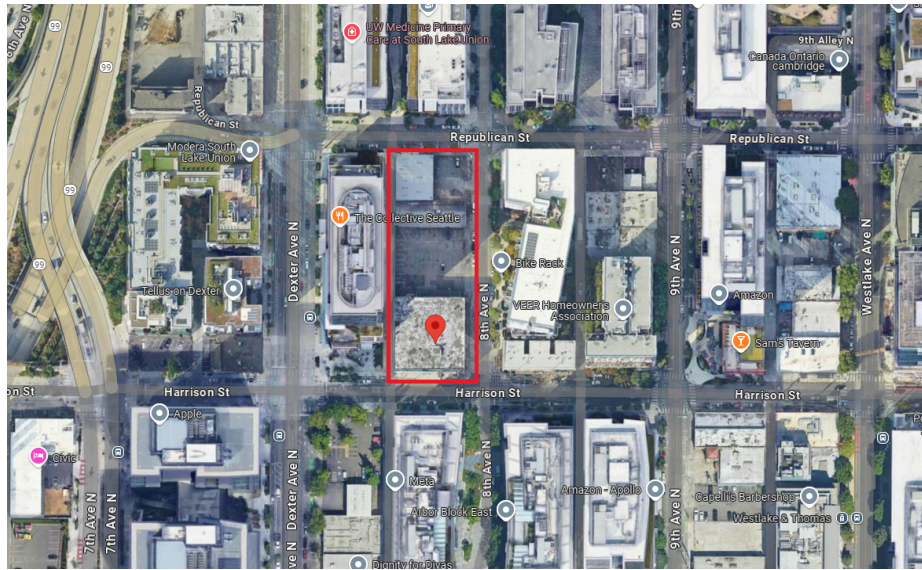


Figure 2.1.1. Location on Google Maps (2024) satellite view

South Lake Union is a rapidly growing neighborhood and over the last few decades has turned into a major technological hub, with Amazon, a global tech conglomerate, situating its main campus here, and Apple and Meta, multinational tech corporations, locating their Seattle offices in this area as well. Additionally, South Lake Union is strengthening its position as the biotechnological center of the US, as various life science organizations are based here. Some of them are: Cancer Research Center, the Allen Institute of Brain Science, the Allen Institute of Cell Science and many others. Furthermore, South Lake Union Campus of the University of Washington School of Medicine includes 4 floors of biotechnology and medical research laboratories.

The background information about the neighborhood suggests that the area is growing rapidly, its vibrancy and bustling city life attracting large corporations, small businesses, fine dining restaurants and other recreational facilities. Building a hotel in this environment would be a huge asset. First, the UW hospital would ensure good flow of residents, be it patients' relatives, or medical conference participants. Second, being located among large tech corporations' office buildings would also suggest good conference room booking opportunities. Third, the hotel would be in a walking distance to popular tourist attractions, like the Space Needle and Museum of Pop Culture, making it more appealing to city visitors.

2.1.2 Design conditions

Seattle is located in a high seismic hazard zone. The Seattle Fault that extends mid-west crossing the middle of the city poses the biggest threat, as it is challenging to estimate the frequency due to a lack of data. According to experts, the Seattle Fault earthquake

could be of magnitude M7.5, which is classified as between “very strong” and “destructive” (Witter, Kelsey and Hemphill-Haley, 2003). Additionally, the Fault line was last active over 1000 years ago and has caused three or four earthquakes in 3500 years (Witter, Kelsey and Hemphill-Haley, 2003).

Seattle experiences mostly shallow earthquakes, which occur closer to the surface (at 0-30 km) and usually cause more damage than deep earthquakes, since the energy would not dissipate heavily before reaching the surface (Washington Military Department, 2013). Moreover, the city lies close to the Cascadia Subduction Zone, a fault that stretches from Northern California to southern British Columbia, affecting primarily the Pacific Northwest region comprising states Washington, Oregon and Idaho. It is the place of interaction of two plates: the North American plate and the Juan de Fuca plate. The earthquakes produced in this region are called “megathrusts” and the magnitude of them could reach over M8.5, the effects of which are described as between “destructive” and “ruinous”. The last event took place in January 1700, and the average recurrence interval is between 570 and 590 years (Witter, Kelsey and Hemphill-Haley, 2003).

Considering the evidence about high seismic activity in Seattle, this would be the primary design condition for this project. Furthermore, the short response acceleration for a return period of 2475 years, S_s , was found to be 1.54g, which is larger than 1.0g and satisfies the high seismicity criterion.

Seattle is not in a “strong wind zone” as the basic ground wind speed for a return period of 50 years does not exceed 40 m/s. However, the value, approximately 35 m/s, is close to the design criterion and will be considered in design.

Similarly, the corrosion map shows that Seattle is in the “Medium” corrosivity category according to the International Organization for Standardization (ISO) (WBDG, n.d.). Since Seattle is a seaport city and is washed by salt water of Puget Sound, the building will be designed to withstand corrosion as well.

2.1.3 Site analysis

Climate in Seattle is temperate, characterized by wet, cool winters and dry, warm summers. In an average year, the temperature is between -5.8°C and $+34.5^{\circ}\text{C}$ (‘Climate of Seattle’, 2024). Furthermore, one of the main features are frequent rainfalls. On average, precipitation exceeds 0.01 in (0.254 mm) on 156 days in a year (‘Climate of Seattle’, 2024). Similarly, snow is not uncommon in Seattle, as the city receives 16.0 mm of snow on average (‘Climate of Seattle’, 2024). However, heavy snowfall is not typical.

The site is currently occupied by a one story wood framed structure, a reinforced concrete structure one story above ground, and an asphalt concrete pavement surface parking lot (GeoEngineers, 2009). It is bound by Republican Street to the north, Harrison Street to the south, an alley to the west and 8th Street Avenue North to the east. The existing structures will be demolished as there are no records of them being occupied for commercial use.

Regarding the wind, the strongest value was recorded as 50.1 mph (22.4 m/s) coming from SW. According to the wind rose provided by the Office of the Washington State Climatologist (no date), south and southwest winds are most common in Seattle.

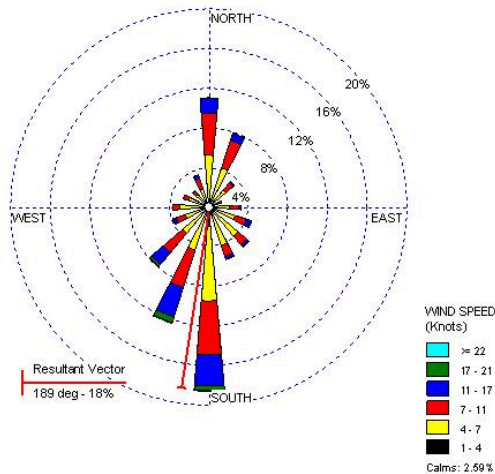


Figure 2.1.2. Wind direction profile (Office of the Washington State Climatologist, n.d.)

As for the sun direction, the sun rises from Dexter Avenue North side and sets in the direction facing 8th Avenue North (SunCalc, 2024). To the west, the sunlight would be partially blocked by a 13-story building nearby. Consequently, the rooms located in the south and east sides would get maximum sunlights, as there are no obstructions, and design considerations would have to be made in order to reduce direct sunlight penetration.

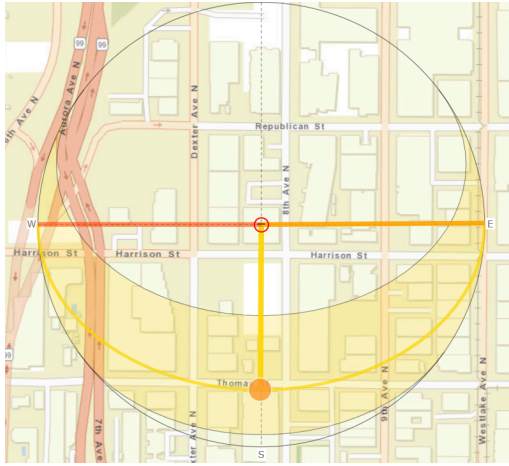


Figure 2.1.3. Sun direction profile (SunCalc, 2024)

According to GeoEngineers' (2019) geotechnical investigation, there is a slight slope to the north, with elevation difference being 7 feet between the southwest and northeast corners of the site. Additionally, glacially consolidated soils were encountered under shallow fill and recent deposits, which consist of very stiff to hard silt and clay. The site class was identified as C based on shear wave velocity data, and it is characterized as very dense soil or/and soft rock (GeoEngineers, 2019).

Regarding the accessibility of the hotel, it is well connected with other areas, including Downtown Seattle, via public transportation. There are more than 3 bus stops in a walking distance, along with light rail stations 2 streets further. This means that the building would be easily accessible for visitors, and would make their travels to other areas more convenient.

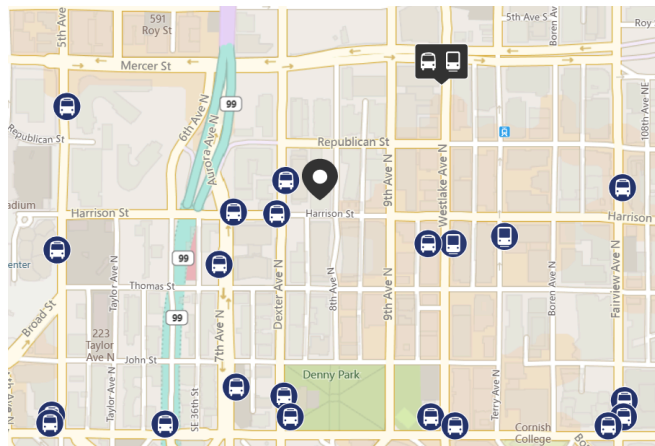


Figure 2.1.4. Public transport stations availability (Google Maps, 2024)

Services, such as power, gas, water, are covered in this area completely. As for stormwater drainage, a combined sewer system, for both wastewater and stormwater, is used (City of Seattle, 2024).

2.1.4 Design concept

The location of the building suggests the need for a design that reflects the vibrant and dynamic nature of the neighborhood. Additionally, since there are many company headquarters and campuses in close proximity, the hotel has to cater mostly to the needs of business travelers, with multiple conference rooms available. Considering frequent rainfalls and low number of sunny days, the facade should include big windows to ensure good sunlight access. Furthermore, for the same reason the facade should be designed in earthy tones, in order to create an inviting atmosphere. Moreover, exterior materials have to easily withstand moisture, be it from rains or snows, and be weather-resistant in general.

Since the hotel will be more business-oriented, lower floors should include rooms for public uses, such as a restaurant and conference rooms. Also, the restaurant should be located to the north, for maximum sunlight availability.

2.1.4 Architectural design

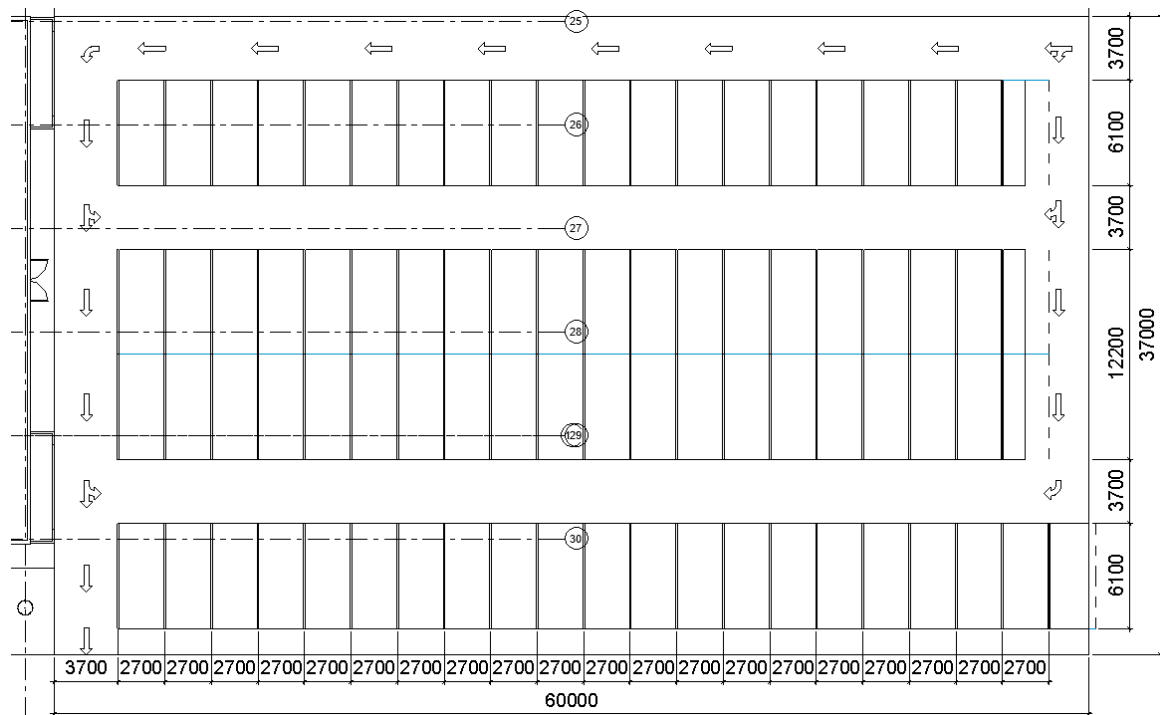
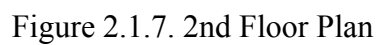
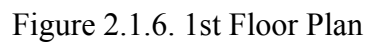
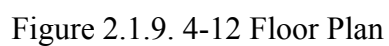
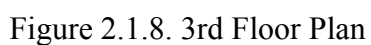


Figure 2.1.5. Site layout plan: traffic flow direction/parking





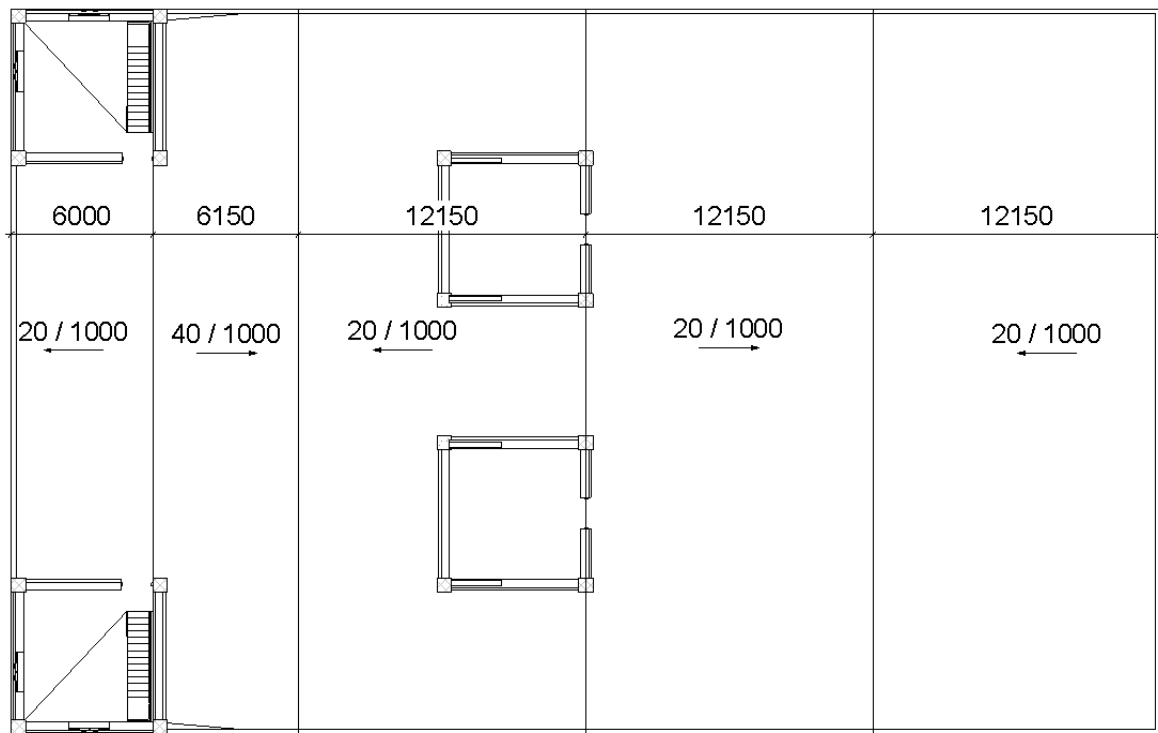


Figure 2.1.10. Roof plan (drainage)



Figure 2.1.11. Basement/Parking

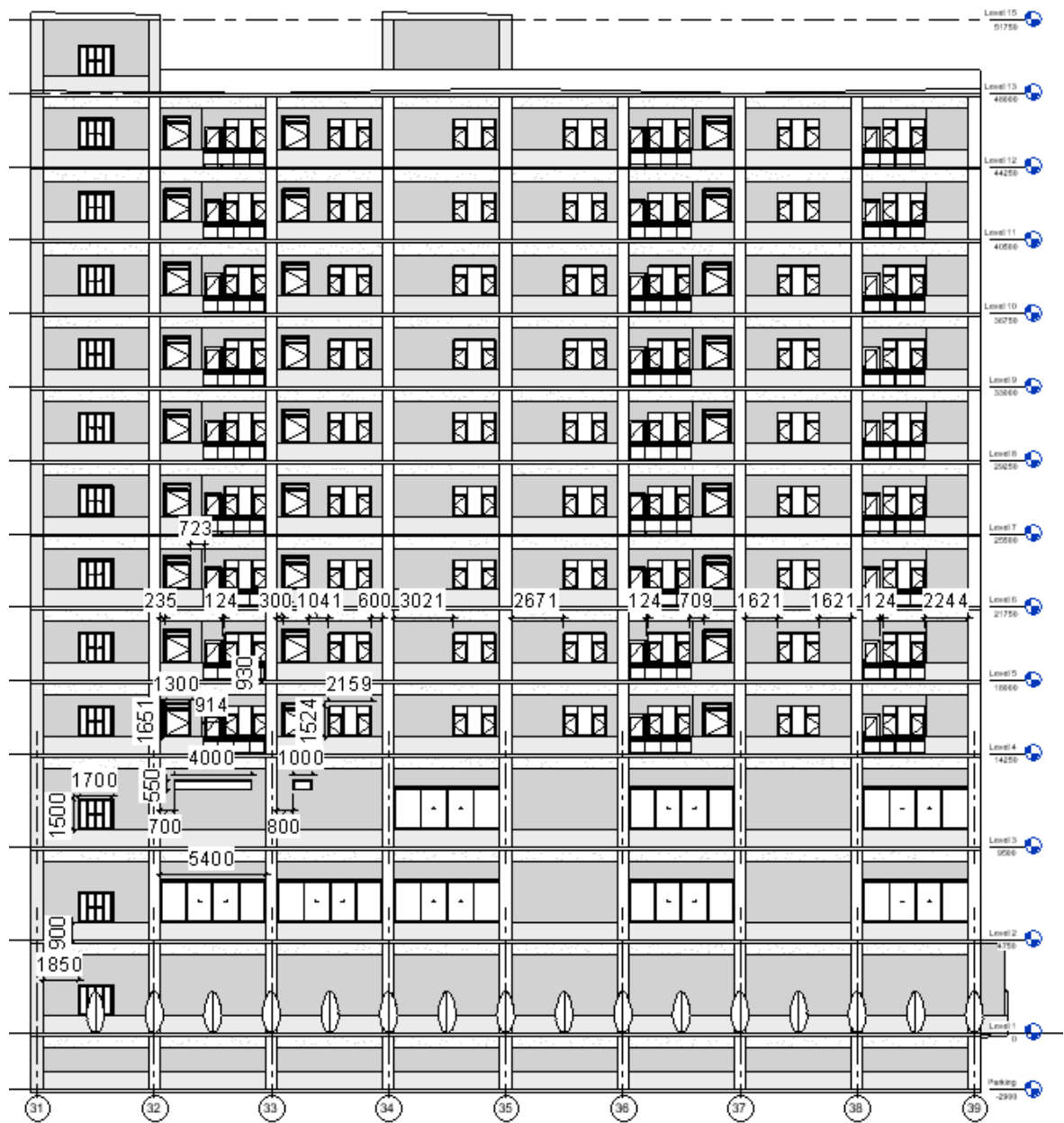


Figure 2.1.12. Elevations: South

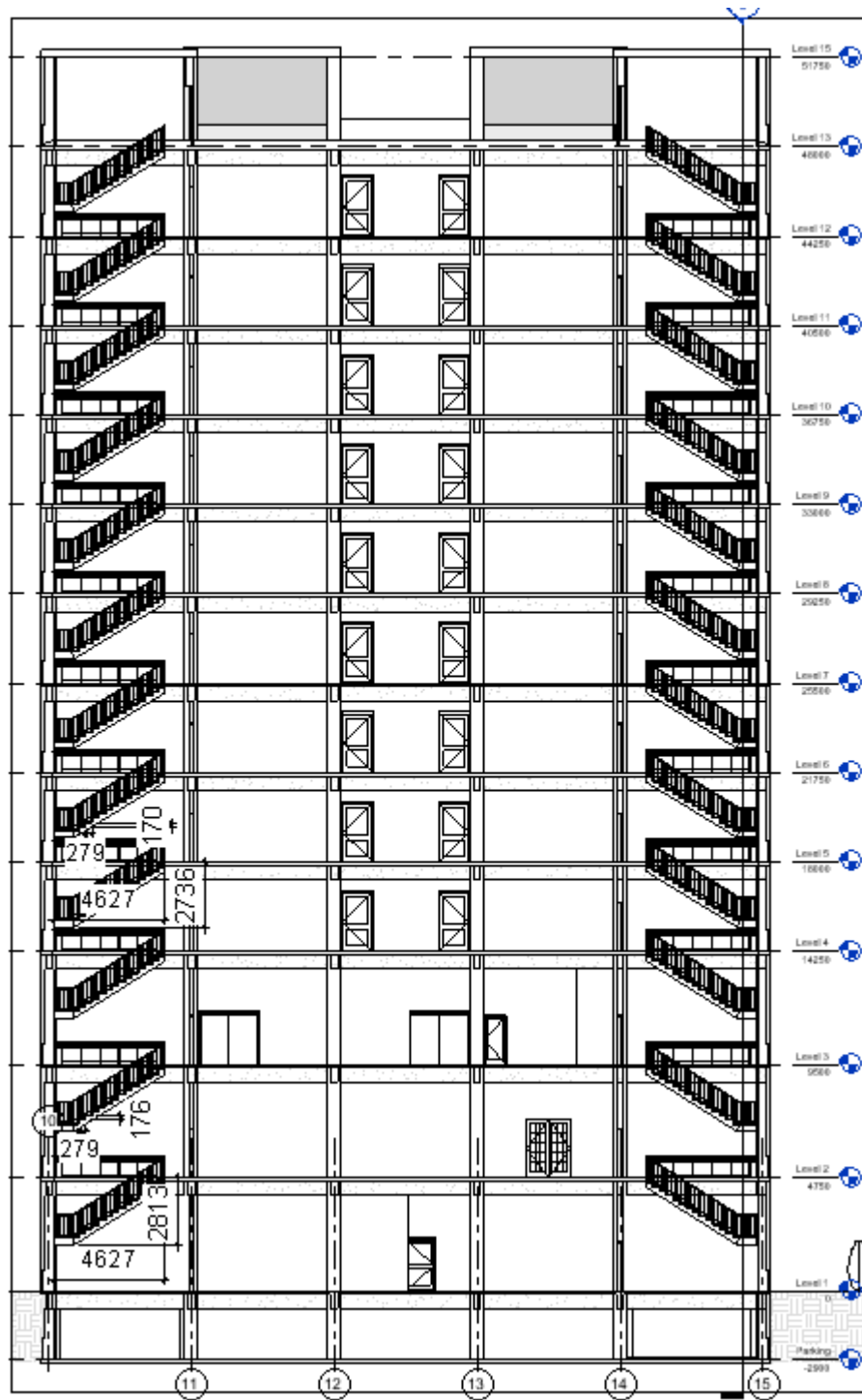


Figure 2.1.13. Section cut 1

Section cut views

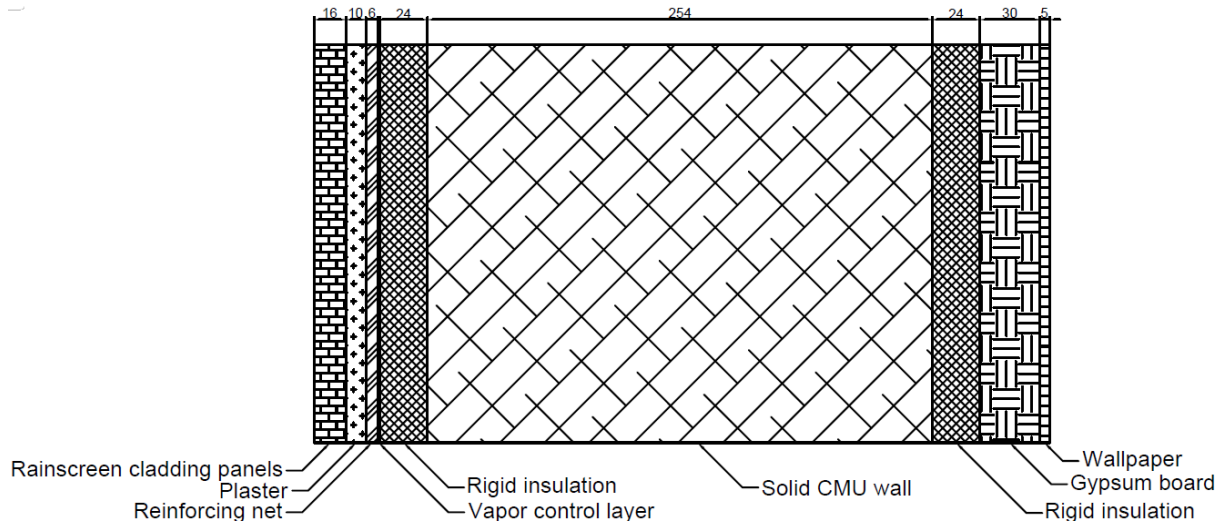


Figure 2.1.14. Exterior wall components

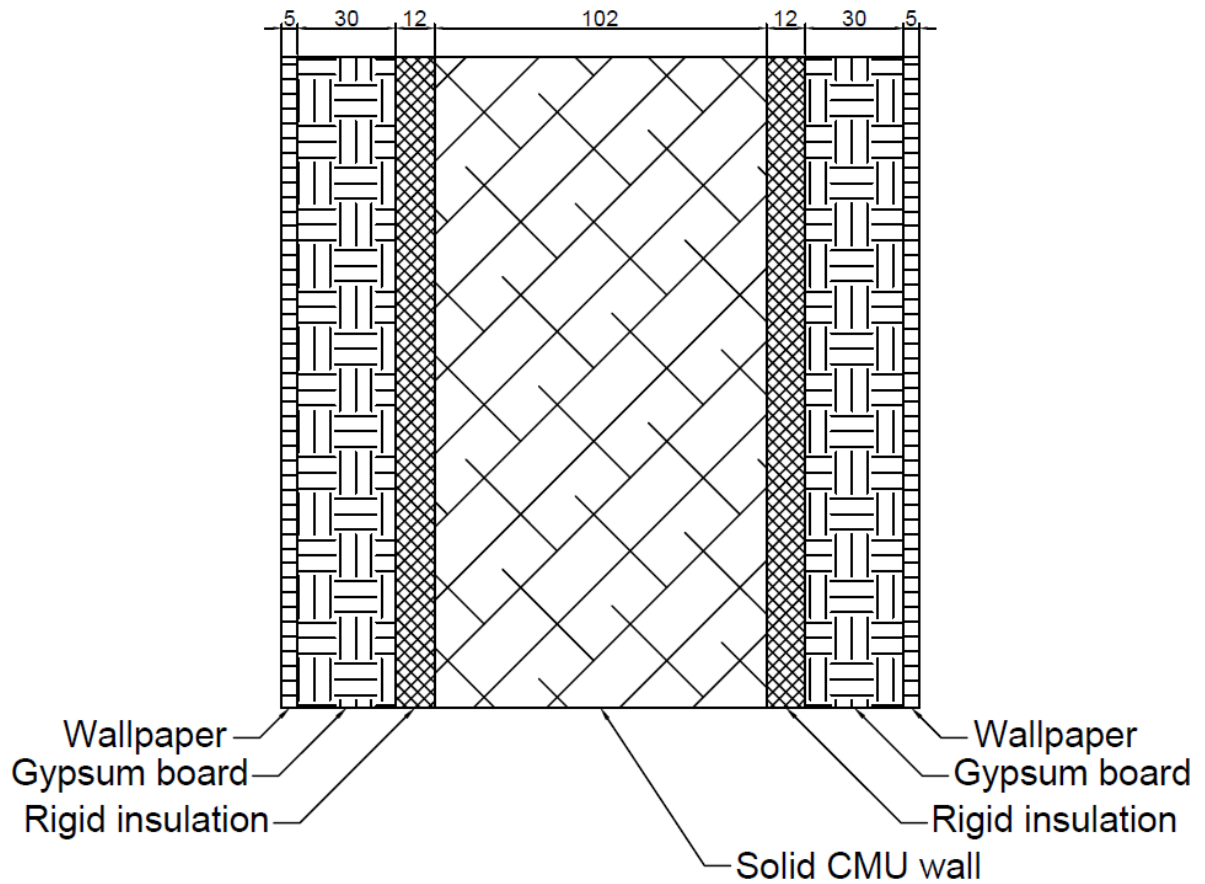


Figure 2.1.15. Interior wall components

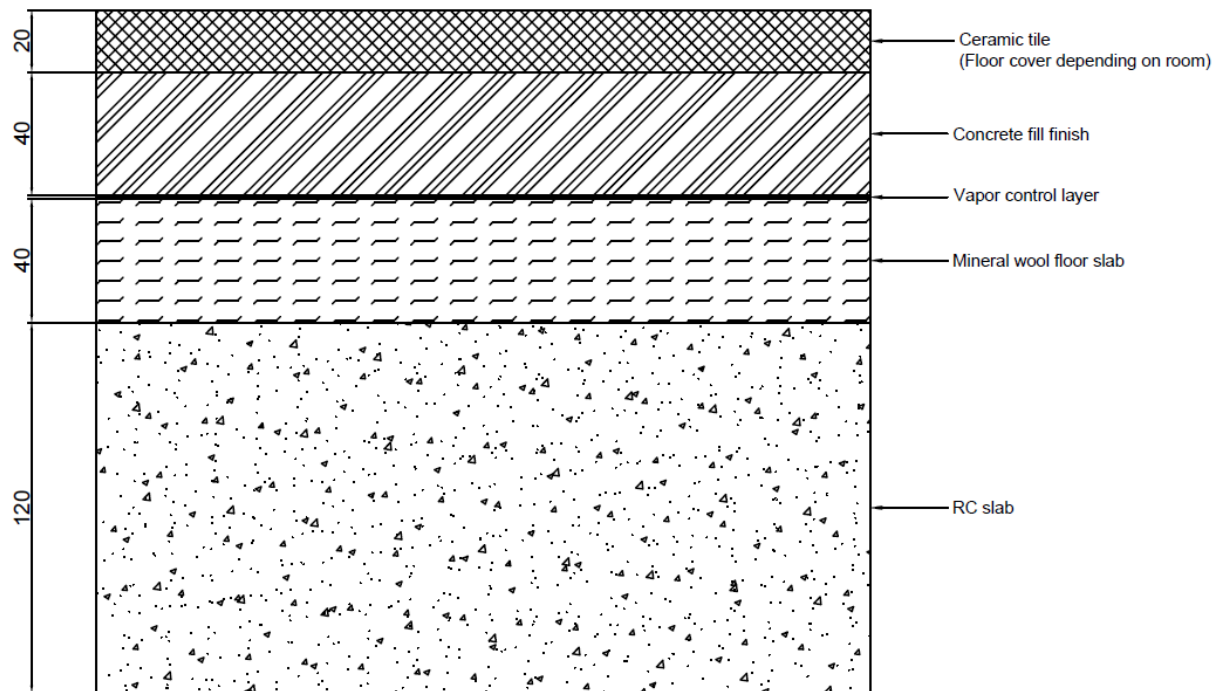


Figure 2.1.16. Floor finishing components

Architecture design calculations

Our building is classified as R-3 building according to IBC 302 and IBC 310. First of all, we have calculated the number of doors required for our building and for each room separately. According to IBC 1004.5 residential areas have an occupant load factor of 200 gross. Our building area is 30.6 meters in width and 48.6 meters in length, which is equal to 1487.16 meters squared. Before dividing it by 200 it should be turned into feet, which is 16008 feet squared. As a result by dividing 16008 by 200 we obtain 80.04 occupant load per story. IBC 1006.2.1 states that to have one exit for a story, occupant load should not exceed 20. Therefore, our building cannot have 1 exit. In IBC 1006.3.2 written that for occupant load from 1 to 500 there should be 2 exits. Therefore, we made 3 exits for the 1st floor and 2 exits for all floors above. IBC 1007.1.1 states that doors should be separated by one-third of the longest diagonal distance of the space for the building with an automatic sprinkler system. All of the door numbers for each floor were calculated in the same way.

Emergency evacuation: According to IBC 1006.2.2.6 common path travel distance should not exceed 125 feet (38100 mm) and IBC 1017.2 states that the total travel distance should not exceed 250 ft (76200 mm). According to IBC 1017.3 travel distance should be calculated from the most remote point to the nearest exit. By applying all of this information

in Figure 2.1.17 it was shown that our building has the longest travel distance of 65017 mm on the 4th floor. These travel distances were calculated by using the tool Path of Travel in Revit.

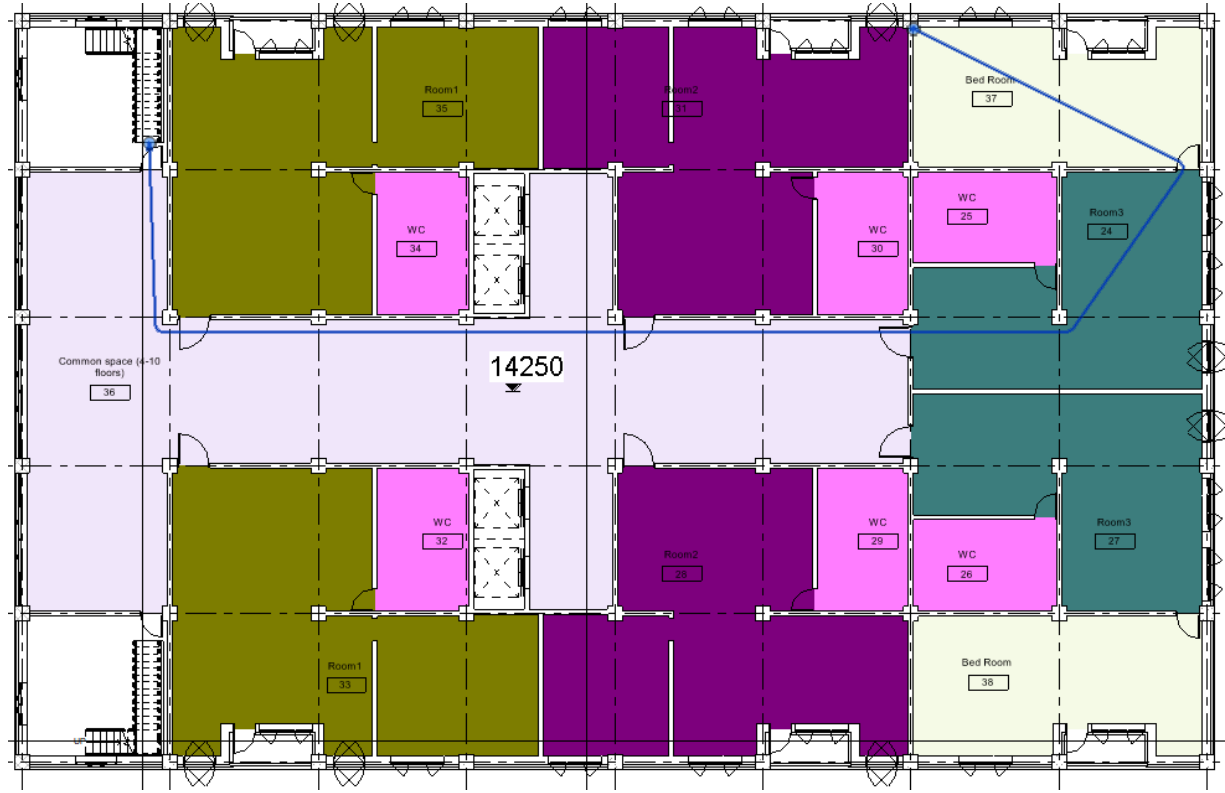


Figure 2.1.17. Longest travel distance

Numbers of stairs and elevators: According to IBC 1009.2.1 buildings with 4 or more accessible floors require at least 1 elevator. Our building is equipped with 4 elevators. Number of stairs was calculated while completing the part for emergency evacuation.

Number of doors and door dimensions: Number of doors were calculated from the previous parts. Door dimensions were designed according to IBC 1010.1.1. Minimum clear width opening should be 813 mm and room with movement of beds 1054 mm and minimum height for all of them should be 2032 mm. Our building considers 4 types of doors. The smallest one has the size of 863.6 mm by 2032 mm. Living area rooms from 4-12 floors placed with the door size of 1219.2mm to 2438.4mm.

Window areas: According to the IBC 1204.2 minimum area of the window should not be less than 8% of the floor area of the room. All of the calculations for the natural light for each room were completed and it can be seen from Figure 2.1.18.

Width	Height	Area	From Room: Area	Windows percent	From Room: Number	From Room: Name
5400	2100	11.34 m²	407.8 m²	8.3	1	Common space (1st)
1: 3				8.3		
2700	1100	2.97 m²	34.9 m²	8.5	2	WC
2: 1				8.5		
2700	1100	2.97 m²	34.9 m²	8.5	3	WC
3: 1				8.5		
1700	1500	2.55 m²	31.0 m²	16.5	8	Emergency exit
8: 2				16.5		
1700	1500	2.55 m²	31.0 m²	16.5	9	Emergency exit
9: 2				16.5		
5400	2100	11.34 m²	331.4 m²	13.7	12	Common space (2nd)
12: 4				13.7		
5400	2100	11.34 m²	192.8 m²	11.8	13	Reception/ waiting area
13: 2				11.8		
5400	2100	11.34 m²	485.4 m²	9.3	14	Restaurant
14: 4				9.3		
5400	2100	11.34 m²	203.8 m²	16.7	15	Kitchen
15: 3				16.7		
<varies>	550	<varies>	31.4 m²	8.8	16	WC
16: 2				8.8		
<varies>	550	<varies>	31.4 m²	8.8	17	WC
17: 2				8.8		
5400	2100	11.34 m²	292.6 m²	11.6	18	Gym
18: 3				11.6		
5400	2100	11.34 m²	416.9 m²	10.9	19	Entertainment area
19: 4				10.9		
5400	2100	11.34 m²	254.8 m²	13.3	20	Common space (3rd)
20: 3				13.3		
5400	2100	11.34 m²	240.0 m²	9.5	21	Spa
21: 2				9.5		
<varies>	550	<varies>	32.5 m²	8.5	23	WC
23: 2				8.5		
<varies>	<varies>	<varies>	78.4 m²	11.1	24	Room3
24: 3				11.1		
<varies>	<varies>	<varies>	78.4 m²	11.1	27	Room3
27: 3				11.1		
<varies>	<varies>	<varies>	123.5 m²	9.7	28	Room2
28: 4				9.7		
<varies>	<varies>	<varies>	123.5 m²	9.7	31	Room2
31: 4				9.7		
<varies>	<varies>	<varies>	125.1 m²	8.7	33	Room1
33: 4				8.7		
<varies>	<varies>	<varies>	125.1 m²	8.7	35	Room1
35: 4				8.7		
5400	2100	11.34 m²	313.1 m²	10.9	36	Common space (4th)
36: 3				10.9		
2159	1524	3.29 m²	60.5 m²	10.9	37	Bed Room
37: 2				10.9		
2159	1524	3.29 m²	60.5 m²	10.9	38	Bed Room
38: 2				10.9		
1700	1500	2.55 m²	32.1 m²	15.9	57	Emergency exit
57: 2				15.9		
1700	1500	2.55 m²	32.1 m²	15.9	58	Emergency exit
58: 2				15.9		
1700	1500	2.55 m²	32.1 m²	15.9	59	Emergency exit
59: 2				15.9		
1700	1500	2.55 m²	32.1 m²	15.9	60	Emergency exit
60: 2				15.9		

Figure 2.1.18. Natural light/Window percentage

Corridor width: According to the IBC 1020.3 the maximum of the minimum requirements for the width of the corridor is 96 inches which is 2438 mm. In our building the smallest corridor width is 2.6 meters, therefore, it meets the requirements.

Clear height of each story: IBC 1208.2 states that height of the ceiling of habitable spaces and corridors should be higher than 2286 mm. Our building covers this requirement as the least clear height of floor in our building is 3 meters.

Number of parking spaces/size: According to the Washington requirements there should be 1 parking lot for each room and 1 for each 500 feet square of public places. Our building contains 8 floors with 6 rooms on each and 2152 meter square (23163,94 square feet) of public places. It is $9 \times 6 + 23163.94 / 500 = 100.328$, so 101 parking lots. According to the IBC 801.3.1 and 801.3.2 the length of the parking lot should be 6.1 meters and width of 2.7 meters. We have 77 outside parking lots and 24 parking spaces in the basement, so in general 101 parking lots.

Roof slope: IBC 1507.10.1 states that roof slope should be at least 2% which is 1 unit vertically for every 50 horizontal units. Roof of our building was divided into 3 parts for water drainage which can be seen from Figure 2.10. The smallest slope is 20/1000 which is exactly 2%.

Ramp design: According to IBC 406.6.3 the ramp design should have a slope of maximum 6.67% which is 1 vertical unit for 15 horizontal units. Our building considers a ramp which allows the entrance to the basement parking. The straight horizontal length is 44 meters and the depth of the basement is 2.9 meters. This gives us $2.9/44 \times 100 = 6.591\%$ and it fits the requirements.

Fire extinguishers: IBC 906.1 states that portable fire extinguishers are required in Group A, B, E, F, H, I, M, R-1, R-2, R-4 and S occupancies.

Roof parapet: The least height of the parapet for the roof is 762 mm according to IBC 705.11.1. The height of the parapet in the roof of our building is 900 mm.

2.1.5 LEED (Leadership in Energy and Environmental Design)

Since one of the main objectives of the project is to perform design in a sustainable and environment-friendly way, a LEED rating system will be used to assess these parameters. Main goals of LEED include reducing contribution to global climate change, protecting and restoring water resources, promoting sustainable and regenerative material cycles, enhancing individual human health, protecting and enhancing biodiversity and ecosystem services and enhancing community quality of life (USGBC, n.d.). These objectives have to be achieved to a certain extent to receive high points across all categories.

Location and Transportation

The hotel is located at the center of the city. It allows people to reach places of their first need in a 10 minute walk. Also, its location provides access to public transport systems. As a result it contributes to the process of decreasing the carbon emission. So, it decreases the level of greenhouse gasses. This section earned 8 out of 16 points and it can be seen from Figure 2.1.19.

0	8	0	Location and Transportation		16
Y			Credit	LEED for Neighborhood Development Location	16
Y	1		Credit	Sensitive Land Protection	1
		N	Credit	High Priority Site and Equitable Development	2
Y	2		Credit	Surrounding Density and Diverse Uses	5
Y	5		Credit	Access to Quality Transit	5
		N	Credit	Bicycle Facilities	1
Y			Credit	Reduced Parking Footprint	1
		N	Credit	Electric Vehicles	1

Figure 2.1.19 Location and Transportation category

Sustainable Sites

The site investigation was conducted and analysed before planning the construction. Nearby the hotel there is a park which provides an open space for interaction with nature and nature protection. The hotel's roof drainage system allows the water cycle system. In general for this part we have earned 6 points out of 10 and it can be seen from Figure 2.1.20.

0	6	0	Sustainable Sites		10
Y			Prereq	Construction Activity Pollution Prevention	Required
Y	1		Credit	Site Assessment	1
Y	1		Credit	Protect or Restore Habitat	2
Y	1		Credit	Open Space	1
Y	2		Credit	Rainwater Management	3
Y	1		Credit	Heat Island Reduction	2
		N	Credit	Light Pollution Reduction	1

Figure 2.1.20 Sustainable Sites category

Water Efficiency

Potable and freshwater reduction is vitally important for life and our future. System of our hotel helps to reduce more than 50% of potable water used outdoors. This requires the use of rainwater. Besides this indoor system can use treated greywater and waste water from showers and sinks for reusing the water as for flushing. Points for this section can be seen in Figure 2.1.21.

0	7	0	Water Efficiency		11
Y			Prereq	Outdoor Water Use Reduction	Required
Y			Prereq	Indoor Water Use Reduction	Required
Y			Prereq	Building-Level Water Metering	Required
Y	1		Credit	Outdoor Water Use Reduction	2
Y	6		Credit	Indoor Water Use Reduction	6
		N	Credit	Optimize Process Water Use	2
		N	Credit	Water Metering	1

Figure 2.1.21 Water Efficiency category

Energy and Atmosphere

The hotel has free access to sunlight which allows it to reduce energy consumption. Also, the building has a good ventilation and air conditioning system which also contributes to energy efficiency. The advanced energy metering will be used in the system to monitor the energy consumption and optimize it. Earned points for this section can be seen from the Figure 2.1.22.

0	19	0	Energy and Atmosphere		33
Y			Prereq	Fundamental Commissioning and Verification	Required
Y			Prereq	Minimum Energy Performance	Required
Y			Prereq	Building-Level Energy Metering	Required
Y			Prereq	Fundamental Refrigerant Management	Required
Y	2		Credit	Enhanced Commissioning	6
Y	16		Credit	Optimize Energy Performance	18
Y	1		Credit	Advanced Energy Metering	1
		N	Credit	Grid Harmonization	2
		N	Credit	Renewable Energy	5
		N	Credit	Enhanced Refrigerant Management	1

Figure 2.1.22 Energy and Atmosphere category

Materials and Resources

In the construction there will be used slag which helps to decrease the carbon emission. Replacing some part of the cement also helps to decrease the cost. Besides this, a waste management plan will be used during the construction. It will help to contribute to environmental improvement. The points were awarded as it is shown in Figure 2.1.23.

0	6	0	Materials and Resources		13
Y			Prereq	Storage and Collection of Recyclables	Required
		N	Credit	Building Life-Cycle Impact Reduction	5
Y	2		Credit	Environmental Product Declarations	2
Y	2		Credit	Sourcing of Raw Materials	2
Y	2		Credit	Material Ingredients	2
		N	Credit	Construction and Demolition Waste Management	2

Figure 2.1.23 Materials and Resources category

Indoor Environmental Quality

The hotel provides good indoor air quality with no chemical and carbon emitting materials. The hotel does not have its own area of plants and trees which could improve the indoor air quality. Nevertheless, it has good access to daylight and accordingly has thermal comfort. Nearby there are no longer buildings than the hotel so it can provide access to quality views. The provided points can be seen from Figure 2.1.24.

0	10	0	Indoor Environmental Quality		16
Y			Prereq	Minimum Indoor Air Quality Performance	Required
Y			Prereq	Environmental Tobacco Smoke Control	Required
Y	2		Credit	Enhanced Indoor Air Quality Strategies	2
Y	2		Credit	Low-Emitting Materials	3
Y	1		Credit	Construction Indoor Air Quality Management Plan	1
Y	1		Credit	Indoor Air Quality Assessment	2
Y	1		Credit	Thermal Comfort	1
Y	1		Credit	Interior Lighting	2
Y	1		Credit	Daylight	3
Y	1		Credit	Quality Views	1
		N	Credit	Acoustic Performance	1

Figure 2.1.24 Indoor Environmental Quality category

Regional Priority

There are 6 regional priorities for this region. They are: Indoor water use reduction, Demand Response, Building product disclosure and optimization - sourcing of raw materials, Renewable energy production, Building product disclosure and optimization - environmental product declarations and Rainwater management. It will be given 1 point for each of the Priorities so this category obtains all 4 points. It can be seen from Figure 2.1.25.

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

Figure 2.1.25 Regional Priority

2.1.6 Life-cycle cost analysis using Life-365

Concrete mix design

Several mix designs were used to make comparisons of costs. The concrete has to be strong to withstand seismic loads and this can be achieved with low water/cement ratio and admixtures. Concrete with high concentrations of fly ash and slag requires more time for curing and early-age strength would be compromised. This would not be suitable for the climate in Seattle, with strong winds and frequent rainfalls, so accelerating admixtures are essential. The following mix designs were considered:

- Base case: w/c ratio 0.4, no admixtures, black steel;
- Alternative 1: w/c ratio 0.4, fly ash 10%, silica fume 10%, slag 30%, epoxy coated;
- Alternative 2: w/c ratio 0.4, fly ash 10%, silica fume 5%, slag 20%, black steel;
- Alternative 3: w/c ratio 0.4, fly ash 10%, silica fume 5%, slag 20%, epoxy coated.

Construction, barrier, repair and life-cycle costs are demonstrated in the table below for different mix designs.

Table 2.1.1 Construction, barrier, repair and life-cycle costs

Mix designs	Construction cost	Barrier cost	Repair cost	Life-cycle cost
Base case	35670	7560	38600	82564
Alternative 1	43250	7560	38600	91570
Alternative 2	43250	7560	38600	92852
Alternative 3	43250	7560	38600	95640

Alternative 3 has highest costs, but the difference with other mix designs is negligible considering its advantages, such as increased corrosion resistance, long-term high strength development and decreased permeability.

2.1.7 Corrosion

As mentioned in sections above, corrosion risk for structures in Seattle is classified as medium. In order to consider corrosion resistance for the structure a few measures were taken.

First, in order to protect rebar from corrosion effects, it was necessary to consider using epoxy-coated rebar in the design instead of regular black steel. Coating acts as a physical barrier between steel and a potentially aggressive environment.

Second, it is important to keep the water/cement ratio low to reduce voids and, thus, permeability. The w/c ratio of 0.4 satisfies this requirement and allows for design of high strength concrete with sufficient workability.

Third, in areas that interact with the ground placing waterproof membranes was considered to protect from moisture seeping into the structure and piles.

2.2 Construction Materials Area

At initial stages of design, some material options were eliminated immediately due to their poor behavior under design conditions. Those are:

- Wood: low fire and moisture resistance; less usable space due to large member sizes;
- Masonry: brittle and hazardous during earthquakes.

Considered options, steel and reinforced concrete, were compared.

Table 2.2.1. Steel and reinforced concrete comparison

Steel	Reinforced concrete
High tensile strength, low compressive strength	Concrete low tensile strength is compensated with steel's high tensile strength
Small cross sections	Large cross sections
High cost and skilled labor	Low cost of labor
Susceptible to corrosion	Less susceptible to corrosion
Fabricated in factories	Can be made both on site and in factories
High resistance to earthquakes	Lower resistance to earthquakes
Lower fire resistance	Good fire resistance
Lower self-weight	Higher self-weight
More expensive	Less expensive

Although steel generally performs better during earthquakes due to its ductility and lower weight of the structure, reinforced concrete was chosen for its durability, high fire and corrosion resistance.

Regarding the concrete grade, considering the height of the building and since heavy loads will be applied, C35 grade concrete was chosen. For reinforcing steel bars, grade 75 was selected. It is generally used for high-rise buildings and buildings subject to large forces or seismic loads (EnduraSteel, 2023).

2.3 Structural Area

2.3.1 Structural load calculations

2.3.1.1 Structural layout

For the structural layout, one option out of three demonstrated in Figure 2.3.1 had to be chosen. The main objective was to determine the most economical option, and the volume of concrete was the critical factor.

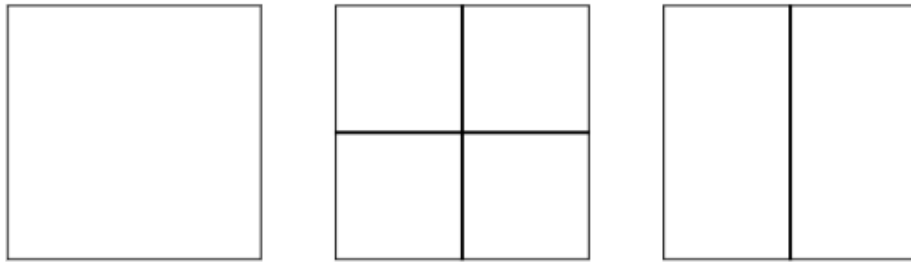


Figure 2.3.1. GLRS slab layout options

Span length is 6 m in both directions, which is the optimal choice considering the available building area. The three options for structural layout are: two-way slab without minor beams, two-way slab with two minor beams and one-way slab with a minor beam. The procedure for sizing members follows ACI 318-19 Chapter 7 to Chapter 10.

For all options following estimations were made:

- Height for major beam is 8% of span;
- Minimum thickness for simply supported minor beams is span divided by 16;
- Width for beams is 40% of height.

Option 1. Two-way slab

In order to estimate the slab thickness, stiffness ratios of members need to be considered. Since all beams have the same moment of inertia and modulus of elasticity, the average stiffness ratio, α_{fm} value corresponds to the individual members' α_f . Additionally,

material properties for slab and beams are the same, hence the equation can be further simplified. It was calculated using the following equation:

$$\alpha_{fm} = \frac{E_{cb} I_b}{E_{cs} I_s} = \frac{I_b}{I_s}$$

Slab thickness was first assumed as 150 mm. Dimensions of major beams are:

$$L = 6 \text{ m};$$

$$h_{beam} = 0.08 * L = 0.08 * 6 = 0.480 \text{ m};$$

$$b_{beam} = 0.4 * h_{beam} = 0.4 * 0.480 = 0.192 \text{ m}.$$

Moments of inertia were calculated as follows:

$$I_b = 0.35 * \frac{1}{12} * h_{beam}^3 * b_{beam} = 0.35 * \frac{1}{12} * 0.480^3 * 0.192 = 6.19 * 10^{-4} \text{ m}^4;$$

$$I_s = 0.25 * \frac{1}{12} * h_{slab}^3 * L = 0.25 * \frac{1}{12} * 0.150^3 * 6 = 4.22 * 10^{-4} \text{ m}^4.$$

Here, 0.35 and 0.25 are moment of inertia reduction factors to account for concrete cracking according to ACI 318-19 Chapter 6. Next, the stiffness ratio was calculated using obtained values.

$$\alpha_{fm} = \frac{6.19 * 10^{-4}}{4.22 * 10^{-4}} = 1.47$$

Since the value is bigger than 0.2 and less than 2.0, the minimum thickness was obtained as follows:

$$h = \frac{l_n (0.8 + \frac{f_y}{200000})}{36 + 5\beta(\alpha_{fm} - 0.2)},$$

where l_n is the biggest span, f_y is the yield strength of rebar, β is the ratio of lengths.

After plugging in the numbers, the minimum thickness is:

$$h = \frac{6 * (0.8 + \frac{75000}{200000})}{36 + 5 * \frac{6}{6} * (1.47 - 0.2)} = 0.1665 \text{ m} > 0.127 \text{ m}.$$

Since there is a significant difference between the initial guess and calculated thickness, more iterations were needed to get closest values.

The same procedure was repeated for the initial slab thickness of 180 mm.

$$I_s = 0.25 * \frac{1}{12} * h_{slab}^3 * L = 0.25 * \frac{1}{12} * 0.180^3 * 6 = 7.29 * 10^{-4} \text{ m}^4;$$

$$\alpha_{fm} = \frac{6.19 * 10^{-4}}{7.29 * 10^{-4}} = 0.85.$$

The stiffness ratio is between 0.2 and 2.0, so minimum thickness is:

$$h = \frac{6 * (0.8 + \frac{75000}{200000})}{36 + 5 * \frac{6}{6} * (0.85 - 0.2)} = 0.1796 \text{ m} > 0.125 \text{ m}$$

This value corresponds to the initial guess, so it will be selected as the slab thickness. Now that all members are sized, volume can be calculated.

$$V_{beam} = 0.480 * 0.192 * 6 = 0.553 \text{ m}^3;$$

$$V_{slab} = 0.180 * 6 * 6 = 6.480 \text{ m}^3;$$

$$V_{1, total} = V_{beam} * 4 + V_{slab} = 0.553 * 4 + 6.480 = 8.692 \text{ m}^3.$$

Option 2. Two-way slab with minor beams

Minor beams' dimensions were estimated as follows:

$$L = 6 \text{ m};$$

$$h_{minor} = \frac{L}{16} = \frac{6}{16} = 0.375 \text{ m};$$

$$b_{minor} = 0.4 * h_{minor} = 0.4 * 0.375 = 0.150 \text{ m}.$$

Starting with the same initial guess of 150 mm, members' moments of inertia were calculated. Slab length was taken as 3 m as the minor beams divide it into 4 two-way slabs and load is distributed accordingly.

$$I_{major} = 6.19 * 10^{-4} \text{ m}^4;$$

$$I_{minor} = 0.35 * \frac{1}{12} * h_{minor}^3 * b_{minor} = 0.35 * \frac{1}{12} * 0.375^3 * 0.150 = 2.3 * 10^{-4} \text{ m}^4;$$

$$I_s = 0.25 * \frac{1}{12} * h_{slab}^3 * L = 0.25 * \frac{1}{12} * 0.150^3 * 3 = 2.11 * 10^{-4} \text{ m}^4.$$

Next, stiffness ratios for major and minor beams were calculated separately and their average was used for further calculations.

$$\alpha_{fm} = \frac{6.19 * 10^{-4}}{2.11 * 10^{-4}} + \frac{2.31 * 10^{-4}}{2.11 * 10^{-4}} = 4.03$$

As this value is larger than 2.0, the minimum thickness was calculated using the following equation:

$$h = \frac{l_n (0.8 + \frac{f_y}{200000})}{36 + 9\beta} = \frac{3 * (0.8 + \frac{75000}{200000})}{36 + 9 * \frac{6}{6}} = 0.0783 \text{ m} < 0.090 \text{ m}$$

The minimum thickness is 90 mm and the value does not change with more iterations, as stiffness ratio only increases. According to IBC Chapter 19, this minimum thickness is

applied for slabs supported directly on the ground. Hence, the slab thickness was chosen as 120 mm for conservative design. Volume for this option is:

$$V_{major} = 0.553 \text{ m}^3;$$

$$V_{minor} = 0.375 * 0.150 * 6 = 0.3375 \text{ m}^3;$$

$$V_{slab} = 0.120 * 3 * 3 = 1.08 \text{ m}^3;$$

$$V_{2, total} = V_{major} * 4 + V_{minor} * 2 + V_{slab} * 4 = 2.212 + 0.3375 * 2 + 1.08 * 4 = 7.207 \text{ m}^3.$$

Option 3. One-way slab with minor beams

Member sizes were estimated as follows:

$$h_{slab} = \frac{L}{20} = \frac{3}{20} = 0.150 \text{ m};$$

$$h_{major} = 0.08 * L = 0.08 * 6 = 0.480 \text{ m};$$

$$b_{major} = 0.4 * h_{beam} = 0.4 * 0.480 = 0.192 \text{ m};$$

$$h_{minor} = \frac{L}{16} = \frac{6}{16} = 0.375 \text{ m};$$

$$b_{minor} = 0.4 * h_{minor} = 0.4 * 0.375 = 0.150 \text{ m}.$$

Span for slab is 3 m since load is transferred between minor and major beams. Finally, volume was calculated for this option as well.

$$V_{major} = 0.553 \text{ m}^3;$$

$$V_{minor} = 0.3375 \text{ m}^3;$$

$$V_{slab} = 0.150 * 3 * 6 = 2.7 \text{ m}^3;$$

$$V_{3, total} = V_{major} * 4 + V_{minor} * 1 + V_{slab} * 2 = 2.212 + 0.3375 + 2.7 * 2 = 7.9495 \text{ m}^3.$$

Member dimensions for all three options are summarized in the table below.

Table 2.3.1. GLRS options member size dimensions

Option	Member	L, m	h, m	b, m	V, m ³	V _{total} , m ³
Two-way slab	Slab	6.000	0.180	6.000	6.480	8.692
	Beam	6.000	0.480	0.192	0.553	
Two-way	Slab	3.000	0.120	3.000	1.080	7.207

slab with minor beams	Major beam	6.000	0.480	0.192	0.553	
	Minor beam	6.000	0.375	0.150	0.338	
One-way slab with minor beams	Slab	3.000	0.150	6.000	2.700	7.9495
	Major beam	6.000	0.480	0.192	0.553	
	Minor beam	6.000	0.375	0.150	0.338	

It is evident that option 2, two-way slab with minor beams, is most economical. Hence, it will be chosen as the gravity load resisting system. For the simplicity of calculations and interpretation of results, dimensions were not rounded to more practical values.

2.3.1.2 Sizing members

Size estimations for beams and slab were demonstrated in the section above. Dimensions have to be adjusted based on the practicality during construction. The final rounded dimensions are presented in Table 2.3.2.

Table 2.3.2. Final member dimensions

Member	L, <i>m</i>	h, <i>m</i>	b, <i>m</i>
Slab	3.000	0.120	3.000
Major beam	6.000	0.500	0.200
Minor beam	6.000	0.400	0.150

Interior column tributary area is the biggest among all members, and, consequently, it will support the biggest amount of loads and will be the biggest among three types of columns. Hence, column design was focused on interior columns only.

Axial load on the interior column includes floor dead loads, interior wall loads, live loads, snow loads and minor and major beams' self-weights, as well as upper columns' self weights. The following load combination was used for factoring the loads:

$$w_u = 1.2D + 1.6L + 0.5S$$

Snow load significantly exceeds roof live load, so it is considered for load calculations. In order to calculate self-weights of members, the unit weight of reinforced concrete was taken as 25 kN/m³. Self-weights of major and minor beams were calculated as follows:

$$SW_{major} = h * b * L * \gamma = 0.5 * 0.2 * 6 * 25 = 15 \text{ kN};$$

$$SW_{minor} = h * b * L * \gamma = 0.4 * 0.15 * 6 * 25 = 9 \text{ kN}.$$

Furthermore, the contribution of the beams' tributary area to the column's tributary area was also considered. For example, in Figure 2.3.8, it can be seen that half of the major beams' tributary area (shaded red area) is included in the interior column's tributary area (dark red borders). Additionally, there are 4 such halves, as there are 4 major beams connected to the column. Therefore, the major beam's self-weight would be multiplied by 4 and 0.5.

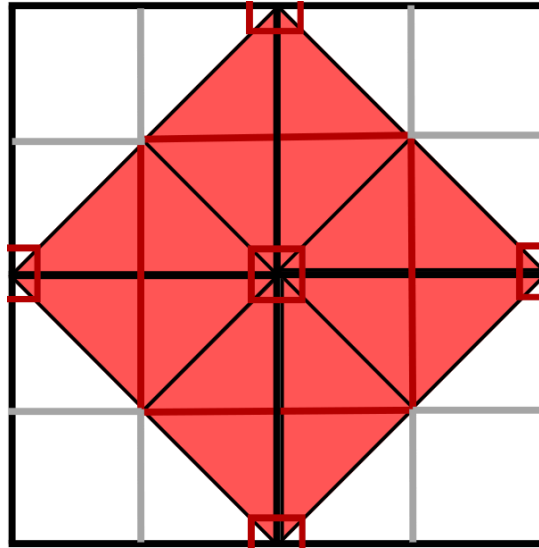


Figure 2.3.2. Minor beam and interior column tributary areas

Axial load was calculated using the following equation:

$$P_u = n * w_u * A_t,$$

where n is the number of stories above the column being considered. Since for our building loads on each floor are not uniform, multiplying by the number of stories is not applicable. Therefore, cumulative axial loads were calculated. For example, for slab 13,

$$w_u = 1.2 * 6.415 + 1.6 * 0 + 0.5 * 1.7 = 8.548 \text{ kPa};$$

$$P_u = 8.548 * 9 = 76.932 \text{ kN};$$

After adding factored self-weight of beams, the total P_u is 134.532 kN. Then, the next slab's axial load will be added to this. Final values are demonstrated in the table below.

Table 2.3.3. Cumulative axial loads on interior column

Location	Axial load, kN
Slab 13	134.532
Slab 12	499.86
Slab 11	903.122
Slab 10	1306.383
Slab 9	1709.645
Slab 8	2112.907
Slab 7	2516.169
Slab 6	2919.431
Slab 5	3322.693
Slab 4	3725.955
Slab 3	4129.217
Slab 2	4743.344
Slab 1	5365.864
Slab 0	5986.177
Slab -1	6436.163

These values were then used for column sizing using the following equation:

$$\phi P_{max} = 0.8\phi(\beta f'_c(A_g - A_{st}) + f_y A_{st})$$

1 % reinforcement was assumed. For top 2 floors, normal strength concrete was used, with 28 MPa (4000 psi) compressive strength and β equal to 0.85. For the remaining columns stronger concrete with 35 MPa (5075 psi) compressive strength and β equal to 0.8 was chosen. Different concrete grades would ensure more economical design. Furthermore, performance of the building would not be affected since the top 2 floors' axial loads are relatively low.

Column sizing for the top three columns will be shown as an example. The initial dimensions were assumed to be 300 mm by 300 mm. Then, ϕP_{max} is:

$$\phi P_{max} = 3 * (1.2 * 0.3^2 * 3 * 25) + (P_u)_{slab 11} = 927.422 \text{ kN}$$

Then, ϕ is 0.65 for tied columns, A_{st} is $0.01A_g$, and after plugging in the numbers and solving for A_g , the calculated size is 250.69 mm. Second iteration with 250 mm by 250 mm was done and the calculated dimensions were 249.68 mm. This is close to the initial guess and with further iterations values converge to 250 mm. So, it was chosen as the final size for the top 3 columns. The same procedure was repeated for the other columns and it was decided to vary column sizes every 4 stories. The final column dimensions are shown in Table 2.3.15.

Table 2.3.4. Final interior column dimensions

Location	Size, mm
Floors 12-13	250
Floors 8-11	400
Floors 4-7	500
Floors 0-3	600

These sizes will be uniform for exterior and corner columns as well. Additionally, total axial loads transferred to foundation had to be calculated for further geotechnical design calculations. For corner and exterior columns, exterior wall loads were considered and multiplied by the span.

$$SW_{exterior} = 14.325 * 6 = 85.95 \text{ kN}$$

Then, the same procedure was repeated and major and minor beams' self-weights were adjusted based on their tributary areas. Finally, the final loads transferred to foundation were calculated as follows:

$$P_u = (P_u)_{slab-1} + 25 * 1.2 * (0.25^2 * 2 + 0.4^2 * 4 + 0.5^2 * 4 + 0.6^2 * 4)$$

The total axial load to the foundation is 6674.74 kN for the interior column, 5414.21 kN for exterior and 3833.85 kN for corner.

2.3.1.3 Gravity load calculations

Dead loads

In order to calculate dead loads, different material types for floor finishing, ceiling, interior and exterior walls, parapet and their loads were taken from ASCE 7-10 Chapter 3.

Table 2.3.5. Floor finishing dead loads

Location	Material type	Thickness, mm	Load, kPa	Total load, kPa
All levels	Concrete fill finish	40	0.92	4.038
	Vapor control layer	1	0.03	
	Mineral wool floor slab	40	0.088	
	RC slab	120	3	
Roof	Three-ply ready roofing	50	2.5	5.71
	Liquid applied waterproofing membrane	0	0.05	
	Vapor control layer	1	0.03	
	RC slab	120	3.125	
	Cellular glass insulation membrane	100	0.13	

Since the hotel has a variety of rooms for different uses, applying one uniform floor finishing material is not possible. Table 2.3.6 shows the floor finishing choices for each room. These values were used to determine the average floor finishing dead load on each floor depending on the area covered.

Table 2.3.6. Floor cover choices for rooms

Room	Material type	Thickness, mm	Load, kPa	Total load, kPa
Private rooms	Hardwood flooring	22	0.19	4.353
Bathroom	Ceramic tile	20	0.77	4.933
Conference room	Linoleum	6	0.05	4.213
Common space	Marble	20	1.58	5.743

Entertainment	Hardwood flooring	22	0.19	4.353
Spa	Ceramic tile	20	0.77	4.933
Restaurant	Granite stone	20	0.518	4.681
Kitchen	Ceramic tile	20	0.77	4.933
Gym	Rubber	20	0.18	4.343

Table 2.3.7. Average floor finishing dead loads

Floor type, m^2	Hardwood flooring	Ceramic tile	Linoleum	Marble	Granite stone	Rubber	Average floor dead load, kPa
Floor 1	38	70	282	408	0	0	5.0651
Floor 2	0	62	0	524	485	0	5.2152
Floor 3	0	66	0	255	0	293	4.9879
Floor 4-12	5446	882	0	2191	0	0	4.7705

Ceiling materials and loads are presented in Table 2.3.8. The total thickness of the ceiling is 400 mm.

Table 2.3.8. Ceiling dead loads

Location	Material type	Thickness, mm	Load, kPa	Total load, kPa
All levels	Acoustic fiber	20	0.05	0.58
	Mechanical duct allowance	150	0.19	
	Suspended steel channel system	200	0.10	
	Gypsum board	30	0.24	

For the core of interior walls solid CMU walls were chosen. The total thickness of interior walls is 196 mm. Material choices are presented in Table 2.3.9.

Table 2.3.9. Interior wall material dead loads

Location	Material type	Thickness, mm	Load, kPa	Total load, kPa
All levels	Wallpaper	5	0.05	2.19
	Gypsum board	30	0.24	
	Rigid insulation	12	0.04	
	Solid CMU wall	102	1.53	
	Rigid insulation	12	0.04	
	Gypsum board	30	0.24	
	Wallpaper	5	0.05	

In order to improve exterior walls' strength and reliability a reinforcing net will be added. Final material selection can be seen in Table 2.3.10. Furthermore, material choices for the roof parapet are demonstrated in Table 2.3.11.

Table 2.3.10. Exterior wall material dead loads

Location	Material type	Thickness, mm	Load, kPa	Total load, kPa
All levels	Wallpaper	5	0.05	4.775
	Gypsum board	30	0.24	
	Rigid insulation	24	0.08	
	Solid CMU wall	254	4.02	
	Rigid insulation	24	0.08	
	Vapor control layer	1	0.03	
	Reinforcing net	6	0.03	

	Plaster	10	0.24	
	Rainscreen cladding panels	16	0.005	

Table 2.3.11. Parapet material dead loads

Location	Material type	Thickness, mm	Load, kPa	Total load, kPa
Roof	Plaster	10	0.24	4.265
	Solid CMU wall	254	4.02	
	Rainscreen cladding panels	16	0.005	

To determine total floor load from interior walls, the total length and height of the interior walls had to be considered. For that, the dead load was multiplied by the area of interior walls on each floor and the resulting value was divided by the total floor area in order to get the load per unit area. Final interior walls' pressure loads are shown in Table 2.3.12.

Table 2.3.12. Total interior wall dead loads

Location	Height, m	Length, m	Total floor area, m^2	Load, kN	Floor load, kPa
Slab 13	0	0	1487.16	0	0
Slab 12	0	0		0	0
Slabs 3-11	3	276		1813.32	1.036
Slab 2	4	168		1471.68	0.928
Slab 1	4	162		1419.12	0.895
Slab 0	4	180		1576.8	0.994
Slab -1	6.5	186		2647.71	1.780

Finally, the total loads for each slab were calculated by summing floor dead load and interior wall load. Exterior wall load was expressed in force per unit length by multiplying the pressure load by wall height. Separating the exterior wall load, as a structural member was done to simplify further calculations for interior column sizing. Final loads are shown in Table 2.3.13.

Table 2.3.13. Total dead load on each slab

Location	Pressure load, kPa	Exterior walls load, kN/m
Slab 13	6.29	0
Slab 12	6.29	14.325
Slabs 3-11	6.445	14.325
Slab 2	6.432	19.1
Slab 1	6.624	19.1
Slab 0	6.580	19.1
Slab -1	6.818	31.04

Live loads

Minimum uniformly distributed live loads were taken from ASCE 7-10 Chapter 4 Table 4-1. For private rooms and corridors serving them in residential occupancy the live load value is 1.92 kPa. Floors 4 to 12 fit this use. This number was further reduced according to the formula below:

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

where L_0 is unreduced live load, A_T is tributary area of a member, K_{LL} is live load element factor. Additionally, following requirements have to be satisfied:

$$L \geq 0.4L_0 = 0.768 \text{ for columns;}$$

$$L \geq 0.5L_0 = 0.96 \text{ for other members;}$$

$$\text{No reduction when: } K_{LL} A_T < 37.16 \text{ m}^2.$$

For floors 1 to 3 L_0 is 4.79 kPa, corresponding to public rooms and corridors serving them. According to ASCE 7-10 Section 4.7.5, live load for this use shall not be reduced. The same applies to the underground parking. For passenger vehicles only garages L_0 is 1.92 kPa and no reduction is permitted for this use.

Final reduced live loads for all levels except roof are shown in Table 2.3.14.

Table 2.3.14. Reduced live loads for floors except roof

Member	A_T, m^2	K_{LL}	$K_{LL} A_T$	L, kPa for floors 4-12	L, kPa for floors 1-3	L, kPa for parking
Column (interior)	36	4	144	1.2112	4.79	1.92
Column (edge)	18	4	72	1.5141	4.79	1.92
Column (corner)	9	1	9	1.92	4.79	1.92
Beam (major, interior)	18	2	36	1.92	4.79	1.92
Beam (major, exterior)	9	2	18	1.92	4.79	1.92
Beam (minor)	18	2	36	1.92	4.79	1.92

Minimum uniformly distributed live load for ordinary flat roof is 0.96 kPa. Reductions were done according to the following formula:

$$L_r = L_0 R_1 R_2, \text{ where } 0.58 \leq L_r \leq 0.96$$

Here, R_1 and R_2 are determined as follows:

$$R_1 = 1, \text{ for } A_T \leq 18.58 \text{ m}^2;$$

$$R_1 = 1.2 - 0.011 A_T, \text{ for } 18.58 \text{ m}^2 < A_T < 55.74 \text{ m}^2;$$

$$R_2 = 1, \text{ for } F \leq 4,$$

where F is roof slope multiplied by 0.12. For a flat roof, slope is zero.

Total reduced live loads for the roof are demonstrated in Table 2.3.15.

Table 2.3.15. Reduced live loads for roof

Member	$A_T, \text{ m}^2$	R_1	$L_r, \text{ kPa}$
Column (interior)	36	0.804	0.77184
Column (edge)	18	1	0.96
Column (corner)	9	1	0.96
Beam (major interior)	18	1	0.96
Beam (major, exterior)	9	1	0.96
Beam (minor)	18	1	0.96

Snow loads

Flat roof snow load was calculated using the formula:

$$p_f = 0.7 C_e C_t I_s p_g$$

Here, ground snow load, p_g , is 15 psf for Seattle as indicated in ASCE 7-10 Chapter 7.

Since the building is located in a densely populated area of the city, the exposure factor, C_e , for terrain category B and sheltered roof was used. According to ASCE 7-10 Table 7.2 this corresponds to the value of 1.2. Thermal factor, C_t , was taken as 1, because the building does not fit any special conditions. Lastly, snow importance factor, I_s , is 1, for risk category II. This risk category applies to all structures that do not pose low or substantial risk to human life in the event of failure and that are not essential facilities.

After plugging in the numbers:

$$p_f = 0.7 * 15 * 1 * 1.2 * 1 = 12.6 \text{ psf}$$

This value has to be compared with minimum snow load:

$$p_m = I_s p_g = 1 * 15 = 15 \text{ psf} > 12.6 \text{ psf}$$

Thus, $p_f = 15 \text{ psf} = 0,718 \text{ kPa}$.

Snow drift

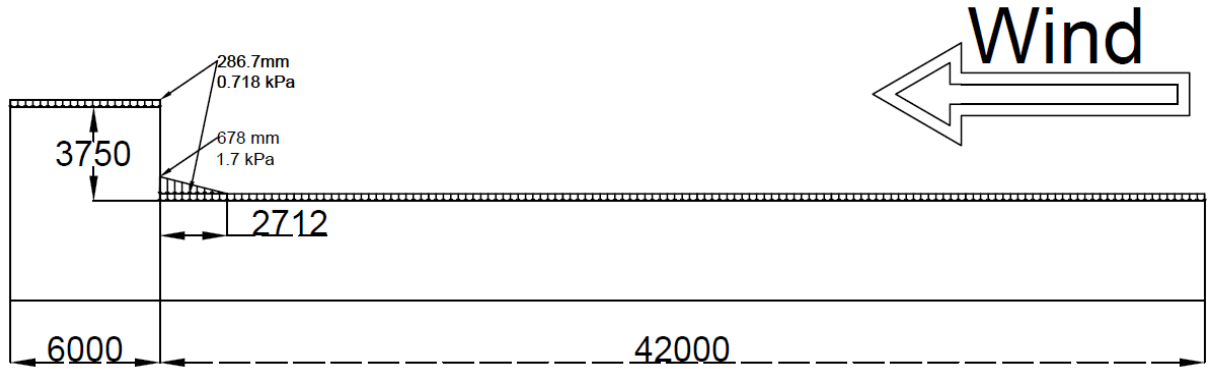


Figure 2.3.3. First case: wind from right

$$Lu = 6 \text{ m} = 19.685 \text{ ft}$$

$$Lu^* = 42 \text{ m} = 137.795 \text{ ft}$$

$$p_f = 15 \text{ psf}$$

$$\text{Windward } h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{Lu^*} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{Lu^*} \times \sqrt[4]{p_g + 10} - 1.5 = \frac{3}{4} \times 0.43 \times \sqrt[3]{137.795} \times \sqrt[4]{15 + 10} - 1.5$$

$$h_d = 2,225 \text{ ft} = 0.678 \text{ m}$$

$$\gamma = 0.426 p_g + 2.2 = 0.426 \times 15 \times 0.0479 + 2.2 = 2.506 \leq 4.7 \text{ kN/m}^3$$

$$h_b = \frac{p_f}{\gamma} = \frac{15 \times 0.0479}{2.506} = 0.2867 \text{ m}$$

$$\text{Leeward } h_d = 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 1.096 \text{ ft} = 0.334 \text{ m}$$

However, as there is no Lu^*_2 it will not be considered.

$$h_c = 3.3 - h_b = 3.3 - 0.2867 = 3.0133 \text{ m}$$

$$h_d \leq h_c \rightarrow w = 4 h_d = 4 * 0.678 = 2.712 \text{ m} \leq 8 h_c$$

$$P_d = h_d \times \gamma = 0.678 \times 2.506 = 1.7 \text{ kN/m}^2$$

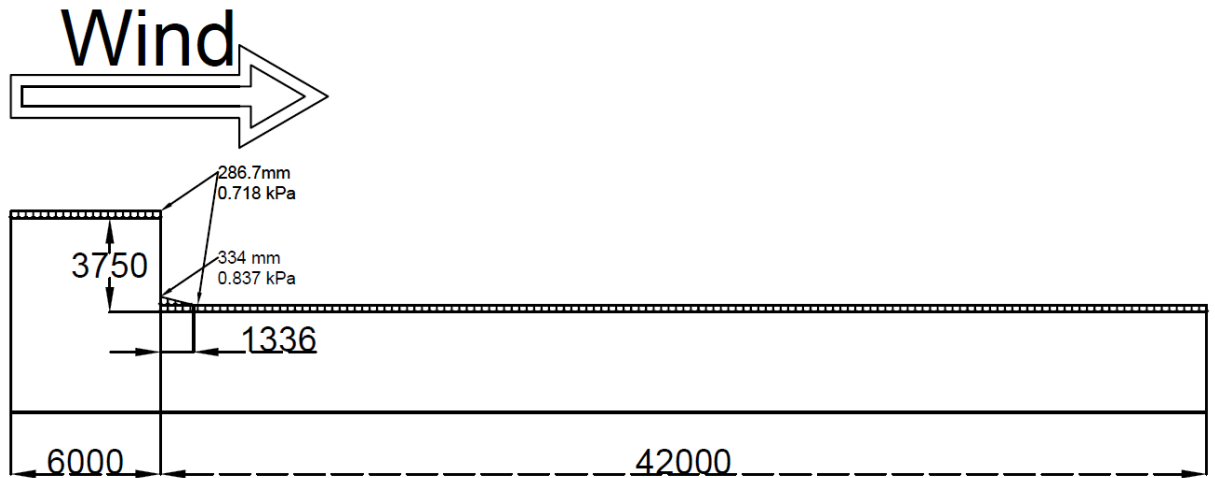


Figure 2.3.4. First case: wind from left

$$\text{Leeward } h_d = 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 1.096 \text{ ft} = 0.334 \text{ m}$$

$$h_d \leq h_c \rightarrow w = 4h_d = 4 \times 0.334 = 1.336 \text{ m} \leq 8h_c$$

$$P_d = h_d \times \gamma = 0.334 \times 2.506 = 0.837 \text{ kN/m}^2$$

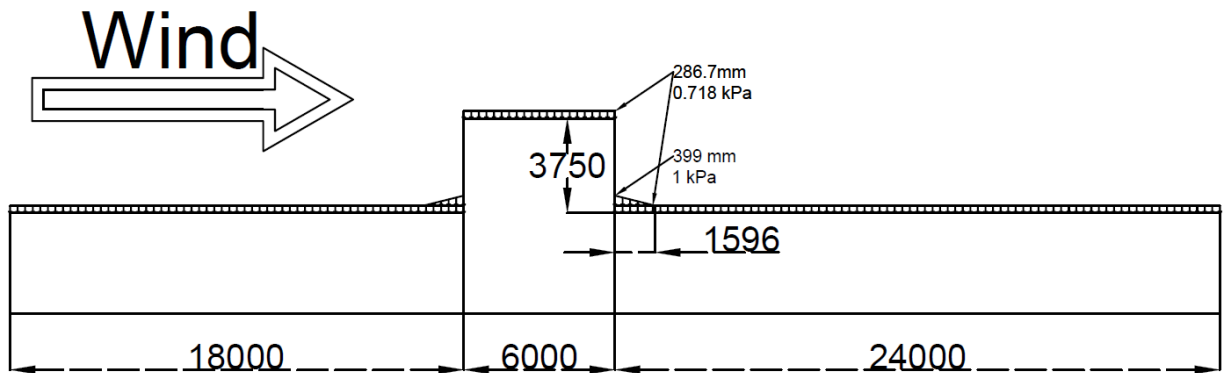


Figure 2.3.5. Second case: wind from left

$$Lu = 6 \text{ m} = 19.685 \text{ ft}$$

$$Lu^*_1 = 18 \text{ m} = 59.055 \text{ ft}$$

$$Lu^*_2 = 24 \text{ m} = 78.74 \text{ ft}$$

$$\text{Windward } h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{Lu^*} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{59.055} \times \sqrt[4]{15 + 10} - 1.5 = 1.308 \text{ ft} = 0.399 \text{ m}$$

$$\text{Leeward } h_d = 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 1.096 \text{ ft} = 0.334 \text{ m}$$

$$h_c = 3.3 - h_b = 3.3 - 0.2867 = 3.0133 \text{ m}$$

$$h_d \leq h_c \rightarrow w = 4h_d = 4 * 0.399 = 1.596 \text{ m} \leq 8h_c$$

$$P_d = h_d \times \gamma = 0.399 \times 2.506 = 1 \text{ kN/m}^2$$

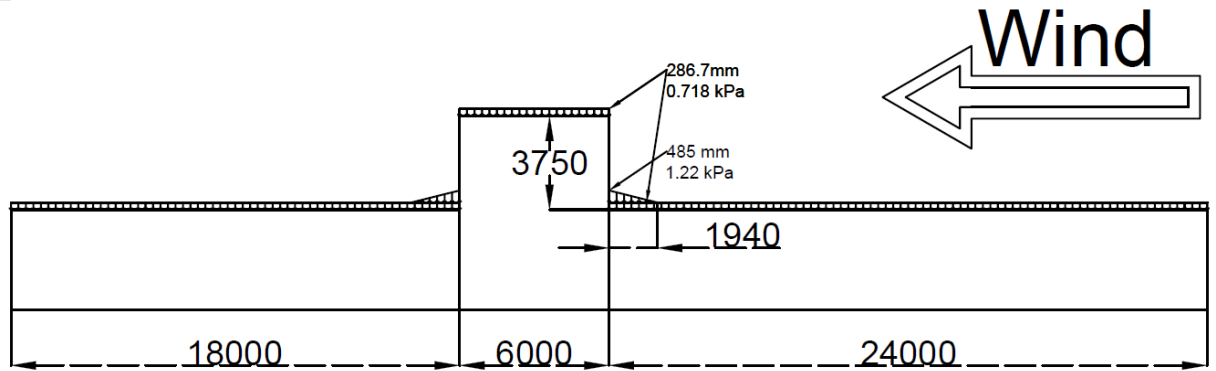


Figure 2.3.6. Second case: wind from right

$$\text{Windward } h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{78.74} \times \sqrt[4]{15 + 10} - 1.5 = 1.591 \text{ ft} = 0.485 \text{ m}$$

$$\text{Leeward } h_d = 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 1.096 \text{ ft} = 0.334 \text{ m}$$

$$h_d \leq h_c \rightarrow w = 4h_d = 4 * 0.485 = 1.94 \text{ m} \leq 8h_c$$

$$P_d = h_d \times \gamma = 0.485 \times 2.506 = 1.22 \text{ kN/m}^2$$

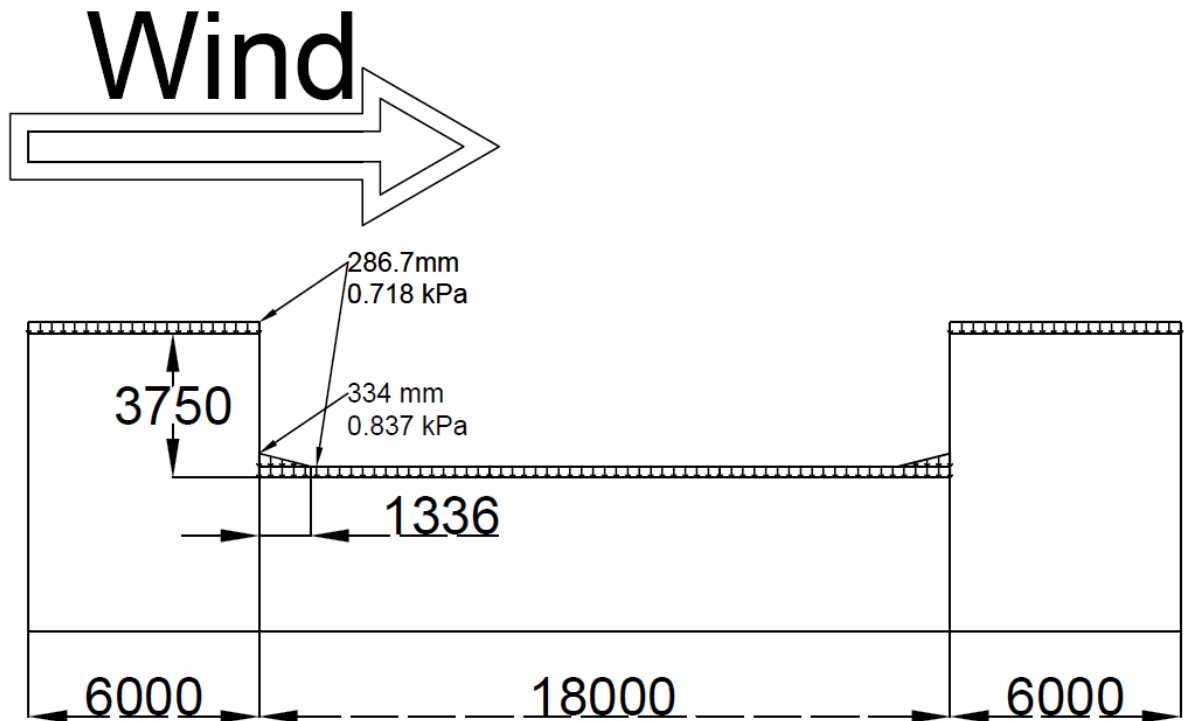


Figure 2.3.7. Third case

$$Lu = 6 \text{ m} = 19.685 \text{ ft}$$

$$Lu^* = 18 \text{ m} = 59.055 \text{ ft}$$

$$\text{Leeward } h_d = 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 1.096 \text{ ft} = 0.334 \text{ m}$$

$$h_d \leq h_c \rightarrow w = 4h_d = 4 * 0.334 = 1.336 \text{ m} \leq 8h_c$$

$$P_d = h_d \times \gamma = 0.334 \times 2.506 = 0.837 \text{ kN/m}^2$$

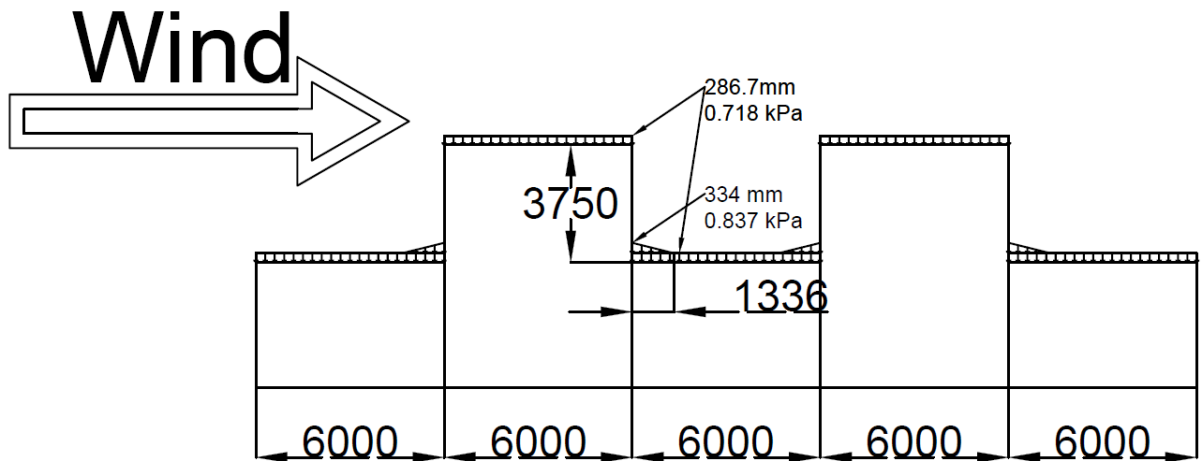


Figure 2.3.8. Fourth case

$$Lu = 6 \text{ m} = 19.685 \text{ ft}$$

$$Lu^*_1 = 6 \text{ m} = 19.685 \text{ ft}$$

$$Lu^*_2 = 6 \text{ m} = 19.685 \text{ ft}$$

$$\text{Windward } h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{Lu^*} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = \frac{3}{4} \times 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 0.447 \text{ ft} = 0.136 \text{ m}$$

$$\text{Leeward } h_d = 0.43 \times \sqrt[3]{Lu} \times \sqrt[4]{p_g + 10} - 1.5$$

$$h_d = 0.43 \times \sqrt[3]{19.685} \times \sqrt[4]{15 + 10} - 1.5 = 1.096 \text{ ft} = 0.334 \text{ m}$$

$$h_d \leq h_c \rightarrow w = 4h_d = 4 * 0.334 = 1.336 \text{ m} \leq 8h_c$$

$$P_d = h_d \times \gamma = 0.334 \times 2.506 = 0.837 \text{ kN/m}^2$$

2.3.1.4 Lateral forces calculations

Wind loads

Basic wind speed in Seattle, WA was identified as 98 mph which is 43.8 m/s.

According to ASCE 7-16 general characteristics were found as follows:

Exposure Category B (city)

Direction effect $K_d=0.85$

Topographical effect $K_{zt}=1$ as it is located in a flat area.

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

Figure 2.3.9. ASCE 7-Wind load: Table 26.9-1

Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All values	-0.7	q_h

Figure 2.3.10. ASCE 7-Wind load: Table 27.4-1

Height effect

To find the velocity pressure exposure coefficient we use the following formulas:

$$\text{For } 15ft \leq z \leq z_g \quad K_z = 2.01 * (z/z_g)^{2/\alpha}$$

$$\text{For } z < 15ft \quad K_z = 2.01 * (15/z_g)^{2/\alpha}$$

By using those formulas K_z values were estimated:

Table 2.3.16. K_z values

Story	h (m)	z (m)	K_z
13	3.75	51.75	1.150
12	3.75	48	1.125
11	3.75	44.25	1.099
10	3.75	40.5	1.072
9	3.75	36.75	1.042
8	3.75	33	1.011
7	3.75	29.25	0.977
6	3.75	25.5	0.939
5	3.75	21.75	0.897
4	3.75	18	0.850
3	4.75	14.25	0.795
2	4.75	9.5	0.708
1	4.75	4.75	0.581

Gust effect

To calculate the gust effect we made calculations for two different wind directions.

First: B=48.6 m and L=30.6 m. Number of frames = 9;

Second: B=30.6 m and L=48.6 m. Number of frames = 6

Case 1: (For First)

For concrete moment resisting frame buildings $n_a = 43.5/h^{0.9}$

H=51.75 m which is 167.78 ft, so $n_1 = 0.428$

$$\bar{z} = 0.6h = 0.6 * 51.75 = 31.05 > 9.14 (z_{min})$$

$$I_z = c * (\frac{10}{z})^{1/6} = 0.3 * (\frac{10}{31.05})^{1/6} = 0.248$$

$$L_z = l * (\frac{\bar{z}}{10})^{\bar{\epsilon}} = 97.54 * (\frac{31.05}{10})^{1/3} = 142.3 \text{ m}$$

$$V_z = \bar{b} * (\frac{\bar{z}}{10})^{\bar{\alpha}} * V = 0.45 * (\frac{31.05}{10})^{0.25} * 43.8 = 26.16 \text{ m/s}$$

$$N_1 = \frac{n_1 * L_z}{\bar{V}_z} = \frac{0.428 * 142.3}{26.16} = 2.329$$

$$R_n = \frac{7.47 * N_1}{(1 + 10.3 * N_1)^{5/3}} = 0.08146$$

$$R_l = \frac{1}{\eta} - \frac{1}{2\eta^2} (1 - e^{-2\eta})$$

$$R_l = R_h \Rightarrow \eta = 4.6n_1 h / \bar{V}_z = 3.895$$

$$R_l = R_B \Rightarrow \eta = 4.6n_1 B / \bar{V}_z = 3.658$$

$$R_l = R_L \Rightarrow \eta = 15.4n_1 / \bar{V}_z = 7.711$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.355$$

$$g_R = \sqrt{2 \ln(3600n_1)} + \frac{0.577}{\sqrt{2 \ln(3600n_1)}} = 3.982$$

$$Q = \sqrt{\frac{1}{1 + 0.63(\frac{B+h}{L_z})^{0.63}}} = 0.815$$

$$G_f = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z} \right) = 0.879$$

External pressure coefficient Cp for this case will be as follows:

Windward=0.8 and Leeward=-0.5 as L/B=0.63<1

Case 1: (For Second)

By repeating the same calculations as before, but by changing the L and B we were able to find the following:

$$G_f = 0.906$$

$$L/B = 48.6/30.6 = 1.588$$

Windward = 0.8

$$\text{Leeward} = \frac{-0.3 - (-0.5)}{2 - 1} * (1.588 - 1) + (-0.5) = -0.38$$

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

$$P = q_z G C_{p, \text{windward}} - q_h G C_{p, \text{leeward}}$$

Force at top floor:

$$F_{top} = (B / \text{number of frames}) * h(\text{top floor}) / 2 * P_{top}$$

Force in the following floors:

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

The results of the calculations are provided in the following tables:

Table 2.3.17. Wind force calculations (Case 1: First side)

Story	h (m)	z (m)	Kz	q (Pa)	Gf	Pw. Pl (Pa)	P (Pa)	Fi (kN)	F per frame (kN)
13	3.75	51.75	1.150	1149.13	0.879	807.81	1312.69	119.62	13.29
12	3.75	48	1.125	1124.70		790.63	1295.51	237.67	26.41
11	3.75	44.25	1.099	1098.86		772.47	1277.35	234.45	26.05
10	3.75	40.5	1.072	1071.41		753.17	1258.05	231.04	25.67
9	3.75	36.75	1.042	1042.07		732.55	1237.43	227.40	25.27
8	3.75	33	1.011	1010.52		710.36	1215.24	223.50	24.83
7	3.75	29.25	0.977	976.28		686.30	1191.18	219.29	24.37
6	3.75	25.5	0.939	938.75		659.92	1164.80	214.69	23.85
5	3.75	21.75	0.897	897.04		630.60	1135.48	209.61	23.29
4	3.75	18	0.850	849.83		597.40	1102.28	203.92	22.66
3	4.75	14.25	0.795	794.96		558.83	1063.71	223.22	24.80
2	4.75	9.5	0.708	708.00		497.70	1002.58	238.50	26.50
1	4.75	4.75	0.581	580.80		408.28	913.16	221.12	24.57

Table 2.3.18. Wind force calculations (Case 1: Second side)

Story	h (m)	z (m)	Kz	q (Pa)	Gf	Pw. Pl (Pa)	P (Pa)	Fi (kN)	F per frame (kN)
13	3.75	51.75	1.150	1149.13	0.906	832.89	1228.52	70.49	11.75
12	3.75	48	1.125	1124.70		815.18	1210.81	139.96	23.33
11	3.75	44.25	1.099	1098.86		796.45	1192.08	137.87	22.98
10	3.75	40.5	1.072	1071.41		776.56	1172.18	135.65	22.61
9	3.75	36.75	1.042	1042.07		755.29	1150.92	133.29	22.21
8	3.75	33	1.011	1010.52		732.42	1128.05	130.76	21.79
7	3.75	29.25	0.977	976.28		707.61	1103.23	128.02	21.34
6	3.75	25.5	0.939	938.75		680.41	1076.03	125.04	20.84
5	3.75	21.75	0.897	897.04		650.18	1045.80	121.74	20.29
4	3.75	18	0.850	849.83		615.96	1011.58	118.04	19.67
3	4.75	14.25	0.795	794.96		576.18	971.81	128.67	21.44
2	4.75	9.5	0.708	708.00		513.16	908.78	136.67	22.78
1	4.75	4.75	0.581	580.80		420.96	816.58	125.39	20.90

In general there are 4 cases from which case 2 and case 4 contain torsional effects.

Those cases can be seen from the following figure:

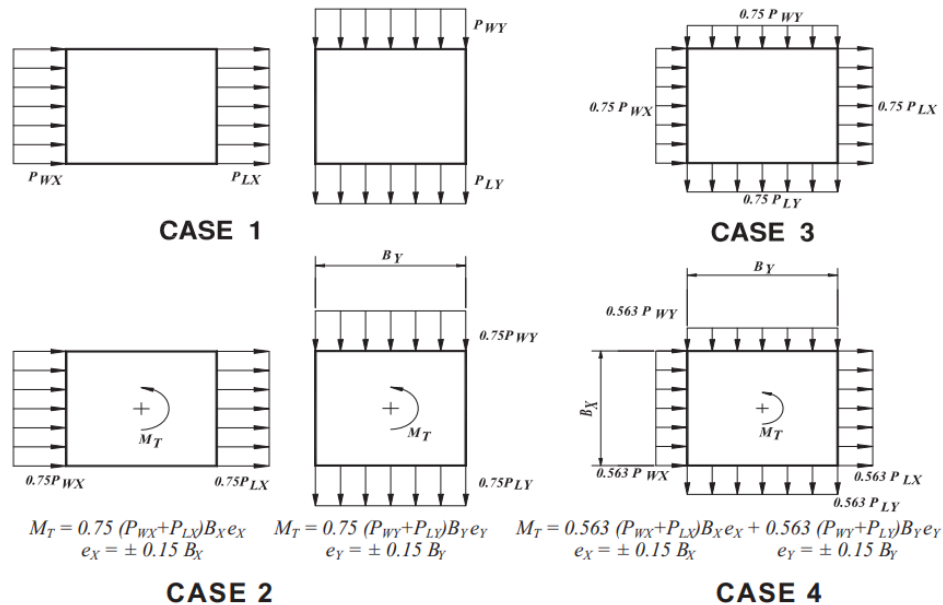


Figure 2.3.11. Design Wind Load Cases

Table 2.3.19. Direct force components (Case 2: First side)

Story	Pw, Pl (Pa)	MT (kN-m)	0.75Fi (kN)	Fdirect (kN)
13	807.81	348.81	89.71	9.97
12	790.63	344.24	178.25	19.81
11	772.47	339.42	175.84	19.54
10	753.17	334.29	173.28	19.25
9	732.55	328.81	170.55	18.95
8	710.36	322.92	167.62	18.62
7	686.30	316.52	164.46	18.27
6	659.92	309.51	161.02	17.89
5	630.60	301.72	157.21	17.47
4	597.40	292.90	152.94	16.99
3	558.83	282.65	167.42	18.60
2	497.70	266.41	178.88	19.88
1	408.28	242.65	165.84	18.43

Table 2.3.20. Torsion and direct force components (Case 2: Second side)

Story	Pw, Pl (Pa)	MT (kN-m)	0.75Fi (kN)	Fdirect (kN)
13	832.89	129.41	52.86	8.81
12	815.18	127.55	104.97	17.49
11	796.45	125.57	103.40	17.23
10	776.56	123.48	101.74	16.96
9	755.29	121.24	99.97	16.66
8	732.42	118.83	98.07	16.34
7	707.61	116.22	96.01	16.00
6	680.41	113.35	93.78	15.63
5	650.18	110.17	91.31	15.22
4	615.96	106.56	88.53	14.76
3	576.18	102.37	96.50	16.08
2	513.16	95.73	102.50	17.08
1	420.96	86.02	94.04	15.67

In order to find the wind force we have to sum up direct force and torsional force. The results for the first floor are shown in the following table (Results for remaining floors will be shown in Appendix A):

Table 2.3.21. Wind force calculations (Case 2: 1st floor)

Case 2. Transverse				Case 2. Longitudinal			
Frame	Ftorsion	Fdirect	Ftotal	Frame	Ftorsion	Fdirect	Ftotal

	(kN)	(kN)	(kN)		(kN)	(kN)	(kN)
A	-0.4398	15.67	15.2341	1	-2.1172	18.43	16.3099
B	-0.2639	15.67	15.4100	2	-1.5879	18.43	16.8392
C	-0.0880	15.67	15.5859	3	-1.0586	18.43	17.3685
D	0.0880	15.67	15.7618	4	-0.5293	18.43	17.8978
E	0.2639	15.67	15.9377	5	0.0000	18.43	18.4271
F	0.4398	15.67	16.1136	6	0.5293	18.43	18.9564
				7	1.0586	18.43	19.4856
				8	1.5879	18.43	20.0149
				9	2.1172	18.43	20.5442

In the same way we can find the results for Case 3 and Case 4.

Table 2.3.22. Wind force calculations (Case 3)

Story	0.75F1 (kN)	F1 per frame (kN)	0.75F2 (kN)	F2 per frame (kN)
13	89.71	9.97	52.86	8.81
12	178.25	19.81	104.97	17.49
11	175.84	19.54	103.40	17.23
10	173.28	19.25	101.74	16.96
9	170.55	18.95	99.97	16.66
8	167.62	18.62	98.07	16.34
7	164.46	18.27	96.01	16.00
6	161.02	17.89	93.78	15.63
5	157.21	17.47	91.31	15.22
4	152.94	16.99	88.53	14.76
3	167.42	18.60	96.50	16.08
2	178.88	19.88	102.50	17.08
1	165.84	18.43	94.04	15.67

Table 2.3.23. Torsion and direct force components (Case 4)

Story	MT (kN-m)	0.563F1 (kN)	0.563F2 (kN)	F1 direct (kN)	F2 direct (kN)
13	358.98	67.35	39.68	7.48	6.61
12	354.16	133.81	78.80	14.87	13.13
11	349.05	132.00	77.62	14.67	12.94

10	343.63	130.07	76.37	14.45	12.73
9	337.84	128.03	75.04	14.23	12.51
8	331.60	125.83	73.62	13.98	12.27
7	324.84	123.46	72.08	13.72	12.01
6	317.43	120.87	70.39	13.43	11.73
5	309.19	118.01	68.54	13.11	11.42
4	299.86	114.80	66.46	12.76	11.08
3	289.02	125.68	72.44	13.96	12.07
2	271.84	134.28	76.95	14.92	12.82
1	246.72	124.49	70.60	13.83	11.77

Table 2.3.24. Wind force calculations (Case 4: 1st floor)

Case 4. Transverse				Case 4. Longitudinal			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	-1.2613	11.77	10.5045	1	-2.1527	13.83	11.6799
B	-0.7568	11.77	11.0090	2	-1.6145	13.83	12.2181
C	-0.2523	11.77	11.5136	3	-1.0763	13.83	12.7562
D	0.2523	11.77	12.0181	4	-0.5382	13.83	13.2944
E	0.7568	11.77	12.5226	5	0.0000	13.83	13.8326
F	1.2613	11.77	13.0272	6	0.5382	13.83	14.3708
				7	1.0763	13.83	14.9089
				8	1.6145	13.83	15.4471
				9	2.1527	13.83	15.9853

Wind force calculations for other floors can be found in Appendix A.

Mass Center

Center of mass needs to be calculated to find the eccentricity and its torsional effect. It can be calculated in the following way:

First of all we have to identify the shaft area of the stairs and the elevator. Total area of two elevators located nearly was found as $2.05 \times 6.12 = 12.546 \text{ m}^2$. Total area of stairs is

$1.1 \times 4.67 = 5.145 \text{ m}^2$. As our building contains two from each of them there will be two different locations of the shafts. Elevators are located at 19.325 meters from the edge of the building in x direction and 8.76 and 21.24 meters in y direction. Similarly location of stairs shaft was also found which are 5.15 meters in x direction and 2.5685 and 27.4315 meters in the y coordinate. So, to find the center of mass we use the following equation:

$$\text{Center of mass} = (1440 \times 24 - (12.546 \times 19.325 \times 2 + 5.145 \times 5.15 \times 2)) / (1440 - 35381) = 24.2215 \text{ v}$$

Seismic loads

In order to calculate seismic loads the equivalent lateral force method was used. First, parameters and constants have to be determined from the ASCE 7-10 Chapter 11. According to the ASCE Hazard Tool website, spectral response acceleration parameters at short periods and at a period of 1 s are 1.343 and 0.52 respectively. In the geotechnical report, soil type was identified as soil class C. Based on the figures below, site coefficients F_a and F_v are 1.0 and 1.3.

Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at Short Period					
Site Class	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Figure 2.3.12. Site coefficient F_a

Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Figure 2.3.13. Site coefficient F_v

Using the following equations:

$$S_{MS} = F_a S_s;$$

$$S_{M1} = F_v S_1;$$

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

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

S_{DS} and S_{D1} parameters were calculated as 0.90 and 0.45, which places the building in Seismic Design Category D, as evident from figures below.

Value of S_{DS}	Risk Category	
	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

Figure 2.3.14. SDC based on S_{DS}

Value of S_{D1}	Risk Category	
	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

Figure 2.3.15. SDC based on S_{D1}

After determining the design category, Seismic Force Resisting System (SFRS) was chosen as special reinforced concrete moment frame, since there are no height limits for category D.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_o^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 ^a	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL

Figure 2.3.16. SFRS, parameters and height limits

Regarding the building period, the approximate fundamental period was calculated as:

$$T_a = C_t h_n^x = 0.0466 * 51.75^{0.9} = 1.625 \text{ sec},$$

where C_t and x values are 0.0466 and 0.9 for concrete moment-resisting frames as shown in Figure 2.3.21. Next, the fundamental period, T was computed as:

$$T \leq C_u T_a = 1.4 * 1.625 = 2.275 \text{ sec.}$$

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 2.3.17. Values of approximate period parameters

For conservative design, the lesser period was chosen. However, this could change with the performance of modal analysis in the future.

Seismic response coefficient was taken as the least of the following:

$$C_s = \frac{S_{DS}}{(R/I)} = 0.1125$$

$$C_{s,max} = \frac{S_{D1}}{T(R/I)} = 0.035$$

However, it should not be taken less than:

$$C_{s,min} = \text{Max}[0.044S_{DS}I, 0.01] = 0.0394$$

Hence, the seismic response coefficient is 0.0394. Next, the effective seismic weight has to be calculated. W is to include all dead loads only, since there are no storage live loads, or partition loads, or large snow loads. For its calculation, the self-weight of columns and beams was multiplied by their quantities on each floor. Furthermore, exterior wall load was multiplied by its total length and then divided by the floor area, to get the smeared value. The total weight of the structure amounted to 199 231 kN. Then, base shear was calculated using the following equation:

$$V_u = C_s W = 0.0394 * 199231 = 7849 \text{ kN}$$

Forces on each frame were obtained by the equations:

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

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k},$$

where k is 1.56 after linear interpolation.

The total forces on each frame are demonstrated in Table 2.3.25. There are 9 frames in transverse and 6 frames in longitudinal directions.

Table 2.3.25. Summarized seismic loads

				Transverse		Longitudinal	
Story	h_x , m	W_x , kN	C_{vx}	F_x , kN	F_x , kN per frame	F_x , kN	F_x , kN per frame
13	51.75	21459	0.236	1853	205.9	1853	308.8
12	48	13840	0.135	1063	118.1	1063	177.1
11	44.25	14194	0.122	960	106.6	960	159.9
10	10.5	14194	0.106	836	92.8	836	139.3
9	36.75	14194	0.091	718	79.8	718	119.6
8	33	14194	0.077	607	67.4	607	101.1
7	29.25	14559	0.066	515	57.3	515	85.9
6	25.5	14559	0.053	416	46.2	416	69.3
5	21.75	14559	0.041	324	36.0	324	54.1
4	18	14559	0.031	241	26.8	241	40.2
3	14.25	15648	0.023	180	20.0	180	30.0
2	9.5	16672	0.013	102	11.3	102	17.0
1	4.75	16599	0.004	34	3.8	34	5.7

Torsional effects have to be considered for seismic loads as well, due to non-uniform stiffness and mass centers. The same procedure as done for wind loads was performed and total forces on each frame were calculated. The forces for the first floor are shown in Table 2.3.26. Tables for remaining floors are included in the appendix.

Table 2.3.26. Torsion, direct and total forces on each frame at floor 1

Transverse				Longitudinal			
Frame	$F_{torsion}$ (kN)	F_{direct} (kN)	F_{total} (kN)	Frame	$F_{torsion}$ (kN)	F_{direct} (kN)	F_{total} (kN)
A	-0.2632	3.8	3.5502	1	-0.7851	5.7	4.9351

B	-0.1579	3.8	3.6555	2	-0.5888	5.7	5.1313
C	-0.0526	3.8	3.7608	3	-0.3925	5.7	5.3276
D	0.0526	3.8	3.8661	4	-0.1963	5.7	5.5239
E	0.1579	3.8	3.9713	5	0.0000	5.7	5.7201
F	0.2632	3.8	4.0766	6	0.1963	5.7	5.9164
				7	0.3925	5.7	6.1127
				8	0.5888	5.7	6.3089
				9	0.7851	5.7	6.5052

2.3.2 Structural Analysis

2.3.2.1 Analytical models development in software

Model properties

Material properties were entered as shown in figures below.

The screenshot shows the 'Material Property Data' dialog box for a material named 'Concrete1'. The dialog is organized into several sections:

- General Data:** Material Name and Display Color (Concrete1), Material Type (Concrete), and Material Notes (Modify/Show Notes...).
- Weight and Mass:** Weight per Unit Volume (23.5631), Mass per Unit Volume (2.4028), and Units (KN, m, C).
- Isotropic Property Data:** Modulus of Elasticity, E (34000000), Poisson (0.2), Coefficient of Thermal Expansion, A (9.900E-06), and Shear Modulus, G (14166667).
- Other Properties for Concrete Materials:** Specified Concrete Compressive Strength, f_c (35000), and a checkbox for Lightweight Concrete (unchecked). Below the checkbox is a field for Shear Strength Reduction Factor.
- Switch To Advanced Property Display:** A checkbox at the bottom left.

At the bottom of the dialog are 'OK' and 'Cancel' buttons.

Figure 2.3.18. Concrete 35 MPa material properties

Material Property Data

General Data

Material Name and Display Color: Concrete2 [Color: Blue]

Material Type: Concrete [Dropdown]

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

Weight and Mass

Weight per Unit Volume: 23.5631 [Text Box]

Mass per Unit Volume: 2.4028 [Text Box]

Units: KN, m, C [Dropdown]

Isotropic Property Data

Modulus of Elasticity, E: 25000000 [Text Box]

Poisson: 0.2 [Text Box]

Coefficient of Thermal Expansion, A: 9.900E-06 [Text Box]

Shear Modulus, G: 10416667 [Text Box]

Other Properties for Concrete Materials

Specified Concrete Compressive Strength, f_c: 28000 [Text Box]

☐ Lightweight Concrete

Shear Strength Reduction Factor: [Text Box]

☐ Switch To Advanced Property Display

[OK] [Cancel]

Figure 2.3.19. Concrete 28 MPa material properties

Frame properties were entered as follows:

Rectangular Section

Section Name: Column1-3 [Text Box] Display Color: [Color: Green]

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

Dimensions

Depth (t₃): 0.6 [Text Box]

Width (t₂): 0.6 [Text Box]

Section

[Diagram: Rectangular section with dimensions and reinforcement layout]

Material

[+] Concrete1 [Dropdown]

Property Modifiers

[Set Modifiers...]

Properties

[Section Properties...]

[Time Dependent Properties...]

[Concrete Reinforcement...]

[OK] [Cancel]

Figure 2.3.20. Bottom 3 column properties

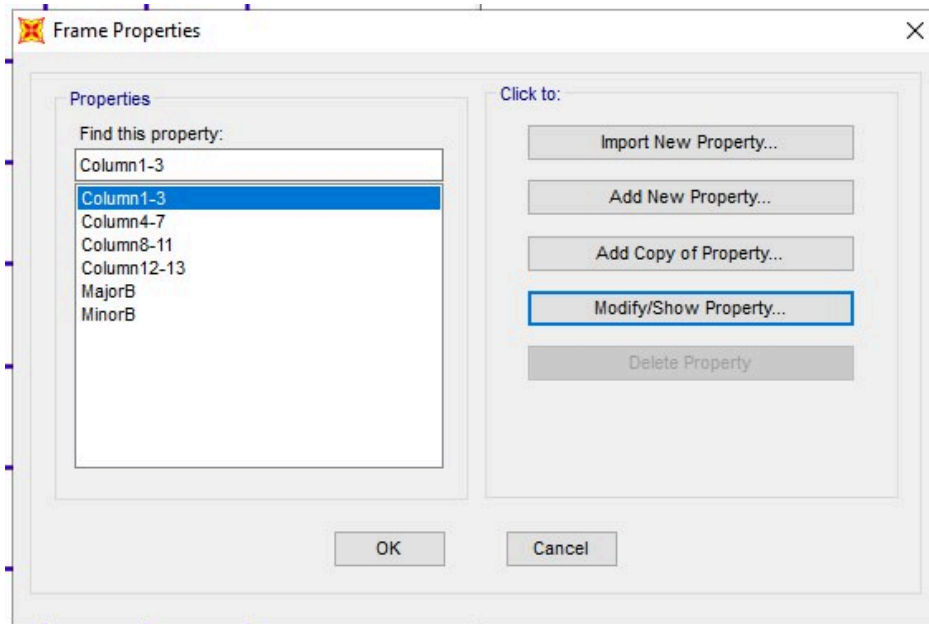


Figure 2.3.21. Frame properties list

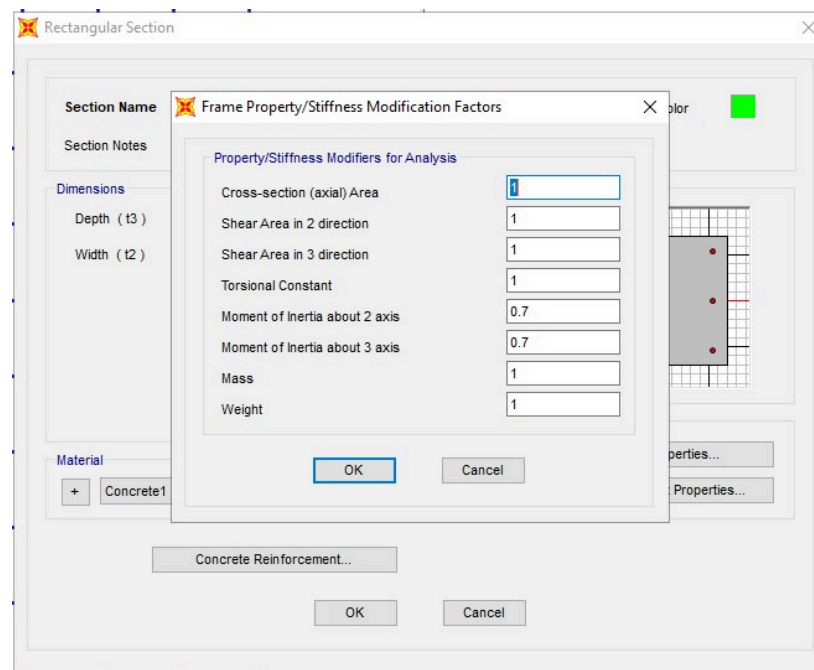


Figure 2.3.22. Moment of inertia reduction factor for column

Gravity loads

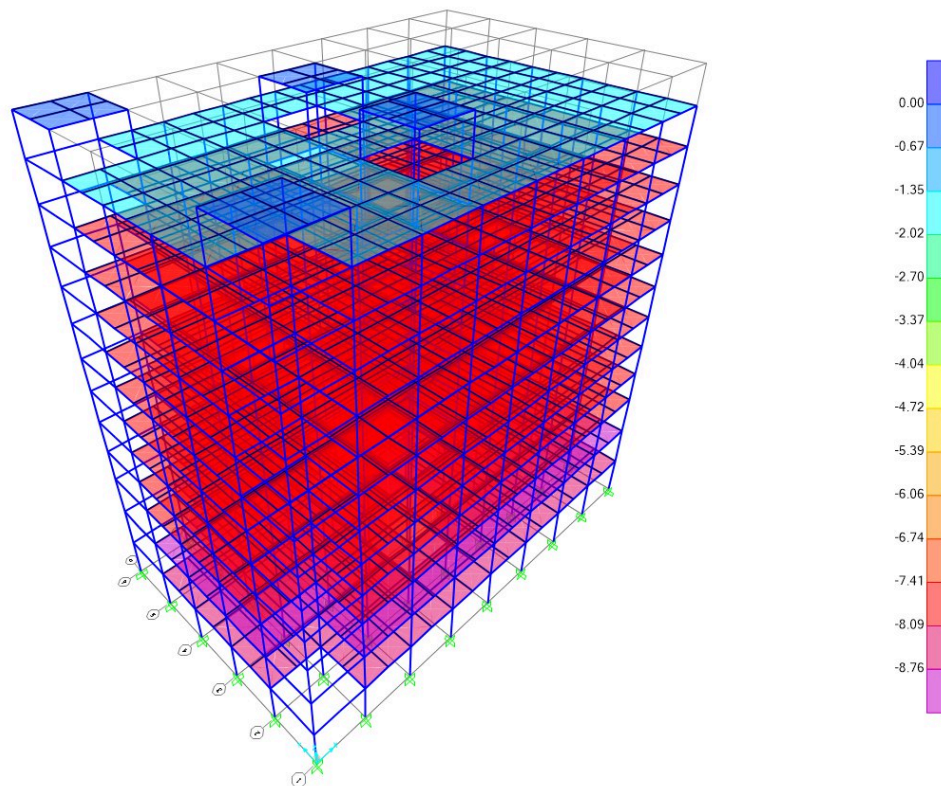


Figure 2.3.23. Applied dead area loads

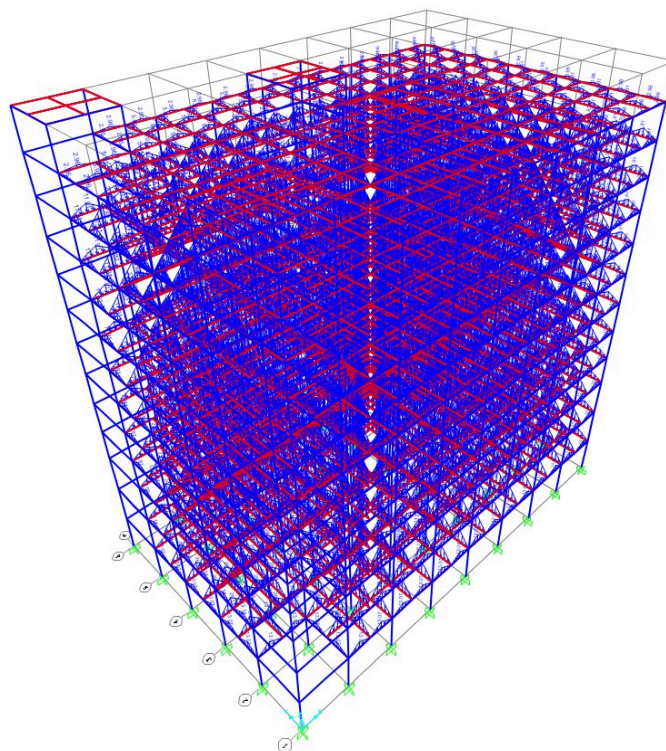


Figure 2.3.24. Applied dead frame loads

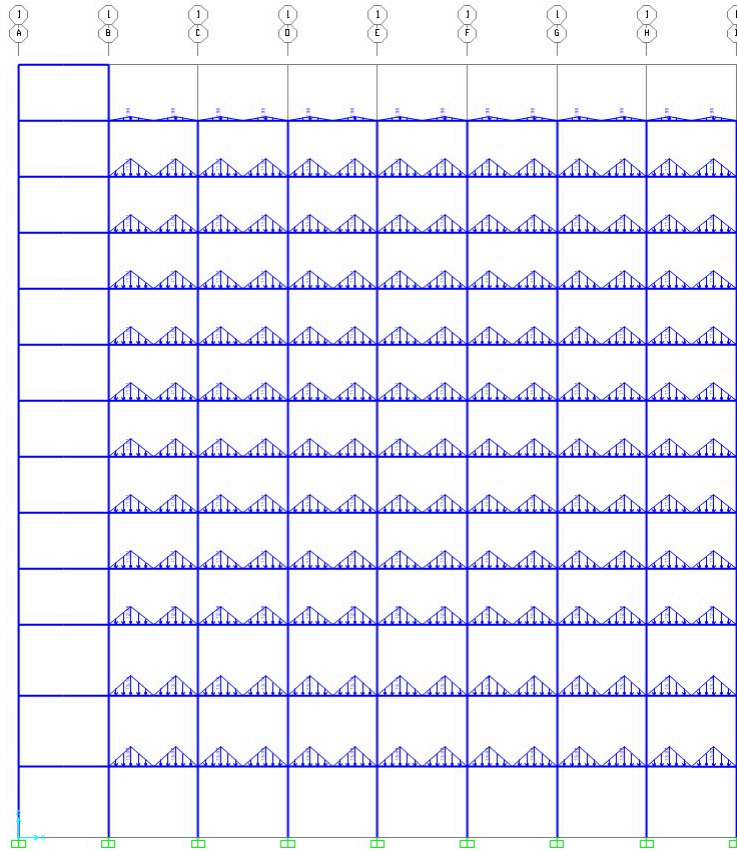


Figure 2.3.25. Applied dead loads in one frame

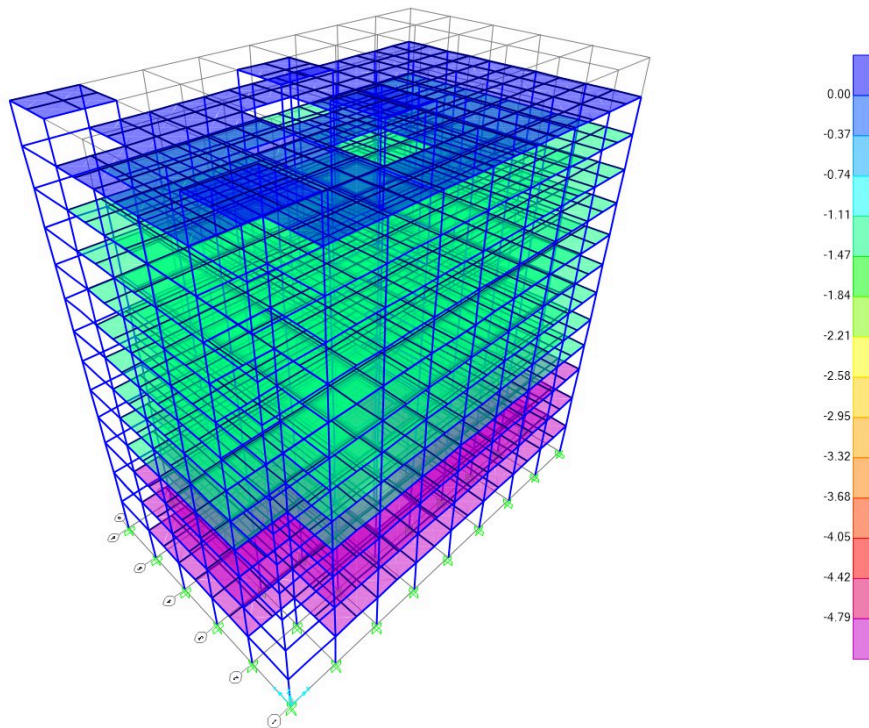


Figure 2.3.26. Applied live area loads

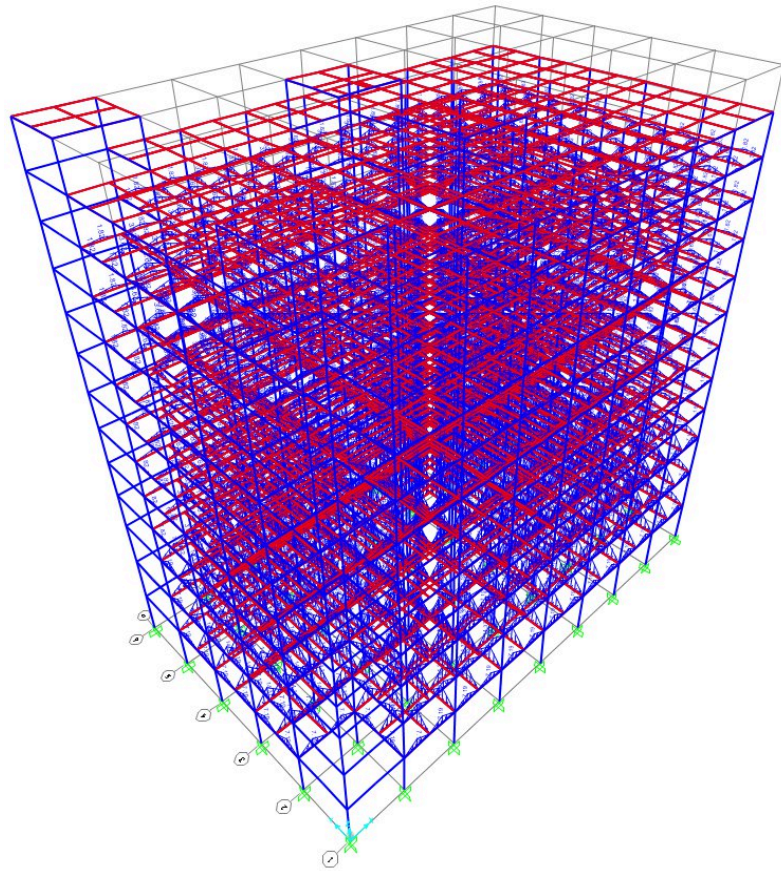


Figure 2.3.27. Applied live frame loads

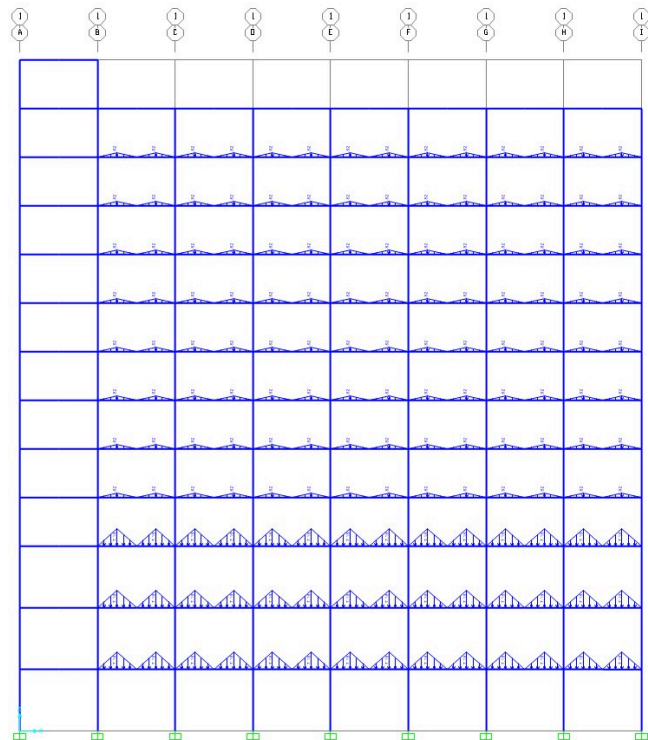


Figure 2.3.28. Applied live loads in one frame

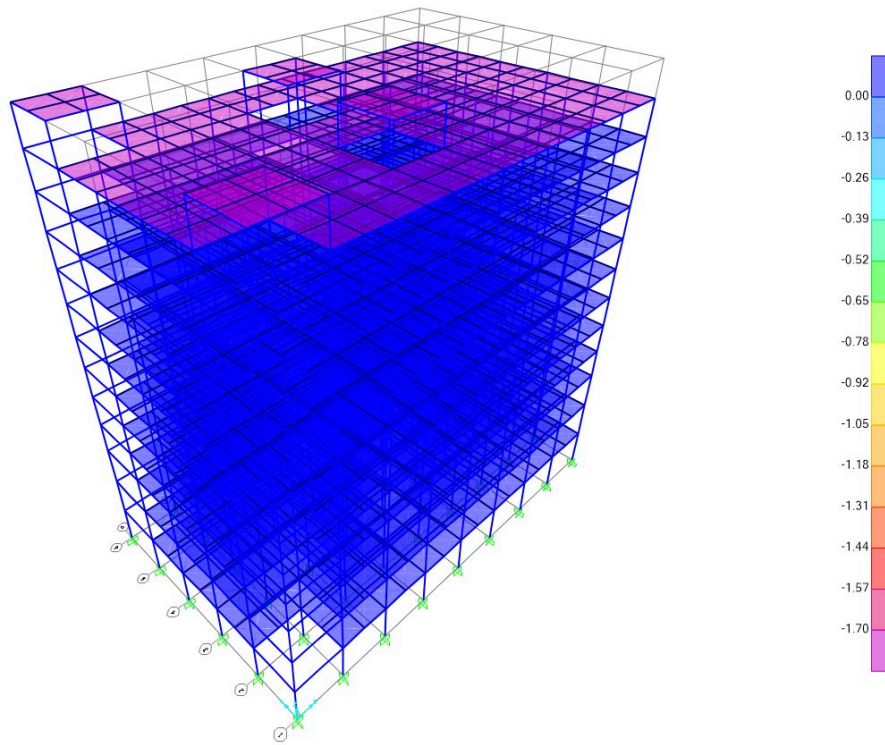


Figure 2.3.29. Applied snow area loads

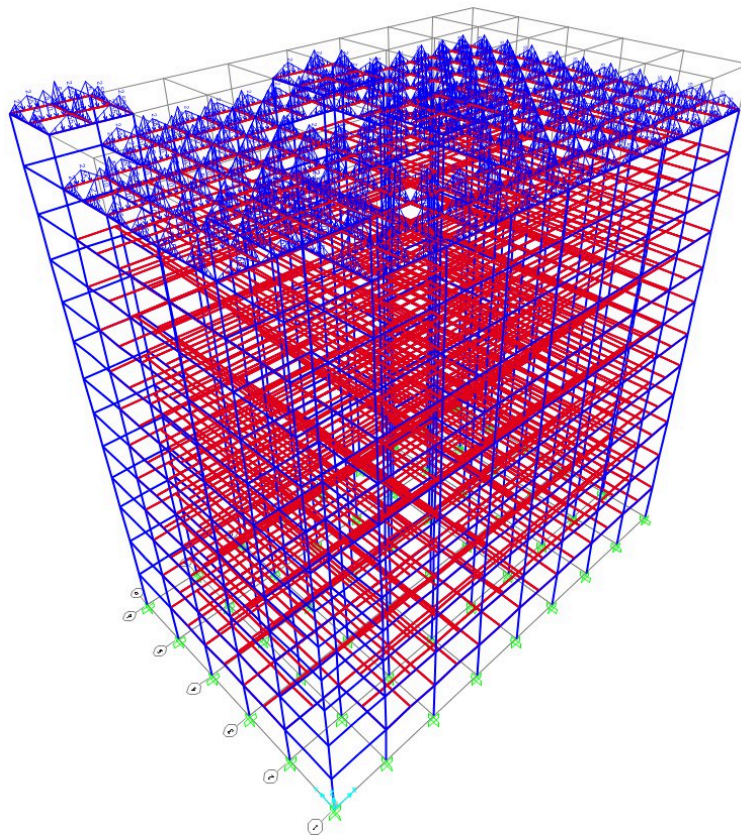


Figure 2.3.30. Applied snow frame loads

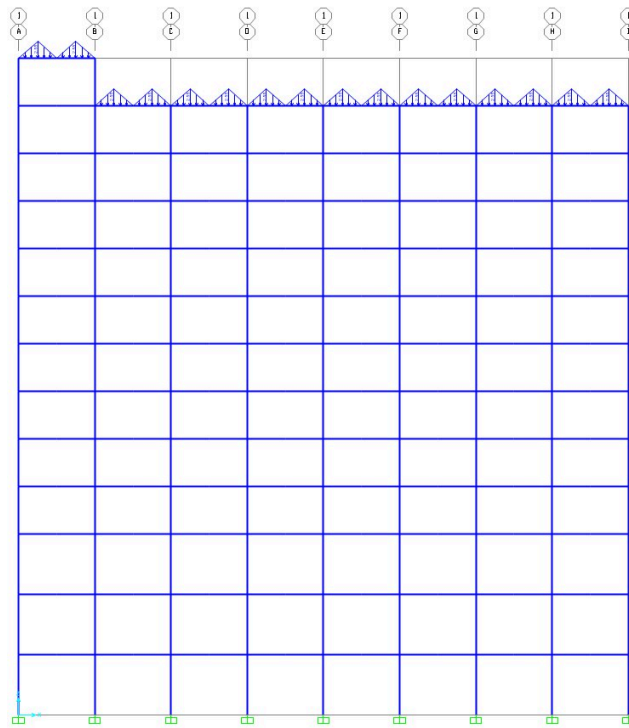


Figure 2.3.31. Applied snow loads in one frame

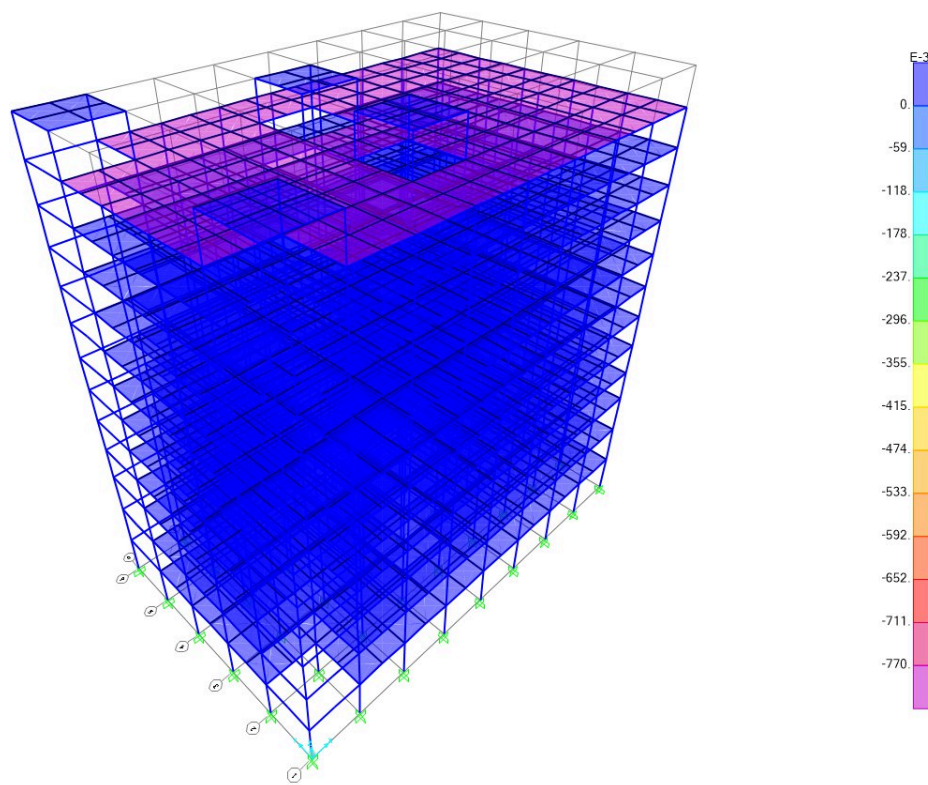


Figure 2.3.32. Applied roof live area loads

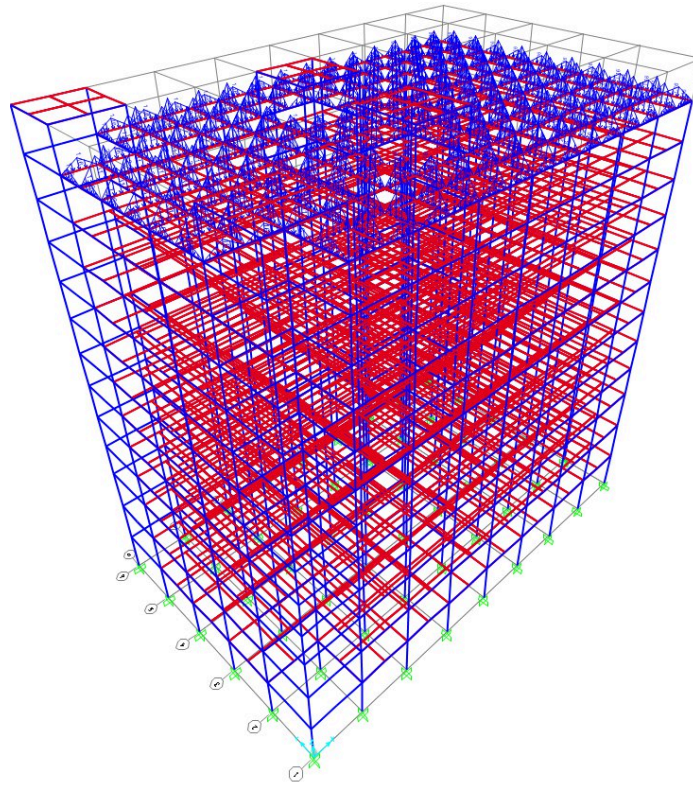


Figure 2.3.33. Applied roof live frame loads

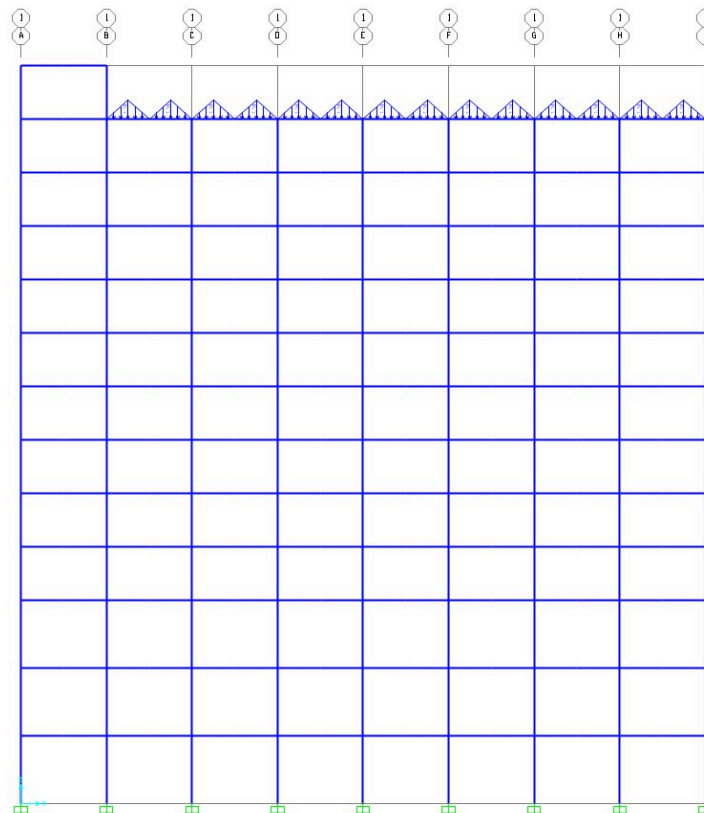


Figure 2.3.34. Applied roof live loads in one frame

Lateral loads

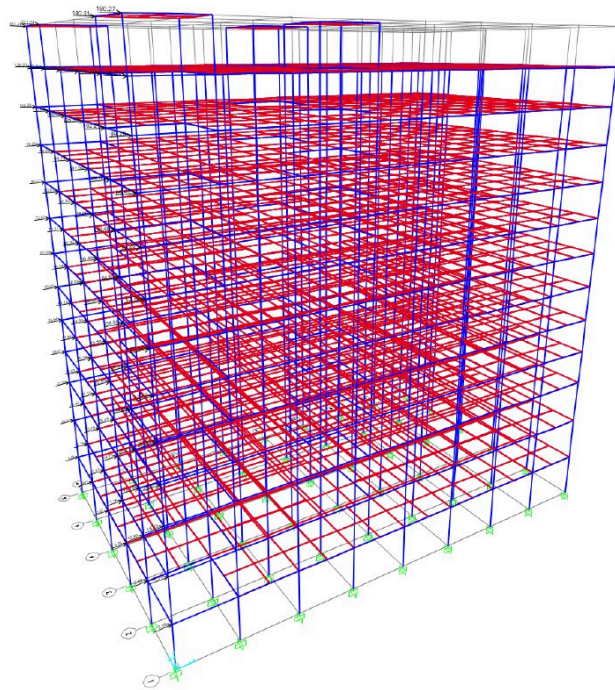


Figure 2.3.35. Applied seismic Transverse loads

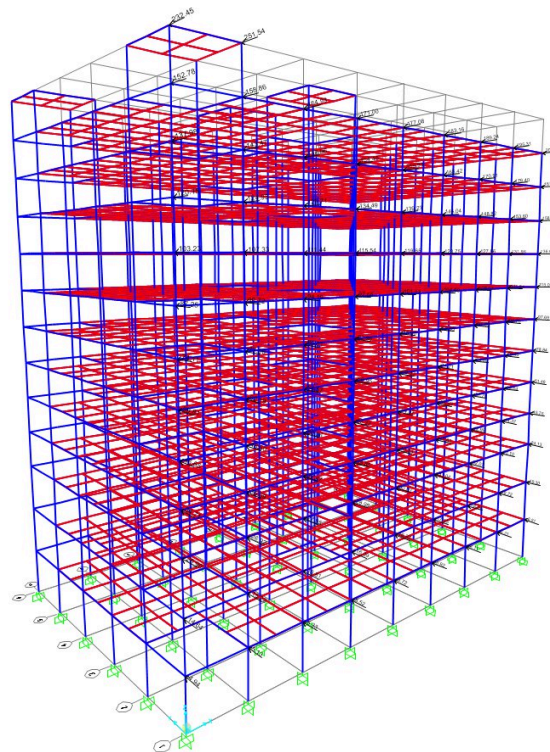


Figure 2.3.36. Applied seismic Longitudinal loads

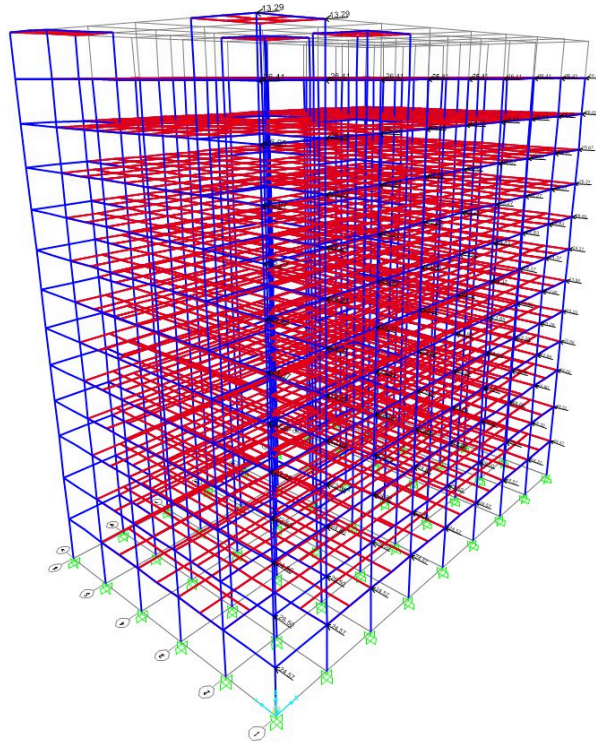


Figure 2.3.37. Applied Wind loads (Case 1 Longitudinal: Most critical)

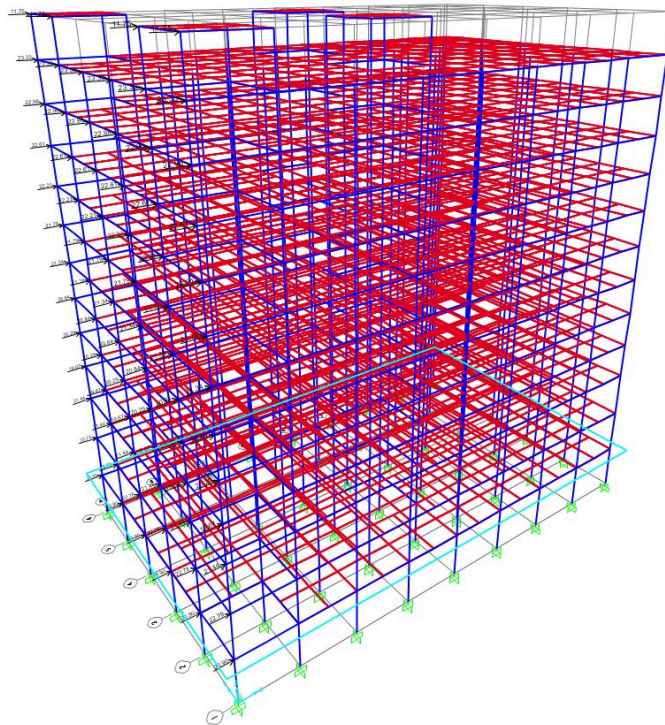


Figure 2.3.38. Applied Wind loads (Case 1 Transverse: Most critical)

Other Wind load cases are shown in Appendix C.

Load combinations used for structural analysis:

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Figure 2.3.39. Load combinations

2.3.2.2 Lateral Drift Analysis

Table 2.3.27. Shear drift under wind load calculations (longitudinal)

Floor	Fi (kN)	Vi col (kN)	Vi avg (kN)	Δt (mm)	Δi (mm)	Δ (mm)
13	11.748	2.3	2.3	0.121	0.466	45.485
12	23.326	7.0	4.7	0.691	0.902	45.018
11	22.978	11.6	9.3	1.113	1.317	44.116
10	22.608	16.1	13.9	1.521	1.767	42.799
9	22.215	20.6	18.4	2.013	2.254	41.032
8	21.793	24.9	22.8	2.496	2.688	38.777
7	21.337	29.2	27.1	2.880	3.067	36.089
6	20.839	33.4	31.3	3.255	3.469	33.022
5	20.290	37.4	35.4	3.683	3.890	29.553
4	19.674	41.4	39.4	4.098	5.562	25.663
3	21.444	45.7	43.5	7.025	7.479	20.101
2	22.779	50.2	47.9	7.933	8.294	12.622
1	20.898	54.4	52.3	8.656	4.328	4.328

Table 2.3.28. Shear drift under wind load calculations (transverse)

Floor	Fi (kN)	Vi col (kN)	Vi avg (kN)	Δt (mm)	Δi (mm)	Δ (mm)
13	13.291	1.7	1.7	0.086	0.330	32.618
12	26.408	5.0	3.3	0.489	0.638	32.288
11	26.050	8.2	6.6	0.788	0.981	31.649
10	25.671	11.4	9.8	1.174	1.365	30.668
9	25.267	14.6	13.0	1.555	1.663	29.304
8	24.833	17.7	16.1	1.770	1.907	27.641
7	24.365	20.7	19.2	2.044	2.178	25.734
6	23.854	23.7	22.2	2.312	2.466	23.555

5	23.290	26.6	25.2	2.619	2.768	21.090
4	22.657	29.5	28.0	2.918	3.963	18.322
3	24.803	32.6	31.0	5.008	5.336	14.359
2	26.500	35.9	34.2	5.663	5.928	9.024
1	24.569	38.9	37.4	6.192	3.096	3.096

Table 2.3.29. Flexural drift under wind load calculations (longitudinal)

Floor	Fi (kN)	Vi (kN)	Mi (kN-mm)	ϕ (1/mm)	θ_i (rad)	Δ_i (mm)	Δ (mm)
13	11.748	11.7	44053.81	1.1E-11	2.7E-05	0.101	1.057
12	23.326	35.1	175580.19	4.6E-11	2.7E-05	0.101	0.956
11	22.978	58.1	393272.49	8.6E-11	2.7E-05	0.100	0.855
10	22.608	80.7	695745.63	1.5E-10	2.6E-05	0.099	0.755
9	22.215	102.9	1081523.63	2.4E-10	2.6E-05	0.097	0.656
8	21.793	124.7	1549023.85	3.4E-10	2.5E-05	0.093	0.559
7	21.337	146.0	2096536.31	3.9E-10	2.4E-05	0.089	0.466
6	20.839	166.8	2722195.79	5.1E-10	2.2E-05	0.083	0.377
5	20.290	187.1	3423942.79	6.4E-10	2.0E-05	0.076	0.294
4	19.674	206.8	4199466.15	7.8E-10	1.8E-05	0.067	0.218
3	21.444	228.3	5283655.94	8.5E-10	1.5E-05	0.071	0.151
2	22.779	251.0	6476044.20	1.0E-09	1.1E-05	0.052	0.080
1	20.898	271.9	7767700.20	1.3E-09	5.9E-06	0.028	0.028

Table 2.3.30. Flexural drift under wind load calculations (transverse)

Floor	Fi (kN)	Vi (kN)	Mi (kN-mm)	ϕ (1/mm)	θ_i (rad)	Δ_i (mm)	Δ (mm)
13	13.29	13.3	49841.12	5.1E-12	1.2E-05	0.045	0.470
12	26.41	39.7	198712.33	2.0E-11	1.2E-05	0.045	0.425
11	26.05	65.7	445271.79	3.8E-11	1.2E-05	0.044	0.380
10	25.67	91.4	788097.10	6.7E-11	1.2E-05	0.044	0.336
9	25.27	116.7	1225672.53	1.0E-10	1.1E-05	0.043	0.292
8	24.83	141.5	1756372.82	1.5E-10	1.1E-05	0.042	0.249
7	24.37	165.9	2378441.92	1.7E-10	1.1E-05	0.039	0.207
6	23.85	189.7	3089964.37	2.3E-10	9.9E-06	0.037	0.168
5	23.29	213.0	3888825.22	2.8E-10	9.0E-06	0.034	0.131
4	22.66	235.7	4772651.02	3.5E-10	7.9E-06	0.030	0.097
3	24.80	260.5	6009976.70	3.8E-10	6.6E-06	0.032	0.067
2	26.50	287.0	7373178.38	4.6E-10	4.8E-06	0.023	0.036
1	24.57	311.6	8853084.79	5.6E-10	2.6E-06	0.013	0.013

Table 2.3.31. Shear drift under seismic load calculations (longitudinal)

Floor	Fi (kN)	Vi col (kN)	Vi avg (kN)	Δt (mm)	Δi (mm)	Δ (mm)
13	206.04	41.2	41.2	2.122	6.038	206.051
12	118.92	65.0	53.1	7.833	8.438	200.013
11	106.54	86.3	75.6	9.044	9.764	191.575
10	92.78	104.9	95.6	10.484	11.430	181.811
9	79.71	120.8	112.8	12.376	13.182	170.381
8	67.37	134.3	127.5	13.989	14.443	157.199
7	57.23	145.7	140.0	14.897	15.268	142.756
6	46.18	155.0	150.3	15.640	16.067	127.487
5	36.02	162.2	158.6	16.495	16.822	111.420
4	26.80	167.5	164.8	17.149	22.885	94.598
3	19.99	171.5	169.5	28.622	28.599	71.713
2	11.31	173.8	172.6	28.576	28.701	43.114
1	3.81	174.5	174.2	28.826	14.413	14.413

Table 2.3.32. Shear drift under seismic load calculations (transverse)

Floor	Fi (kN)	Vi col (kN)	Vi avg (kN)	Δt (mm)	Δi (mm)	Δ (mm)
13	309.06	38.6	38.6	1.989	5.661	193.930
12	178.38	60.9	49.8	7.343	7.911	188.269
11	159.82	80.9	70.9	8.479	9.596	180.358
10	139.16	98.3	89.6	10.713	11.679	170.763
9	119.56	113.2	105.8	12.646	12.880	159.083
8	101.05	125.9	119.6	13.115	13.540	146.203
7	85.84	136.6	131.2	13.966	14.314	132.663
6	69.28	145.3	140.9	14.662	15.063	118.348
5	54.03	152.0	148.6	15.464	15.770	103.285
4	40.20	157.0	154.5	16.077	20.870	87.515
3	29.99	160.8	158.9	25.662	26.226	66.645
2	16.96	162.9	161.9	26.790	26.907	40.419
1	5.72	163.6	163.3	27.024	13.512	13.512

Table 2.3.33. Flexural drift under seismic load calculations (longitudinal)

Floor	Fi (kN)	Vi (kN)	Mi (kN-mm)	ϕ (1/mm)	θ_i (rad)	Δi (mm)	Δ (mm)
13	206.04	206.0	772654.961	2.0E-10	1.4E-04	0.541	5.348
12	118.92	325.0	1991257.74	5.2E-10	1.4E-04	0.538	4.807
11	106.54	431.5	3609403.19	7.9E-10	1.4E-04	0.530	4.269
10	92.78	524.3	5575459.87	1.2E-09	1.4E-04	0.519	3.739
9	79.71	604.0	7840419.68	1.7E-09	1.3E-04	0.502	3.220

8	67.37	671.4	10358012.1	2.3E-09	1.3E-04	0.478	2.717
7	57.23	728.6	13090209.3	2.4E-09	1.2E-04	0.446	2.239
6	46.18	774.8	15995599.9	3.0E-09	1.1E-04	0.412	1.793
5	36.02	810.8	19036068.7	3.6E-09	9.9E-05	0.370	1.382
4	26.80	837.6	22177036.3	4.1E-09	8.5E-05	0.320	1.012
3	19.99	857.6	26250571.3	4.2E-09	7.0E-05	0.331	0.692
2	11.31	868.9	30377809.8	4.9E-09	5.0E-05	0.236	0.361
1	3.81	872.7	34523148.9	5.6E-09	2.6E-05	0.125	0.125

Table 2.3.34. Flexural drift under seismic load calculations (transverse)

Floor	Fi (kN)	Vi (kN)	Mi (kN-mm)	ϕ (1/mm)	θ_i (rad)	Δ_i (mm)	Δ (mm)
13	309.06	309.1	1158982.44	1.2E-10	8.4E-05	0.317	3.133
12	178.38	487.4	2986886.60	3.0E-10	8.4E-05	0.315	2.817
11	159.82	647.3	5414104.79	4.6E-10	8.3E-05	0.311	2.502
10	139.16	786.4	8363189.81	7.2E-10	8.1E-05	0.304	2.191
9	119.56	906.0	11760629.5	1.0E-09	7.8E-05	0.294	1.886
8	101.05	1007.0	15537018.1	1.3E-09	7.5E-05	0.280	1.592
7	85.84	1092.9	19635314.0	1.4E-09	7.0E-05	0.261	1.312
6	69.28	1162.2	23993399.8	1.8E-09	6.4E-05	0.241	1.051
5	54.03	1216.2	28554103.0	2.1E-09	5.8E-05	0.217	0.810
4	40.20	1256.4	33265554.4	2.4E-09	5.0E-05	0.187	0.593
3	29.99	1286.4	39375857.0	2.5E-09	4.1E-05	0.194	0.406
2	16.96	1303.3	45566714.7	2.9E-09	2.9E-05	0.138	0.212
1	5.72	1309.1	51784723.3	3.3E-09	1.5E-05	0.073	0.073

The total inter-story drift was calculated and the values from hand calculations, SAP 2D and SAP 3D models were compared as shown in figures below. For drift under seismic load, drift was amplified by s_d factor of 5.5. All values are within limits and are acceptable.

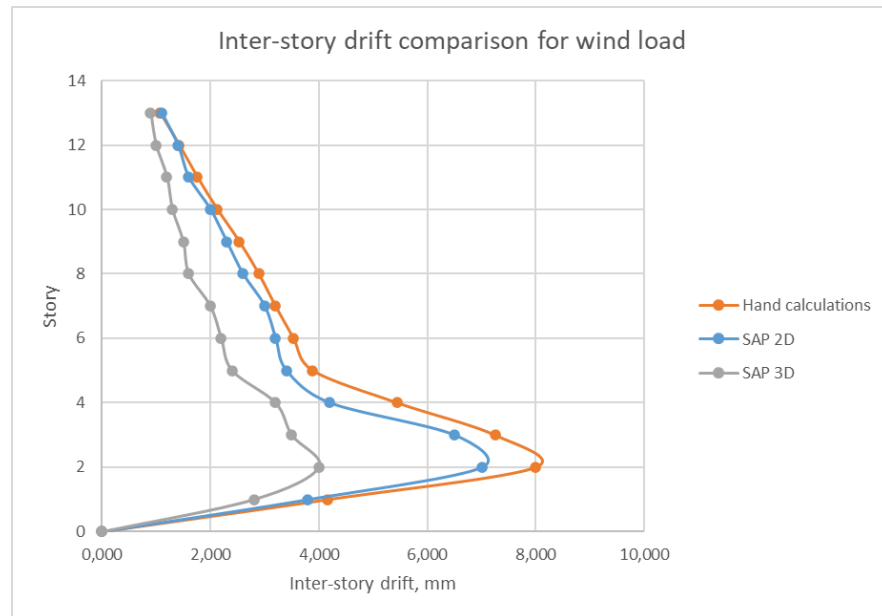


Figure 2.3.40 Inter-story drift comparison for wind load

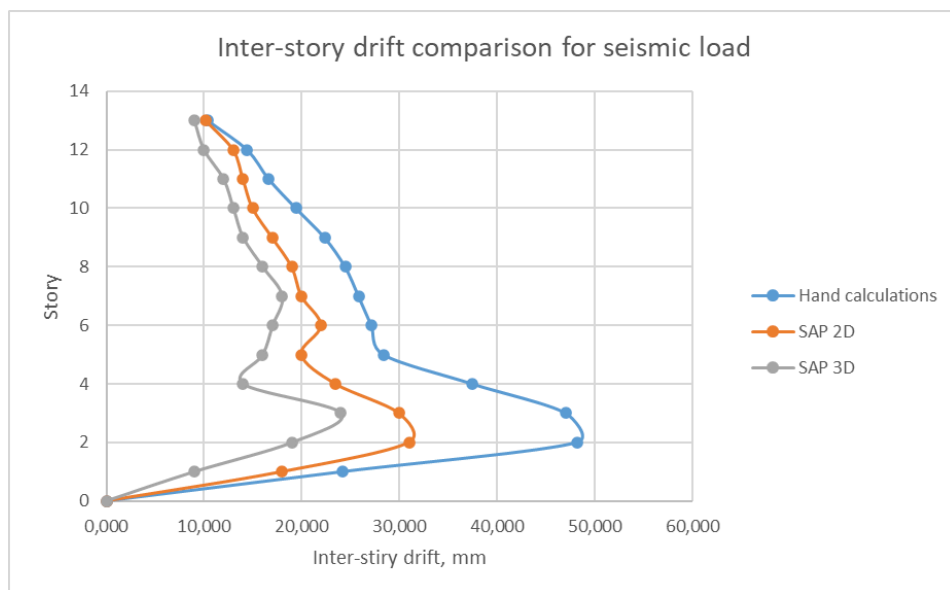


Figure 2.3.41 Inter-story drift comparison for seismic load

As it is evident from the graphs, hand calculations yield the most conservative results, while software results present more economical and precise results, since interaction between members is considered.

2.3.2.3 Internal force analysis

First step before calculating the load distribution on frames is to find the self weight of the minor and major beams:

$$SW_{major\ beam} = 0.48m * 0.2m * 6m * 25kN/m^3 = 14.4kN$$

$$SW_{minor\ beam} = 0.38m * 0.15m * 6m * 25kN/m^3 = 8.55kN$$

The building has different floor loads for the basement, 1, 2, 3, 4, 11, 12, 13 floors. Therefore there will be 6 different floor load estimations.

Basement:

$$D_{basement} = 6.523\ kN/m^2$$

$$L_{basement} = 1.92\ kN/m^2$$

Maximum distributed loads: w_{max}

$$w_{max_D\ (for\ dead\ load)} = 6.523 * 1.5 * 2 = 19.57\ kN/m$$

$$w_{max_L\ (for\ live\ load)} = 1.92 * 1.5 * 2 = 5.76\ kN/m$$

Reaction force from minor beam: R_{minor}

$$R_{minor_D\ (for\ dead\ load)} = 0.5 * 19.57 * 0.5 * 3 * 2 + \frac{SW_{minor\ beam}}{2} = 33.6\ kN$$

$$R_{minor_L\ (for\ live\ load)} = 0.5 * 5.76 * 0.5 * 3 * 2 = 8.64\ kN$$

Point loads from minor beams to major beams: P_{major}

$$P_{major_D\ (for\ dead\ load)} = 33.6 * 2 = 67.3\ kN$$

$$P_{major_L\ (for\ live\ load)} = 8.64 * 2 = 17.28\ kN$$

Reaction force from major beam to column: R_{major}

$$R_{major_D} = 0.5 * 19.57 * 0.5 * 3 * 2 + \frac{SW_{minor\ beam}}{2} = 36.55\ kN$$

Point load from major beam to column: P_{major}

$$P_{major_D} = 0.5 * 2 * 0.5 * 2 * 3 * 19.57 + 36.55 * 2 = 131.8\ kN$$

Point live load exerted on exterior column:

Reduction for exterior column = 0.789

Reduction for interior column = 0.631

$$P_{L_{ex}} = 0.5 * 2 * 0.5 * 2 * 3 * 5.76 + 0.5 * 2 * 17.28 * 0.789 = 27.3 \text{ kN}$$

$$P_{L_{in}} = 0.5 * 2 * 0.5 * 2 * 3 * 5.76 + 0.5 * 2 * 17.28 * 0.631 = 21.8 \text{ kN}$$

Same calculations were completed for 1st, 2nd, 3rd, 4-11, 12, 13 floors. In the following table we have the obtained results:

Table 2.3.35. Force calculations

	Dead load	Live load	Wma x_D	Wma x_L	Rmin or_D	Rmin or_L	Pmaj or_D	Pmaj or_L	Rmaj or_D	Pmaj or_D	P_L_ ex	P_L_ int
Basement	6.523	1.92	19.6	5.8	33.6	8.6	67.3	17.3	36.6	131.8	27.3	21.8
1 floor	6.639	4.79	19.9	14.4	34.2	21.6	68.3	43.1	37.1	133.9	68.0	54.4
2 floor	6.690	4.79	20.1	14.4	34.4	21.6	68.8	43.1	37.3	134.8	68.0	54.4
3 floor	6.496	4.79	19.5	14.4	33.5	21.6	67.0	43.1	36.4	131.3	68.0	54.4
4-11 floors	6.387	1.211	19.2	3.6	33.0	5.5	66.0	10.9	35.9	129.4	17.2	13.8
12 floor	6.415	0.77	19.3	2.3	33.1	3.5	66.3	6.9	36.1	129.9	10.9	8.7
13 floor	6.415	0.77	19.3	2.3	33.1	3.5	66.3	6.9	36.1	129.9	10.9	8.7

Now, we can find the distributed and point loads assigned to beams and point loads affecting the interior and exterior columns.

Basement:

$$w_u = 1.2 * w_{max_D} + 1.6 * w_{max_L} = 1.2 * 19.57 + 1.6 * 5.76 = 32.7 \text{ kN/m}$$

$$P_{major\ beam} = 1.2 * P_{major_D} + 1.6 * P_{major_L} = 1.2 * 67.3 + 1.6 * 17.28 = 108.4 \text{ kN}$$

$$P_{ex} = 1.2 * P_{major_D} + 1.6 * P_{L_{ex}} = 1.2 * 131.8 + 1.6 * 27.3 = 201.8 \text{ kN}$$

$$P_{in} = 1.2 * P_{major_D} + 1.6 * P_{L_{in}} = 1.2 * 131.8 + 1.6 * 21.8 = 193.1 \text{ kN}$$

After repeating the same calculations for each floor we obtain the following results:

Table 2.3.36. Load distribution on frame members

	Wu	Pmajor_beam	Pex_col	Pint_col
Basement	32.6988	108.3564	201.805344	193.068576
Beams under 1 floor	46.89276	150.93828	269.528688	247.732272
Beams under 2 floor	47.07672	151.49016	270.632448	248.836032
Beams under 3	46.37724	149.39172	266.435568	244.639152

floor				
Beams under 4-11 floor	28.80516	96.67548	182.7507398	177.2392954
Beams under 12 floor	24.249	83.007	161.31177	160.21683
Beams under 13 floor	24.249	83.007	161.31177	160.21683

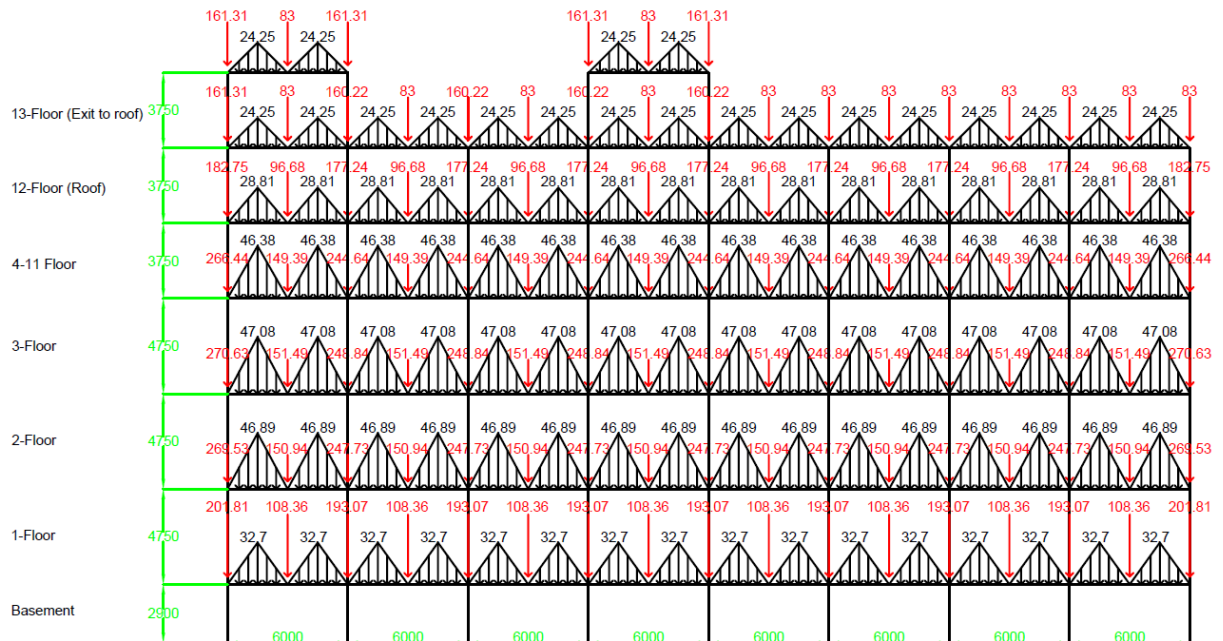


Figure 2.3.42. Load distribution on frame members

Hand calculations of reaction forces

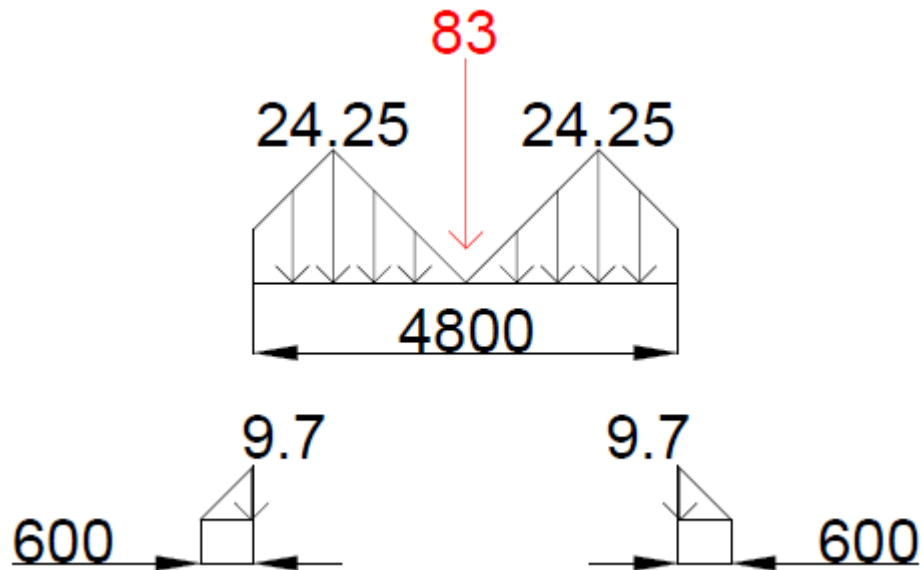


Figure 2.3.43. Load distribution for moment calculation (13 floor)

To calculate the moment on a major beam we have to divide it into separate pieces:
0.1L and 0.8L.

$$0.1L = 6 \times 0.1 = 0.6\text{m and } 0.8L = 6 \times 0.8 = 4.8\text{m}$$

Distributed load at 0.1L can be calculated as follows:

$$24.25 \times \frac{0.6}{1.5} = 9.7 \text{ kN/m}$$

To calculate the moment at mid first of all we have to find the area of each shape of distributed load:

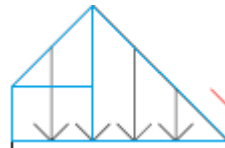


Figure 2.3.44. Separate pieces of distributed load

$$\text{Area of smaller triangle: } (24.25 - 9.7) \times (1.5 - 0.6)/2 = 6.55 \text{ kN}$$

$$\text{Area of rectangle: } 9.7 \times (1.5 - 0.6) = 8.73 \text{ kN}$$

$$\text{Area of triangle: } 24.25 \times 1.5/2 = 18.19 \text{ kN}$$

$$M_{mid} = 6.55 \times (1.5 - 0.6) \times 2/3 + 8.73 \times (1.5 - 0.6)/2 + 18.19 \times \\ \times (1.5 - 0.6 + 1.5 \times 1/3) + 83 \times (1.5 - 0.6 + 1.5) = 232.5 \text{ kN} \cdot \text{m}$$

To calculate the moment at 0 and L we have to find the reaction forces at both ends of 0.8L.

$$R = ((6.55 + 8.73 + 18.19) \times 2 + 83)/2 = 74.97 \text{ kN}$$

$$M = 9.7 * 0.6/2 * 0.6 * 2/3 + 74.97 * 0.6 = 46.14 \text{ kN} * m$$

To find the shear we complete the following calculations:

$$V_{\text{exterior}} = 9.7 * 0.6/2 + 74.97 + 161.31 = 239.19 \text{ kN}$$

$$V_{\text{interior}} = 9.7 * 0.6/2 + 74.97 + 160.22 = 238.09 \text{ kN}$$

$$V_{\text{in column}} = (46.14 + 46.14/2)/\text{floor height (3.75m)} = 18.46 \text{ kN}$$

In the same way we have calculated moment and shear for all other floors. The results are in the following table:

Table 2.3.37 Moment results

	w at 0.1L	Area of small triangle	Area of rectangle	Area of triangle	R at 0.8L	M	M_mid
Basement	13.1	8.8	11.8	24.5	99.3	61.2	305
Beams under 1 floor	18.8	12.7	16.9	35.2	140.2	86.4	426.7
Beams under 2 floor	18.8	12.7	16.9	35.3	140.7	86.7	428.3
Beams under 3 floor	18.6	12.5	16.7	34.8	138.7	85.4	422.3
Beams under 4-11 floor	11.5	7.8	10.4	21.6	88.1	54.2	271.6
Beams under 12 floor	9.7	6.5	8.7	18.2	75.0	46.1	232.5
Beams under 13 floor	9.7	6.5	8.7	18.2	75.0	46.1	232.5

Table 2.3.38 Shear results

	Vex	Vin	V in column
Basement	305.0	296.3	31.6
Beams under 1 floor	415.3	393.5	27.3
Beams under 2 floor	417.0	395.2	27.4
Beams under 3 floor	410.7	388.9	27.0
Beams under 4-11 floor	274.3	268.8	21.7
Beams under 12 floor	239.2	238.1	18.5
Beams under 13 floor	239.2	238.1	18.5

Table 2.3.39 Axial forces

Location	Axial load (kN)
Slab 13	134.532
Slab 12	499.86
Slab 11	903.12192
Slab 10	1306.38384
Slab 9	1709.64576
Slab 8	2112.90768
Slab 7	2516.1696
Slab 6	2919.43152
Slab 5	3322.69344
Slab 4	3725.95536
Slab 3	4129.21728
Slab 2	4743.34416
Slab 1	5365.8648
Slab 0	5986.17792
Slab -1	6436.16352

2.3.2.4 Hand calculation verification for internal forces

To find the internal forces for Wind load we have used a portal method to calculate all of the values (The method can be found in Appendix D). For the calculation we have used the short side with 5 frames. The results of the calculation are shown in the following table:

Table 2.3.40 Hand calculation of wind load

Story	Axial Force, kN	Shear Force, kN	Moment, kNm
12	1.1	2.2	3.3
11	4.39	4.37	6.56
10	9.83	6.51	9.77
9	17.39	8.62	12.93
8	27.04	10.69	16.03
7	38.74	12.72	19.08
6	52.45	14.7	22.06
5	68.13	16.65	24.97
4	85.72	18.53	27.8
3	105.29	20.6	30.9

2	126.99	22.81	34.21
1	154.79	24.86	43.5

Table 2.3.41 Hand calculation of Dead load

Story	Axial Force, kN	Shear Force, kN	Moment, kNm
12	75	54	111
11	406	54	111
10	740	54	111
9	1076	54	111
8	1299	54	111
7	1745	54	111
6	2050	54	111
5	2400	54	111
4	2780	54	111
3	3210	54	111
2	3740	54	111
1	4215	54	111

Internal forces comparison of the values from 2D and 3D models with hand calculation:

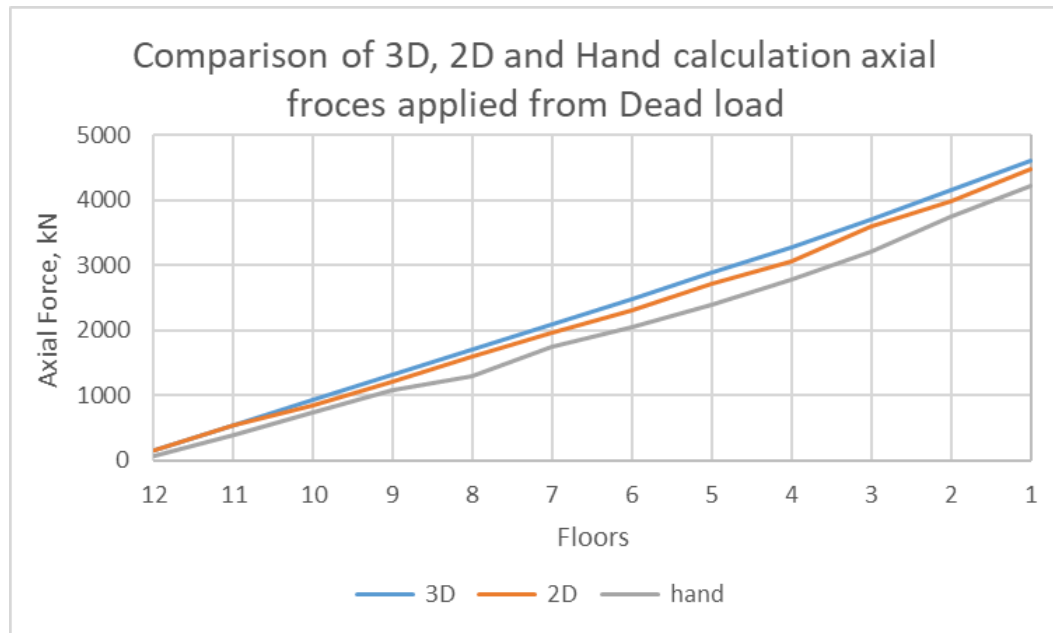


Figure 2.3.45. Axial forces from dead load

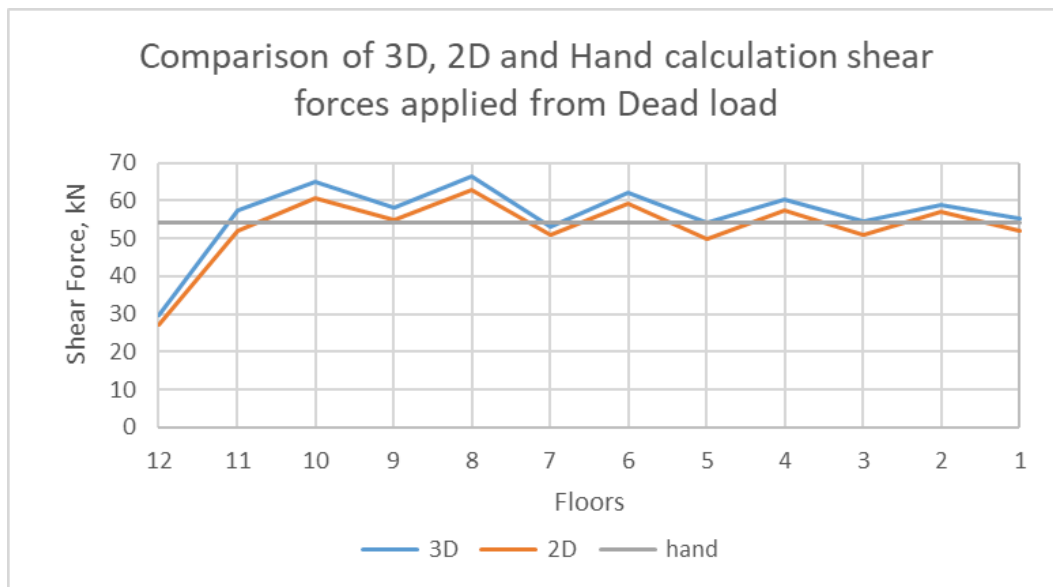


Figure 2.3.46. Shear forces from dead load

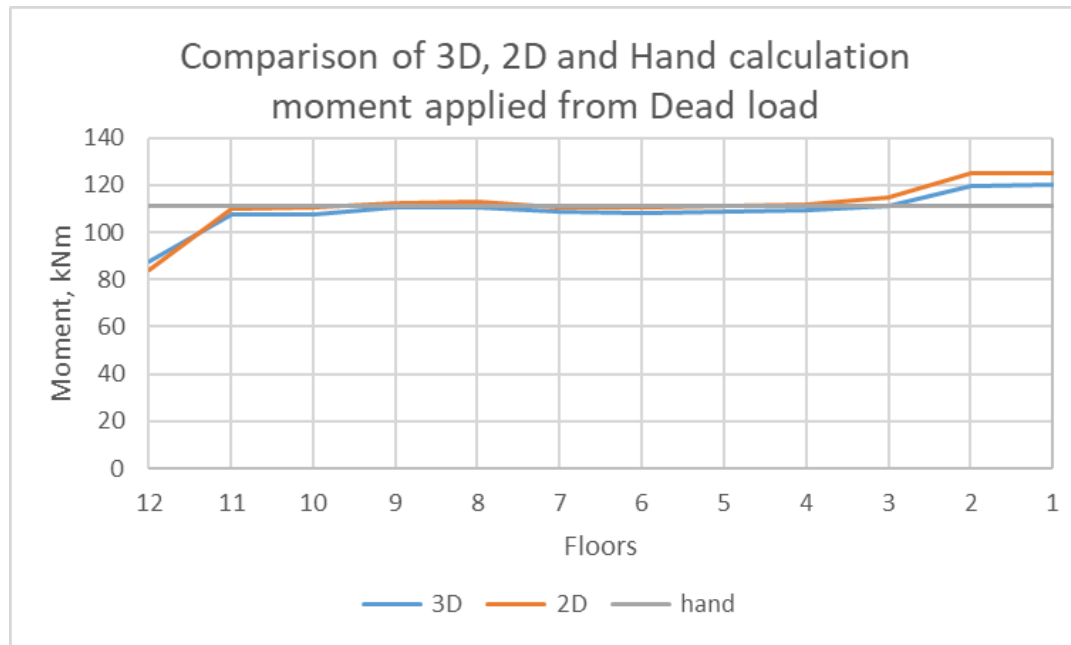


Figure 2.3.47. Moment from dead load

From the graph it can be seen that the results are approximately the same. However, 2D and 3D results from the shear force diagram are fluctuating by every floor.

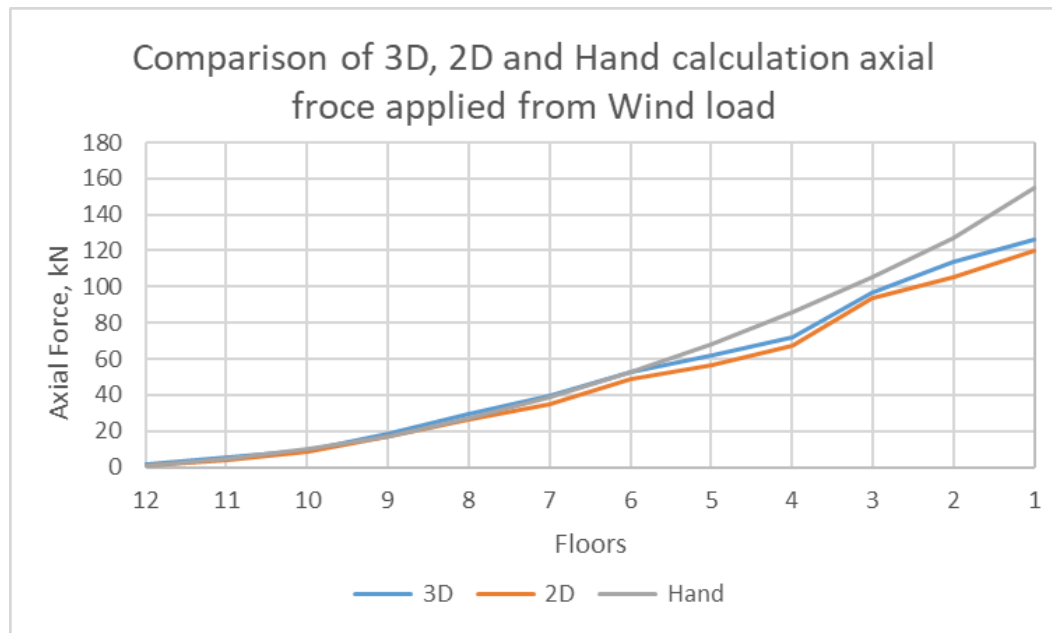


Figure 2.3.48. Axial forces from wind load

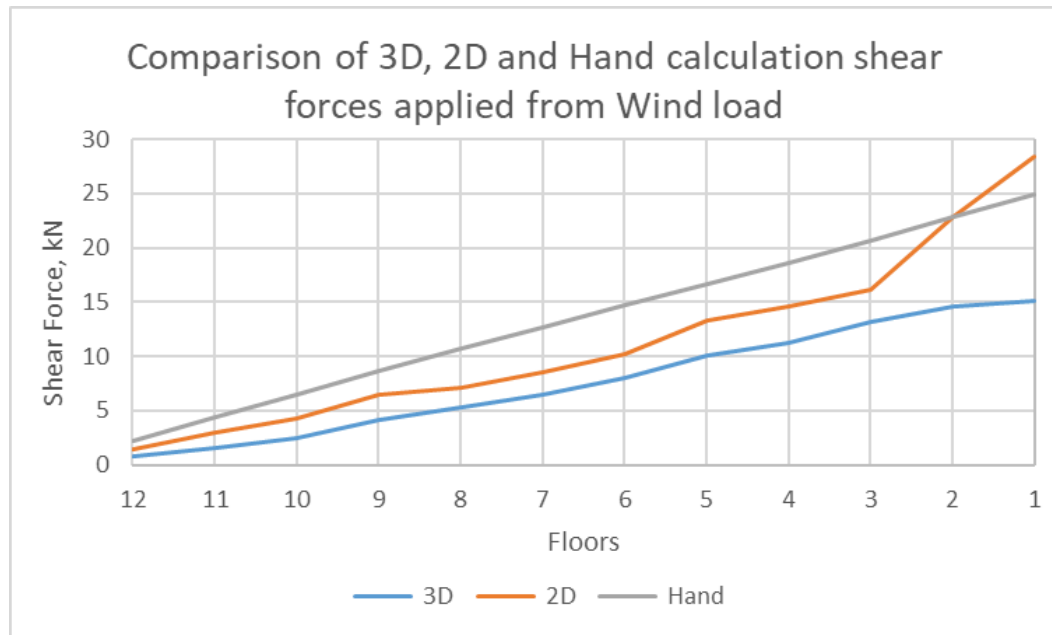


Figure 2.3.49. Shear forces from wind load

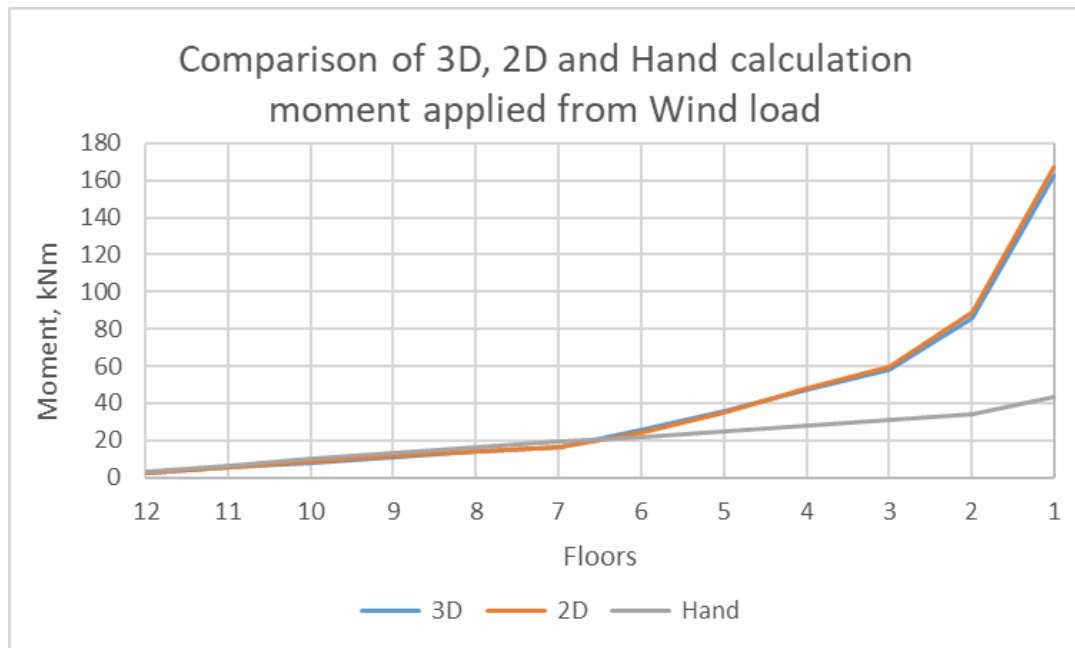


Figure 2.3.50. Moment from wind load

From the wind load results it can be seen that the shear force diagram has low precision between the results. Also, moments diagram hand calculation is significantly different from SAP2000 models.

2.3.3 Structural member design (size or check reinforcement) using software

After performing design or check of structure in SAP 2000, it was found that most of the members did not pass multiple design checks. Some of the examples are shown in figures below.

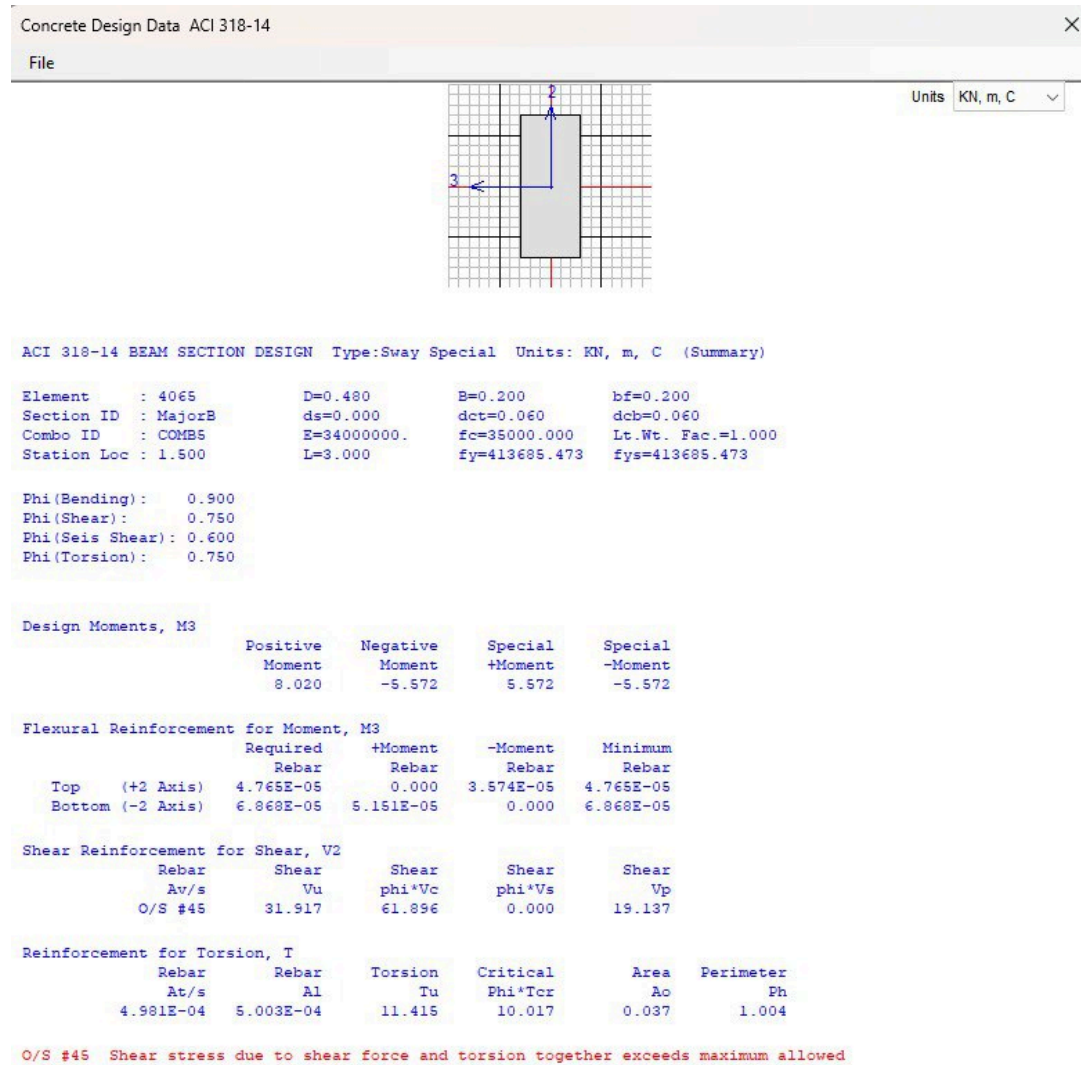


Figure 2.3.51. Beam section shear stress check fail

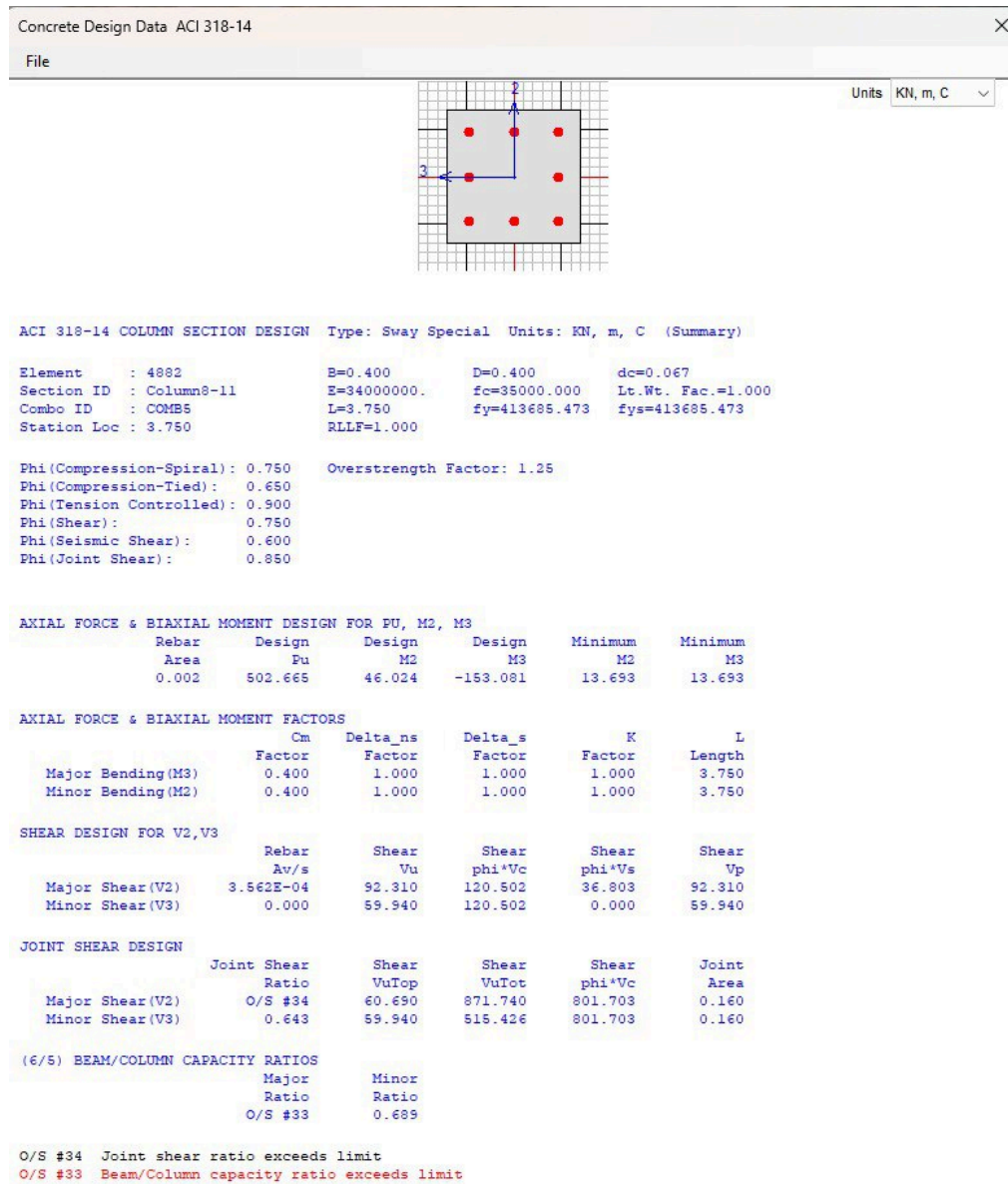


Figure 2.3.52. Column section capacity ratio check fail

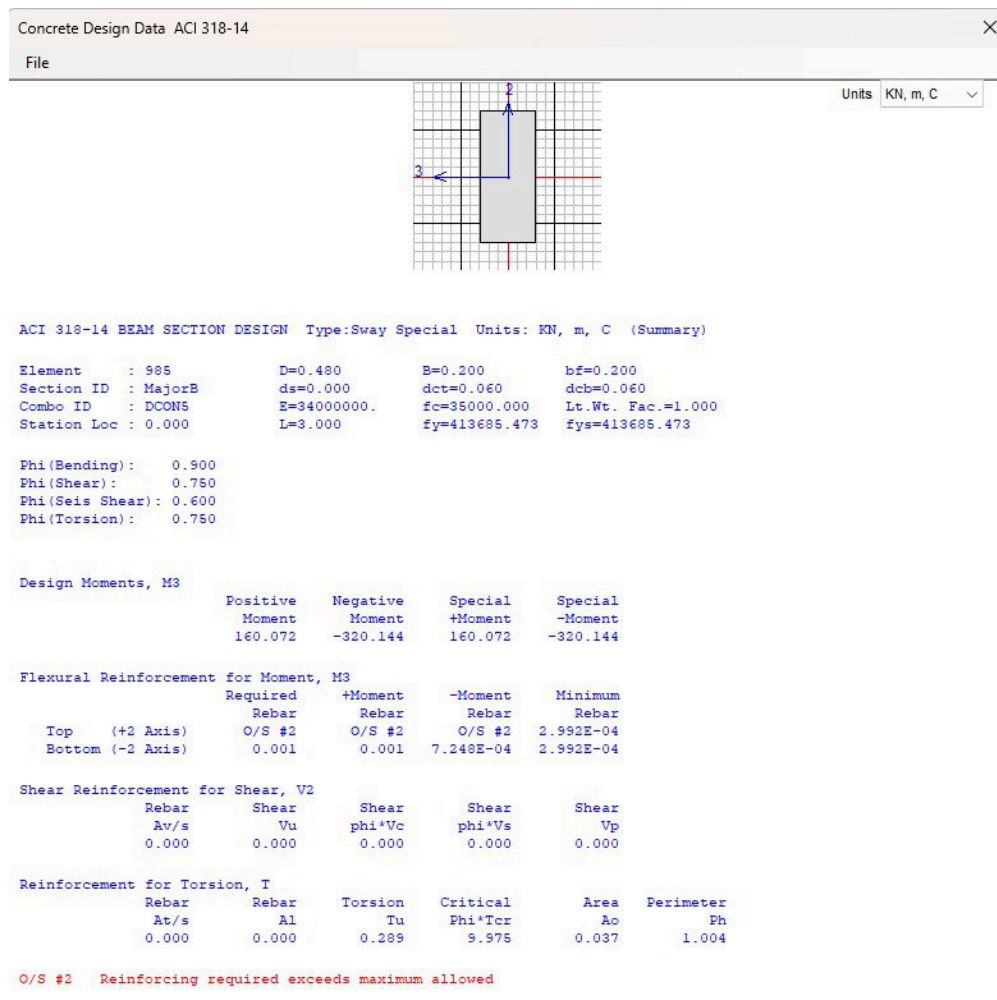


Figure 2.3.53. Beam section reinforcement limit check fail

After increasing member sizes step by step, the dimensions were finalised as follows.

Table 2.3.42. Final member dimensions after software design

Member	Height, mm	Width, mm
Major beam	650	350
Minor beam	400	200
Column 1-3 levels	700	700
Column 4-7 levels	650	650
Column 8-11 levels	600	600
Column 12-13 levels	550	550

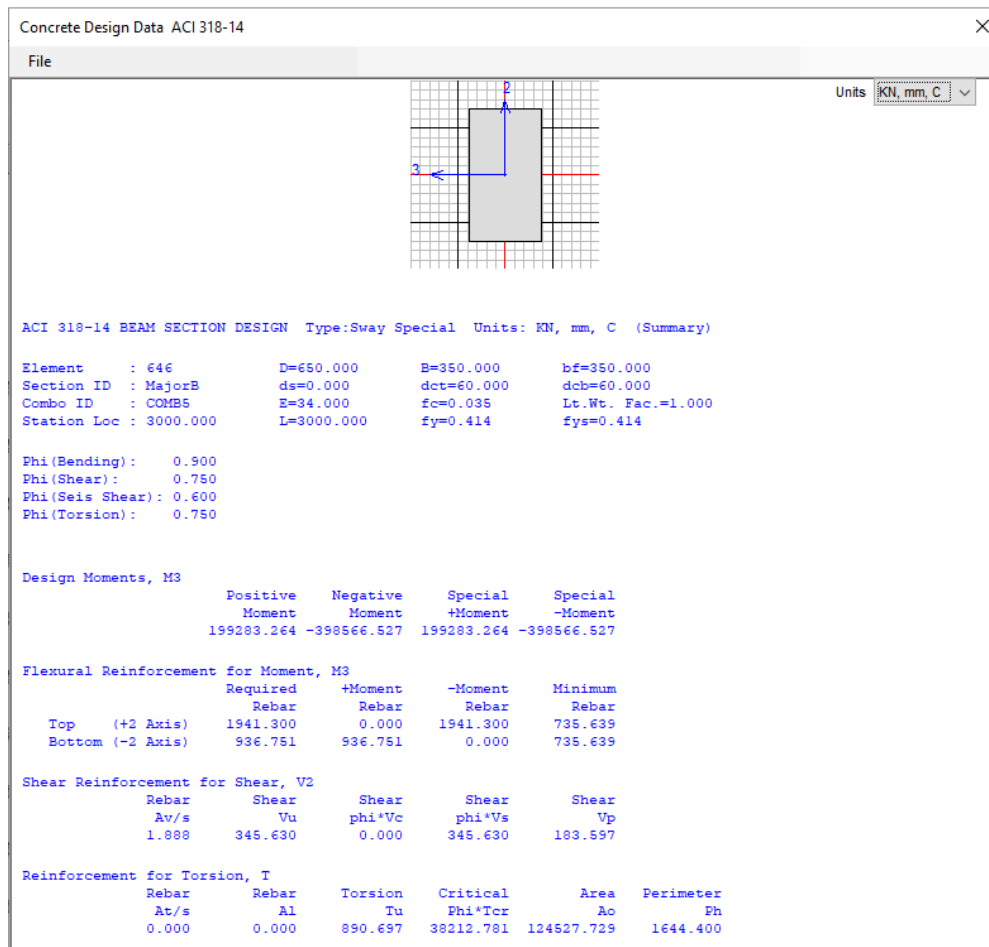


Figure 2.3.54. Major Beam section design (Negative moment)

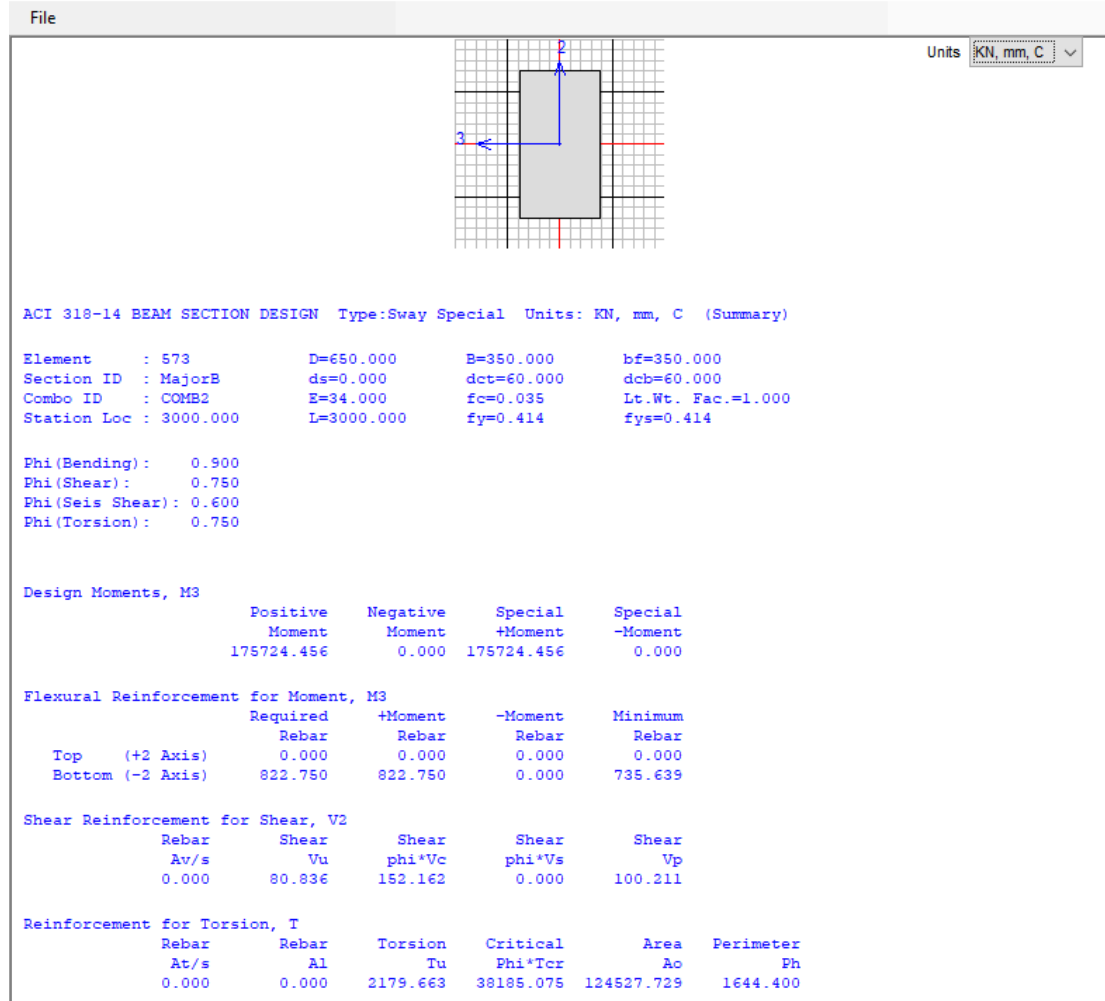


Figure 2.3.55. Major Beam section design (Positive moment)

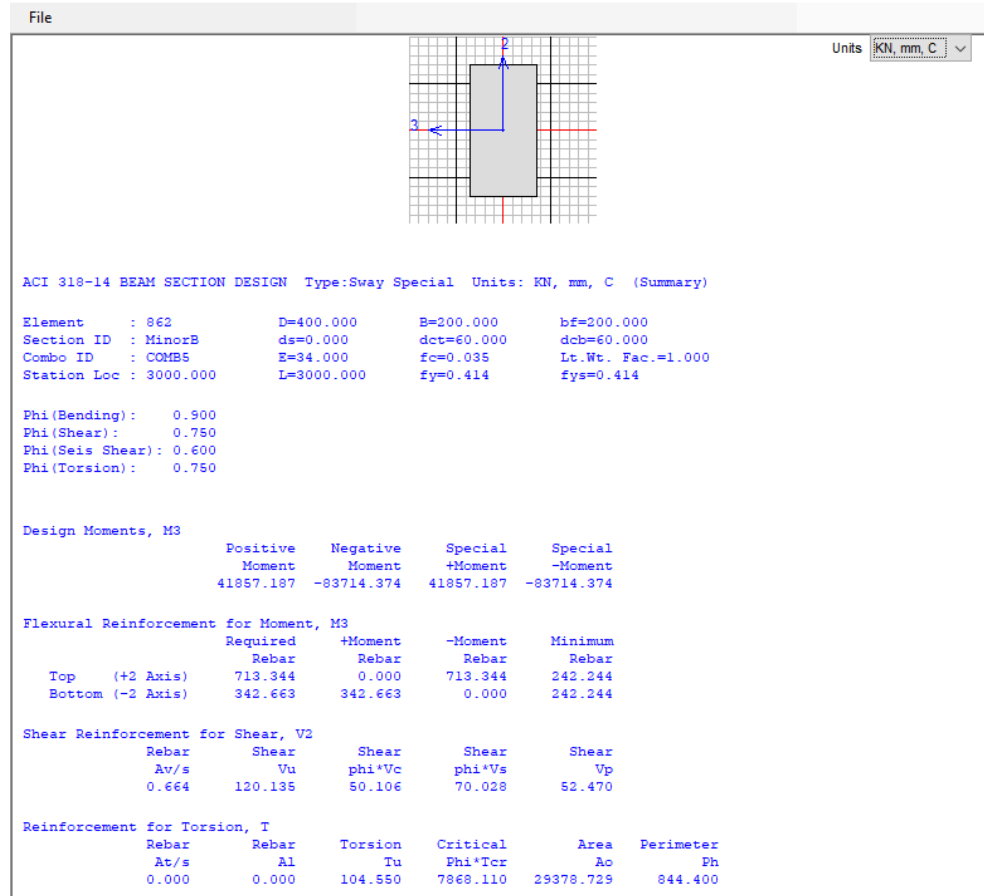


Figure 2.3.56. Minor Beam section design (Negative moment)

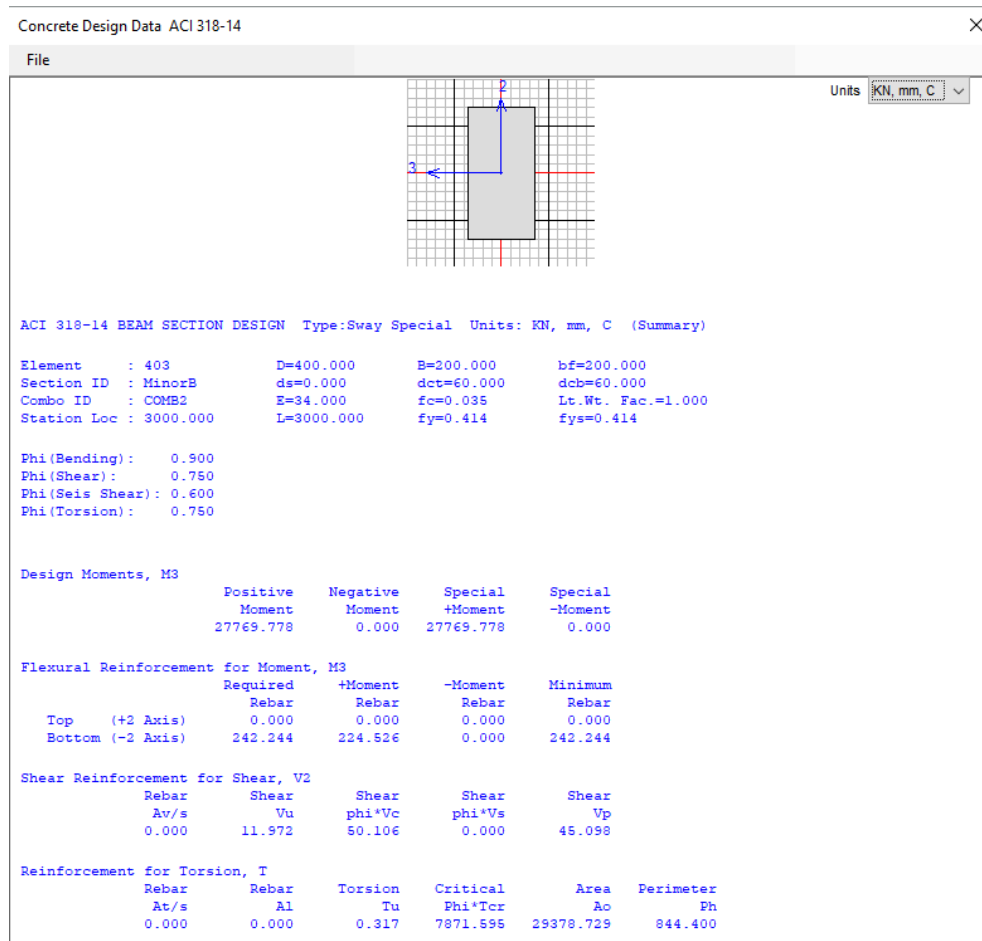


Figure 2.3.57. Minor Beam section design (Positive moment)

2.3.4 Hand calculation verification for structural member design

Given members in section above were chosen from the SAP2000 model and they have the highest positive and negative moments for each floor (the given pictures represent moments for the 2nd floor. Other floors can be found in Appendix E). For this reason we have used them to calculate the reinforcement for the upper and lower parts of the beam. The same procedures will be completed for the entire structure members.

2.3.4.1 Major beam design

Negative moment / Major beam

Flexural analysis

$$M_u = 398.567 \text{ kNm} \Rightarrow \text{from SAP2000}$$

$$b = 350 \text{ mm}, h = 650 \text{ mm}, d = 650 - 2.5 \text{ in} * 25.4 \text{ mm/in} \approx 580 \text{ mm}$$

$$R_n = M_u / \phi b d^2 = 3761.27 \text{ kN/m}^2 \quad (\phi = 0.9)$$

$$\rho = 0.85 * f'_c / f_y * (1 - \sqrt{1 - 2R_n / (0.85f'_c)}) = 0.0078$$

$$A_s = \rho b d = 0.0078 * 580 * 350 \approx 1685 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \Rightarrow a = \frac{1585 * 517}{0.85 * 35 * 350} = 78.7 \text{ mm}$$

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

$$c = \frac{\alpha}{\beta_1} = 98.87 \text{ mm}$$

$$\varepsilon_t = 0.003 * \left(\frac{d-c}{c}\right) = 0.0146 > 0.005 - \text{Tension controlled}$$

$$\text{Error: } \frac{1942 - 1685}{1942} * 100 = 13\%, \text{ Which was decided as acceptable}$$

$$A_{s,min} = 3 \sqrt{f'_c} b d / f_y \geq 200 b d / f_y \Rightarrow 578.63 > 541.44$$

$$A_s = 1942 \text{ mm}^2 > 580 \text{ mm}^2 \Rightarrow \text{meets the requirement}$$

$$1942 \text{ mm}^2 = 3.01 \text{ in}^2 \quad 4\#8: 3.14 \text{ in}^2 @ 48 \text{ mm}$$

Positive moment / Major beam

$$M_u = 175.724 \text{ kNm} \Rightarrow \text{from SAP2000}$$

$$R_n = M_u / \phi b d^2 = 1658.31 \text{ kN/m}^2 (\phi = 0.9)$$

$$\rho = 0.85 * f'_c / f_y * (1 - \sqrt{1 - 2 R_n / (0.85 f'_c)}) = 0.0035$$

$$A_s = \rho b d = 0.0035 * 580 * 350 \approx 711 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \Rightarrow a = \frac{711 * 517}{0.85 * 35 * 350} = 35.3 \text{ mm}$$

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

$$c = \frac{\alpha}{\beta_1} = 44.35 \text{ mm}$$

$$\varepsilon_t = 0.003 * \left(\frac{d-c}{c}\right) = 0.036 > 0.005 - \text{Tension controlled}$$

$$\text{Error: } \frac{823 - 711}{823} * 100 = 13\%, \text{ Which was decided as acceptable}$$

$$A_{s,min} = 3 \sqrt{f'_c} b d / f_y \geq 200 b d / f_y \Rightarrow 578.63 > 541.44$$

$$A_s = 823 \text{ mm}^2 > 580 \text{ mm}^2 \Rightarrow \text{meets the requirement}$$

$$823 \text{ mm}^2 = 1.28 \text{ in}^2 \quad 3\#6: 1.28 \text{ in}^2 @ 90 \text{ mm}$$

Shear analysis

From SAP2000 $\rightarrow V_u = 345.63 \text{ kN}$ (Maximum V_u). For normal – weight concrete $\lambda = 1$

$$\phi V_c = \phi * 2\sqrt{f'_c} b_w d = 0.75 * 2 * \sqrt{35000} * 350 * 580 = 72.34 \text{ kN}$$

$V_u > \phi V_c$, so, we should provide shear reinforcement.

$$V_{c1} = 4\sqrt{f'_c} b_w d = 192.9 \text{ kN and } V_{c2} = 8\sqrt{f'_c} b_w d = 385.8 \text{ kN}$$

$$V_s = (V_u - \phi V_c) / \phi = 364.4 < V_{c2} \rightarrow \text{proceed}$$

$$A_v / s = V_s / f_{yt} d = 364.4 / (0.414 * 580) = 1.518 \text{ mm}^2 / \text{mm}$$

A_v / s From SAP2000 is 1.888

Error: $\frac{1.888 - 1.518}{1.888} * 100 = 20\%$, which is quite big. It may be connected with the structure built in SAP2000. Not all of the values are totally correct as it had the same problems in some other sections.

Spacing of stirrups: $S_1 = A_v f_{yt} d / V_s = 119 \text{ mm}$ (Assumed as #3 stirrup)

$$S_2 = d / 2 / 1000 = 368 \text{ mm}$$

$$S_3 = A_v f_{yt} / 50 b_w \geq A_v f_{yt} (0.75 \sqrt{f'_c} b_w) \Rightarrow 4.19 \geq 1.49$$

$S = S_1 = 119 \text{ mm}$ #3 stirrups @ 100 mm

Torsional analysis

From SAP2000 $\rightarrow 2180 \text{ Nm}$

$$A_{cp} = b * h = 0.28 \text{ m}^2, P_{cp} = 2 * (b + h) = 2.3 \text{ m}$$

$$T_n = \phi \lambda \sqrt{f'_c} A_{cp}^2 / P_{cp} = 4.78 \text{ kNm} > 2.18 \text{ kNm} \rightarrow \text{reinforcement is not needed}$$

2.3.4.2 Minor beam design

Negative moment / Minor beam

Flexural analysis

$$M_u = 83.714 \text{ kNm} \Rightarrow \text{from SAP2000}$$

$$b = 200 \text{ mm}, h = 400 \text{ mm}, d = 400 - 2.5 \text{ in} * 25.4 \text{ mm/in} \approx 330 \text{ mm}$$

$$R_n = M_u / \phi b d^2 = 4270.69 \text{ kN/m}^2 (\phi = 0.9)$$

$$\rho = 0.85 * f'_c / f_y * (1 - \sqrt{1 - 2R_n / (0.85 f'_c)}) = 0.00896$$

$$A_s = \rho b d = 0.00896 * 330 * 200 \approx 592 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \Rightarrow a = \frac{592 * 517}{0.85 * 35 * 200} = 51.44 \text{ mm}$$

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

$$c = \frac{\alpha}{\beta_1} = 64.62 \text{ mm}$$

$$\epsilon_t = 0.003 * (\frac{d-c}{c}) = 0.0123 > 0.005 - \text{Tension controlled}$$

$$A_{s,min} = 3 \sqrt{f'_c} b d / f_y \geq 200 b d / f_y \Rightarrow 189 > 171$$

$$A_s = 714 \text{ mm}^2 > 189 \text{ mm}^2 \Rightarrow \text{meets the requirement}$$

$$714 \text{ mm}^2 = 1.11 \text{ in}^2 \quad 2\#7: 1.20 \text{ in}^2 @ 50 \text{ mm}$$

Positive moment / Minor beam

$$M_u = 27.769 \text{ kNm} \Rightarrow \text{from SAP2000}$$

$$b = 200 \text{ mm}, h = 400 \text{ mm}, d = 400 - 2.5 \text{ in} * 25.4 \text{ mm/in} \approx 330 \text{ mm}$$

$$R_n = M_u / \phi b d^2 = 1416.64 \text{ kN/m}^2 (\phi = 0.9)$$

$$\rho = 0.85 * f'_c / f_y * (1 - \sqrt{1 - 2R_n / (0.85 f'_c)}) = 0.00281$$

$$A_s = \rho b d = 0.00281 * 330 * 200 \approx 186 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \Rightarrow a = \frac{186 * 517}{0.85 * 35 * 200} = 16.16 \text{ mm}$$

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

$$c = \frac{\alpha}{\beta_1} = 20.3 \text{ mm}$$

$$\epsilon_t = 0.003 * (\frac{d-c}{c}) = 0.0458 > 0.005 - \text{Tension controlled}$$

$$A_{s,min} = 3 \sqrt{f'_c} b d / f_y \geq 200 b d / f_y \Rightarrow 191.83 > 179.5$$

$$A_s = 243 \text{ mm}^2 > 192 \text{ mm}^2 \Rightarrow \text{meets the requirement}$$

$$243 \text{ mm}^2 = 0.377 \text{ in}^2 \quad 2\#4: 0.39 \text{ in}^2 @ 60 \text{ mm}$$

Shear analysis:

$$\text{From SAP2000} \rightarrow V_u = 120.135 \text{ kN (Maximum } V_u). \text{ For normal - weight concrete } \lambda = 1$$

$$\phi V_c = \phi * 2 \sqrt{f'_c} b_w d = 0.75 * 2 * \sqrt{35000} * 200 * 330 = 24.13 \text{ kN}$$

$V_u > \phi V_c$, so, we should provide shear reinforcement.

$$V_{c1} = 4\sqrt{f'_c} b_w d = 64.36 \text{ kN and } V_{c2} = 8\sqrt{f'_c} b_w d = 128.713 \text{ kN}$$

$$V_s = (V_u - \phi V_c)/\phi = 128 < V_{c2} \rightarrow \text{proceed}$$

$$A_v/s = V_s/f_{yt} d = 128/(0.414 * 330) = 0.719 \text{ mm}^2/\text{mm}$$

A_v/s From SAP2000 is 0.664

$$\text{Error: } \frac{0.664-0.719}{0.664} * 100 = 8\%$$

Spacing of stirrups: $S_1 = A_v f_{yt} d / V_s = 247 \text{ mm}$ (Assumed as #3 stirrup)

$$S_2 = d/2/1000 = 215 \text{ mm}$$

$$S_3 = A_v f_{yt} / 50 b_w \geq A_v f_{yt} (0.75 \sqrt{f'_c} b_w) \Rightarrow 7.34 \geq 2.62$$

$$S = S_2 = 215 \text{ mm} \quad \#3 \text{ stirrups @ } 200 \text{ mm}$$

Torsional analysis:

From SAP2000 $\rightarrow 104.55 \text{ Nm}$

$$A_{cp} = b * h = 0.1 \text{ m}^2, P_{cp} = 2 * (b + h) = 1.4 \text{ m}$$

$$T_n = \phi \lambda \sqrt{f'_c} A_{cp}^2 / P_{cp} = 1.0022 \text{ kNm} > 0.105 \text{ vkNm} \rightarrow \text{reinforcement is not needed}$$

Same procedure was used to calculate the beam design for other floors. The results are as shown in the following figure:

Major Beam (12th floor)				Minor Beam (12th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-90.294	47.155	-126.915	Mu (kN-m)	-14.232	6.736	-20.436
As (mm2)	555.87	288.321	735.639	As (mm2)	151.676	71.341	218.926
As (in2)	0.8615468072	0.4468707378	1.14017204	As (in2)	0.235083695	0.1105719157	0.3393149411
Compression ba	4#3	2#3	3#4	Compression ba	2#3	2#3	2#3
Tension bars	2#6	5#3	6#4	Tension bars	3#3	2#3	2#4
Stirrups	#3@150mm			Stirrups	#3@100mm		
Major Beam (11th floor)				Minor Beam (11th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-184.453	115.647	-248.052	Mu (kN-m)	-44.193	17.823	-55.832
As (mm2)	864.88	714.859	1175.76	As (mm2)	362.554	190.518	462.978
As (in2)	1.340483571	1.107964972	1.822318661	As (in2)	0.5619249845	0.2952851829	0.7175728456
Compression ba	4#4	2#4	3#5	Compression ba	3#3	2#3	2#4
Tension bars	5#5	6#4	6#5	Tension bars	3#4	3#3	4#4
Stirrups	#3@250mm			Stirrups	#3@200mm		
Major Beam (10th floor)				Minor Beam (10th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-188.258	113.958	-269.893	Mu (kN-m)	-42.817	17.541	-57.159
As (mm2)	883.288	704.23	1284.174	As (mm2)	350.83	187.461	474.571
As (in2)	1.369014259	1.091491011	1.990350279	As (in2)	0.5437538748	0.2905471172	0.7355409175
Compression ba	4#4	2#4	6#4	Compression ba	3#3	2#3	2#4
Tension bars	5#5	6#4	2#9	Tension bars	5#3	3#3	4#4
Stirrups	#3@250mm			Stirrups	#3@200mm		
Major Beam (9th floor)				Minor Beam (9th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-185.991	114.285	-286.293	Mu (kN-m)	-41.398	17.59	-58.608
As (mm2)	872.32	706.288	1366.146	As (mm2)	338.77	187.988	487.266
As (in2)	1.352014879	1.094680719	2.117399256	As (in2)	0.5250619963	0.2913639182	0.755216987
Compression ba	4#4	2#4	6#4	Compression ba	3#3	2#3	2#4
Tension bars	5#5	6#4	5#6	Tension bars	3#4	3#3	4#4
Stirrups	#3@150mm			Stirrups	#3@200mm		
Major Beam (8th floor)				Minor Beam (8th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-183.232	114.406	-300.361	Mu (kN-m)	-40.17	17.609	-59.838
As (mm2)	858.98	707.047	1436.863	As (mm2)	328.36	188.196	498.071
As (in2)	1.33133912	1.095857099	2.22700403	As (in2)	0.5089274644	0.2916862988	0.7719637322
Compression ba	4#4	2#4	6#4	Compression ba	3#3	2#3	2#4
Tension bars	5#5	6#4	4#7	Tension bars	5#3	3#3	4#4
Stirrups	#3@100mm			Stirrups	#3@200mm		
Major Beam (7th floor)				Minor Beam (7th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-183.435	113.421	-314.009	Mu (kN-m)	-38.981	17.444	-60.958
As (mm2)	859.959	700.847	1505.825	As (mm2)	318.3	186.409	507.928
As (in2)	1.332856479	1.086247675	2.333888717	As (in2)	0.4933353999	0.288916615	0.7872411655
Compression ba	4#4	2#4	6#4	Compression ba	3#3	2#4	2#4
Tension bars	5#5	6#4	4#7	Tension bars	5#3	3#3	4#4
Stirrups	#3@100mm			Stirrups	#3@200mm		

Figure 2.3.58. Major and minor beam design 7-12th floors

Major Beam (6th floor)				Minor Beam (6th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-182.578	112.624	-323.118	Mu (kN-m)	-38.086	17.312	-61.782
As (mm2)	855.82	695.833	1552.053	As (mm2)	310.747	184.979	515.198
As (in2)	1.326441414	1.078476441	2.405537818	As (in2)	0.4816289523	0.286700248	0.7985089895
Compression ba	6#3	2#3	3#6	Compression ba	3#3	2#3	4#3
Tension bars	3#6	6#4	4#7	Tension bars	5#3	3#3	2#6
Stirrups	#3@100mm			Stirrups	#3@200mm		
Major Beam (5th floor)				Minor Beam (5th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-177.725	112.82	-328.807	Mu (kN-m)	-37.35	17.35	-62.489
As (mm2)	832.395	697.067	1581.002	As (mm2)	304.537	185.385	521.447
As (in2)	1.290134842	1.080389027	2.450406076	As (in2)	0.4720040298	0.2873295102	0.8081943583
Compression ba	6#3	2#3	4#5	Compression ba	3#3	2#3	4#3
Tension bars	3#6	6#4	6#6	Tension bars	5#3	3#3	2#6
Stirrups	#3@100mm			Stirrups	#3@200mm		
Major Beam (4th floor)				Minor Beam (4th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-173.737	112.35	-335.893	Mu (kN-m)	-36.527	17.276	-63.247
As (mm2)	813.179	694.112	1617.155	As (mm2)	297.609	184.581	528.157
As (in2)	1.260351829	1.075809051	2.506439864	As (in2)	0.461266274	0.286083385	0.8185942343
Compression ba	6#3	2#3	3#6	Compression ba	3#3	2#3	4#3
Tension bars	3#6	6#4	6#6	Tension bars	5#3	3#3	2#6
Stirrups	#3@100mm			Stirrups	#3@200mm		
Major Beam (3th floor)				Minor Beam (3th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-205.38	144.764	-386.643	Mu (kN-m)	-50.244	22.526	-79.749
As (mm2)	966.407	735.639	1879.011	As (mm2)	414.483	241.751	676.837
As (in2)	1.49784098	1.14017204	2.912292312	As (in2)	0.6424101054	0.3746915685	1.049034408
Compression ba	4#4	2#4	5#5	Compression ba	3#3	2#3	5#3
Tension bars	5#5	6#4	5#7	Tension bars	6#3	2#4	6#4
Stirrups	#3@150mm			Stirrups	#3@200mm		
Major Beam (2nd floor)				Minor Beam (2nd floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-236.2559	175.724	-398.567	Mu (kN-m)	-80.0223	27.769	-83.714
As (mm2)	1117.57	822.75	1941.3	As (mm2)	679.335	242.244	713.344
As (in2)	1.732129572	1.275185989	3.00883447	As (in2)	1.052906076	0.3754556727	1.105616863
Compression ba	2#6	2#3	2#8	Compression ba	3#4	2#3	3#4
Tension bars	4#6	3#6	4#8	Tension bars	6#4	2#4	6#4
Stirrups	#3@100mm			Stirrups	#3@200mm		
Major Beam (1th floor)				Minor Beam (1th floor)			
Beam part	End	Mid	End	Beam part	End	Mid	End
Mu (kN-m)	-194.725	152.058	-349.375	Mu (kN-m)	-55.846	23.733	-80.347
As (mm2)	914.622	735.639	1686.211	As (mm2)	463.095	242.244	682.316
As (in2)	1.417579045	1.14017204	2.613470242	As (in2)	0.7177541847	0.3754556727	1.057526348
Compression ba	4#4	3#6	3#6	Compression ba	2#4	2#3	3#4
Tension bars	5#5	6#4	6#6	Tension bars	4#4	2#4	6#4
Stirrups	#3@150mm			Stirrups	#3@200mm		

Figure 2.3.59. Major and minor beam design 1-6th floors

2.3.4.3 Slab panel design

Minimum thickness for slab is:

$$t_{min} = \frac{l_n}{30} = \frac{6-0.7}{30} = 0.177 \text{ m}$$

Direct Design Method was used for two-way slab design.

$$q_D = 6.6391 \text{ kN/m}^2$$

$$q_L = 4.79 \text{ kN/m}^2$$

$$q_u = 1.2 \times q_D + 1.6 \times q_L = 15.631 \text{ kN/m}^2$$

$$M_0 = \frac{q_u \times l_2 \times l_n^2}{8} = \frac{15.631 \times 6 \times (6-0.7)^2}{8} = 329.30 \text{ kN} \cdot \text{m}$$

$$M_{negative} = 0.65 \times M_{total} = 214.05 \text{ kN} \cdot \text{m}$$

$$M_{positive} = 0.35 \times M_{total} = 115.26 \text{ kN} \cdot \text{m}$$

Table 2.3.43 Interior panel design

	Column Strip		Middle Strip	
	Negative	Positive	Negative	Positive
%	75	75	25	25
Mu (kN*m)	160.54	86.44	53.51	28.81
Mu for slab (kN*m)	24.08	12.97	8.03	4.32
b (m)	1.5	1.5	1.5	1.5
Rn (kPa)	2066.78	1112.88	688.93	370.96
ρ	0.0041	0.0022	0.0013	0.00072
A_s (in ²)	0.89574	0.47398	0.29123	0.15596
$A_{s,min}$ (in ²)	0.61938	0.61938	0.61938	0.61938

Table 2.3.44 Exterior panel design

	Column Strip			Middle Strip		
	Negative	Positive	Negative	Negative	Positive	Negative
%	96.57	75	75	3.43	25	25
Mu (kN*m)	206.71	86.44	160.54	7.34	28.81	53.51
Mu for	31.01	12.97	24.08	1.10	4.32	8.03

slab (kN*m)						
b (m)	1.85	1.85	1.85	1.5	1.5	1.5
Rn (kPa)	2442.88	902.33	1675.76	94.50	370.96	688.93
ρ	0.0049	0.0018	0.0033	0.0002	0.0007	0.0013
A_s (in ²)	1.31515	0.47222	0.88926	0.3955	0.15596	0.29123
$A_{s,min}$ (in ²)	0.61938	0.61938	0.61938	0.61938	0.61938	0.61938

Choose 7#4 bars ($A_s=1.37$ in²) with spacing 0.25 m.

$$P_u = 5358.497 \text{ kN}, M_u = 363.379 \text{ kN}$$

$$A_{g,required} = \frac{P_u}{0.6 \times f'_c} = 0.2552 \text{ m}^2$$

$$A_{g,prov} = 0.7 * 0.7 = 0.49 \text{ m}^2$$

$$e = \frac{M_u}{P_u} = 0.0678 \text{ m}$$

$$P_n = \frac{P_u}{\phi} = 8243.8415 \text{ kN}$$

$$K_n = \frac{P_n}{f'_c \times A_{g,prov}} = 0.481$$

$$R_n = \frac{P_n \times e}{f'_c \times A_{g,prov} \times h} = \frac{8243.8415 \times 0.0678}{35000 \times 0.49 \times 0.7} = 0.0466$$

From interaction diagram $\rho=0.01=1\%$.

$$A_s = 0.49 * 0.01 = 0.0049 \text{ m}^2 = 7.595 \text{ in}^2$$

2.3.4.4 Edge column design

Edge column (550 mm x 550 mm)

$$P_u = 143.73 \text{ kN}$$

$$I_c = 0.005338 \text{ m}^2; I_b = 0.0028035 \text{ m}^2$$

$$\psi = \frac{\Sigma(EI/L)_{column}}{\Sigma(EI/L)_{beam}} = 3.05; k = 1.562$$

$$\frac{kl_u}{r} = 28.4 > 22 \Rightarrow \text{Slender column}$$

Moment magnification for sway slender columns

$$(EI)_{eff} = \frac{0.4EI}{1+\beta_{ds}} = 0.4EI = 8.48 * 10^{13} \text{ N} * \text{mm}^2$$

$$P_c = \frac{\pi^2(EI)_{eff}}{(kl_u)^2} = 38120.32 \text{ kN}$$

$$\delta_s = \frac{1}{1 - \frac{\Sigma P_u}{0.75 \Sigma P_c}} = \frac{1}{1 - \frac{4*75.102 + 22*180.314 + 28*326.025}{0.75*54*38120.32}} = 1.00875$$

$$M_{1,s} = 40.9098 \text{ kN} * \text{m}; M_{2,s} = 22.4961 \text{ kN} * \text{m}; (1E)$$

$$M_{1,ns} = 110.4072 \text{ kN} * \text{m}; M_{2,ns} = 106.0721 \text{ kN} * \text{m}; (1.2D+1L+0.2S)$$

$$M_1 = M_{1,ns} + \delta_s M_{1,s} = 151.675 \text{ kN} * \text{m}; M_2 = 128.765 \text{ kN} * \text{m}$$

Design of edge column on top floor:

$$P_n = P_u / 0.65 = 143.73 / 0.65 = 221.123 \text{ kN}$$

$$M_n = M_u / 0.65 = 128.765 / 0.65 = 198.100 \text{ kN} * \text{m}$$

$$e = \frac{M_u}{P_u} = \frac{128.765}{143.73} = 0.896 \text{ m}$$

$$\gamma = \frac{0.550 - 0.040*2}{0.550} = 0.855$$

$$K_n = \frac{P_n}{f_c A_g} = 0.021; R_n = \frac{P_n e}{f_c A_g h} = 0.034 \Rightarrow$$

$$\rho = 0.01 \Rightarrow A_s = 0.01 A_g = 0.003025 \text{ m}^2 = 4.68 \text{ in}^2 \text{ (Choose 6\#8; } A_s = 4.71 \text{ in}^2)$$

Biaxial bending check was performed as follows:

$$M_{ux} = 107.056 \text{ kN} * \text{m}; M_{nx} = M_{ux} / 0.65 = 164.702 \text{ kN} * \text{m}$$

$$M_{uy} = 128.765 \text{ kN} * \text{m}; M_{ny} = 198.100 \text{ kN} * \text{m}$$

$$\gamma_x = \gamma_y = 0.746; e_x = 0.745; e_y = 0.896$$

$$\frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_{n0}} = \frac{1}{20034.81} + \frac{1}{9935.96} - \frac{1}{11192.12} = \frac{1}{16337.44} \text{ 1/kN}$$

$$P_n > 0.1 P_{n0} \Rightarrow \text{the method is valid}$$

$$\phi P_n = 10619.34 \text{ kN} > 143.73 \text{ kN} \Rightarrow OK!$$

Shear strength check showed following results:

$$V_c = 0.17(1 + \frac{P_u}{14A_g})\lambda\sqrt{f_c}b_w d = 0.17(1 + \frac{143.73}{14*0.003025}) * \sqrt{35000} * 0.55 * 0.47 = 311.689 \text{ kN}$$

$$\phi V_c / 2 = 116.88 \text{ kN} > V_u = 76.412 \text{ kN} \Rightarrow \text{Minimum reinforcement}$$

Hand calculations verify the results from software, since the reinforcement ratio was found to be 1% for all columns in SAP2000.

2.3.5 Joint design

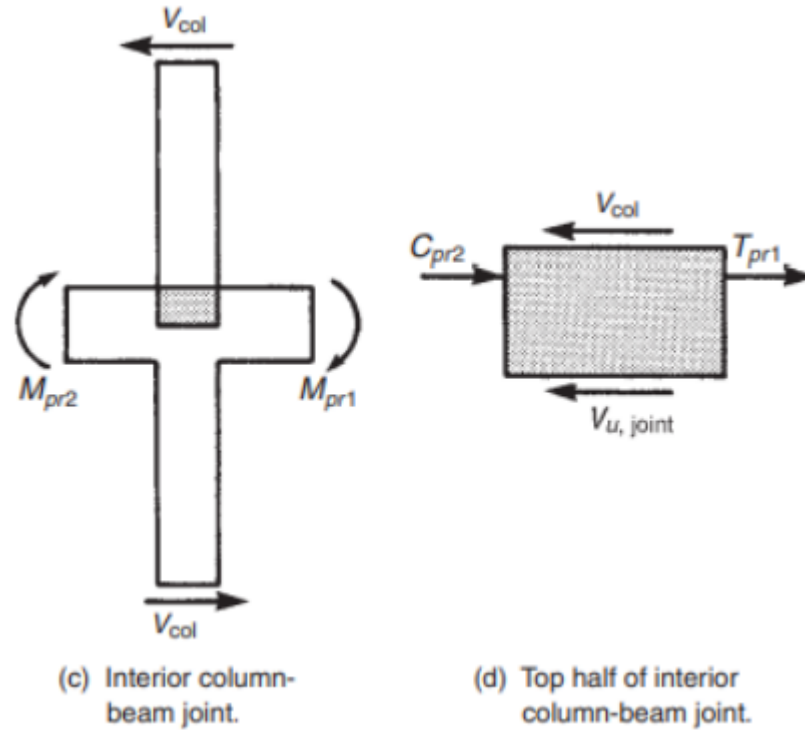


Figure 2.3.60. Beam column joint

Shear in the joint was calculated as follows:

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

$$T_{pr1} = \alpha A_s f_y = 1.25 * 1942 * 0.414 = 1004.985 \text{ kN}$$

$$C_{pr1} = \alpha A_s f_y = 1.25 * 823 * 0.414 = 425.9 \text{ kN}$$

$$V_{col} = 423.54 \text{ kN} \rightarrow \text{From SAP2000}$$

So, the shear demand is:

$$V_{u, joint} = 1004.985 + 425.9 - 423.54 = 1007.345$$

$$\phi V_n \geq V_u, \text{ where } \phi = 0.85$$

$$V_n = \gamma \sqrt{f'_c} A_j, A_j = b_j * h_{col}, \gamma = 1.7$$

Effective width of joint:

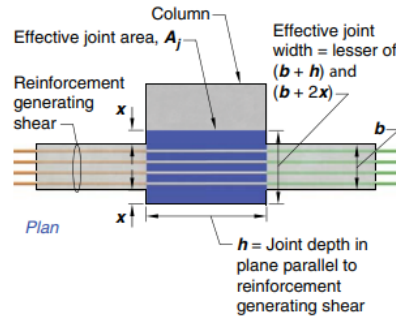


Figure 2.3.61. Effective joint width (ACI 318-19. Chapter 15)

$$b_j = (b_b + b_{col})/2 = (350 + 700)/2 = 525mm$$

From Figure 2.3.65

$$b_j < b_b + h_{col} = 350 + 700 = 1050 mm \text{ and } b_j < b_b + 2x = 350 + 700 = 1050 mm$$

$$\text{So, } V_n = 1.7 * \sqrt{35} * 525 * 700 = 3696.1 kN$$

$$0.85 * 3696.1 = 3141.7 kN \geq V_u = 1007.345 kN$$

From this result we can conclude that the joint is safe against shear forces. In the same way we can find other directions. The results showed that they are safe too.

The next thing is to find the spacing, also can be found using the ACI 318-19. Chapter 15

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

For the bars #10 and less it is used #3 stirrups, accordingly by using the formulas above it was found that the spacing is:

$$s = \min\left(\frac{2*71*414}{0.062\sqrt{35}*700}, \frac{A_v f_{yt}}{0.35b_c}\right) = \min(228.96, 239.95)$$

$$s = 220 mm$$

2.3.6 Seismic detailing

Beams

The place where the building is planned to be constructed is considered as a high seismic region, therefore, beam structures requirements should fit the requirements of a high seismic region. The beam contains longitudinal bars which were properly chosen according to the load applied to them. The bottom and top bars can resist the tension and compression strength applied to them. Overall, the tension bars are greater and can resist more force than compression bars. We have used hoops that are not longer than $d/4$, $6db$ and $150 mm$.

Columns

In the columns we have used reinforcement bars which build up 3-4% from the area of the column. This number is enough to provide needed strength for the structure. Those bars were tied with #3 stirrups at each 400mm which provides additional strength to the structure. During the design of columns for high seismic regions there were considered other requirements too from the ACI code section 18.7.

Joints

During the design of joints we have considered the following condition:

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

Figure 2.3.62 Development length from ACI

This represents the development length of the bars used in the joint connection of columns with beams. Also, the depth $h > 20d_b$ to provide stronger connections.

2.3.7 Structural detailing

2.3.7.1 Development length

For bars in tension the following formula can be used to find the development length:

$$\frac{l_d}{d_b} = \frac{f_y \psi_t \psi_e \psi_s}{20 \lambda \sqrt{f'_c}} \Rightarrow l_d = 52.51 \text{ in} = 1333.77 \text{ mm} > 700 \text{ mm}$$

Since the development length is greater than the column dimension we should use hooked bars:

$$l_{dh} = \frac{f_y \psi_e \psi_s \psi_o \psi_c}{55 \lambda \sqrt{f'_c}} d_b^{1.5} = 21 \text{ in} = 533.51 \text{ mm} > 8d_b, 6 \text{ in}$$

The hook distance:

$$12d_b = 12 \text{ in} = 304.8 \text{ mm}, \text{ and stirrups spacing is } \leq 3d_b = 76.2 \text{ mm}$$

For bars in compression the following formula can be used:

$$l_{dc} = \left(\frac{f_y \psi_r}{50 \lambda \sqrt{f'_c}} \right) d_b \geq 0.0003 f_y \psi_r d_b \Rightarrow l_{dc} = 15.753 \geq 16.875 = 428.625 \text{ mm}$$

2.3.7.2 Lap splices

For Beam:

$$l_{sc} = 0.0005 f_y d_b = > l_{sc} = 953 \text{ mm for compression}$$

$$l_{st} = l_{dh} = 530 \text{ mm for tension}$$

For Column:

$$l_{sc} = 0.0005 f_y d_b = > l_{sc} = 953 \text{ mm}$$

Table 2.3.45. Lap splice for all floors

Floor, Bar Size	Lap Splice, mm
12, 6#8 (1 in)	953
8-11, 6#9 (1.128 in)	1074.42
4-7, 6#10 (1.27 in)	1209.675
1-3, 6#10 (1.27 in)	1209.675

2.3.8 Structural serviceability design

2.3.8.1 Vertical deflection

To identify if the members used in the building pass the serviceability check for deflection we can use the following formula:

$$\delta = wl^4/384EI$$

$$\text{Total deflection: } \delta = wl^4/384EI = 18.234 \text{ mm}$$

$$\text{Deflection from DL: } \delta_{DL} = 8.563 \text{ mm}$$

$$\text{Deflection due to LL: } \delta_{LL} = 9.671 \text{ mm}$$

Maximum permissible deflection according to ACI code:

$$\delta_{DL} = \frac{l}{240} = \frac{6000}{240} = 25 \text{ mm} > 8.563 \text{ mm}$$

$$\delta_{LL} = \frac{l}{360} = \frac{6000}{360} = 16.67 \text{ mm} > 9.671 \text{ mm}$$

In the same way we have calculated the deflection for different beam dimensions, and as a result the deflection limit is not exceeded, so the structure passes the deflection check.

2.3.8.2 Crack width

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

$$d_c = 1.89 \text{ in}$$

$$A = \frac{5 \times 18.6}{6} = 15.5 \text{ in}^2$$

$$w = 0.076 * 1.2 * 2/3 * 0.414 * \sqrt[3]{1.89 * 15.5} = 0.0147 \text{ in}$$

Allowable crack width for the building is 0.016 in, therefore it is acceptable.

The spacing for longitudinal reinforcement is:

$$s = 15 * \frac{40}{f_s} - 2.5C_c = 15 * \frac{40}{2/3 \times 75} - 2.5 * 1.89 = 7.275 \text{ in} = 184.785 \text{ mm}$$

This value for spacing can be taken as 180 mm.

3. Geotechnical Design

3.1. Site Location. Literature review on design codes

The building location is in Seattle, state Washington in the USA, which means that it must follow the design criterias of American Society of Civil Engineers (ASCE-7). According to GeoEngineers (2019), the site is designated as Class C in the International Building Code (IBC) 2018, which means that further seismic design studies are necessary to be conducted with all the earthquake time histories for the future structural models. As a geotechnical structure, the building must follow the design and construction requirement of the Seattle Building Code (SBC) that was adopted in 2018. According to Chapter 17 of the SBC 2018, a lot of criterias for material testing, inspections and fire-resistance code requirements are provided.

For this project the geotechnical investigation report of GeoEngineers company was used. In general, 6 boring results were used for this projects, where B-56-4, B-1, B-2 borings were done by GeoEngineers, HC-1 and HC-3 were done by Hart Crowser in 2014 and BB-9 was done by HWA GeoSciences in 1998.

3.1.1 Site description

The chosen location is in Seattle, Washington. The exact address is 401 8th Avenue N, Seattle, as shown with the red mark on Figure 1. The city is located in the northwest region of the US, surrounded by two major mountain ranges. The Cascade Range is to the east of Seattle, including Mount Rainier and Mount Baker volcanic peaks, while the Olympic Mountains are to the west of the city [1]. These mountains make the location prone to earthquakes and mudslides. Seattle is also located between the Puget Sound and Lake Washington waters, making the city being built next to the hills, water and valleys [1].

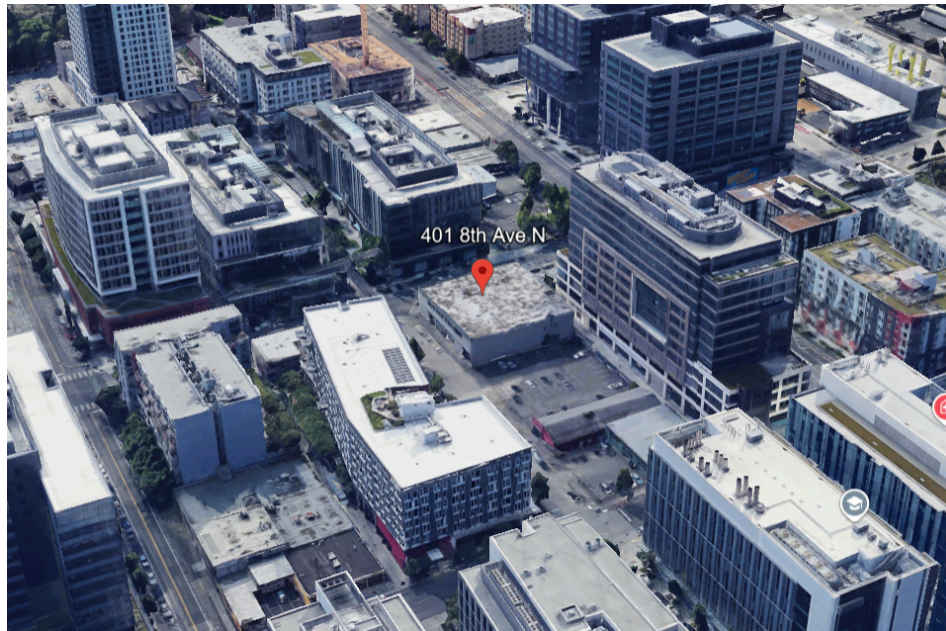


Figure 3.1.1.1. Site location

The site is between Harrison and Republican streets. The total area of the land for this project including parking is 4300 meters square. Two thirds of the land will be allocated to the 15 storey hotel buildings and 2 storey conference buildings, while one third of the land is allocated to the parking lot.

In order to assess the subsurface conditions of the soil, 3 borings were drilled, and their depths ranged from 8 meters to 18.6 feet below the ground level. Using the Standard Penetration Test (SPT) results, Atterberg limit, fines and moisture content tests were conducted. Moreover, shear wave testing results were obtained with the use of a high resolution micrometer that collected data across the site. Soil samples were taken from the borings at depths of 0.76 and 1.52 meters using a 5 cm split-barrel tool. A 63.5 kg hammer was dropped from 0.76 meters to drive the tool 0.46 meters into the ground, and the number of hammer strikes needed for every 15 cm was recorded.

Currently, the site is surrounded by 8th Avenue North and Harrison Street, and a one-story building with the parking lots occupies the location. The site slopes gently down to the north, with the elevation going from about 68 feet in the southwest to around 61 feet in the northeast. Considering the subsurface conditions of the site, the soils are relatively shallow and glacially consolidated. B-3 results showed stiff silt with sand and some gravel, while B-2 had medium dense silty sand and very stiff silt with sand.

Groundwater conditions for this site were observed for more than 5 years and according to the automatic data logger results, the groundwater depth in the B-1 ranged from

Elevation 2.26 m to 2.38 m in 1 month. After 5 years, groundwater level raised to Elevation 5.24 m, and currently it is recommended by the GeoEngineers group to design groundwater Elevation 6 meters. Considering the liquefaction of the soil, which is the condition of the soil where excess pore pressures develop in a saturated soil usually due to vibrations of earthquakes, it exists on the location. The analysis done by the group of engineers showed that the soil on the given area is under a low risk of liquefaction due to its high density and gradation.

3.2 Field Investigation

Geographic location of Seattle city is the city that is surrounded by mountain ranges often being the reason for earthquakes and seismic activity. According to the USGS (United States Geological Survey) Seismic Hazard Map, Seattle is located at the highest hazard location compared to other parts of the USA. According to the report data, the shear wave velocity was identified to be 602 m/sec, as shown in the Table 3.1 below.

Table 3.2.1. Change of shear wave velocity by depth [4].

Depth (m)		S-wave velocity (m/sec)			
top	bottom	Array A	Array B	Array C	Array D
0	1.3716	283.0678	244.5106	414.8633	169.4078
1.3716	3.2004	279.8064	262.951	473.6592	318.455
3.2004	5.6388	288.0665	352.5317	519.1658	385.4196
5.6388	8.6868	308.6405	349.5446	530.0777	465.643
8.6868	12.192	438.7596	368.1984	530.0472	478.8103
12.192	20.7264	476.5548	478.5665	585.155	525.1399
20.7264	31.3944	603.1992	663.8849	602.1324	630.936
31.3944	49.04232	623.2246	644.9568	608.6246	602.1324
49.04232	68.2752	617.4334	635.9957	607.4969	640.4762

The location was divided into 4 arrays, which can be seen in Figure 3.2 below. These arrays represent different shear waves.



Figure 3.2.1. Seismicity analysis [4]

In order to better visualize the shear wave results, the graphs were built. Figure 3.3 shows the combination of all 4 arrays used in this testing. Here, Array C and D show similar results.

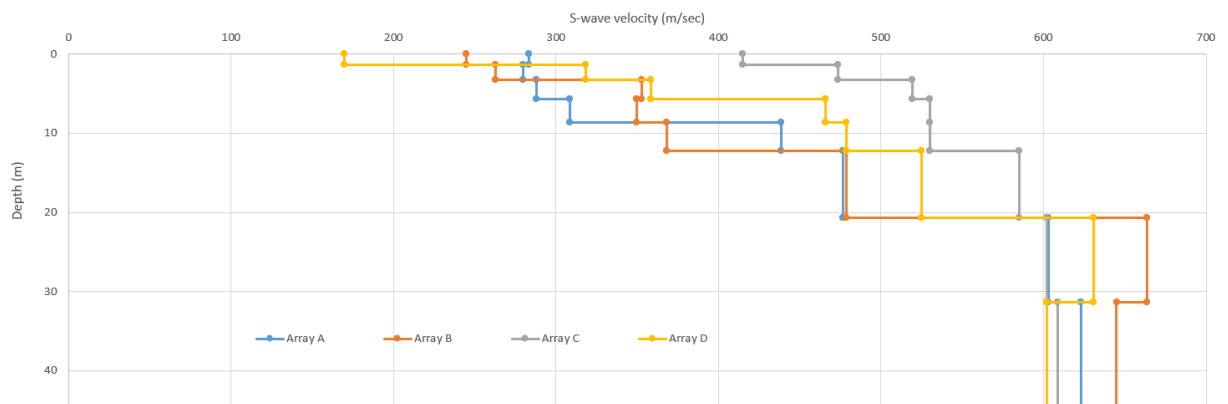


Figure 3.2.2. Combination of Array A, B, C and D shear wave velocity

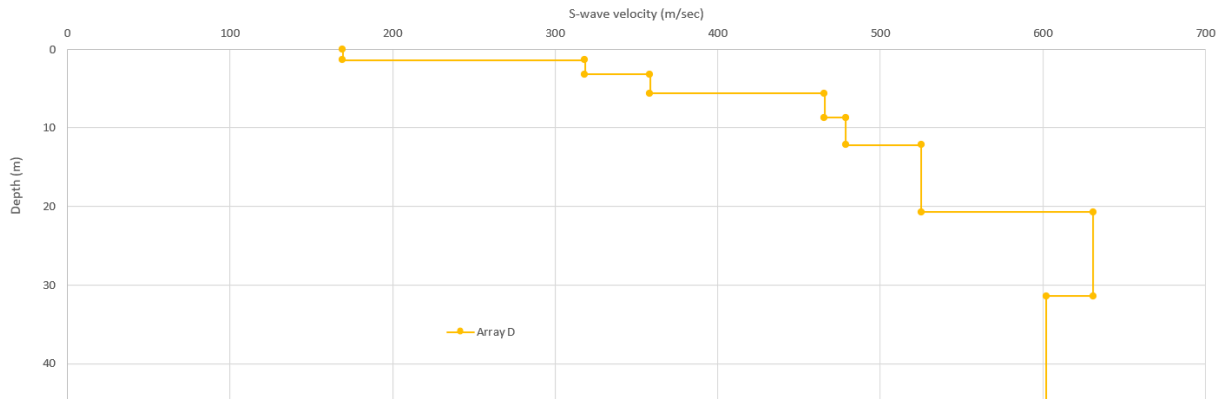


Figure 3.2.3. Array D shear wave velocity

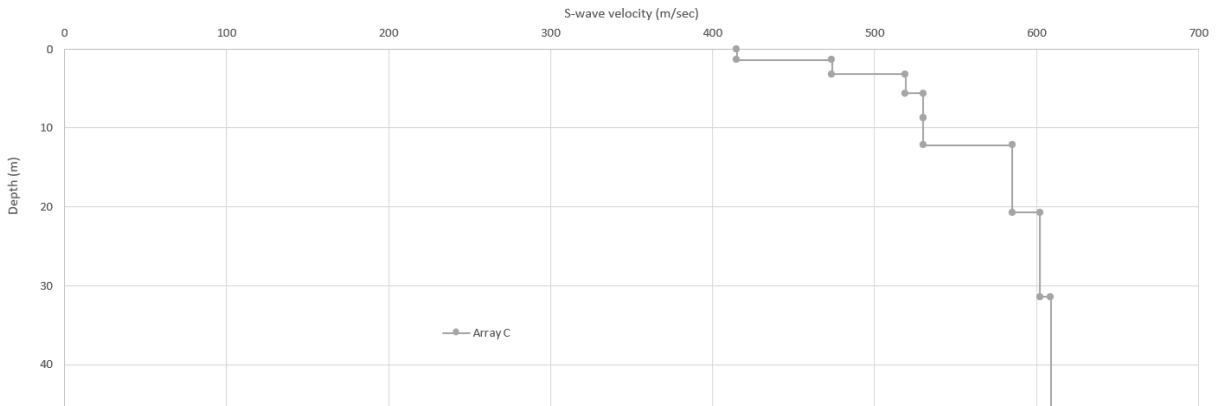


Figure 3.2.4. Array C shear wave velocity

When transforming 103 feet into meters, values of shear wave velocity for 30 meters can be obtained. The values of shear wave velocity for 30 meters are 2044.7 feet/sec or 623m/sec (A array), 2116.0 feet/sec or 645 m/sec (B array), 1996.8 feet/sec or 609 m/sec(C array), 1975.5 feet/sec or 602 m/sec (D array).

Among these values, it is necessary to get the least value, as it indicates the weakest soil part in the site, which is 602 m/sec, which belongs to the D array.

Taking the average of shear wave velocity values for A-D arrays in the chosen location, 620 m/sec value is obtained.

3.3 Soil Profile Description

The geotechnical report by GeoEngineers (2019) includes the data for soil characteristics from the field exploration in which the 4 boring holes were drilled by the same company in 2014-2015 years. Additionally, this report covers the data of other 6 borings that are located in near areas of construction. They were done by several companies in different periods: Hart Crowser (2015), GeoEngineers (2013) and HWA GeoSciences (1998). In order to obtain more

detailed information about the soil all of 10 boreholes' data were analyzed. The summary of the soil profile according to the analysis could be seen in the Table 3.2 below:

Table 3.3.1. Soil profile data from geotechnical report

Depth, m	ΔH , m	SPT N value, blows/foot	Soil group classification	Description	Unit weight saturated, kN/m^3	Moisture content, %
3.0	3.0	38	SM	Silty SAND	20.8	18
6.7	3.7	47	ML	Inorganic clayey silty SAND with slight plasticity	19.5	11.2
12.2	5.5	45	CL	Inorganic CLAY of low to medium plasticity	17.8	20.9
14.1	1.9	29	CH	Inorganic CLAY of high plasticity	20	34.6
17.0	2.9	44	SC	Dense, gravelly clayey fine to coarse SAND, moist	21.5	10.8
19.7	2.7	49	SM	Dense to very dense, gravelly, silty fine to coarse SAND, wet	17.5	13.4
24.1	4.4	50	SP SM	Very dense, slightly gravelly, slightly fine to coarse sand, wet	22.3	17.3
24.7	0.6	50	SM	Very dense, silty gravelly sand, wet	17.8	12

For the soil profile analysis data obtained from the geotechnical report was not sufficient. Therefore the soil properties such as its friction angle, cohesion, Poisson ratio and modulus of elasticity were calculated. During calculations the corrected values of the SPT were used.

The correction procedure was started with the Cestari (1996) method in which the following formulas were used:

$$N_{60} = N_{SPT} \cdot CE$$

$$CE = \frac{ER}{60}$$

$$N_{1(60)} = N_{SPT} \cdot CE \cdot CN \cdot CR \cdot CB \cdot CS$$

In which the factors could be spelled out in a way showed below (Catanzariti, 1969):

- CE - Energy correction factor;

→ ER - Hammer energy transfer ratio according to ASTM D-4633-86;

Automatic hammer ER for North and South America ER(%) = 55-83

ER/60 = 0.92 - 1.38; ave.= 1.15;

- CN - Overburden correction factor;

In order to calculate the CN the methods of Liam & Withman (1986) and Skempton (1986) were used respectively:

$$CN = \frac{Pa}{\sqrt{\sigma'_v}}$$

$$CN = \frac{2}{1 + \sigma'_v/100}$$

- CR - Rod length correction factor; taken as 1 because length of the cathead is 30 inches in drop;
- CB - Borehole diameter correction factor; taken as 1 because the diameter of boreholes is 2 inches;
- CS - Liner correction factor; taken as 1 because it is assumed that used sampler is standard.

For the calculation of the friction angle three different equations were used. The research article which reviews the correlation of friction angle value with SPT-N values was the source for these equations (AbuSerriya and Osman, 2023). The formulas for Hatanaka & Uchida (1996), Briaud & Gibbens (1997), Tand et al. (1994) and Mahmoud (2013) methods are written respectively:

$$\phi = (20 \cdot N)^{0.5} + 20$$

$$\phi = 7.1 \cdot \log_{10} N + 29.8$$

$$\phi = 0.209 \cdot N + 19.68$$

In the process of obtaining values for Poisson ratio, cohesion and Young Modulus of soil the method was different. In case of Poisson ratio and cohesion value the article has included the graphical relation of the value with SPT N results which consisted of 300 data points (Kumar et al., 2016). For the calculation of the Poisson ratio and cohesion the method of the best fitting curve based on the graphical relation was used. The case for Young Modulus calculation is similar to the method described above but additionally, it has the divisions by the country in relation to soil profiles (Yamamoto and Togtokhbuyan, 2018). All the results for the calculated values are shown in Table 3.2.

Table 3.3.2. Calculation of the soil properties.

Soil type		SM	ML	CL	CH	SC	SM	SP SM	SM
N_{60}		43	54	51	33	50	56	57	57
$(N_1)_{60}$		39	49	46	30	46	51	52	52
$\phi, ^\circ$	Hatanaka & Uchida (1996)	47.9	51.3	50.3	44.5	50.3	51.9	52.2	52.2
	Briaud & Gibbens (1997), Tand et al. (1994)	41.1	41.8	41.6	40.3	41.6	41.9	42	42
	Mahmoud (2013)	27.8	29.9	29.3	26.0	29.3	30.3	30.6	30.6
Poisson ratio		0.40	0.45	0.43	0.35	0.43	0.46	0.46	0.46
Young Modulus, MPa		11.7	14.7	13.8	9.0	13.8	15.3	15.6	15.6
Cohesion, kN/m^3		22.7	24.1	23.7	21.3	23.7	24.5	24.6	24.6

As the used methods were empirical it was preferred to choose the least values, for example for friction angle the method which had high results in comparison was excluded from taking it into the consideration. Consequently, the average of the other two methods were calculated and was chosen to be used in following calculations procedures in the geotechnical design part. Moreover, as one of the needed variables further the effective stress was

calculated for each layer of soil (Das and Sivakugan, 2019). The example shown below is done for the second layer and the rest of the layers follow the same procedure.

$$\sigma' = \gamma h_1 + \gamma' h_2 = 20.8 * 3 + (19.5 - 9.81) * 3.7 = 98.3 \text{ kN/m}^2$$

The summary of the calculated values with the data obtained from the geotechnical report can be seen from Table 3.3.3:

Table 3.3.3. Soil profile.

Depth, m	ΔH , m	N	N_{60}	$(N_1)_{60}$	USCS	Soil type	γ , $\frac{kN}{m^3}$	w, %	ϕ , °	c' , $\frac{kN}{m^2}$	μ	E , MPa	$\sigma'_{0'}$, $\frac{kN}{m^2}$
0-3	3	38	43	39	SM	Silty SAND	20.8	18	34.5	22.7	0.40	11.7	62.4
3-6.7	3.7	47	54	49	ML	SILT	19.5	11.2	35.9	24.1	0.45	14.7	98.3
6.7-12.2	5.5	45	51	46	CL	CLAY of low plasticity	17.8	20.9	35.5	23.7	0.43	13.8	142.2
12.2-14.1	1.9	29	33	30	CH	CLAY of high plasticity	20	34.6	33.2	21.3	0.35	9.0	161.6
14.1-17	2.9	44	50	46	SC	Clayey SAND	21.5	10.8	35.5	23.7	0.43	13.8	195.5
17-19.7	2.7	49	56	51	SM	Silty SAND	17.5	13.4	36.1	24.5	0.46	15.3	216.2
19.7-24.1	4.4	50	57	52	SP SM	Poorly graded, silty SAND	22.3	17.3	36.3	24.6	0.46	15.6	271.2
24.1-24.7	0.6	50	57	52	SM	Silty SAND	17.8	12	36.3	24.6	0.46	15.6	276.0

3.4 Foundation Design

3.4.1 Shallow foundation

For the shallow foundation the analysis of resistance towards the loads was done for the pad and mat types of foundation. The total load exerted on foundation was calculated as 6674.74 kN for the interior, 5414.21 kN for exterior and 3833.85 kN for corner columns in the structural area of the project. Firstly, the ultimate bearing capacity was calculated based on the soil properties; followingly, it was used to check the result obtained with the factor of safety.

According to Das and Sivakugan (2019) the factor of safety value for the shallow foundation design that could be considered to be appropriate is 3.

$$q_{all} = \frac{q_u}{FS}$$

Bearing capacity calculation

The basic form of the bearing capacity equation for shallow foundations was originally developed by Terzaghi and later modified by Meyerhof (1963). The ultimate bearing capacity q_u expressed as:

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

Where,

c' = cohesion

q = effective stress at the foundation bottom

γ = unit weight of soil

B = foundation width

N_c, N_q, N_γ = bearing capacity factors

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

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors

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

Shape factors calculation formulas:

Meyerhof (1963) introduced shape, depth, and inclination factors to refine the basic Terzaghi equation for different types of foundations (such as strip, square, or circular footings):

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

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

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

Depth factors calculation formulas when $\phi' \geq 0$:

$$F_{cd} = F_{qd} - \frac{1-F_{qd}}{N_c \tan \phi'}$$

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

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

Load inclination factors:

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

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

Where,

q = effective stress at the foundation bottom

$$q = \gamma D_f$$

D_f = foundation depth

B = foundation width

L = foundation length

Bearing capacity factors:

Bearing capacity factors are dimensionless coefficients used in bearing capacity equations to account for the effects of soil strength, particularly cohesion, overburden pressure, and unit weight, on the ultimate load a foundation can support. These factors depend primarily on the soil's internal friction angle and are essential for determining the contributions of cohesion, surcharge, and self-weight of the soil to the total bearing capacity.

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

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

$$N_i = 2(N_q + 1) \tan \phi$$

3.4.1.1 Pad foundation

Calculations:

The specificity of the calculation process for pad foundation is that length and width of the foundation footings will be the same and can be taken as B. During the calculation this B will be found in order to assess the safety of the foundation. The needed values can be seen from the list below. Embedded depth for the pad foundation (square shaped footings) was assumed as 1.5 m:

$$D_f = 1.5 \text{ m}$$

$$FS = 3$$

$$\beta = 0$$

$$\phi' = 17.5^\circ$$

$$\gamma = 20.2 \text{ kN/m}^3$$

$$c = 22.7 \text{ kN/m}^2$$

Bearing capacity factors:

$$N_c = 12.7$$

$$N_q = 5.0$$

$$N_\gamma = 3.8$$

Shape factors:

$$F_{cs} = 1 + \frac{5.0}{12.7} = 1.39$$

$$F_{qs} = 1 + \tan 17.5^\circ = 1.31$$

$$F_{\gamma s} = 1 - 0.4(B/B) = 0.6$$

Depth factors when $\phi' \geq 0$:

$$F_{qd} = 1 + 2 \tan 17.5^\circ (1 - \sin 17.5^\circ)^2 \left(\frac{1.5}{B}\right) = 1 + 0.462555/B$$

$$F_{cd} = \frac{B+0.57807}{B}$$

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

Load inclination factors:

$$F_{ci} = F_{qi} = F_{\gamma i} = 1$$

$$q = \gamma D_f = 20.2 \text{ kN/m}^3 * 1.5 \text{ m} = 30.3 \text{ kN/m}^2$$

$$\begin{aligned} q_u &= [22.7 * 12.7 * 1.39 * \frac{B+0.57807}{B} * 1] + [30.3 * 5 * 1.31 * (1 + \frac{0.462555}{B}) * 1] + \\ &+ [\frac{1}{2} * 20.2 * B * 3.8 * 0.6 * 1 * 1] = \\ &= [\frac{400.7231B+231.6460}{B}] + [\frac{198.465B+91.8001}{B}] + [23.028B] = \\ &= 23.028B^2 + 599.1881B + 323.4461 \end{aligned}$$

$$q_{all} = \frac{q_u}{3} = \frac{Q_{all}}{B^2}$$

Interior column

$$\frac{23.028B^2+599.1881B+323.4461}{3} = \frac{6674.74}{B^2}$$

$$B = 2.95 \text{ m}$$

The ultimate bearing capacity:

$$q_u = 23.028(2.95)^2 + 599.1881(2.95) + 323.4461 = 2291.45 \text{ kN/m}^2$$

The ultimate bearing load:

$$Q_u = q_u * A = 2291.45 \text{ kN} * 2.95\text{m} * 2.95\text{m} = 19941.34 \text{ kN}$$

The allowable load:

$$Q_{all} = \frac{Q_u}{FS} = \frac{19941.34 \text{ kN}}{3} = 6647.12 \text{ kN}$$

Exterior column:

$$\frac{23.028B^2+599.1881B+323.4461}{3} = \frac{5414.21}{B^2}$$

$$B = 2.75 \text{ m}$$

The ultimate bearing capacity:

$$q_u = 23.028(2.75)^2 + 599.1881(2.75) + 323.4461 = 2145.36 \text{ kN/m}^2$$

The ultimate bearing load:

$$Q_u = q_u * A = 2145.36 \text{ kN} * 2.75\text{m} * 2.75\text{m} = 16224.29 \text{ kN}$$

The allowable load:

$$Q_{all} = \frac{Q_u}{FS} = \frac{16224.29 \text{ kN}}{3} = 5408.1 \text{ kN}$$

Corner column:

$$\frac{23.028B^2 + 599.1881B + 323.4461}{3} = \frac{3833.85}{B^2}$$

$$B = 2.44 \text{ m}$$

The ultimate bearing capacity:

$$q_u = 23.028(2.44)^2 + 599.1881(2.44) + 323.4461 = 1922.57 \text{ kN/m}^2$$

The ultimate bearing load:

$$Q_u = q_u * A = 1922.57 \text{ kN} * 2.44\text{m} * 2.44\text{m} = 11446.21 \text{ kN}$$

The allowable load:

$$Q_{all} = \frac{Q_u}{FS} = \frac{11446.21 \text{ kN}}{3} = 3815.4 \text{ kN}$$

In order to check the safety of pad foundation the obtained allowable loads for the columns were compared to the design load. As we can see from Table 3.3.1 below square footings with the calculated measurements are not suitable for resisting the loads that are exerted by the building structures.

Table 3.4.1.1. Safety check for the pad foundation.

Column type	Interior	Exterior	Corner
B, m	2.95	2.75	2.44
Allowable load, kN	6647.1	5408.1	3815.4
^		^	^
Design load, kN	6674.7	5414.21	3833.9
NOT SAFE			

3.4.1.2 Mat foundation

In comparison to the pad type mat foundation consists of a single continuous slab. The area of the mat foundation is the same with the hotel area: 50 m x 30 m. The smaller side of 30 m is considered as B (Das and Sivakugan, 2019) which means that L is 50 m. Also, the embedded depth for the mat foundation is assumed as 1m. Besides the equation to identify the ultimate bearing capacity, in analysis of mat foundation the net ultimate capacity was calculated by using equations:

$$q_{u(net)} = q_u - q$$
$$q_{u(net)} = \frac{N_{60}}{0.08} F_d \left(\frac{S_e}{25} \right)$$

Calculations:

FS = 3;

Maximum design load: interior column = 6674.7 kN;

Columns in total = 54;

Bearing capacity factors

$$N_c = 12.7$$

$$N_q = 5.0$$

$$N_\gamma = 3.8$$

Shape factors:

$$F_{cs} = 1 + \frac{30}{50} + \frac{5.0}{12.7} = 1.99$$

$$F_{qs} = 1 + \left(\frac{30}{50} \right) \tan 17.2^\circ = 1.19$$

$$F_{\gamma s} = 1 - 0.4 \left(\frac{30}{50} \right) = 0.76$$

Depth factors when $\phi' \geq 0$:

$$F_{qd} = 1 + 2 \tan 17.2^\circ (1 - \sin 17.2^\circ)^2 \left(\frac{1.0}{30} \right) = 1.01$$

$$F_{cd} = 1.01 - \frac{1-1.01}{12.7 \tan 17.2^\circ} = 1.01$$

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

Load inclination factors:

$$F_{ci} = F_{qi} = F_{\gamma i} = 1$$

$$q = \gamma D_f = 20.8 \text{ kN/m}^3 * 1.0 \text{ m} = 20.8 \text{ kN/m}^2$$

$$q_u = [22.7 * 12.7 * 1.99 * 1.01 * 1] + [20.8 * 5.0 * 1.19 * 1.01 * 1] + 0.5[20.8 * 30 * 3.8 * 0.76 * 1 * 1] = 1605.487671 = 1605.5 \text{ kN/m}^2$$

$$q_u = 1605.5 \text{ kN/m}^2$$

$$q_{u(net)} = q_u - q = 1605.5 - 20.8 = 1584.7 \text{ kN/m}^2$$

$$q_{all(net)} = \frac{q_{u(net)}}{FS}$$

$$q_{all(net)} = 528.2 \text{ kN/m}^2$$

$$1500 \text{ m}^2 / 54 = 27.78$$

$$q_{design \text{ load}} = 6674.7 / 27.78 = 240.3 \text{ kN/m}^2$$

Embedded depth is on 1 m = SM which is silty sand therefore the settlement is considered:

$$F_d = 1 + 0.33(D_f/B) = 1 + 0.33(1/30) = 1.011$$

$$q_{u(net)} = \frac{N_{60}}{0.08} [1 + 0.33(\frac{D_f}{B})](\frac{S_e}{25}) = \frac{39}{0.08} [1 + 0.33(\frac{1}{30})](\frac{50}{25}) = 985.725 \text{ kN/m}^2$$

The ultimate bearing load:

$$Q_u = q_u * A = 985.725 \text{ kN/m}^2 * 30 \text{ m} * 50 \text{ m} = 1478587.5 \text{ kN} = 1478.6 \text{ MN}$$

The allowable load:

$$Q_{all} = \frac{Q_u}{FS} = \frac{1478587.5 \text{ kN}}{3} = 492862.5 \text{ kN} = 492.9 \text{ MN}$$

$$Q_{design} = q_u * A = 528.2 * 30 * 50 = 792300 \text{ kN} = 792.3 \text{ MN}$$

$$Q_u = q_u * A = 985.725 \text{ kN/m}^2 * 30 \text{ m} * 50 \text{ m} = 1478.6 \text{ MN}$$

$$Q_{all} = \frac{Q_u}{FS} = 492.9 \text{ MN}$$

$$Q_{design} = q_u * A = 528.2 * 30 * 50 = 792300 \text{ kN} = 792.3 \text{ MN}$$

$$Q_{all} < Q_{design}$$

As design load is greater than the allowable load for mat foundation also it can be concluded that shallow foundation is not safe to construct the hotel building. For this reason the foundation design continues with the deep foundation type.

3.4.2. Deep foundation selection

When designing a structure's foundation, several key factors must be taken into account, including the type of structure, soil characteristics, depth of the water table, and the loads it must support. Previously, pad, mat, and pile foundations were analyzed in terms of their load-bearing capacity and settlement behavior. Although the shallow foundation demonstrated sufficient load-bearing strength, it exhibited excessive settlement. As a result, the pile foundation was selected to effectively transfer the structural load to the underlying soil. Additionally, a deep foundation was preferred due to its superior resistance to lateral forces, particularly those caused by seismic activity.

3.4.2. Foundation Design under Axial Loading

3.4.2.1. Hand calculations of axial bearing capacity

It is important to build a deep foundation to support the weight of the construction. For instance, deep foundations can be empirical in cases of weak soils, heavy loads from the

structures, or in challenging situations considering structural stability. For these cases, the ultimate load (Q_u) for the piles can be calculated from the sum of shaft resistance (Q_s) and point bearing capacity (Q_p) as follows:

$$Q_u = Q_s + Q_p$$

These values are essential in the deep foundation calculation process as they determine how much of the loads can a pile carry to ensure the stability of the structure. Skin friction capacity determines the friction between the surface of the pile and soil to handle cohesive soils being the primary load-bearing mechanism. Point bearing capacity, in turn, provides the load-bearing capacity at the pile's tip when it touches dense soil layers.

In order to properly design a deep foundation, pile's diameter and length, multiple iterations and calculations need to be done. It is important that the design loads are less compared to the allowable loads. As a factor of safety (FoS) is usually taken as the value ranging from 2 to 3, it is taken to be 3.

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

Reinforced concrete piles are chosen in the design of deep foundations as they can support heavy loads by transferring high building loads to deeper soils. Reinforcement with steel is necessary to enhance the pile's strength and durability as they are usually applied in the construction of high-rise buildings and heavy structures. While the concrete provides compressive strength to allow piles to bear axial loads, steel reinforcements provide tensile strength to withstand lateral loads.

There are multiple types of reinforced concrete piles such as precast concrete piles, cast-in-place concrete piles and prestressed concrete piles. Precast concrete piles are usually manufactured in an off-site factory and transported to the construction site, and driven into the ground. On the other hand, cast-in-place concrete piles are made directly on the construction site by first drilling the hole, installing reinforcement and pouring concrete into the hole. Another type is prestressed concrete piles that are prestressed before the pouring of concrete to handle higher bending forces. It can be concluded that the precast concrete piles are the best option for this building as they have a higher quality and consistency due to the factory production. Moreover, they are installed fast, can still handle large loads, and cause minimal soil disturbance as they are directly driven into the ground.

Construction engineers often choose between the two the most appropriate shapes of piles as circular piles and rectangular piles. Circular piles are more advantageous for this specific project due to the uniform load distribution, pile driving and high axial loads. Due to their shapes, circular piles can distribute loads across their cross section by both reducing soil displacement during the installation and higher resistance to the axial loads. Comparatively, rectangular piles are suited for carrying high lateral loads. As the building contains many floors, axial loads are the priority and therefore, circular piles are chosen for this project.

For the excavation purposes, basement and pile cap thicknesses are necessary. Building's basement is 3.5 m in height, and the thickness of the design pile cap is taken to be 1.2 m. Taking into account the rule of thumb. The minimum thickness of the pile should be twice the pile's diameter to ensure correct load distribution. It is recommended in the geotechnical report provided by GeoEngineers (2019) that the embedded portion of the soldier piles need to be at least 0.4 m in diameter and extend to a minimum distance of 3 m below the basement. Therefore, it is necessary that the excavation is more than 4.2 m.

For a circular pile with a diameter of 0.4 m, the area and perimeter would be as follows:

$$A = \pi * \frac{d^2}{4} = \pi * \frac{0.4^2}{4} = 0.1256 \text{ m}^2$$

$$P = \pi D = 1.256 \text{ m}$$

In order to properly estimate the pile's bearing capacity, more than 1 method is needed. The point-bearing capacity is calculated using Meyerhof's, Vesic's, and Coyle & Castello's methods.

3.4.2.1.1 Single pile

Point bearing capacity in sand

Point bearing capacity calculations are done due to the pile length of 12 meters, but the tip of the pile will be located at the depth of 16.2 m. However, calculations to the other lengths are also done.

1. Meyerhof's method.

Friction angle for the soil layer at a depth of 16.2 m is 35° , N_q^* is 143, p_a is 101.325 kPa. In order to calculate other factors, following equations are used:

$$Q_p = A_p q_p = A_p q' N_q^* \leq A_p q_l$$

$$q_l = 0.5 p_a N_q^* \tan \phi'$$

Where,

p_a = atmospheric pressure (101.325kPa);

N_q^* = bearing capacity factor;

ϕ' = effective angle of soil's internal friction;

$$q' = 3.7 * 19.5 + 5.5 * 17.8 + 1.9 * 20 + (10 + 4.2 - 14.1) * 21.5 = 210.2 \text{ kN/m}^2$$

$$Q_p = A_p q' N_q^* = 0.1256 \text{ m}^2 * 210.2 \text{ kN/m}^2 * 143 = 3500 \text{ kN}$$

$$Q_l = A_p q_l = 0.1256 * 0.5 * 101.325 * 143 * \tan(35^\circ) = 637.15 \text{ kN} < 3500 \text{ kN}$$

Final value of the point bearing capacity is $Q_p = 3500 \text{ kN}$

2. Vesic's method

Point bearing capacity of the soil will be calculated using the following equation:

$$Q_p = A_p q_p = A_p \bar{\sigma}_o' N_\sigma^*$$

Where, average effective vertical stress is calculated as follows and is measured in $[\text{kN/m}^2]$

and N_σ^* is a bearing capacity factor:

$$\bar{\sigma}_o' = \left(\frac{1+2(1-\sin\phi')}{3} \right) q'$$

$K_0 = (1 - \sin\phi')$ - earth pressure coefficient;

Reduced rigidity index of the soil is calculated using the formula below:

$$I_{rr} = \frac{I_r}{1+I_r \Delta}$$

$$I_r = \frac{E_s}{2(1+\mu_s) q' \tan \phi'}$$

$E_s = m * p_a$ - $m=200$, which is a medium dense soil. Therefore, $E_s = 20000 \text{ MPa}$;

Poisson ratio for the friction angle at a range of 25-45 degrees:

$$\mu_s = 0.1 + 0.3 \left(\frac{\phi' - 25}{20} \right)$$

Average volumetric strain is calculated as follows:

$$\Delta = 0.005(1 - \frac{\phi' - 25}{20}) \frac{q'}{p_a}$$

Calculations:

$$K_0 = (1 - \sin \phi') = 1 - \sin 35^\circ = 0.41221$$

$$\mu_s = 0.1 + 0.3(\frac{35-25}{20}) = 0.25$$

$$\Delta = 0.005(1 - \frac{35-25}{20}) \frac{q'}{p_a} = 0.005(1 - 0.5) * 210.2/101.325 = 0.005255$$

$$I_r = \frac{E_s}{2(1+\mu_s)q' \tan \phi'} = \frac{20000}{2(1+0.25)*210.2*\tan(35^\circ)} = 51.277$$

$$I_{rr} = \frac{I_r}{1+I_r \Delta} = \frac{51.277}{1+51.277*0.005255} = 40.393$$

$$\sigma_o' = (\frac{1+2*0.426424}{3}) * 210.2 = 129.823 \text{ kN/m}^2$$

$$N_\sigma^* = 53.899$$

$$Q_p = A_p \sigma_o' N_\sigma^* = 0.1256 * 129.823 * 53.899 = 3203.3 \text{ kN}$$

Therefore, the value is $Q_p = 3203.3 \text{ kN}$

3. Coyle and Castello's method

$$Q_p = q' N_q^* A_p$$

Embedment ratio (L/D) is taken into account in this method, where $10/0.4=25$. N_q^* was identified to be 48 according to the graph in the book. Therefore, the calculations are as follows:

$$Q_p = q' N_q^* A_p = 210.2 * 48 * 0.1256 = 1267.25 \text{ kN}$$

4. SPT Meyerhof's method

$$Q_p = A_p q_p = A_p * 0.4 * p_a N_{60} \frac{L}{D} \leq 4 p_a N_{60}$$

Average value of the standard penetration number was taken to determine the point bearing capacity at 4D below and 10D above distances.

$$N_{60} = \frac{46+30+46+51+52}{5} = 45$$

$$Q_p = A_p * 0.4 * p_a N_{60} \frac{L}{D} = 0.1256 * 0.4 * 101.325 * 45 * 25 = 5726.89 \text{ kN}$$

$$Q_p = 4p_a N_{60} = 4 * 101.325 * 45 = 5154.2 \text{ kN}$$

$$5726.89 \text{ kN} \geq 5154.2 \text{ kN}, \text{ therefore } Q_p = 5726.89 \text{ kN}$$

5. SPT Briaud method

$$Q_p = A_p q_p = A_p * 19.7 p_a N_{60}^{0.36}$$

N60 value is also taken as average, therefore,

$$Q_p = 0.1256 * 19.7 * 101.325 * 45^{0.36} = 987.03 \text{ kN}$$

In order to compare the results obtained using 5 different methods, the table is provided below:

Table 3.4.1.1. Point bearing capacity in sand.

Pile length, m	Depth, m	Layer	Meyerhof, kN	Vesic, kN	Coyle and Costello, kN	SPT Meyerhof, kN	SPT Briaud, kN
10	14.2	Clayey SAND SC	3500.62	1310.94	1041.85	5684.47	1014.77
11	15.2	Clayey SAND SC	3954.29	1596.29	1176.87	6252.91	1014.77
12	16.2	Clayey SAND SC	4407.96	1847.74	1311.892	7075.89	1028.54
13	17.2	Silty SAND SM	4199.06	1727.64	1249.72	8989.10	1090.63

The Coyle and Costello's method was chosen for the bearing capacity values as other methods deviate from the calculated values.

Frictional resistance of Sand

Frictional resistance in sand is the force that opposes the movement of an object or surface due to the interaction between sand particles and the object. Here, it is calculated using general equation 12.15 and Coyle and Costello's methods.

1. Coyle and Costello's method

$$Q_s = f_{av} pL = (\overline{K\sigma'_o \tan \delta'}) p \Delta L$$

Where,

σ'_o - average effective vertical stress, kN/m²;

K - effective earth pressure coefficient;

Soil-pile friction angle is calculated as follows:

$$\delta' = 0.8\phi'$$

$$Q_s = (\overline{K\sigma'_o \tan \delta'}) p \Delta L = 2 * 48.75 * \tan(0.8 * 35^\circ) * 1.256 * 1.884 * 10 = 345.611 \text{ kN}$$

2. Equation 12.15 method

$$Q_s = \sum f_{av} pL = \sum K \sigma'_o \tan \delta' p \Delta L$$

$K = 1.5$, which is effective earth pressure coefficient for a pile concrete;

f_{av} - frictional resistance [kN/m²];

ΔL - delta soil layer [m];

σ'_o - effective vertical stress [kN/m²];

$$\text{For } z=0 \text{ to } L': f = K \sigma'_o \tan(\delta')$$

$$\text{For } z=L' \text{ to } L: f = f_{z=L'}$$

$$f = 1.5 * 72.15 * \tan(0.8 * 36^\circ) = 59.5 \text{ kN/m}^2$$

$$Q_s = 112.8 * 3.149 * 0.6 * 3.7 = 788.48$$

The values calculated for each of the layer are presented in the table below:

Table 3.4.2.1.2. Skin friction due to Eq12.15.

Layer #	Depth, m	Soil Layer	Delta H, m	Effective stress, kN/m ²	$f, \text{ kN/m}^2$	$f_{av}, \text{ kN/m}^2$	$Q_s, \text{ kN}$
1	3	SAND	3	0	0	56.39	319.65
2	6.7	SAND	3.7	72.15	59.5	112.8	788.48
3	12.2	CLAY	5.5	170.05	135.6	166.1	172.5.8
4	14.1	CLAY	1.9	208.05	154.9	196.5	705.53

5	17	SAND	2.9	270.4	215.7	238.2	1304.9
6	19.7	SAND	2.7	317.65	261.9	260.6	1329.6
7	24.1	SAND	4.4	415.77	342.8	259.3	2156.1
8	24.7	SAND	0.6	426.45	351.6	175.8	199.33
						ΣQ_s	8529

3. Meyerhof's SPT method

$$f_{av} = 0.02 * 100 * N_{60} ave$$

$$Q_s = L * p * f_{av}$$

4. Briaud's SPT method

$$f_{av} = 0.224 * 100 * N_{60} ave^{0.29}$$

$$Q_s = L * p * f_{av}$$

Skin friction for the sand layer is calculated using 4 different methods and results are shown below for piles with different lengths:

Table 3.4.2.1.3. Qs in Sand layers.

Pile length, m	Depth, m	Layer	Meyerhof SPT, kN	Briaud SPT, kN	Coyle and Castello, kN	Eq 12.15, kN
10	14.2	SAND	1122.03	846.71	3360.69	736.2
11	15.2	SAND	1234.23	931.37	3631.32	3477.9
12	16.2	SAND	1396.67	1026.9	4066.99	4807.6
13	17.2	SAND	1774.31	1165.06	3878.60	6232.4

Point bearing capacity of Clay layer.

The pile length in this case was decided to be 9 m in length and 13.2 m in depth, and is located in a clay layer. This value was calculated using 2 different methods:

1. Meyerhof's method

$$Q_p = A_p c_u N_c^* = 9 A_p c_u$$

$c_u = 21.3 \text{ kN/m}^2$ at the depth of 14.2 m;

Therefore, $Q_p = 9 A_p c_u = 9 * 0.1256 * 21.3 = 24.077 \text{ kN}$

2. Vesic's method

This method for clay is similar to that of the sand. Reduced rigidity index is also calculated, together with Young's Modulus.

$$Q_p = A_p c_u N_c^*$$

$$I_{rr} = I_r = \frac{E_s}{3c_u} = \frac{9000}{3*21.3} = 140.845$$

$$N_c^* = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1 = \frac{4}{3} (\ln(140.845) + 1) + \frac{\pi}{2} + 1 = 10.5$$

$$Q_p = A_p c_u N_c^* = 0.1256 * 21.3 * 10.5 = 28.09 \text{ kN}$$

Table 3.4.2.1.4. Point bearing capacity of clay layer.

Pile length, m	Depth, m	Layer	Meyerhof's, kN	Vesic's, kN
7	11.2	CLAY	26.79	32.53
8	12.2	CLAY	24.08	28.09
9	13.2	CLAY	24.08	28.09

Frictional resistance of clay layer.

For the calculation of the frictional resistance in clay, alpha, lambda and beta methods were used.

1. λ method

During this method, vertical stresses and embedment lengths are used to calculate skin resistance of the soil layer.

$$f_{av} = \lambda(\bar{\sigma}_o + 2c_u)$$

$$Q_s = p L f_{av}$$

$$\bar{\sigma}_o = \frac{c_{u1}L_1 + c_{u2}L_2 + c_{u3}L_3 + \dots}{L}$$

Where,

c_u - undrained cohesion value [kN/m²];

$\bar{\sigma}_o$ - average effective vertical stress value [kN/m²];

λ - value from the table, dependent on the length of embedment;

$$\bar{\sigma}_o = \frac{769.785 + 648.96 + (208.05 + (9 + 4.2 - 12.2) * 20 + 208.05) / 2 * (9 + 4.2 - 12.2)}{9 + 4.5} = 123.99 \text{ kN/m}^2$$

$$\sigma_o' = \frac{0 + 133 + 666 + 359 + 693 + 793 + 1613 + 252}{24.7 \text{ m}} = 189.69 \text{ kN/m}^2$$

$$c_u = \frac{21.3 * (9 + 4.2 - 12.2)}{9 + 4.2 - 12.2} = 21.3 \text{ kN/m}^2$$

$$f_{av} = 0.2632(182.69 + 2 * 21.3) = 59.297$$

$$Q_s = pL f_{av} = 1.884 * 9 * 59.29 = 1005.45 \text{ kN}$$

2. α method

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

$$Q_s = 0.909785 * 21.3 * 1.256 * (9 + 4.2 - 12.2) * 2 = 48.68 \text{ kN}$$

Together with the previous layers, this would be $Q_s = 815.35 \text{ kN}$

3. β method

$$Q_s = \sum (1 - \sin \phi_R') \tan \phi_R' \sigma_o' p \Delta L$$

$$f_{av} = 0.2957 * ((121.1 + 121.1 + (9 + 4.2 - 12.2) * 20 / 2)) = 38.768 \text{ kN/m}^2$$

$$Q_s = (1 - \sin 35) \tan 35 * 121.1 * 1.884 * (9 + 4.2 - 12.2) = 68.128 \text{ kN}$$

Summing Q_s values of the previous layers, $Q_s = 8573 \text{ kN}$

Table 3.4.2.1.5. Skin friction in Clay layer.

Pile length, m	Depth, m	Layer	Lambda method, kN	Alpha method, kN	Beta method, kN
7	11.2	CLAY	9438.6	8747.0	8555.1
8	12.2	CLAY	9505.3	8747.0	8558.1
9	13.2	CLAY	9534.9	8598.8	8573.2

Summary of all the loads calculated are shown below. The allowable bearing capacity of single piles are calculated and their value is less than column load values. Therefore they are not suitable as a single piles.

Table 3.4.2.1.6. Summary of all the loads.

Pile length, m	Depth, m	Layer	Qp, kN	Qs, kN	Qu, kN	Qall, kN
7	11.2	CLAY	66.7	8913.6	8980.3	2993.4
8	12.2	CLAY	58.7	8936.8	8995.5	2998.5
9	13.2	CLAY	58.7	8902.3	8961.0	2986.9
10	14.2	SAND	2851.3	3360.69	6212.0	2070.7
11	15.2	SAND	3273.9	3631.32	6905.2	2301.7
12	16.2	SAND	3720.8	4066.99	7787.8	2595.9
13	17.2	SAND	4566.8	3878.60	8445.1	2815.0

Table 3.4.2.1.7. Structural loads table.

Column	At, m²	Pu, kN
Exterior	18	3742.59
Interior	36	6106.19
Corner	9	2278.18

Concluding the analysis, it would be better to use the deep pile with a length of 12 m, as the interior columns need to carry extra load.

It was determined before that the shallow foundation and deep foundation with a single pile is not satisfactory for the loads applied to the structure. The critical loads to take into account can be seen below:

Table 3.4.2.1.8. Loads applied to the column.

Column type	Area, m ²	Pu, kN	Wu, kN/m ²
Exterior	18	5069.61	241.74
Interior	36	7761.80	215.61
Corner	9	2990.59	252.48

Group Piles

To undergo the axial compression applied the pile group configuration was designed. The pile group efficiency that carries the load-bearing capacity calculated by formula:

$$\eta = \frac{Q_{g(u)}}{\sum Q_u}$$

where,

η = group efficiency

$Q_{g(u)}$ = ultimate load – bearing capacity of the group pile

Q_u = ultimate load – bearing capacity of the each pile without the group effect

Based on the spacing within the members of the pile group piles could act in different ways: as a block and as individual piles. As the Capstone I single pile analysis showed it is not suitable for carrying the design load. Therefore, in this case the pile group acts as a block with dimensions $L_g \times B_g \times L$, and frictional capacity as $f_{av} p_g L \approx Q_{g(u)}$ (here the p_g is the perimeter of the cross section for the block = $2(n_1 + n_2 - 2)d + 4D$; f_{av} = average unit frictional resistance). Noting this the simplified analysis for group efficiency calculated as:

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} = \frac{f_{av}[2(n_1+n_2-2)d+4D]L}{n_1 n_2 p L f_{av}} = \frac{2(n_1+n_2-2)d+4D}{p n_1 n_2}$$

In case center spacing d is large enough group efficiency will be greater than 1. If $\eta < 1$, it

means $Q_{g(u)} = \eta \sum Q_u$ and if $\eta \geq 1$, then $Q_{g(u)} = \sum Q_u$.

Allowable bearing capacity of the pile group $Q_{g(all)}$ is dependent on the ultimate bearing capacity $Q_{g(u)}$ of the pile group and factor of safety.

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

Group piles under pile cap with dimensions:

$$L_g = (n_1 - 1)d + 2(D/2)$$

$$B_g = (n_2 - 1)d + 2(D/2)$$

Table 3.4.2.1.9. Obtained group pile parameters.

		Exterior Column	Interior Column	Corner Column
Pile length	L, m	12	12	12
Pile diameter	D, m	0.4	0.4	0.4
Minimum spacing	d, m	1.2	1.2	1.2
N of piles	n1	3	3	2
N of piles	n2	2	2	2
Width of pile group	Bg, m	1.6	1.6	1.6
Length of pile group	Lg, m	2.8	2.8	1.6
Perimeter	P, m	4.0	4.0	3.2
Area	A, m2	0.13	0.13	0.13

The efficiency of the groups of piles are calculated using 3 different methods and the average is taken:

1. Simplified method

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

2. Los Angeles GA method

$$\eta = 1 - \frac{D}{\pi dn_1n_2} [n_1(n_2 - 1) + n_2(n_1 - 1) + \sqrt{2}(n_1 - 1)(n_2 - 1)]$$

3. Converse-Laborre method

$$\eta = 1 - \left[\frac{(n_1-1)n_2 + (n_2-1)n_1}{90n_1n_2} \right] \theta$$

$$\theta(deg) = \tan^{-1}(D/d)$$

Table 3.4.2.1.10. Efficiency with different methods.

Group	$\eta_{Simplified}$	$\eta_{Los\ Angeles\ GA}$	$\eta_{Converse-Laborre}$	$\eta_{average}$
-------	---------------------	---------------------------	---------------------------	------------------

3x3	0.99	0.79	0.73	0.84
3x2	1.0	0.83	0.76	0.86
2x2	1.0	0.86	0.79	0.88
2x1	1.0	0.95	0.9	0.95

$$Q_{g(u)} = \sum Q_u = 3 * 2 * (3952 + 3562) = 45084 \text{ kN for the exterior column}$$

$$Q_{g(all)} = \frac{Q_{g(u)}}{FS} = \frac{45084 \text{ kN}}{3} = 15028 \text{ kN} > 5069.61 \text{ kN}$$

Therefore, the bearing capacities of the pile groups are more than the design and are suitable for this project.

Table 3.4.2.1.11. Group piles bearing capacity.

	η	Qp, kN	Qs, kN	Qu, kN	sum(Qu), kN	Qall, kN	Qdesign, kN
Exterior	0.86	3952	3562	7514	45084	15028	3742.60
Interior	0.86	3952	3562	7514	45084	15028	6336.20
Corner	0.88	3325	3105	6430	25720	8573	2278.19

The following Figures represent the placing of each pile group relying on the building's layout.

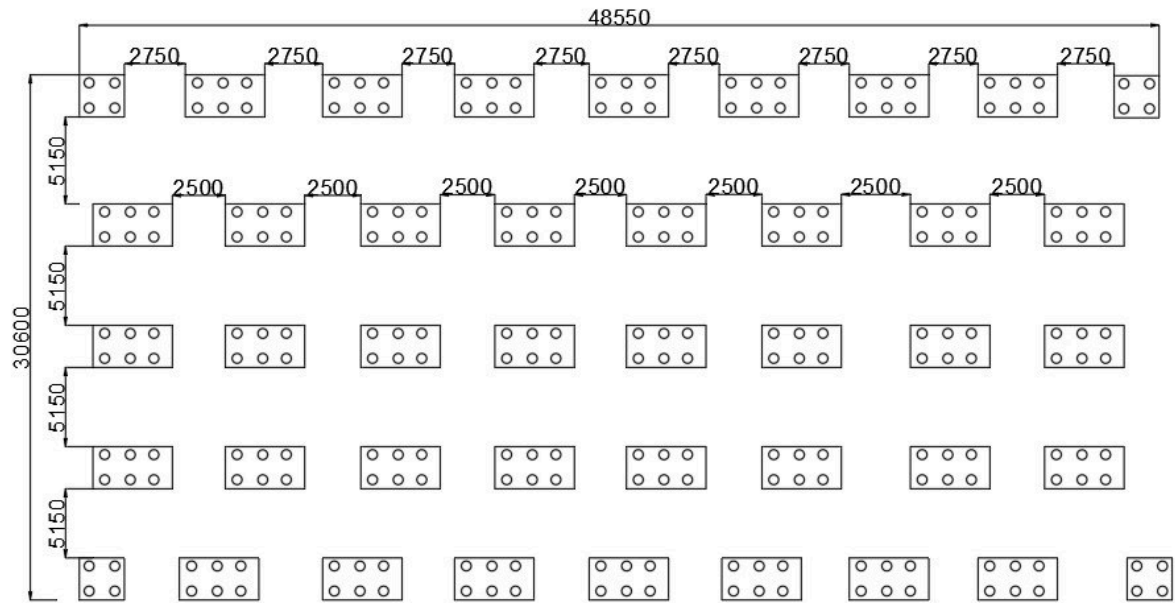


Figure 3.4.1. Group Piles distribution in the building.

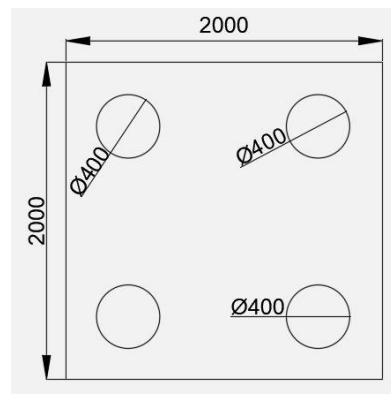


Figure 3.4.2. Group pile under Corner Column

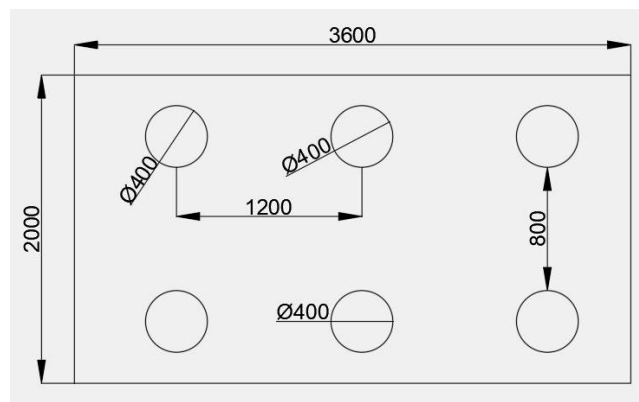


Figure 3.4.3. Group Pile under Exterior and Interior Columns

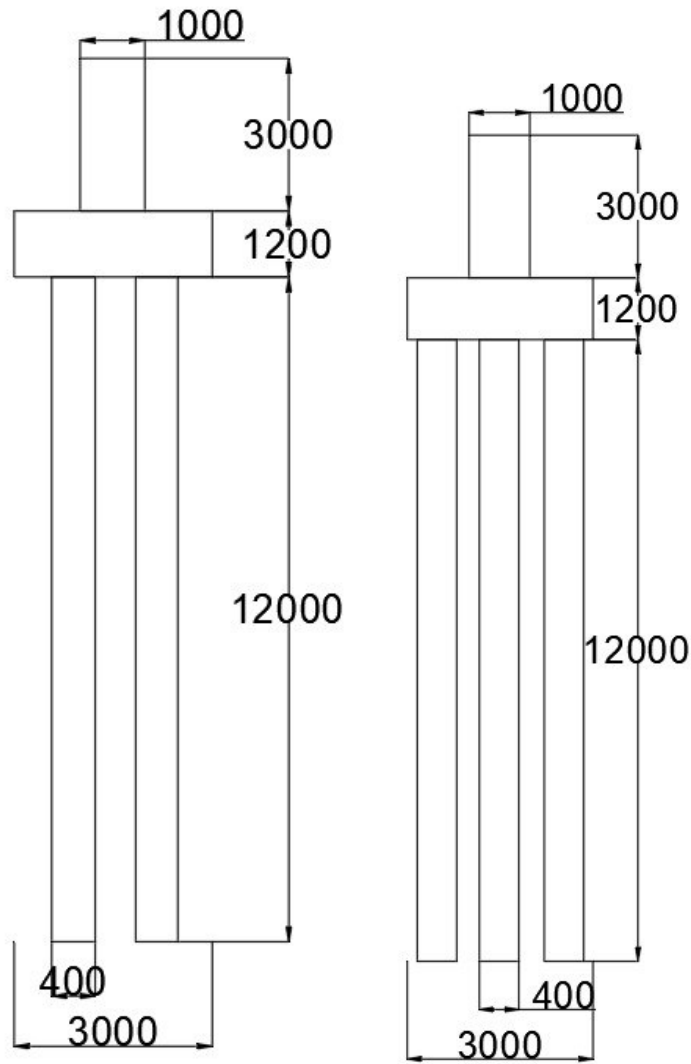


Figure 3.4.4. Side view of the pile groups.

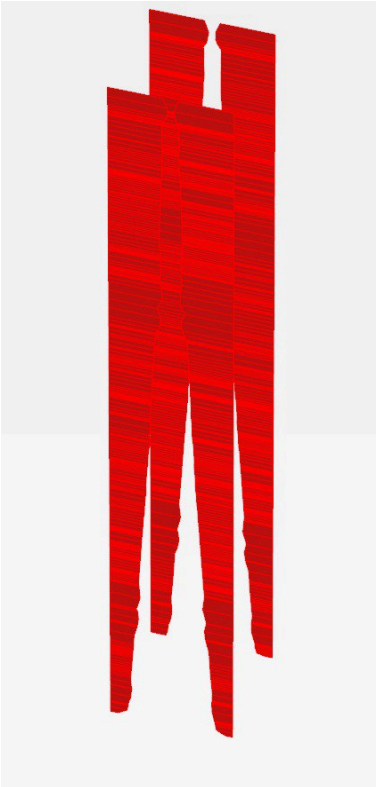
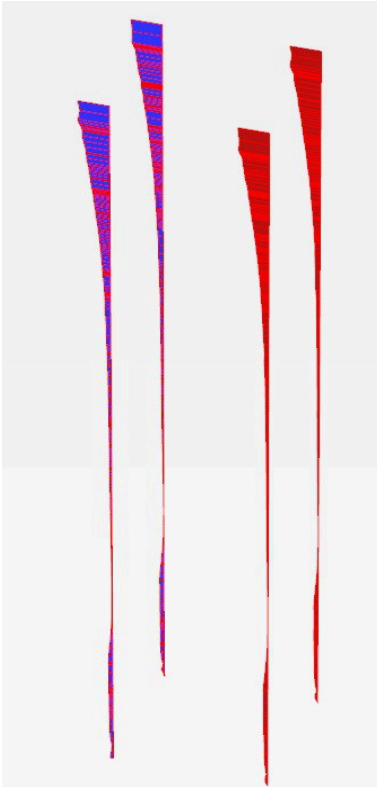
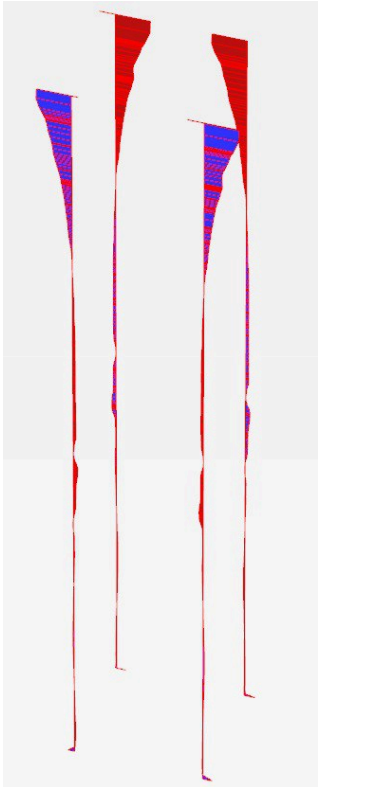
3.4.2.2 Software analysis of pile groups: PLAXIS 3D

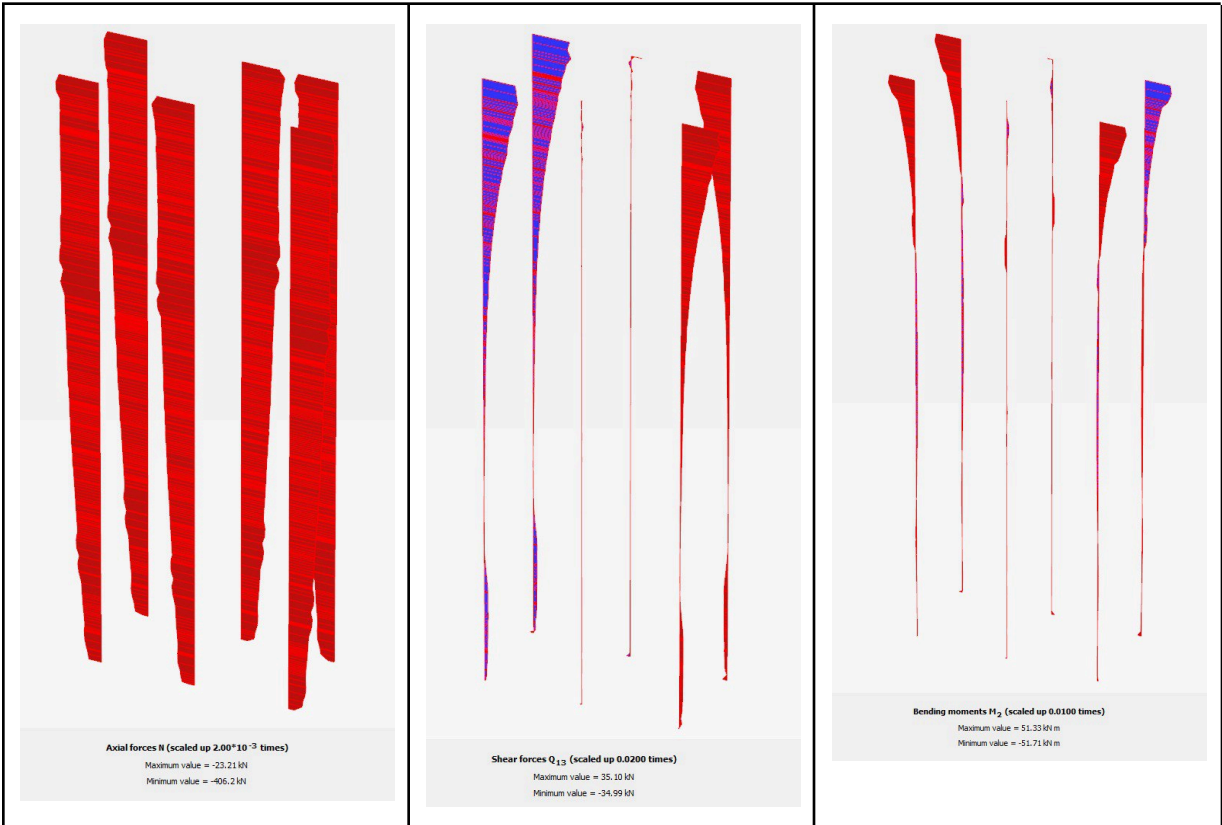
Plaxis 3D was used to check the loads and displacements of the pile group in the project. The pile group was modeled, and various load conditions were applied to assess the response of the piles. The displacements and stresses in the piles were analyzed to ensure that they remained within acceptable limits. The results were evaluated to verify the stability and performance of the pile group under different loading scenarios.

1. The soil profile was created in **Plaxis 3D**, defining soil layers with their properties such as strength, stiffness, and depth. This provided the foundation for modeling pile interactions with the soil.
2. A 3D model of the pile group was developed, and various load conditions (vertical, lateral, and moment loads) were applied to simulate real-world forces on the piles.

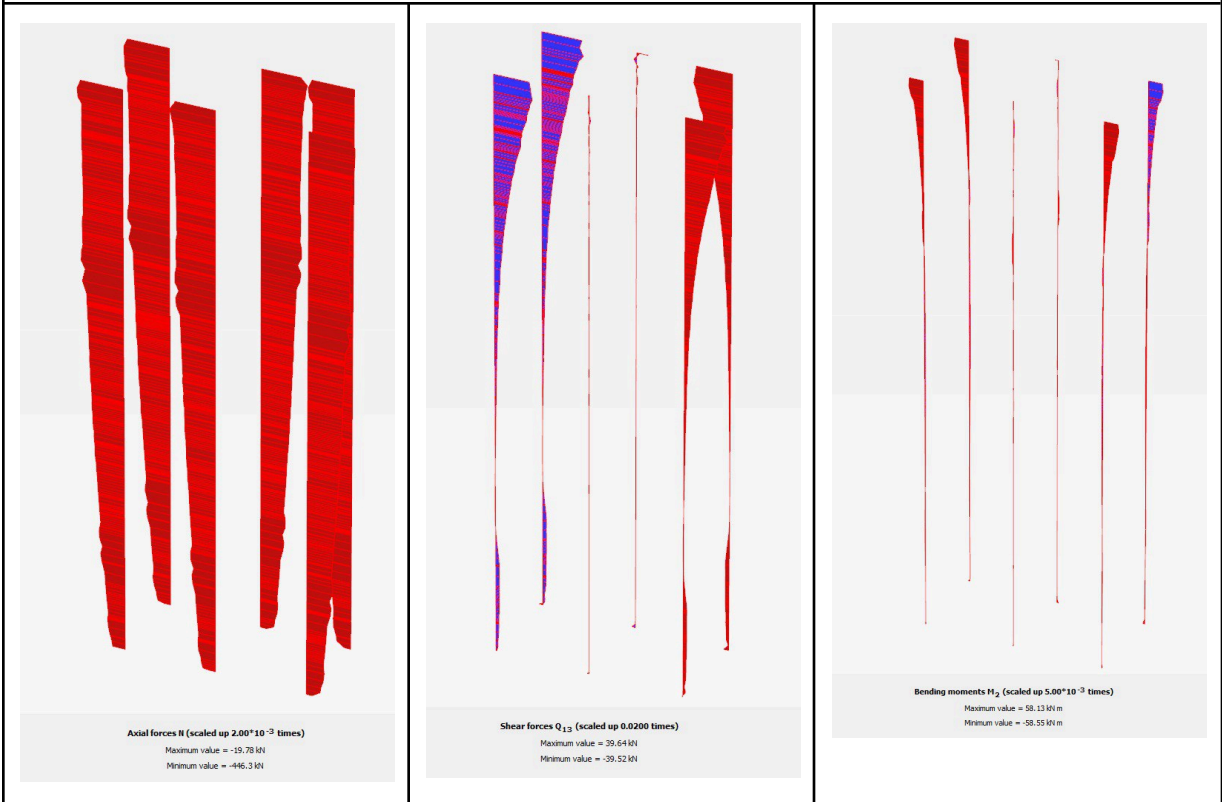
3. The simulation was run to analyze the resulting displacements, stresses, and load distribution in the pile group. The results were reviewed to ensure the pile group performed within acceptable limits.
4. The pile group's behavior was evaluated based on the results, and adjustments were made to optimize the design, ensuring it could safely support the expected loads.

Table 3.4.2.2. Pile Groups analysis results from Plaxis 3D

Corner column		
 Axial forces N (scaled up $2.00 \cdot 10^{-3}$ times) Maximum value = -53.84 kN Minimum value = -426.0 kN	 Shear forces Q_{13} (scaled up 0.0200 times) Maximum value = 17.88 kN Minimum value = -17.88 kN	 Bending moments M_3 (scaled up 0.0200 times) Maximum value = 24.09 kN m Minimum value = -24.34 kN m
Exterior column		



Interior column



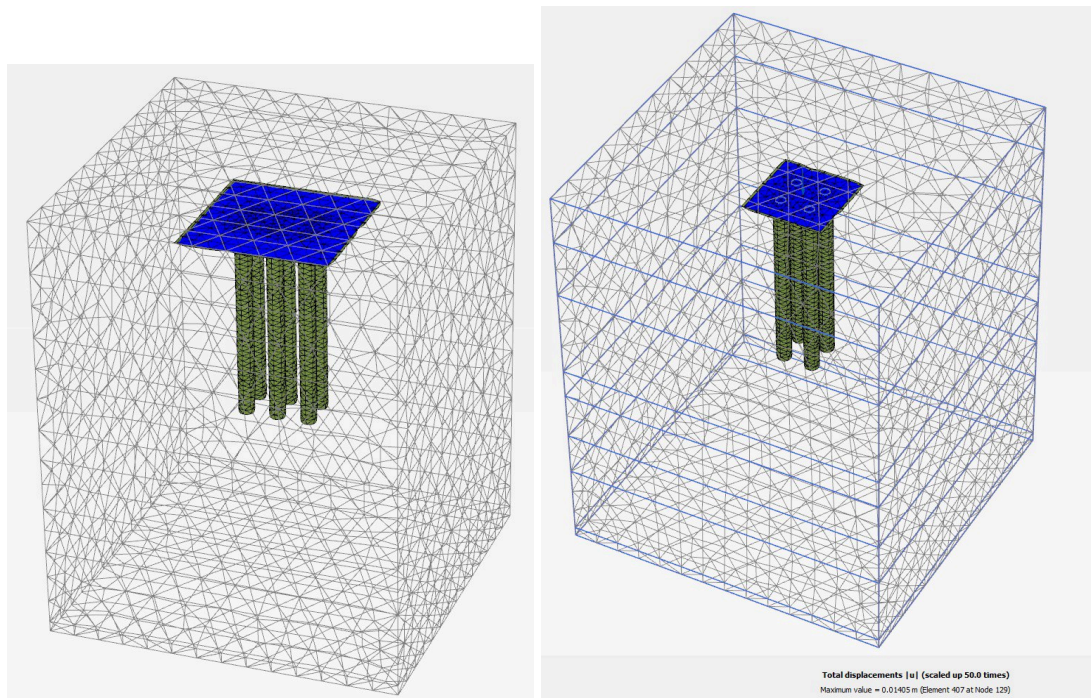


Figure 3.4.2.2. Pile Groups Visualization in Plaxis 3D

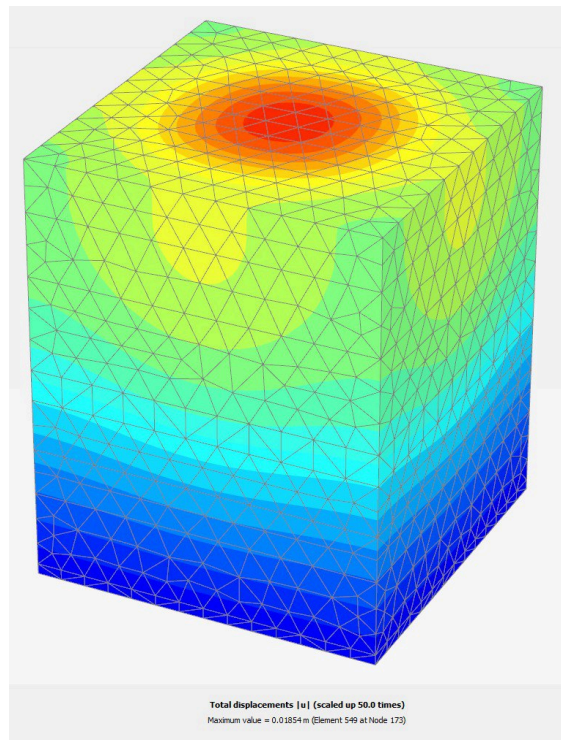


Figure 3.4.2.2.1. Total Displacement of the Pile Group in Soil

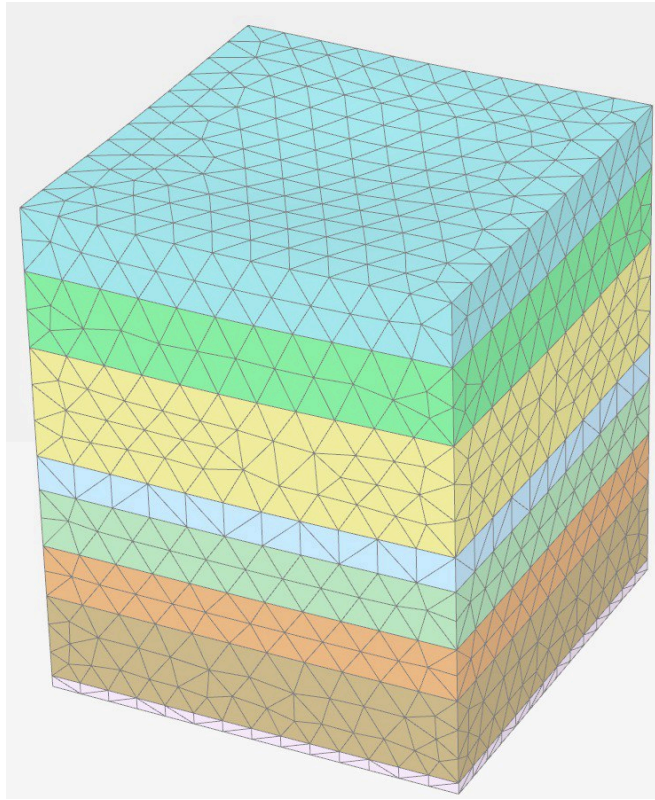


Figure 3.4.2.2.2. Soil Displacement with the Loads

3.4.2.3 Software analysis of pile groups: GEO5

At the start the soil profile data was entered to the software in accordance with the elevation and layers of soil. After that the pile group parameters were selected such as diameter, length and formation of the piles. The water table is introduced also. The next step was to enter the loads from the structure in respective columns. The demonstration of the soil profile data with the pile group could be seen from the figure below.

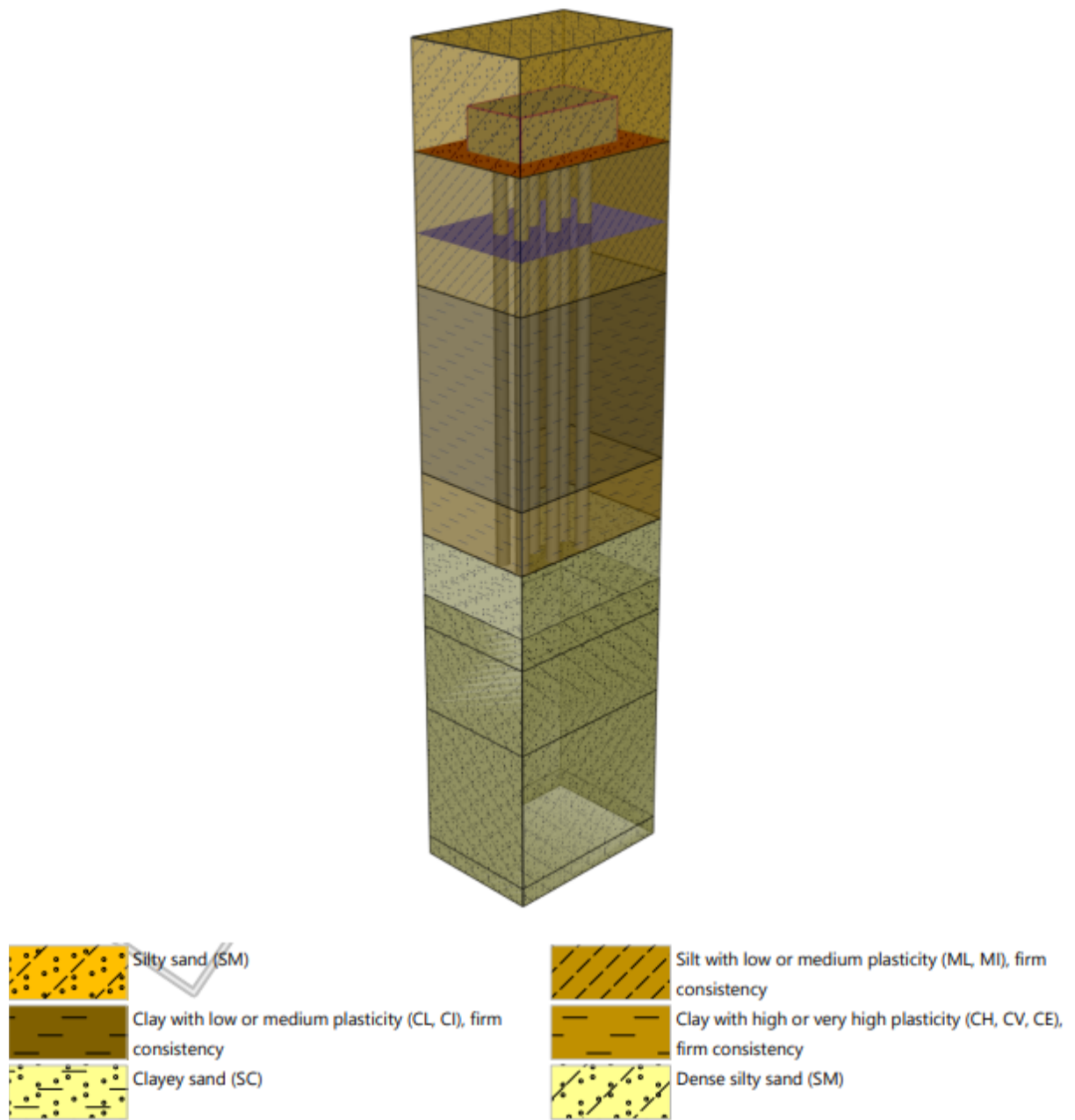


Figure 3.4.2.3.1. Soil profile by GEO5 with pile group.

As all the data was entered the analysis for the bearing capacity check was done. The results are satisfactory for all columns as can be seen below:

Average undrained shear strength along the piles $c_{us} = 23,25$ kPa
 Undrained shear strength at base of pile group $c_{ub} = 21,30$ kPa
 Cohesion group bearing capacity factor $N_{cg} = 8,36$
 Vertical bearing capacity of pile group $R_g = 6153,19$ kN
 Maximum vertical force $V_d = 2529,69$ kN
 Safety factor = $2,43 > 2,00$

Vertical bearing capacity of pile group is SATISFACTORY

Figure 3.4.2.3.2. Bearing capacity check for interior column by GEO5.

Average undrained shear strength along the piles $c_{us} = 23$,
 Undrained shear strength at base of pile group $c_{ub} = 21$,
 Cohesion group bearing capacity factor $N_{cg} = 8$,
 Vertical bearing capacity of pile group $R_g = 6153$,
 Maximum vertical force $V_d = 2603$,
 Safety factor = $2,36 > 2,00$

Vertical bearing capacity of pile group is SATISFACTORY

Figure 3.4.2.3.3. Bearing capacity check for exterior column by GEO5.

Average undrained shear strength along the piles $c_{us} = 23,25$ kPa
 Undrained shear strength at base of pile group $c_{ub} = 21,30$ kPa
 Cohesion group bearing capacity factor $N_{cg} = 9,00$
 Vertical bearing capacity of pile group $R_g = 4274,29$ kN
 Maximum vertical force $V_d = 1875,82$ kN
 Safety factor = $2,28 > 2,00$

Vertical bearing capacity of pile group is SATISFACTORY

Figure 3.4.2.3.4. Bearing capacity check for corner column by GEO5.

3.4.2.4 Hand calculations of settlement

As the deep foundation type was chosen for the hotel construction it is significant to calculate and check the settlement. By doing that we can justify the measurements for the pile parameters:

$$L = 12 \text{ m}$$

$$D = 0.4 \text{ m}$$

3.4.1 Settlement for single pile

The elastic settlement for single pile was calculated using the method by Vesic (1977) which consists of 3 components (Das and Sivakugan, 2019):

The three elastic components are:

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)}$$

1. Elastic shortening of the pile $[s_{e(1)}]$:

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} = \frac{(4267 + 0.67 * Q_{ws})L}{A_p E_p}$$

$$s_{e(1)} = \frac{(1522.28 + 0.67 * 2077.48) * 12}{0.126 * 30 * 10^6} = 9.25 * 10^{-3} m$$

Where,

Q_{wp} = working load at the pile point = $4566.84/3 = 1522.28 \text{ kN}$

Q_{ws} = working load along the pile shaft = $6232.44/3 = 2077.48 \text{ kN}$

A_p = cross – sectional area of the pile point = $\pi * R^2 = 0.126 \text{ m}^2$

L = pile length = 12 m

E_p = modulus of elasticity of the pile material = 30 GPa

ξ = a constant in the range of 0.50–0.67, depending on the skin friction distribution

2. Settlement of the pile due to the working load (Q_{wp}) at the pile point [$s_{e(2)}$]:

$$s_{e(2)} = \frac{q_{wp} D}{E_s} (1 - \mu_s^2) I_{wp}$$

$$s_{e(2)} = \frac{12081.59 * 0.4}{15600} (1 - 0.46^2) 0.85 = 0.208 \text{ m}$$

Where,

D = width or diameter of pile

q_{wp} = point load per unit area at pile point = $Q_{wp}/A_p = 1522.28/0.126 = 12081.59 \text{ kN/m}^2$

E_s = modulus of elasticity of soil at or below the pile point

μ_s = Poisson's ratio of soil

I_{wp} = influence factor ≈ 0.85

3. Settlement of the pile due to working load (Q_{ws}) along the pile shaft [$s_{e(3)}$]:

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws}$$

$$s_{e(3)} = \left(\frac{12081.59}{1.26 \times 12}\right) \frac{0.4}{15600} (1 - 0.46^2) 3.92 = 0.0633 \text{ m}$$

Where,

p = *perimeter of the pile*

L = *embedded length of pile*

I_{ws} = *influence factor*

$$I_{ws} = 2 + 0.35 \sqrt{\frac{L}{D}} = 2 + 0.35 \sqrt{\frac{12}{0.4}} = 3.92$$

The total settlement is calculated by summing obtained 3 components:

$$s_e = 9.25 \times 10^{-3} + 0.208 + 0.0633 = 0.281 \text{ m} = 281 \text{ mm}$$

According to the Skempton and MacDonald (1956) for adjacent columns the maximum permissible settlement is 25 mm. As 152 mm is too high value the building can not have a foundation consisting of only single piles.

3.4.2 Settlement for group of piles

For determining the settlement for group of piles the Meyerhof method (1976) was used:

$$s_{g(e)}(mm) = \frac{0.96q\sqrt{B_g I_g}}{N_{60}}$$

Where,

$$q = Q_g / (L_g B_g)$$

L_g and B_g = *length and width of the group pile section, respectively (m)*

N_{60} = *average standard penetration number within seat of settlement*

($\approx B_g$ deep below the tip of the piles)

I = influence factor

$$I = 1 - L/8B_g \geq 0.5$$

$$I = 1 - L/8B_g \geq 0.5 = 1 - 12/8 * 2 = 0.25$$

L = length of embedment of piles (m)

Calculations:

- Interior:

$$q = 6674.74/(3.6 * 2) = 926.99 \text{ kN/m}^2$$

$$s_{g(e)}(mm) = \frac{0.96 * 926.99 * \sqrt{2 * 0.25}}{57} = 11.04 \text{ mm}$$

- Exterior:

$$q = 5414.21/(3.6 * 2) = 751.97 \text{ kN/m}^2$$

$$s_{g(e)}(mm) = \frac{0.96 * 751.97 * \sqrt{2 * 0.25}}{57} = 8.96 \text{ mm}$$

- Corner:

$$q = 3833.85/(2 * 2) = 958.46 \text{ kN/m}^2$$

$$s_{g(e)}(mm) = \frac{0.96 * 958.46 * \sqrt{2 * 0.25}}{57} = 11.41 \text{ mm}$$

All settlement values for columns are less than 25 mm which means that the design loads could be carried safely by the deep foundation consisting of a group of piles. Moreover, the fact that settlement values are in the range of permissible values justifies that the diameter of the piles was chosen correctly.

3.4.2.1. Software analysis of settlement

The pile groups for each column type were constructed in the softwares GEO5 and PLAXIS 3D. After entering the loads applied onto structure the analysis done by the software. The figures provided below demonstrates the procedure and the obtained results:

Max. vertical force includes self-weight of pile cap.

The depth of substitute found. $d = 8,00$ m
Maximum service load $N = 558,72$ kN
Depth of influence zone $h = 2,35$ m
Settlement of pile group $s = 8,7$ mm

Figure 3.4.2.1.1. Settlement of pile group for interior column by GEO5.

The depth of substitute found. $d = 8,00$ m
Maximum service load $N = 545,72$ kN
Depth of influence zone $h = 2,32$ m
Settlement of pile group $s = 8,4$ mm

Figure 3.4.2.1.2. Settlement of pile group for exterior column by GEO5.

The depth of substitute found. $d = 8,00$ m
Maximum service load $N = 448,72$ kN
Depth of influence zone $h = 2,08$ m
Settlement of pile group $s = 6,4$ mm

Figure 3.4.2.1.3 Settlement of pile group for corner column by GEO5.

All obtained values for settlement of pile groups are less than the allowable maximum settlement of 25 mm for all columns. From comparison of values obtained from hand calculation and software analysis the GEO5 results showed minimum settlement. It could be explained by the features of the software including the opportunity to construct a soil profile with layers which have different soil parameters, design code parameters for steel and concrete structures and the presence of the water table and its effect.

Table 3.4.2.1.1. Comparison of the settlement results.

Column type	Settlement in columns		
	Hand calculations, mm	GEO5, mm	PLAXIS 3D, mm
Interior	11.04	8.7	20.41
Exterior	8.96	8.4	18.54
Corner	11.41	6.4	14.02

3.4.3 Foundation Design under Lateral Loading

After the axial loading the piles should be analyzed for lateral loading as well. The analysis starts with the lateral loads description applied to the structure which can be summarized in Table below:

Table 3.4.3.1. Lateral loads on critical joints.

Combination	Column	V, kN	F_x, kN	F_y, kN	$M_x, kN \cdot m$	$M_y, kN \cdot m$
1.2D+1Ey+L+0.25	Corner	2334	-93.4	-13.6	6.7	-502.8
1.2D+1Ey+L+0.25	Exterior	2325	-111.2	4.2	-21.1	-527.1

1.2D+1.6L+0.55	Interior	2292	0.3	-1.1	1.6	0.6
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3.4.3.1 Calculation of the lateral bearing capacity

The laterally loaded piles are analyzed for ultimate bearing capacity by method of Broms (1965). According to the method the next assumptions are applied: (a) for short piles shear failure in soil and (b) for long piles bending of a pile. The calculations for both cases could be seen in the procedure described below.

Method of Broms

- Short pile: calculating the ultimate load resistance $Q_{u(g)}$

Rankine passive earth coefficient

$$K_p = \tan^2(45 + \phi'/2)$$

$$K_p = \tan^2(45 + 35.4'/2) = 3.75$$

Undrained cohesion

$$c_u \approx \frac{0.75q_u}{FS} = 0.375q_u$$

where,

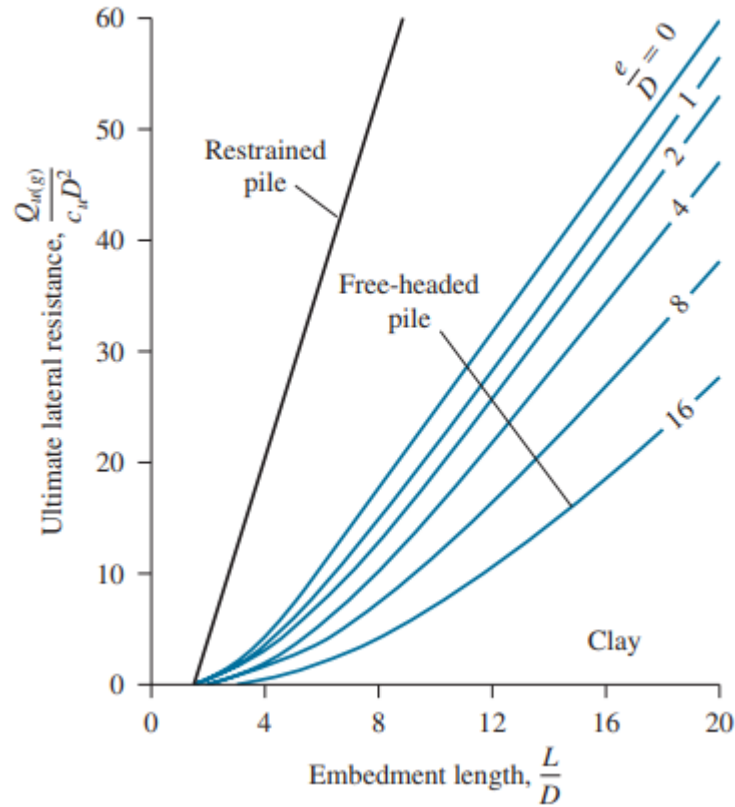
FS = factor of safety (which is taken as 2)

q_u = unconfined compression strength

Short pile

Firstly, the ratio of ultimate lateral resistance is found in respect to the ratio of length and diameter of pile. As for 12 m long piles with a diameter of 0.4 m the L/D ratio equals to 30 the maximum available 60 is taken for ultimate lateral resistance ratio from a graph (Das and Sivakugan, 2019). After using this ratio the ultimate and allowable bearing capacity values were found for each column. As can be seen from the graph the maximum value of 60 is used for both free-headed and restrained piles.

$$Q_{u(g)}/c_u D^2 \approx 60$$



- Exterior column case:

$$Q_{u(g)} = 60c_u D^2 = 60 * 0.375 * 357.01 * 0.4^2 = 1223.54 \text{ kN}$$

$$Q_{all} = \frac{Q_{u(g)}}{FS} = \frac{1223.54}{3} = 407.85 \text{ kN}$$

- Interior column case:

$$Q_{u(g)} = 60c_u D^2 = 60 * 0.375 * 546.61 * 0.4^2 = 1967.80 \text{ kN}$$

$$Q_{all} = \frac{Q_{u(g)}}{FS} = \frac{1967.80}{3} = 655.93 \text{ kN}$$

- Corner column case:

$$Q_{u(g)} = 46c_u D^2 = 60 * 0.375 * 210.60 * 0.4^2 = 758.16 \text{ kN}$$

$$Q_{all} = \frac{Q_{u(g)}}{FS} = \frac{758.16}{3} = 252.72 \text{ kN}$$

Long pile

For the long pile the yield moment is found:

$$M_y = SF_y$$

where,

S = section modulus of the pile section

F_y = yield stress of the pile material

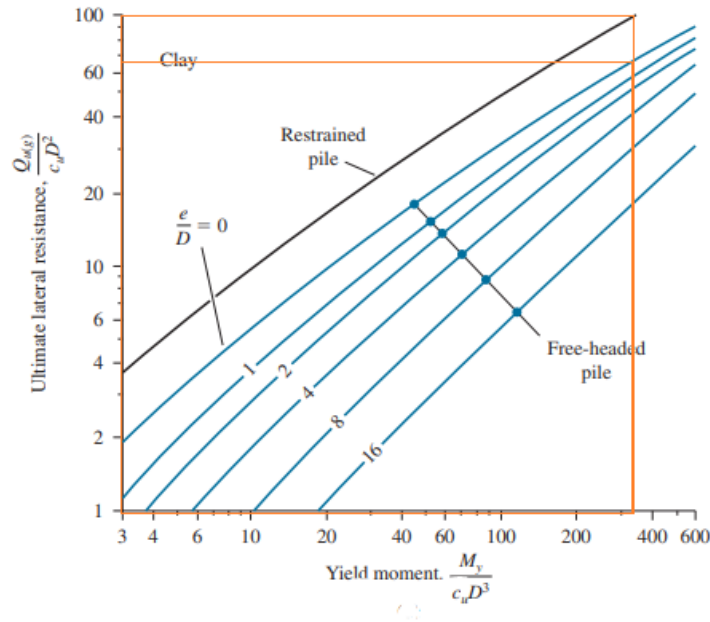
$$S = \pi D^3 / 32 = 0.006 \text{ m}^3$$

$$M_y = 2673 \text{ kN} \cdot \text{m}$$

The ratio of yield moment is found for each column in accordance with loads applied, and then this ratio is used for finding the ultimate lateral bearing capacity value similarly to the procedure above from the graph provided.

- Exterior column case:

$$\frac{M_y}{c_u D^3} = 311.96$$



$$\text{Restrained pile: } \frac{Q_{u(g)}}{c_u D^2} = 100$$

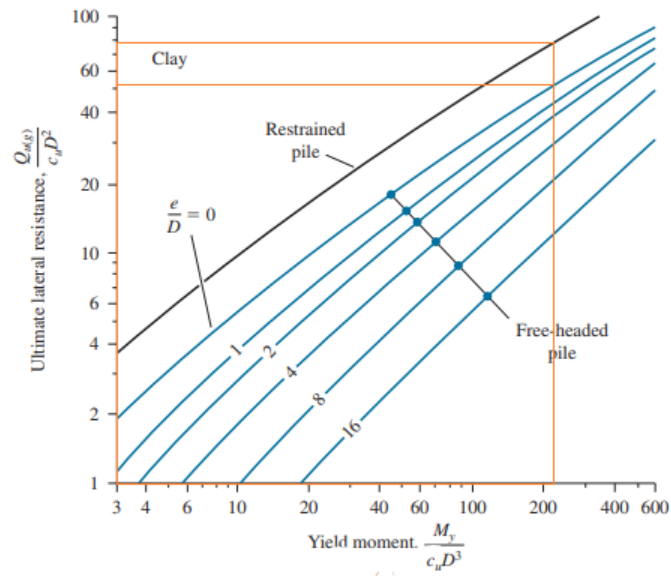
$$Q_{u(g)} = 4819.635 \text{ kN}; Q_{all} = 1606.545 \text{ kN}$$

$$\text{Free-headed pile: } \frac{Q_{u(g)}}{c_u D^2} = 67$$

$$Q_{u(g)} = 3229.156 \text{ kN}; Q_{all} = 1076.385 \text{ kN}$$

- Interior column case:

$$\frac{M_y}{c_u D^3} = 213.44$$



$$\text{Restrained pile: } \frac{Q_{u(g)}}{c_u D^2} = 79$$

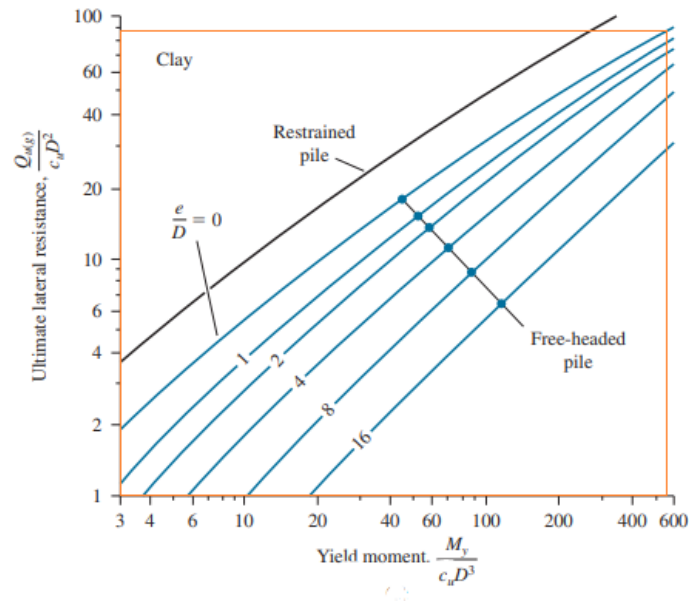
$$Q_{u(g)} = 5829.596 \text{ kN}; Q_{all} = 1943.499 \text{ kN}$$

$$\text{Free-headed pile: } \frac{Q_{u(g)}}{c_u D^2} = 51$$

$$Q_{u(g)} = 3763.410 \text{ kN}; Q_{all} = 1254.47 \text{ kN}$$

- Corner column case:

$$\frac{M_y}{c_u D^3} = 553.97$$



$$\text{Restrained pile: } \frac{Q_{u(g)}}{c_u D^2} = \text{max} = 100$$

$$Q_{u(g)} = 2843.1 \text{ kN}; Q_{all} = 947.7 \text{ kN}$$

$$\text{Free-headed pile: } \frac{Q_{u(g)}}{c_u D^2} = 89$$

$$Q_{u(g)} = 2530.359 \text{ kN}; Q_{all} = 947.7 \text{ kN}$$

Table 3.4.3.2. Lateral bearing capacity analysis comparison.

	Short pile		Long pile	
	Restrained	Free-headed	Restrained	Free-headed
$Q_{u(g)}, \text{ kN}$	1967.80	1967.80	5829.6	3763.4
$Q_{all}, \text{ kN}$	655.93	655.93	1943.5	1254.5
Lateral load, kN	236.0			

As the lateral loads applied onto the interior column are the highest, the calculation results obtained for the interior column are summarized in Table above. As it can be seen the lateral load is less than the allowable bearing capacity calculated it can be concluded that the design is safe.

3.4.3.2 Deflection hand calculation

Broms method

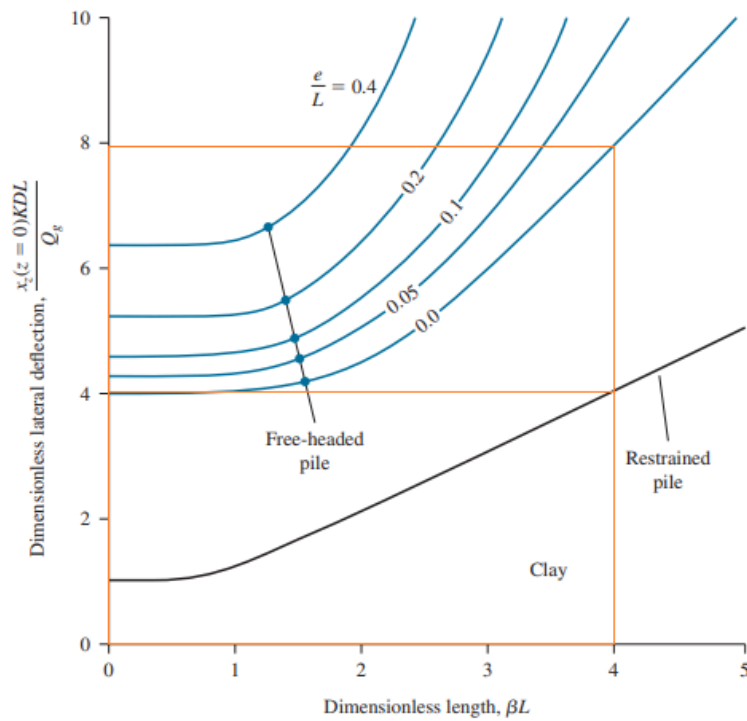
Continuing the use of Broms method the deflection at pile head could be estimated by the graphical representation.

$$\beta = \sqrt[4]{\frac{KD}{4E_p I_p}}$$

where,

$K = 30,000 \text{ kN/m}^3$ in regard to unconfined compressive strength of soil.

$$\beta = 0.39 \approx 0.4$$



Restrained pile:

$$\frac{x_z(z=0)KDL}{Q_g} = 4$$

Free-headed pile:

$$\frac{x_z(z=0)KDL}{Q_g} = 8$$

Elastic solution for deflection

According to Winkler's model (Das and Sivakugan, 2019) the soil is considered as a series of infinitely close independent elastic springs.

$$k = \frac{p' (kN/m)}{x (m)}$$

where,

k = modulus of subgrade reaction

p' = pressure on soil

x = deflection

Based on this the pile deflection at any point could be found using the formula:

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

T is the characteristics length of the soil-pile system:

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

- Long pile (when $L \geq 5T$)
- Rigid pile (when $L \leq 2T$)

$E_p = 30 \text{ GPa}$

I_p (moment of inertia for a circular cross-section):

$$I_p = (\pi D^4 / 64) = \pi * 0.6^4 / 64 = 6.36 * 10^{-3} m^4$$

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} = \sqrt[5]{\frac{30 * 10^6 * 6.36 * 10^{-3}}{10500}} = 1.79 m$$

$8.9 m = 5T < L = 10 m$ therefore considered as long pile

The calculation details for a deflection could be seen in Table below in which the coefficients used can be seen for all columns. The obtained results were interpreted by the graph of deflection by the depth.

Table 3.4.3.2.1. Deflection on each soil layer by elastic solution.

			Deflection, mm		
Z	Ax	Bx	Corner	Interior	Exterior
0.5586	1.644	0.873	9.017616004	12.8945603	11.58328864
1.1173	0.738	0.223	3.017647515	4.75802762	4.169391073

1.6759	0.381	0.029	1.032444984	1.9309339	1.6270443
2.2346	0.155	-0.0738	-0.102200679 6	0.2633263072	0.1396966799
2.7933	0.197	-0.0852	-0.077441771 2	0.3871312378	0.2300019696
3.3519	-0.083	-0.068	-0.601234080 3	-0.796967886 2	-0.730766214 8
3.9106	-0.05	-0.028	-0.283098019 5	-0.401009950 7	-0.361129425 8
4.4693	-0.0295	-0.014	-0.151653563 5	-0.221221602 9	-0.197692093 2
5.0279	-0.009	0	-0.020209107 1	-0.041433255 1	-0.034254760 6
5.5866	0	0	0	0	0
6.1453	0	0	0	0	0

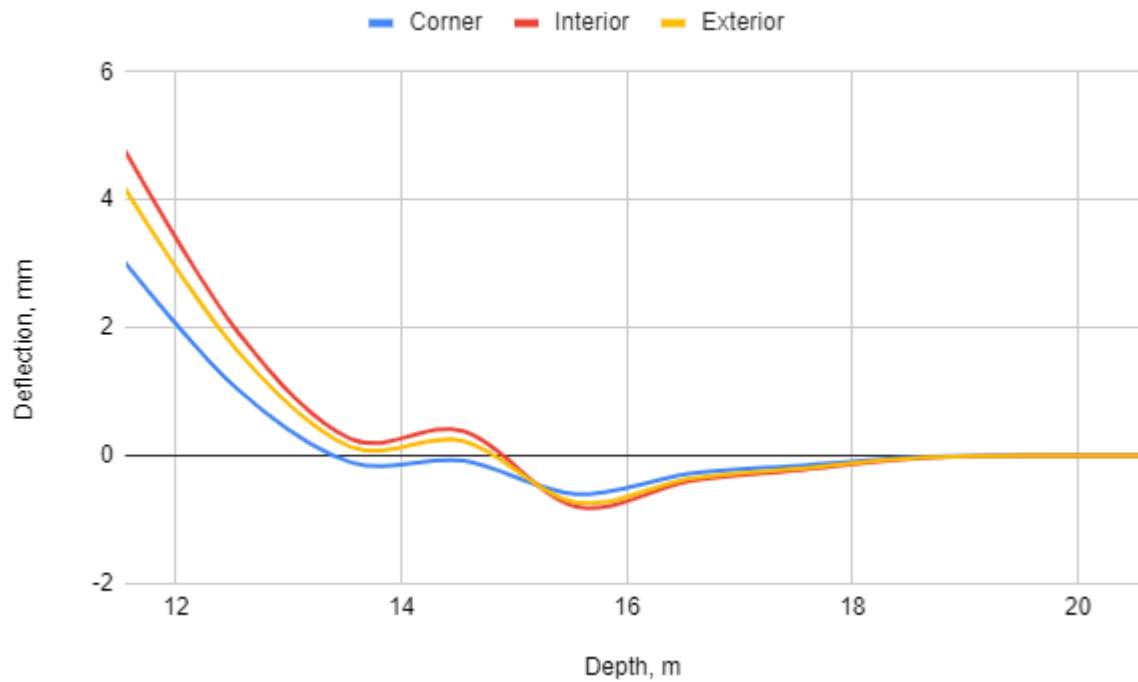


Figure 3.4.3.2.1. Graph of deflection by depth for each column.

The hand calculation results for deflection by Broms and elastic solution were obtained. Overall, all values for columns are in acceptable range for a design. It can be shown in Table below.

Table 3.4.3.2.2. Deflection hand calculation results.

	Deflection on the pile head, mm		
Pile group	Restrained	Free-headed	Elastic solution
Corner	5.3	4.7	9.0
Exterior	8.9	6.0	11.6
Interior	10.8	6.9	12.9

3.4.4 Design of Group Piles.

3.4.4.1 Pile Cap Design.

The design of pile design starts with the determination of the pile cap size. In the structure there are two types of the pile groups: three by two and two by two. The following calculations will be done for the second type for the corner columns. The method used is reinforcement design of pile cap and piles by Ray (1994).

2*2 piles: square shape with $L=B$;

$$B = d(n_{1,2} - 1) + 2(1.5D)$$

The minimum height defined by diameter:

$$h_{min.} = 1.5D$$

The loads applied to the group piles should be analysed. The following combinations will be used:

$$LC_1 = 1.2D + 1.6L$$

$$LC_2 = 1.2D + 1.0L + 1.0W$$

$$LC_3 = 1.2D + 1.0L + 1.0E_x$$

Dead load = 90.3 kN;

Live load = 31.07 kN;

$$Weight\ of\ pile\ cap = W_{pc} = \gamma_{concrete} * h = 25 * 1.2 = 30\ kN/m^2$$

$$\text{Weight of backfill} = W_{bf} = \gamma_{soil} * h = 19.65 * 0.5 = 9.825 \text{ kN/m}^2$$

$$\text{Weight of surcharge} = W_{sc} = DL + LL = 121.37 \text{ kN/m}^2$$

$$W = W_{pc} + W_{bf} + W_{load} = 30 + 9.825 + 121.37 = 161.195 \text{ kN/m}^2$$

Load factors applied:

$$1.2W = 193.434 \text{ kN/m}^2$$

$$1.4W = 225.673 \text{ kN/m}^2$$

Bending moment due to the load of pile on section 1-1:

$$M'_{11} = \frac{2.4 * 193.434 * 1.5^2}{2} = 522.2718 \text{ kN} \cdot \text{m}$$

$$\text{or } M'_{22} = \frac{2.4 * 225.673 * 1.5^2}{2} = 609.3171 \text{ kN} \cdot \text{m}$$

Bending moment due to the load on pile on section 2-2:

$$M'_{22} = \frac{3.6 * 193.434 * 0.9^2}{2} = 282.026772 \text{ kN} \cdot \text{m}$$

$$\text{or } M'_{22} = \frac{3.6 * 225.673 * 0.9^2}{2} = 329.031234 \text{ kN} \cdot \text{m}$$

The reaction forces by:

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

$$Q_1 = 3807.404833 \text{ kN}, Q_2 = 3309.984666 \text{ kN}$$

$$Q_3 = Q_4 = 2812.5645 \text{ kN}$$

Q_1, Q_2, Q_3 and Q_4 respectively for the group of piles in where:

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

$$P = 1.4 * 7761.8 + 30 + 9.825 = 10906.345 \text{ kN}$$

$$M_{xx} = M_x + Ne_y + H_y h + M_x^* - \text{moment about the } x \text{ axis on pile group};$$

$$M_{xx} = 1.5785 + 2647 * 0.6 + 2.4 * 1.2 = 1592.6585 \text{ kN} \cdot \text{m}$$

$$M_{yy} = M_y + Ne_x + H_x h + M_y^* - \text{moment about the } y \text{ axis on pile group};$$

$$M_{yy} = 0.5532 + 2647 * 1.2 + 1 * 2.4 = 3179.8332 \text{ kN} \cdot \text{m}$$

$$I_{xx} = \sum y^2 \text{ about the } x \text{ axis};$$

$$I_{xx} = 4 * 0.6^2 = 1.44$$

$$I_{yy} = \sum x^2 \text{ about the } y \text{ axis}.$$

$$I_{yy} = 4 * 1.2^2 = 5.76$$

The bending moment caused by the reaction forces are calculated by the formula:

$$M''_{11} = 1.4(Q_3 + Q_4) = 1.4(2812.5645 + 2812.5645) = 7875.1806 \text{ kN} \cdot \text{m}$$

$$\begin{aligned} M''_{22} &= 0.5(Q_1 + Q_2 + Q_3) = 0.5(3807.404833 + 3309.984666 + 2812.5645) \\ &= 4964.977 \text{ kN} \cdot \text{m} \end{aligned}$$

These bending moments are combined followingly:

$$M_{11} = M'_{11} + M''_{11} = 609.3171 + 7875.1806 = 8484.4977 \text{ kN} \cdot \text{m}$$

$$M_{22} = M'_{22} + M''_{22} = 329.031234 + 4964.977 = 5294.008234 \text{ kN} \cdot \text{m}$$

The pile cover value is chosen based on the concentration of the sulfur trioxide in the soil.

SO₃ is 1.27% which is class exposure 4 and minimum cover on blinding 60 mm; ,minimum cover elsewhere is 100 mm.

Table 7.4 Minimum cover to reinforcement for class of exposure.

Class of exposure	Total SO ₃ percentage	Minimum cover on blinding (mm)	Minimum cover elsewhere (mm)
1	<0.2	35	75
2	0.2 to 0.5	40	80
3	0.5 to 1.0	50	90
4	1.0 to 2.0	60	100
5	>2.0	60	100

After the area of reinforcement for a pile is calculated. The reinforcing bars' dimensions were selected based on the analysis above. The cover provided is the minimum dimension according to the ACI requirements. The area of the reinforcement in the pile cap determined using the following formulas:

$$d_x = h_{pile\ cap} - cover - 0.5d_{bar} = 1200 - 100 - 0.5 * 28 = 1086\ mm$$

Diameter of the bar is assumed to be 28 mm;

$$f_{cu} = 30\ N/mm^2$$

$$f_y = 460\ N/mm^2$$

x-direction:

$$K = \frac{M_{11}}{f_u b d^2} = \frac{8485 * 10^6}{30 * 3600 * 1086^2} = 0.0666$$

$$z = d(0.5 + \sqrt{0.25 - \frac{K}{0.9}}) = 1086(0.5 + \sqrt{0.25 - \frac{0.0666}{0.9}}) = 998.6 < 0.95d = 1031.7$$

$$A_{st.} = \frac{M_{11}}{0.87 f_y z} = \frac{8485 * 10^6}{0.87 * 460 * 998.6} = 21231.623\ mm^2$$

$$\# of bars = \frac{A_{st.}}{A_{bar}} = \frac{21231.623}{615.752} = 34.5 \approx 35$$

$$Spacing = \frac{L_g}{\# of bars} = \frac{3600}{34.5} = 104.347 \approx 105\ mm$$

Use 35 bars with the spacing of 105 mm.

y-direction:

$$d_y = h_{pile\ cap} - cover - 0.5d_{bar} = 1200 - 100 - 0.5 * 24 = 1088\ mm$$

$$K = \frac{M_{22}}{f_u b d^2} = \frac{5294 * 10^6}{30 * 3600 * 1088^2} = 0.0414$$

$$z = d(0.5 + \sqrt{0.25 - \frac{K}{0.9}}) = 1088(0.5 + \sqrt{0.25 - \frac{0.0414}{0.9}}) = 1035.41 > 0.95d$$

$$z = 0.95 * 1088 = 1033.6$$

$$A_{st} = \frac{M_{22}}{0.87f_y z} = \frac{5294*10^6}{0.87*460*1033.6} = 12798.36088 \text{ mm}^2$$

$$\# \text{ of bars} = \frac{A_{st}}{A_{bar}} = \frac{12798.36088}{452.389} = 28.3 \approx 29$$

$$\text{Spacing} = \frac{L_g}{\# \text{ of bars}} = \frac{3600}{28.3} = 127.209 \approx 128 \text{ mm}$$

Use 29 bars with the spacing of 128 mm.

Table 3.4.4.1.1. Pile cap design reinforcement.

Pile group	h , mm	d_x , mm	Design in x-direction	d_y , mm	Design in y-direction
Interior	1200	1086	28 mm #35 @ 105 mm	1088	28 mm #29 @ 128 mm
Exterior	1200	1075	28 mm #20 @ 180 mm	1026	28 mm #24 @ 150 mm
Corner	600	486	24 mm #16 @ 158 mm	464	24 mm #36 @ 68 mm

3.4.4.2 Single Pile Reinforcement

For this project, the reinforcement of a single pile was designed and modeled using Revit software in accordance with relevant structural standards as ACI 318 (Building Code Requirements for Structural Concrete). The reinforcement was included to ensure that the pile would have adequate strength and durability to carry structural loads and resist potential cracking or deformation.

A reinforcement cage was modeled, consisting of vertical bars to take axial loads and horizontal ties to provide confinement and stability. The arrangement was designed to maintain proper spacing, concrete cover, and structural integrity. The reinforcement layout was carefully planned to reflect practical construction methods and to ensure the pile could perform effectively under the expected loading conditions. The design was coordinated with the structural and architectural models to avoid clashes and ensure consistency throughout the project.

While our work focused only on the design stage, the reinforcement was modeled with practical construction methods in mind to help ensure the foundation would be safe and effective. Reinforcement of deep pile foundations is essential because concrete exhibits high

compressive strength but low tensile strength. Piles are subjected to both vertical and lateral forces, such as those caused by wind, seismic activity, and uneven soil pressure, which may induce bending or cracking. The reinforcement ensures the pile can withstand these forces, maintaining structural integrity. Additionally, it enhances the durability and overall stability of the foundation system, ensuring long-term performance. Figures 3.4.4.2.1 and 3.4.4.2.2 provide a visualization of the reinforcement of the single pile and corner, exterior, interior pile groups.

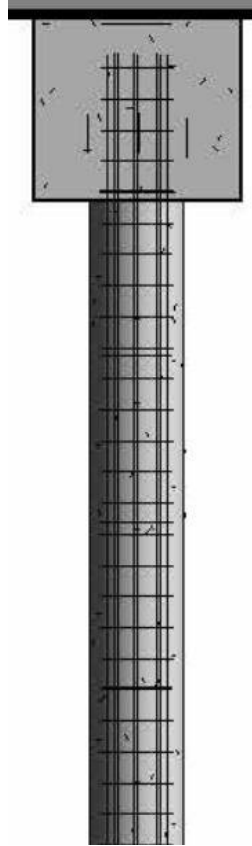


Figure 3.4.4.2.1. Single Pile Reinforcement.

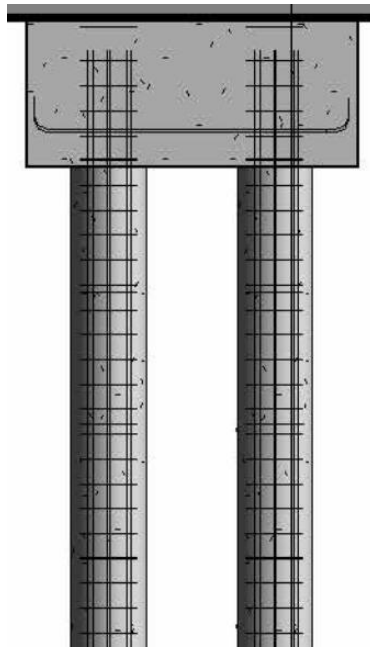


Figure 3.4.4.2.2. Pile Group Reinforcement.

Table 3.4.4.2.1. Individual pile reinforcement design.

Group of piles		D of pile, mm	Ast, mm ²	Cover, mm	Reinforcement, %	Design
Interior	3 × 2	400	1626.8	100	2	# 10 @ 14
Exterior	3 × 2	400	1519.0	100	1.8	# 10 @ 14
Corner	2 × 2	400	1279.8	100	1.6	# 8 @ 14

Single Pile Reinforcement software analysis.

The obtained results were analyzed through the software GEO5 and all design parameters for shear, bending moment and reinforcement ratio in acceptable.

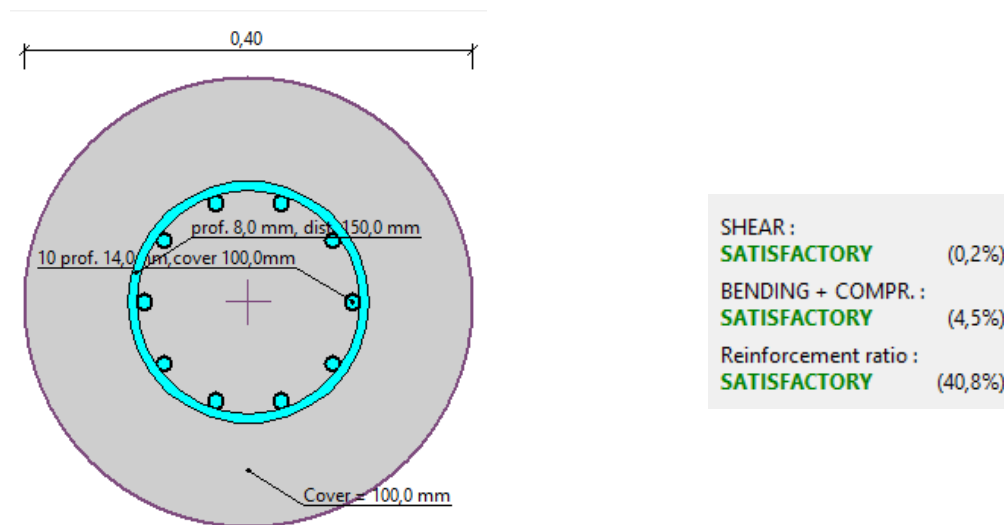


Figure 3.4.4.2.3. The reinforcement of a single pile for the interior column.

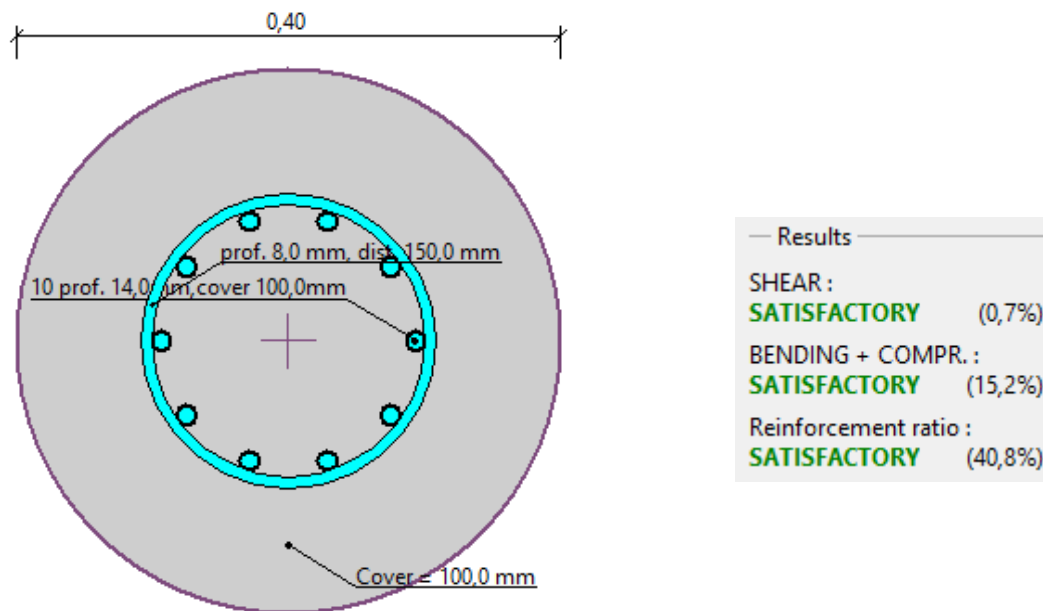


Figure 3.4.4.2.4. The reinforcement of a single pile for the exterior column.

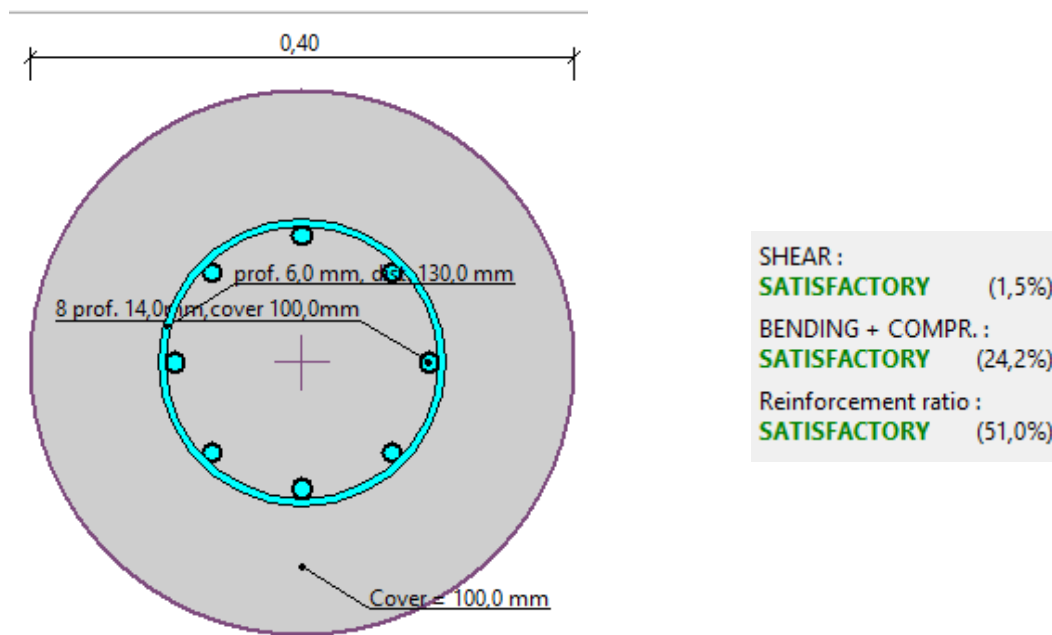


Figure 3.4.4.2.5. The reinforcement of a single pile for the corner column.

3.5 Earth retaining structures

The earth retaining structures such as braced-cuts, sheet-pile walls, retaining walls support the vertical and near vertical slope stability of soil (Das and Sivakugan, 2019). Therefore, it is important to construct these types of structures in order to provide the safety for building by preventing the risks related to the unstable soil including sliding, earthquakes and other occurrences which influence the foundation of the building. This process needs the results of lateral earth pressure calculations.

3.5.1 Lateral earth pressure

The active and passive earth pressures represent the maximum loading when the soil is at failure (Das and Sivakugan, 2019). For this reason these values will be calculated further.

3.5.1.1. Active Rankine earth pressure

The Rankine active-pressure coefficient is calculated by:

$$K_a = \tan^2(45 - \phi'/2)$$

This value is used to calculate the Rankine active pressure applying the equation below:

$$\sigma'_a = \gamma H K_a$$

Similarly the active force per unit length is calculated applying the equation below:

$$P_a = 0.5 \gamma H^2 K_a$$

The height of the base excavation is 3.5 m and it was divided into 1.5 m, 1.5 m and 0.5 m layers of soil in order to analyze the active earth pressure change by excavation height. The calculation results can be seen in Table 3.5.1:

Table 3.5.1.1.1. Active earth pressure calculation.

Depth, m	$\gamma, kN/m^3$	$\phi', ^\circ$	K_a	$\sigma', kN/m^3$	$\sigma'_a, kN/m^2$
0	0.0	34.5	0.28	0	0
1.5-	20.8	34.5	0.28	31.20	8.64
1.5+	20.8	34.5	0.28	31.20	8.64
3-	20.8	34.5	0.28	31.20	17.28
3+	20.8	34.5	0.28	31.20	17.28
3.5	19.5	35.9	0.26	31.20	25.42

The active force per unit length is calculated by summing the area values in the pressure distribution diagram:

$$P_a = 0.5 * 1.5 * 8.64 + 1.5 * 8.64 + 0.5 (8.64 + 8.14)/2 = 23.6 kN/m$$

The location height of the force is calculated by taking the moments about the bottom of the wall and resulted as $z = 1.48 m$.

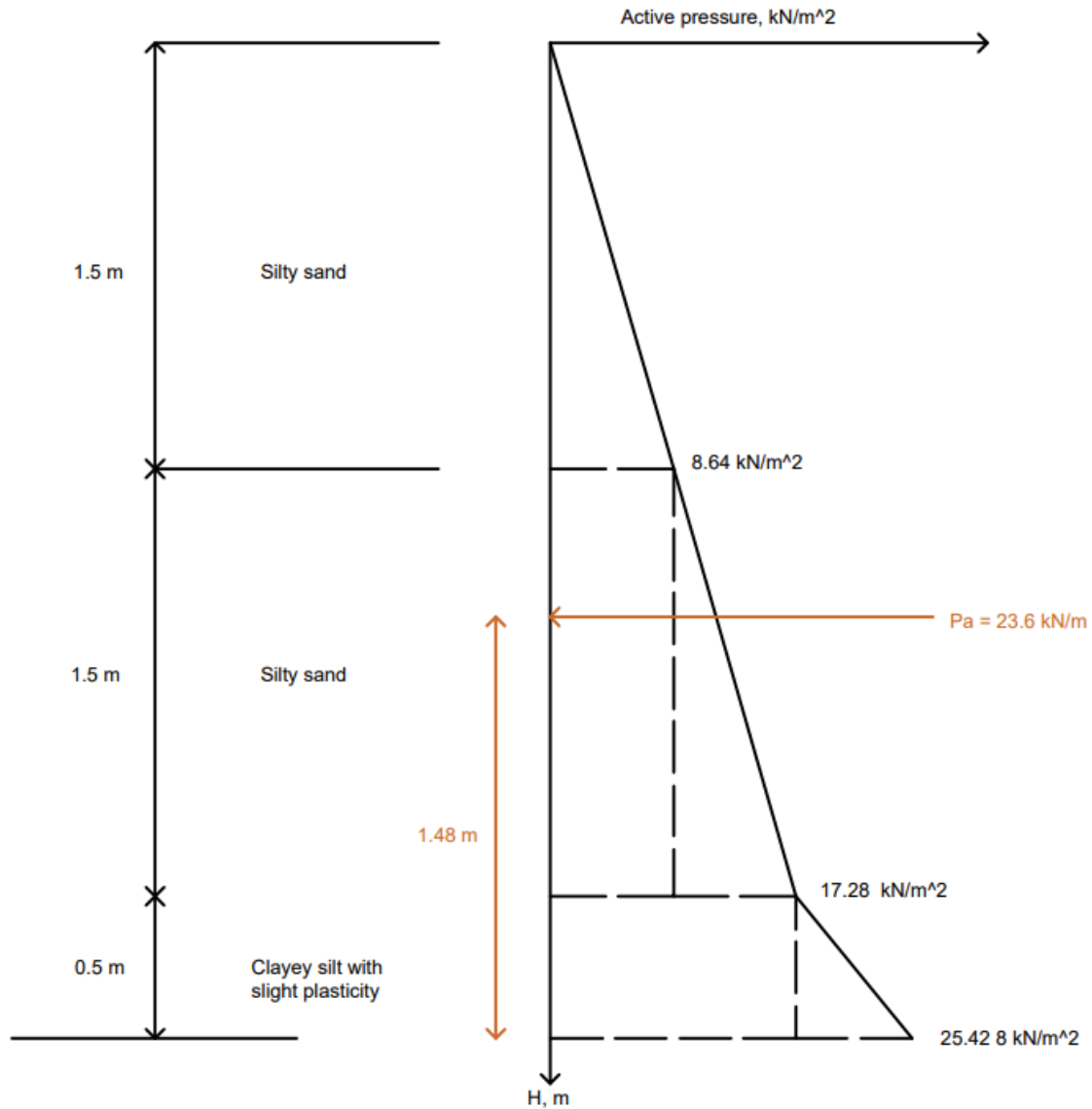


Figure 3.5.1.1.1. Active earth pressure distribution diagram.

3.5.1.2 Passive Rankine earth pressure

The Rankine passive-pressure coefficient is calculated by:

$$K_a = \tan^2(45 + \phi'/2)$$

Besides the coefficient formula other equations for calculation of passive earth pressure are the same as for the active pressure. The results can be shown in Table 3.5.2:

Table 3.5.1.2.1. Passive earth pressure calculation.

Depth, m	$\gamma, \text{ kN/m}^3$	$\phi', ^\circ$	K_p	$\sigma', \text{ kN/m}^3$	$\sigma'_p, \text{ kN/m}^2$
0	0.0	34.5	3.61	0	0

1.5-	20.8	34.5	3.61	31.20	112.71
1.5+	20.8	34.5	3.61	31.20	112.71
3-	20.8	34.5	3.61	62.40	225.43
3+	20.8	34.5	3.61	93.60	338.14
3.5	19.5	35.9	3.84	103.35	396.37

Accordingly the passive force was calculated as 521.77 kN/m and its height from bottom as 1.22 m . All results are represented below:

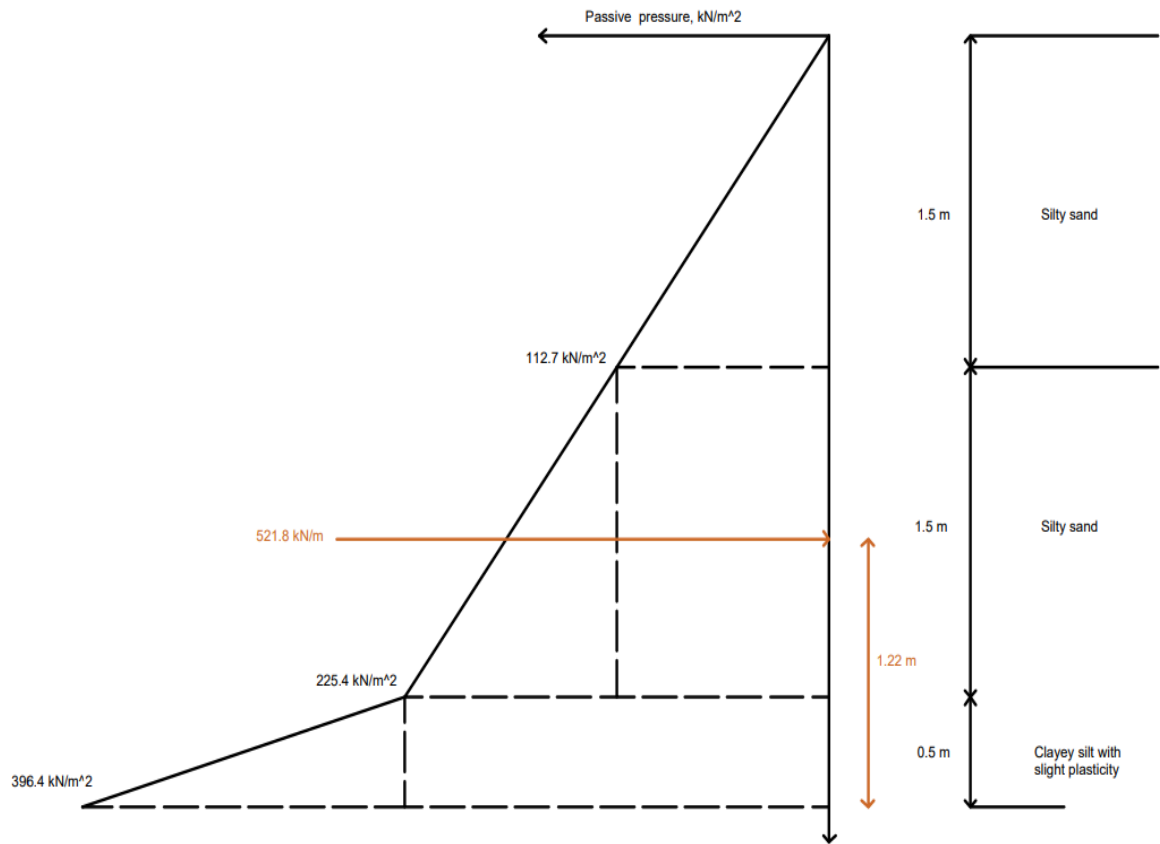


Figure 3.5.1.2.1 Passive earth pressure distribution diagram.

As mentioned above, the active and passive pressure values are extreme loading on soil at failure. Comparing these two cases it can be seen that the passive force exceeds the passive earth pressure values and therefore there is a high probability of soil failing.

3.5.2 Hand calculations of sheet pile.

For the sheet-pile wall it was recommended to choose the cantilever type of pile walls (Das and Sivakugan, 2019). For the calculation of sheet-pile design the obtained pressure values were used. Also, for the case when there is no water table until reaching the dredge line Das and Sivakugan (2019) give these equations to calculate the required variables in order to draw the diagram of sheet pile penetrating the soil:

$$\sigma'_2 = \gamma L K_a = 21.8 * 3 * 0.28 = 18.312 \text{ kN/m}^2$$

$$21.364$$

$$L_3 = \frac{LK_a}{(K_p - K_a)} = \frac{3*0.28}{(3.61-0.28)} = 0.252 \text{ m}$$

$$0.294$$

$$\sigma'_5 = \gamma L K_p + \gamma L_3 (K_p - K_a) = 21.8 * 3 * 3.61 + 21.8 * 1.04(3.61 - 1.04) = 113.322 \frac{\text{kN}}{\text{m}^2}$$

$$P = 0.5\sigma'_2 L + 0.5\sigma'_5 L_3 = 0.5 * 20.384 * 3.5 + 0.5 * 20.384 * 1.04 = 48.129 \text{ kN/m}$$

$$\bar{z} = \frac{L(2K_a + K_p)}{3(K_p - K_a)} = \frac{3.5(2*0.28+3.61)}{3(3.61-0.28)} = 2.211 \text{ m}$$

$$L_4^4 + A_1' L_4^3 - A_2' L_4^2 - A_3' L_4 - A_4' = 0$$

Where,

$$A_1' = \frac{\sigma'_5}{\gamma(K_p - K_a)} = \frac{113.322}{21.8(3.61-0.28)} = 5.589$$

$$A_2' = \frac{8P}{\gamma(K_p - K_a)} = \frac{8*48.129}{21.8(3.61-0.28)} = 18.991$$

$$A_3' = \frac{6P[2\bar{z}\gamma(K_p - K_a) + \sigma'_5]}{\gamma^2(K_p - K_a)^2} = \frac{6*48.129[2*2.211*20.8(3.61-0.28) + 113.322]}{21.8^2(3.61-0.28)^2} = 142.6$$

$$A_4' = \frac{P(6\bar{z}\sigma'_5 + 4P)}{\gamma^2(K_p - K_a)^2} = \frac{48.129(6*2.211*113.322 + 4*48.129)}{21.8^2(3.61-0.28)^2} = 198.59$$

So, the equation will look like:

$$L_4^4 + 5.589L_4^3 - 18.991L_4^2 - 142.6L_4 - 198.59A_4' = 0$$

By solving which the $L_4 = 3.82 \text{ m}$

$$D_{theory} = L_3 + L_4 = 4.86 \text{ m}$$

$$Total \text{ length} = 3.5 + 1.3 * 4.86 = 9.82 \text{ m}$$

As now we have find the L_4 :

$$\sigma'_3 = L_4(K_p - K_a)\gamma = 3.82(3.61 - 0.28)21.8 = 77.45 \text{ kN/m}^2$$

$$\sigma'_4 = \sigma'_5 + \gamma L_4(K_p - K_a) = 113.322 + 21.8 * 3.82(3.61 - 0.28) = 190.77 \text{ kN/m}^2$$

Based on the calculated values above the sheet pile design diagram was constructed and can be seen from Figura 3.5.2:

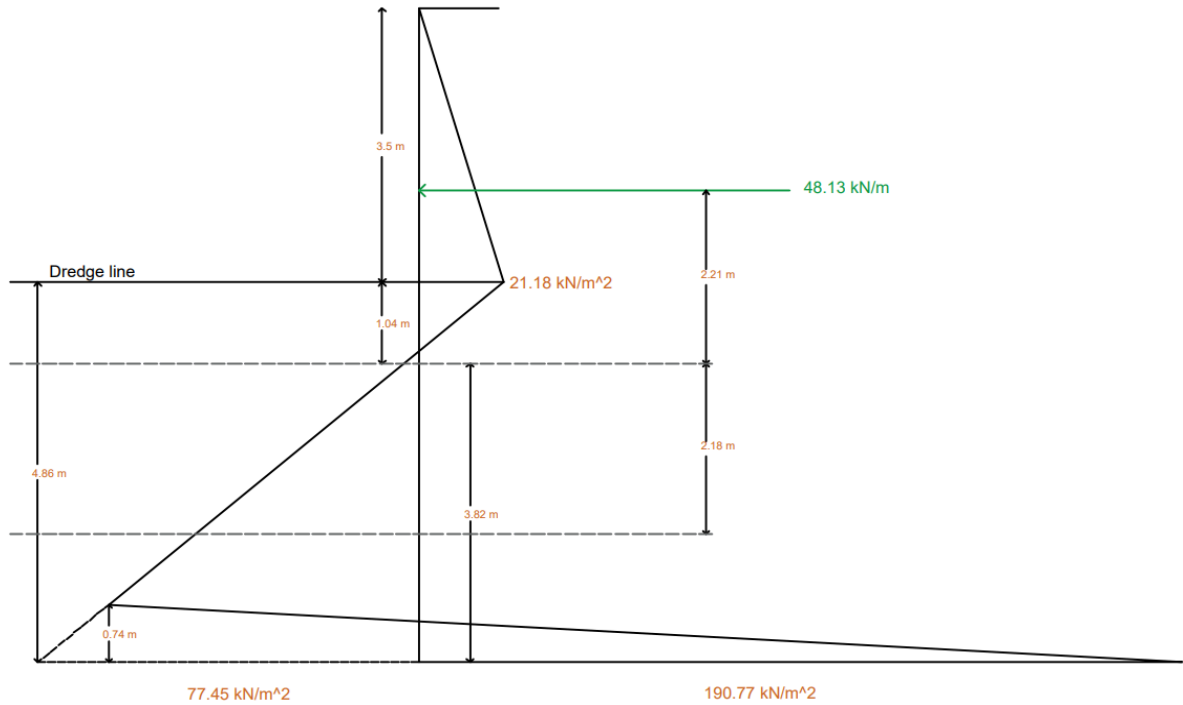


Figure 3.5.2.1. Sheet pile design diagram.

After the magnitude of the maximum moment can be calculated as:

$$M_{max} = P(L + z') - \frac{\gamma z'^3(K_p - K_a)}{6} = 48.129(3.5 + 1.15) - \frac{21.8 * 1.15^3(3.61 - 0.28)}{6} = 205.4 \text{ kN} \cdot \text{m}$$

Where,

$$z' = \sqrt{\frac{2P}{\gamma(K_p - K_a)}} = \sqrt{\frac{2 * 48.129}{21.8(3.61 - 0.28)}} = 1.15 \text{ m}$$

The section modulus of the pile calculated by:

$$S = \frac{M_{max}}{\sigma_{all}} = \frac{205.4}{225000} = 9.13 * 10^{-4} \text{ m}^3/\text{m}$$

The allowable stress value taken as 225 MN/m^2 because ASTM A-572 type of steel was chosen (Das and Sivakugan, pp. 754, 2019). The obtained section modulus is close to the

Section designation of PZ-22. To build the sheet-pile structure the properties for Available Sheet-Pile Sections (Adapted after Hammer and Steel, Inc., Hazelwood, Missouri, USA) (Das and Sivakugan, pp. 755, 2019) will be used.

3.5.2.1 Software analysis of sheet pile design

The sheet pile design was analyzed by GEO5 software. The same soil profile used for the axial bearing capacity was used. The design parameters such as length of excavation and water table was included.

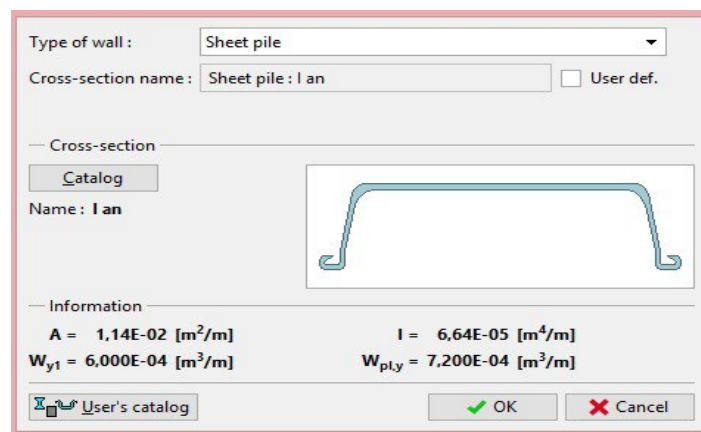


Figure 3.5.2.1.1. Sheet pile material by GEO5.

The overall length of sheet pile by the software equals to 7.97 m, it can be considered that this value is close to the hand calculation of 8.36 m. The difference between these two could be explained by the software features which allows entering each soil layer with its properties. In the case of hand calculation the method only focuses in granular soil.

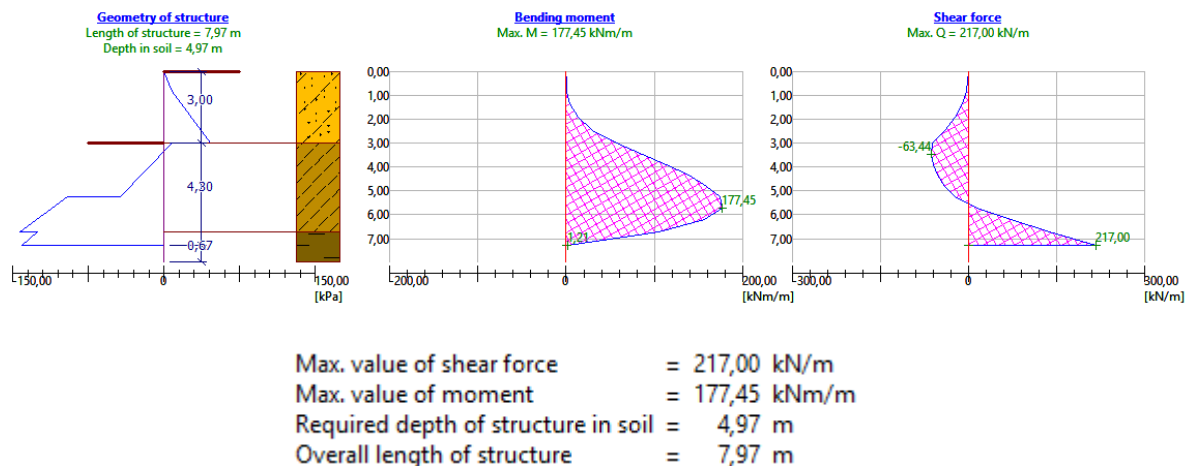


Figure 3.5.2.1.2. Sheet pile structure with bending moment and shear force analysis by GEO5.

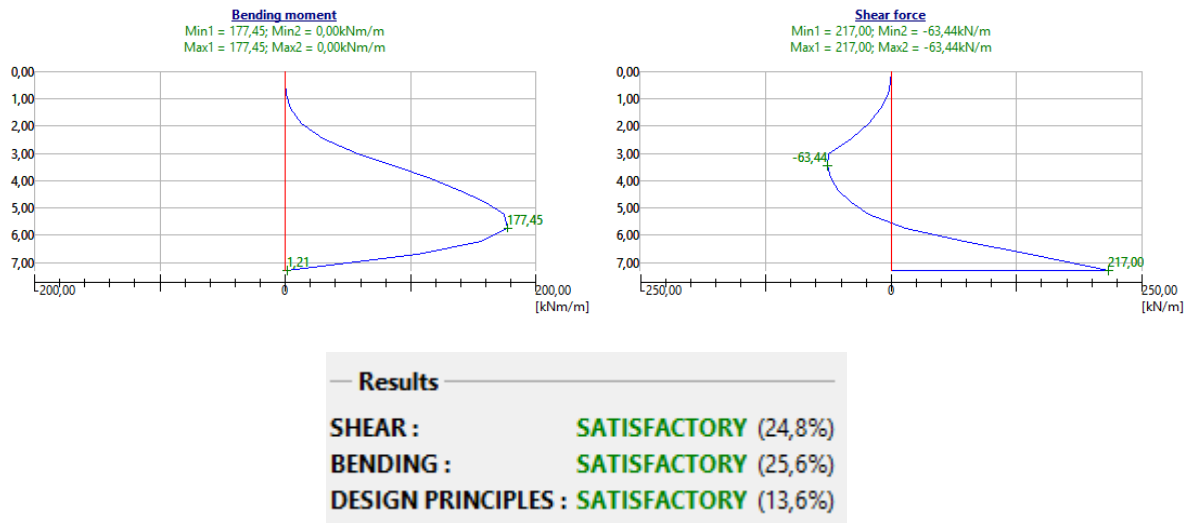


Figure 3.5.2.1.3. Maximum bending moment and shear force analysis for sheet pile by GEO5.

3.6. Site Response Analysis

Site response analysis was performed using Plaxis 2D to evaluate the seismic behavior of the site based on the local soil conditions. The soil profile was defined using geotechnical investigation data, and each layer was assigned appropriate material properties, including stiffness, density, and damping characteristics. A finite element model was created, and the boundaries were set to allow for free-field conditions and absorb seismic energy.

Seismic loading was applied in the form of acceleration time histories at the base of the model to simulate earthquake effects according to the seismicity in 1979 in Seattle, Washington. The dynamic response of the soil was analyzed over time to observe how seismic waves travel through the different layers. Ground motion amplification and changes in shear strain and stress were assessed at various depths. The results were used to understand potential impacts on the foundation system and to support safe structural design under seismic conditions.

The Figure 3.6.1 below illustrates the variation of ground acceleration over time as recorded or applied in the site response analysis. The x-axis shows time in seconds (s), while the y-axis represents acceleration in meters per second squared (m/s^2). This accelerogram was used as seismic input at the base of the Plaxis 2D model to simulate earthquake loading conditions. Peaks in the curve indicate the strongest shaking periods, which are critical for evaluating the dynamic response of the soil layers and the performance of the foundation system.

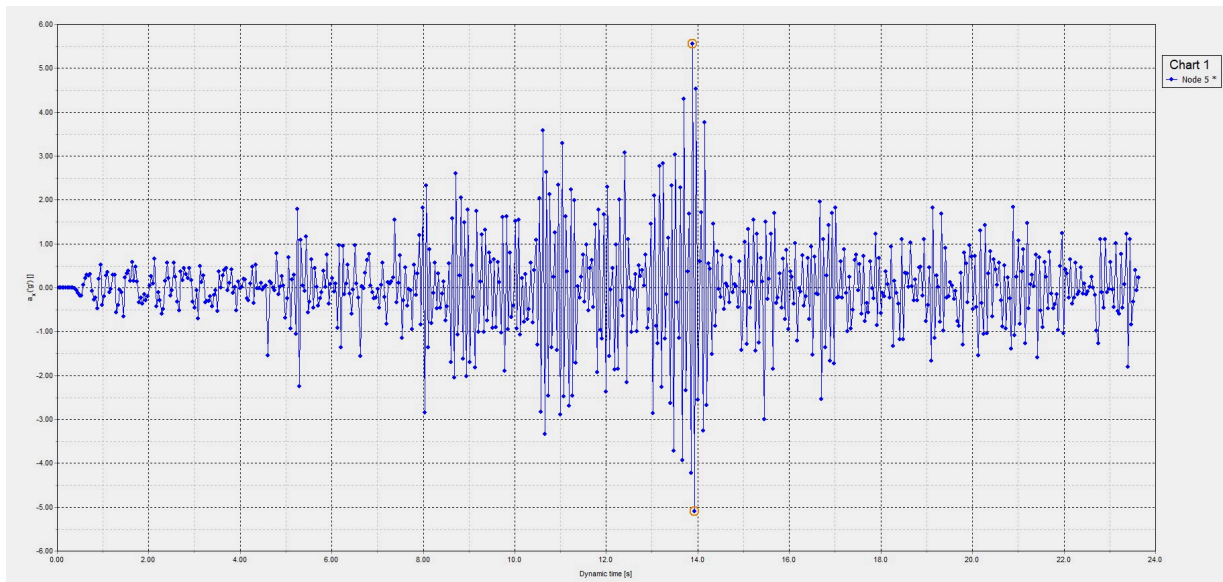


Figure 3.6.1. Accelerogram Chart

The PSA graph below in Figure 3.6.2 displays the variation of pseudo-spectral acceleration with respect to frequency of a system subjected to seismic loading. The x-axis represents the natural period (T) in seconds, while the y-axis shows the pseudo-spectral acceleration (PSA) in meters per second squared (m/s^2). This curve is derived from the input accelerogram and is used to assess the expected acceleration demand on structures with varying dynamic characteristics. Peaks in the PSA curve indicate periods at which structures are most likely to experience maximum acceleration response, making it a critical tool for evaluating seismic vulnerability and guiding structural design.

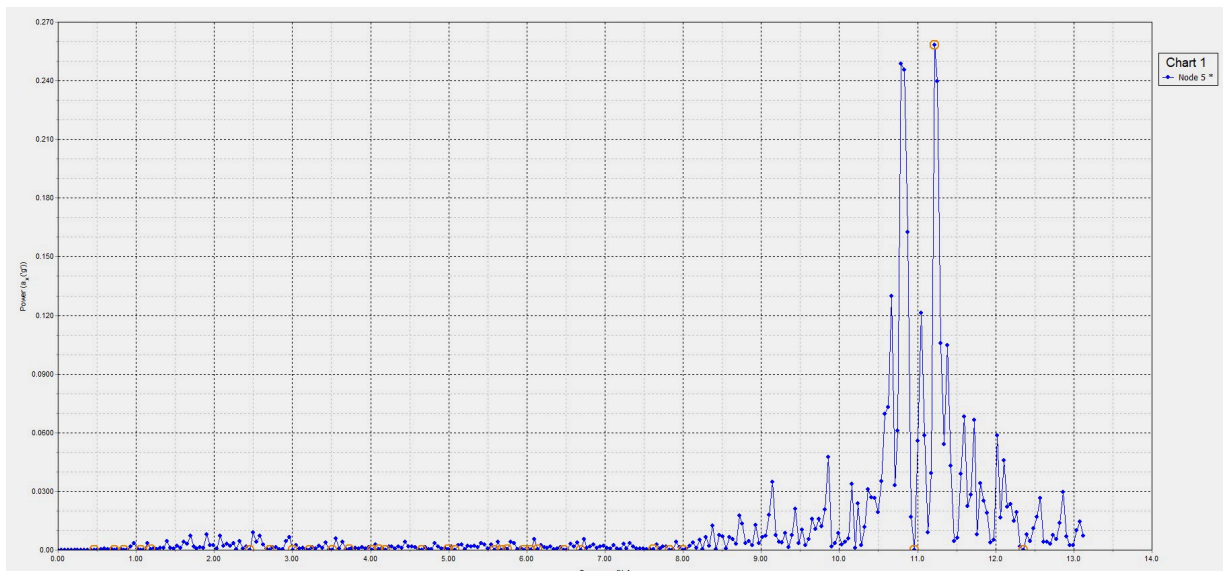


Figure 3.6.2. PSA Graph (Peak Acceleration Response)

Figure 3.6.3 below shows how the amplitude of ground motion varies with frequency during a seismic event. The x-axis represents frequency (Hz), while the y-axis indicates the amplitude of acceleration or Fourier amplitude. This graph helps identify dominant frequencies in the ground motion, which is important for understanding potential resonance effects with structures.

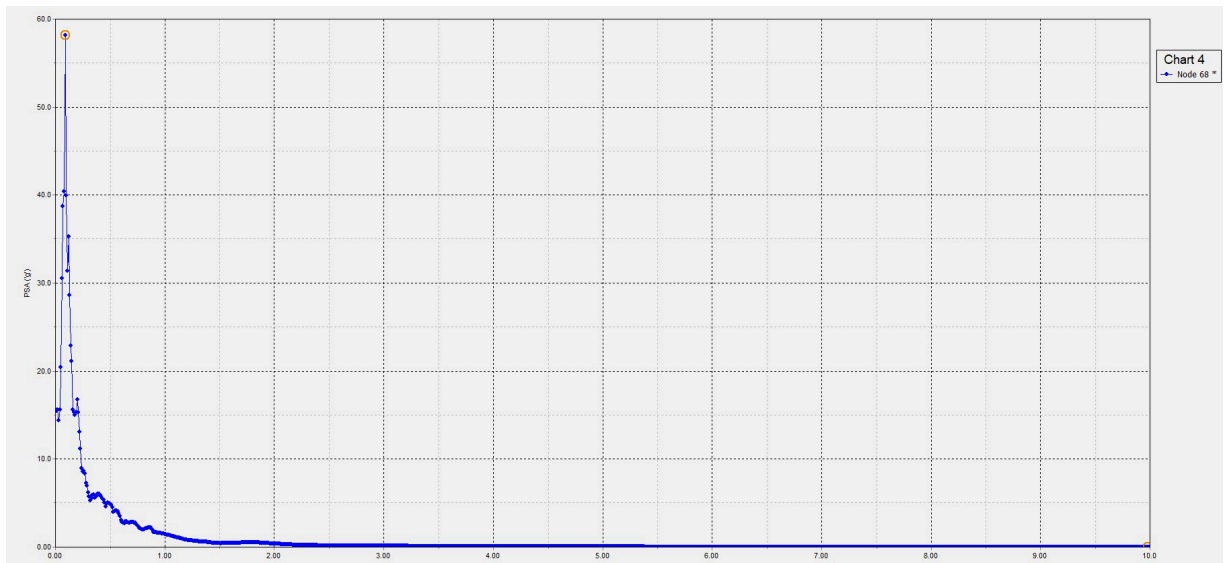


Figure 3.6.3. Frequency Spectra Graph

Figure 3.6.4 below shows the cumulative energy content of ground motion over time during an earthquake. The x-axis shows time (seconds), while the y-axis represents Arias Intensity (m/s), which is a measure of the total seismic energy transmitted to the ground. This graph is useful for evaluating the duration and destructiveness of shaking, especially in site response and liquefaction analyses.

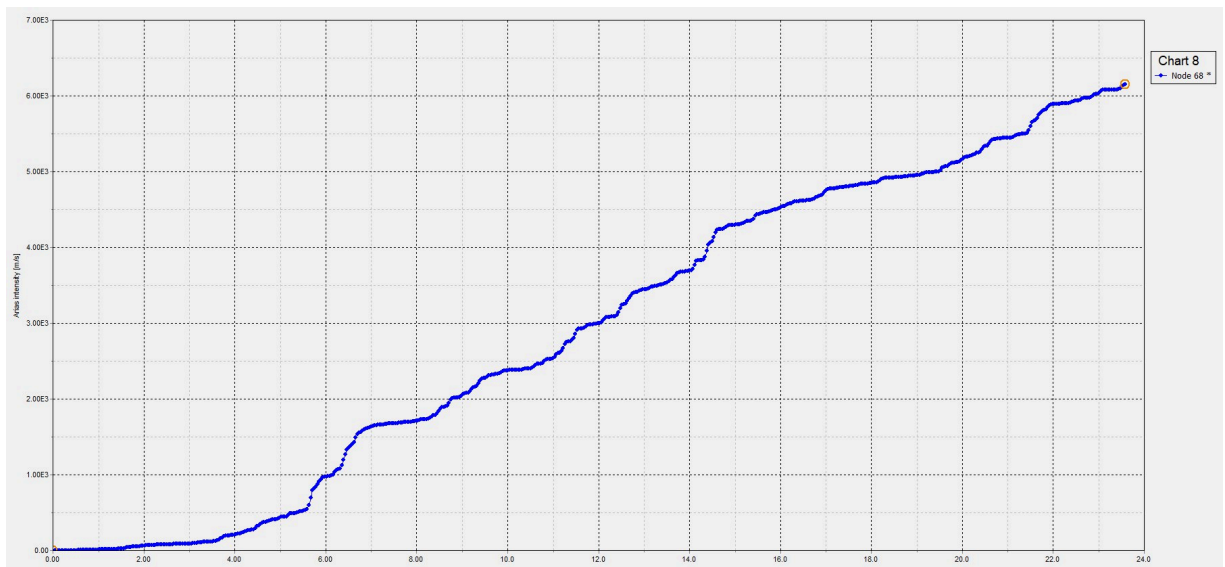


Figure 3.6.4. Arias Intensity Graph

3.7 Excavation Process

The excavation procedure is a critical step in construction that involves digging, removing, and managing soil, rock, or other materials from a construction site. The excavation process begins with the removal of soil, rock, or other materials within the marked area. Heavy machinery, like excavators or bulldozers, is used to dig and move the material, while

dump trucks carry it off-site or to a designated stockpile. Throughout the excavation, the depth and slope of the excavation is monitored to ensure it meets the design requirements. In deeper excavations, shoring or bracing may be needed to prevent the walls of the excavation from collapsing. This is particularly important in areas with loose or unstable soil. Temporary supports, like sheet piles or timber, are installed to stabilize the excavation and ensure worker safety. **Plaxis 3D** was used to model the excavation process in the project, allowing the behavior of the soil during excavation to be visualized. The soil conditions were simulated to assess the stability of the excavation and identify potential issues such as soil movement or groundwater flow. Appropriate support systems, such as shoring or sheet piles, were designed to ensure the safety and stability of the excavation area. Valuable insights into the construction process were gained, helping to optimize the excavation phase of the project. For instance, Figure X below shows the total displacement of 0.0072 mm during the excavation and installation of shoring.

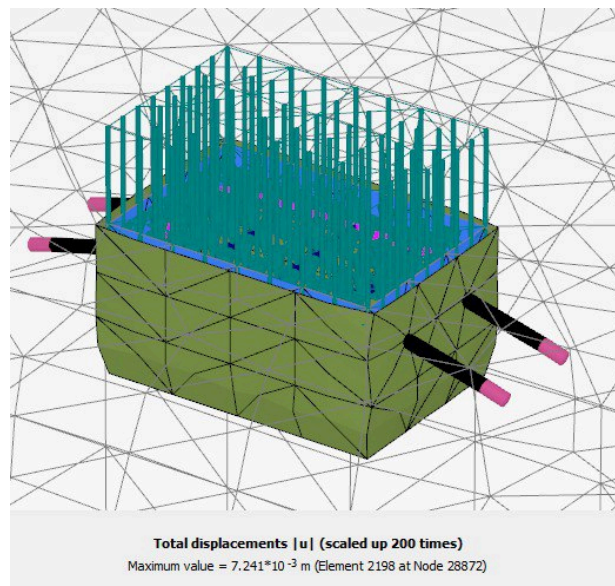


Figure 3.7.1. Excavation model in Plaxis 3D

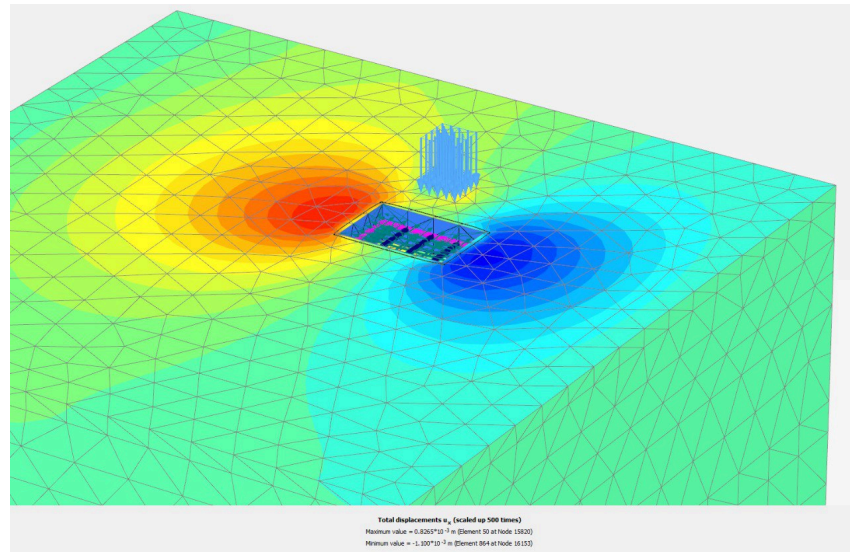


Figure 3.7.2. Distribution of forces in excavation in Plaxis 3D

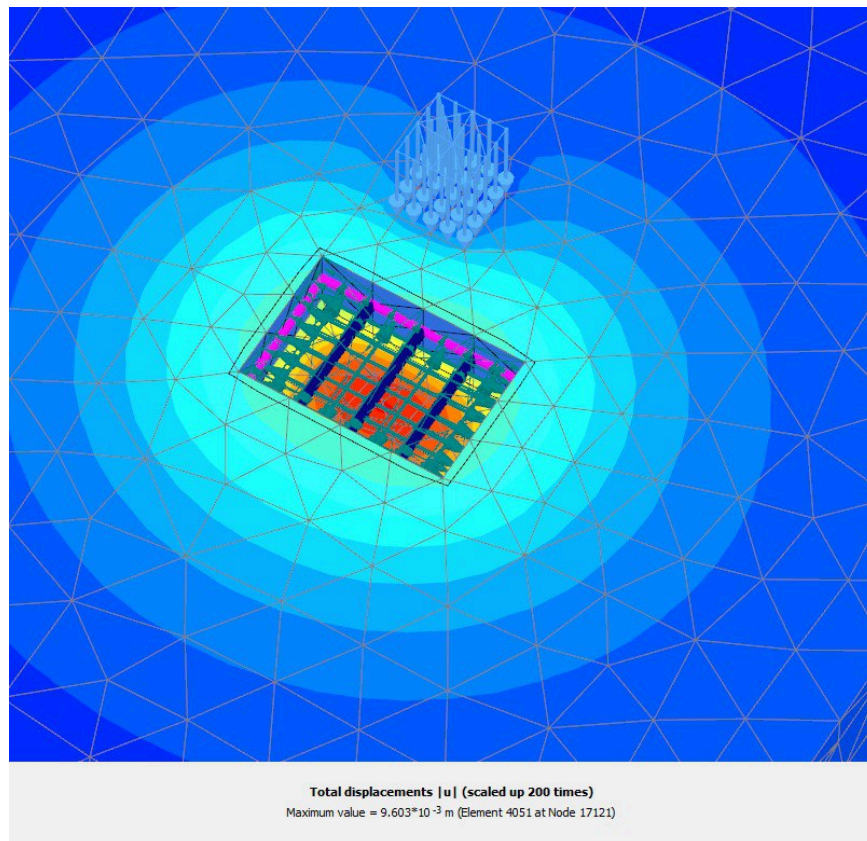


Figure 3.7.3. Loads distribution in Plaxis 3D

3.8 Detailed Construction Procedure

Besides the site surveying and soil investigation processes, site preparation process before the actual construction is crucial. Its importance lies in ensuring the safety, stability, and efficiency of the entire project. When a site is properly prepared, it removes potential hazards such as tree stumps, debris, and loose soil that could otherwise cause accidents or

hinder construction. It also allows for accurate layout and leveling, which ensures that the building is positioned exactly as planned in the design documents.

Site surveying and Soil testing. Site surveying is the process of measuring and mapping the land to determine boundaries, ground levels, and the location of existing features like roads or utilities. It helps ensure that the building is placed accurately according to the design plans. Soil testing, also called geotechnical investigation, checks the type, strength, and condition of the soil beneath the site. Engineers drill boreholes and take soil samples to test in a lab. The results help decide what kind of foundation is safe and suitable for the building.

Soil Plan Design is the process of creating a plan for how the building's foundation will interact with the soil, based on the results of soil testing. It includes choosing the right type of foundation (such as shallow footings, raft, or deep piles) depending on the soil's strength and water level. The design also shows the depth and size of the foundation and includes any special measures like soil compaction, drainage layers, or ground improvement if the soil is weak.

Site Clearing and Grubbing is one of the primary steps in the site preparation process. It includes preparing the land by removing all natural and manmade obstacles that could interfere with the building work. The importance of this step is due to ensure a stable, clean, and obstruction-free site for excavation and foundation construction. Site clearing refers to the removal of surface-level vegetation and materials. This includes cutting down trees, removing shrubs, bushes, tall grass, and scraping away topsoil that contains organic matter like roots and leaves.

Site grading and leveling means shaping the ground to make it even and smooth for building. Workers cut soil from higher spots and move it to lower spots to get the right height and slope. This helps rainwater flow away from the building so there is no flooding or water damage. After that, the soil is pressed down to make it strong and steady. It also makes it easier and safer for machines and workers to move around. Good grading helps keep the building safe and strong over time.

Sheet pile installation involves driving interlocking panels into the ground to create a barrier. This method is used to support excavation sites and prevent soil movement, especially in waterlogged or deep excavations. The panels are driven vertically into the ground to form a continuous wall that helps with soil retention and water control. Sheet piles are commonly used in foundation work, retaining walls, and waterfront structures.

Excavation is the process of digging or removing soil and materials from a site to prepare for construction. It is done to create space for foundations, basements, or utilities. Machinery or manual labor is used to dig, and the material is either removed or reused for backfilling. Excavation is important because it prepares the site by creating the necessary space for a stable foundation and ensuring the structure is built on solid ground.

Substructure construction refers to the part of a building that is below ground level, including foundations, basement floors, and retaining walls. It involves the construction of footings, slabs, columns, and walls that support the structure above. This phase includes tasks like excavation, pouring concrete, installing reinforcement, and ensuring the foundation is

stable and level. Substructure work is critical for distributing the building's weight to the ground and preventing settlement or shifting of the structure over time.

Pile installation is the process of driving or drilling long, strong columns (piles) into the ground to transfer a building's weight to deeper, more stable soil or rock. This method is used when the surface soil cannot support the load of the structure. Piles can be made of materials like concrete, steel, or wood, and they are either driven into the ground or placed by drilling and then filled with concrete. This technique provides stability for structures in weak or shifting soils, ensuring the building's foundation is secure.

The detailed construction procedure is essential for ensuring that a building is constructed safely, efficiently, and according to design specifications. Each step, from site preparation to final finishes, contributes to the stability, functionality, and durability of the structure. By following a systematic process, potential issues can be identified and addressed early, reducing the risk of costly delays and structural problems. Proper planning and execution at each stage also ensure compliance with safety standards and regulations, ultimately leading to the successful completion of a reliable, long-lasting building.

4. Environmental Area

1.1. Introduction

This report presents the comprehensive design and analysis of the stormwater and drainage system for a 12-story hotel project located in Seattle, WA. As part of the environmental component of the Capstone project, the goal is to ensure efficient, sustainable, and code-compliant stormwater management. The scope includes the hydrologic analysis, open channel (gutter) design, storm sewer pipe sizing, time of concentration (Tc) calculations, and grading plan. Seattle's rainfall patterns and dense urban environment necessitate well-planned drainage systems to prevent flooding, erosion, and property damage.

Stormwater runoff consists of the building roof (Area 1), paved driveways (Area 1-4), and surrounding hardscape (Area 2 and 7). These impervious areas contribute to rapid runoff, requiring properly designed conveyance elements to capture, direct, and safely discharge stormwater into the existing city's drainage infrastructure.

The design is governed by local and national standards, including the Seattle Stormwater Manual, NOAA Atlas 14 (for rainfall intensity data), NRCS TR-55 (for time of concentration calculations), and FHWA HEC-22 (for gutter and open channel design).

1.2. Site description

The project site is located in a developed urban area with an existing slope and elevation change of approximately 4 meters. The drainage design must consider both natural grading and engineered elevation adjustments to ensure all runoff is captured and conveyed efficiently.

- Building Area (1): 1487.16 m²
- Area 2: 250 m²
- Area 3: 1110 m²
- Area 4: 555 m²
- Area 5: 555 m²
- Area 6: 555 m²
- Area 7: 100 m²
- Slope:
 - Roof: 2%
 - Parking area: 1.5% slope.

The Figure below illustrates the elevations, the direction of water flow with gutters, the place where it connects to the existing sewer system and divided areas.



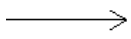
Figure 4.1 Site plan.

19.5



- Proposed elevation

1.5%



- Slope with its direction

21

- Natural elevation

1.3. Grading and Drainage Plan

A grading plan was developed to guide surface water toward designated drainage points. This strategy is essential to avoid water ponding, erosion, and overloading of downstream infrastructure.

1.3.1. Key grading features:

- Building pad elevation set at 21.5 meters to match existing contours
- Cut-and-fill strategy: southern areas are cut (22-21 m), and northern zones (18-20 m) are filled to balance earthwork
- Surface runoff directed through trapezoidal gutters toward inlet to city's pipelines
- Rooftop downspouts are connected to storm sewers via catch basins

The elevation goes down around the building so all runoff will go far from it. The site naturally drains toward the northeast, where a connection to the city's storm sewer network exists at 8th Avenue.

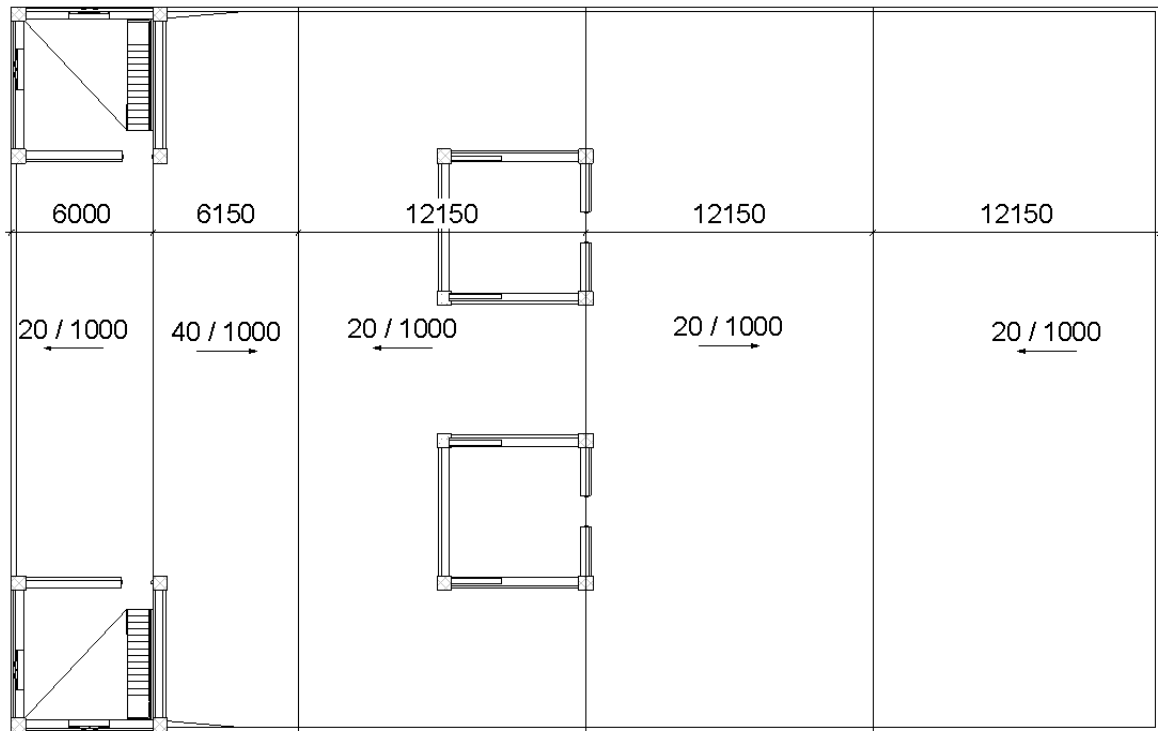


Figure 4.2. Roof Plan.

1.3.2. Existing infrastructure integration

The storm sewer system is quite integrated and compatible with the current functional layout of the city. Taking into consideration the elevation map and the current sewer drainage layout in the area, pipes for water flow will be delivered to the site's northeastern section. From there, water will link to the city's mainline coming across 8th Avenue.

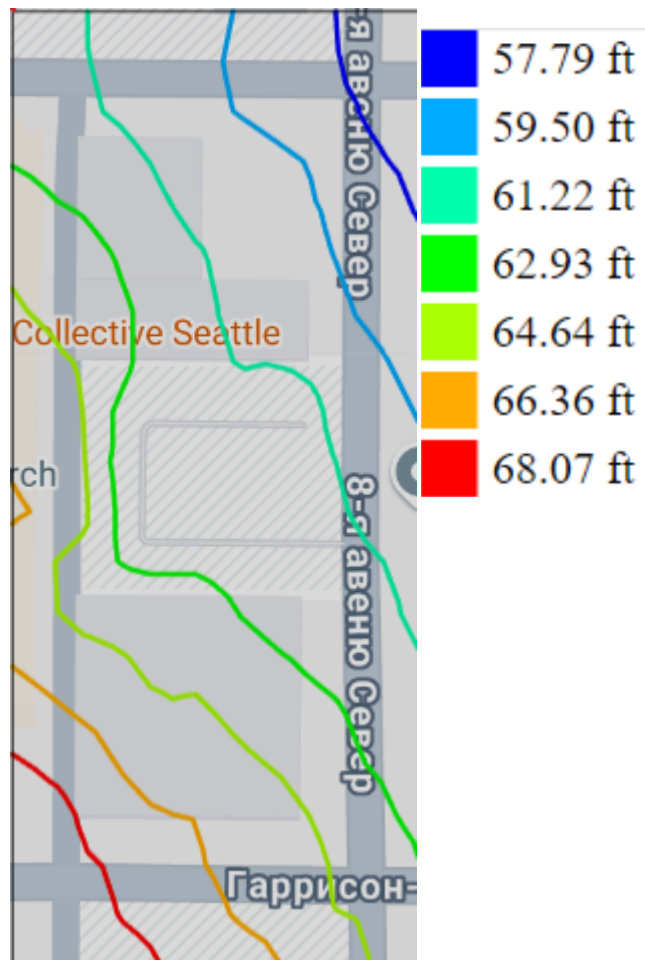


Figure 4.3. Elevation map of construction site.



Figure 4.4. Existing sewer drainage map.

The stormwater drainage and grading system proposed for this type of project is definitely one postmodern gesture toward ecological responsibility in site design. Co-ordinated integration involves features such as; natural contours, grading slopes, catch basins as elements which allow collection and redirection of runoff water to prevent floods and impacts on buildings. In the grading and drainage plan the natural elevation and contour line is adopted for the efficient drainage and flow lines of water to the drainage points. The design thus facilitates efficient transfer of water from parking lots, rooftops and other surfaces to catch basins. Slopes are 1.5 % to 2% hence the site should not have water logging and the effects of excessive runoff are negligible.

Cut-and-fill operations are planned in a way that construction and development activity obtains a level ground that does not interfere with natural drainage systems. In essence, the more elevated areas are being synchronized with the lower areas in order to improve flow of water across the site.

In other words, this stormwater management plan lacks novelty for managing runoff and achieving functional objectives for drainage, but also considers principles of sustainability. Gradation and slope design indicate system compatibility with corresponding care for environment and urban sustainability, ensuring long performance and compliance with local standards. Of course, this project will represent a good example of the high level of engineering that meets current requirements for drainage in urban projects.

1.4. Calculation of Time of Concentration T_c

The time of concentration (T_c) represents the time required for water from the most hydraulically distant point of a drainage area to reach the outlet. T_c affects the selection of rainfall intensity in peak flow calculations.

The NRCS TR-55 method was used:

- **Sheet Flow:** over short distances (≤ 100 ft), modeled using Manning's equation
- **Shallow Concentrated Flow:** flow after initial sheet phase, velocity-based using standard tables

We use the following formulas to calculate time of concentration:

$$Tt1 = \left(\frac{0.42}{P(24)^{0.5}} \right) * \frac{(nL)^{0.8}}{(\sqrt{S})^{0.6}} - \text{for sheet flow (min)}$$

$$V = k * S^{0.5}; Tt2 = \frac{L}{60V} - \text{for shallow concentrated flow (min)}$$

For parking zone 1, we use:

$L=30$ m = 98.43 ft - length of flow

$n=0.011$ - roughness coefficient for smooth asphalt according to Seattle's stormwater manual

$s=1.5\%$ (0.015) - slope

$P(24) = 2.5$ in - 2-year, 24-hour rainfall

V - velocity of flow

$k = 30$ (according to Seattle's stormwater manual) - velocity factor for paved areas

Rainfall Intensity (I): Based on Hydrologic analysis and design for Seattle, assuming the 10-year recurrence of storm event with 5 minute duration we get the following data.

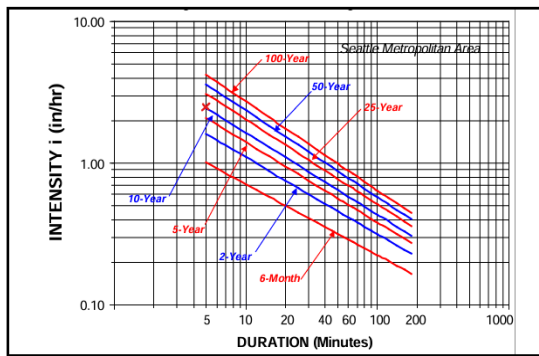


Figure F.18. Intensity-Duration-Frequency Curves for the City of Seattle.

Duration (minutes)	Precipitation Intensities (in/hr)							
	Recurrence Interval (years)							
	6-mo	2-yr	5-yr	10-yr	20-yr	25-yr	50-yr	100-yr
5	1.01	1.60	2.08	2.45	2.92	3.08	3.61	4.20
6	0.92	1.45	1.87	2.21	2.62	2.76	3.23	3.75
8	0.80	1.24	1.59	1.87	2.21	2.32	2.71	3.13
10	0.71	1.10	1.40	1.64	1.93	2.03	2.36	2.72
12	0.65	1.00	1.27	1.48	1.74	1.82	2.11	2.43
15	0.58	0.88	1.12	1.30	1.52	1.60	1.84	2.11

Figure 4.5. Intensity rate.

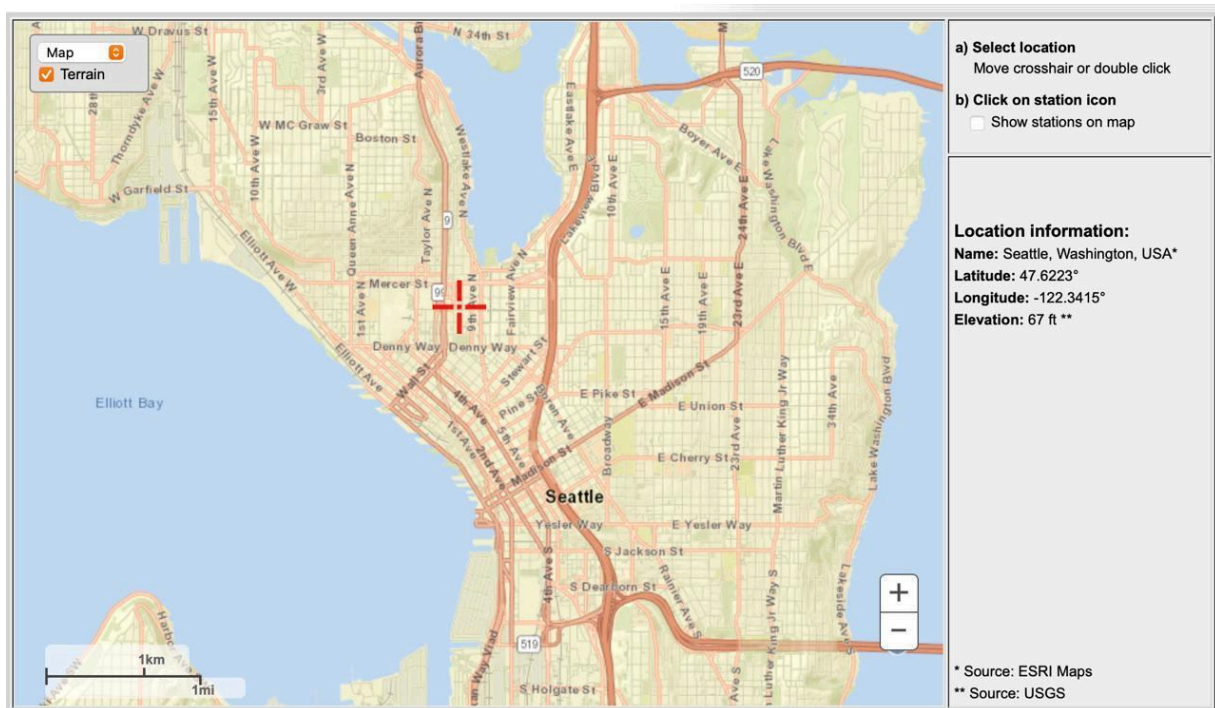


Figure 4.6. Intensity of the location of the building.

By using the previous equations and adding concentration time for sheet and shallow flow to determine Total time of Concentration we get the following results.

Table 4.1 Result of Time of Concentrations.

Area	n	L, ft	s	k	V	T1, min	T2, min	T
A1	0.011	98.43	0.02	30	4.242640687	1.354437952	0.3866695582	1.741107511
A2	0.011	16.405	0.015	30	3.674234614	0.3624212124	0.07441459117	0.4368358036
A3	0.011	98.43	0.015	30	3.674234614	1.51961863	0.446487547	1.966106177
A4	0.011	60.6985	0.015	30	3.674234614	1.032226461	0.2753339873	1.307560449
A5	0.011	98.43	0.015	30	3.674234614	1.51961863	0.446487547	1.966106177
A6	0.011	98.43	0.015	30	3.674234614	1.51961863	0.446487547	1.966106177
A7	0.011	6.562	0.015	30	3.674234614	0.1741251895	0.02976583647	0.2038910259

1.5. Peak Flow Calculations

The Rational Method was used to estimate peak flows:

$Q = C \cdot I \cdot A$ – peak flow

$C = 0.9$ Runoff Coefficients for both parking and roof area was taken as this as for impervious pavement and Solid CMU,

$I = 2.45 \text{ in/hr}$

$A = 4070 \text{ m}^2$. (37x110 m)

Below is shown the example for Area 3:

$Q_{\text{cfs}} = 0.0002491661 \cdot 0.9 \cdot 2.45 \cdot 555 = 0.305 \text{ CFS}$

0.0002491661 - is the unit convection coefficient.

Table 4.2 Peak runoff results.

Area	A, m ²	i, in/hr	C	unit coefficient	Q, cfs
A1	1487.16	2.45	0.9	0.0002491661	0.8170624353
A2	250	2.45	0.9	0.0002491661	0.1373528126
A3	555	2.45	0.9	0.0002491661	0.304923244
A4	555	2.45	0.9	0.0002491661	0.304923244
A5	555	2.45	0.9	0.0002491661	0.304923244
A6	555	2.45	0.9	0.0002491661	0.304923244
A7	100	2.45	0.9	0.0002491661	0.05494112505
Total					2.229049349

1.6. Design

The storm sewer system was designed using the Rational Method to determine peak runoff, and Manning's equation for pipe sizing, according to Seattle Stormwater Code and the City of Seattle Drainage Manual.

1.6.1. Pipe design

The total impervious area of the project site is approximately 1.006 acres, including roof and parking surfaces. Using a rainfall intensity of 2.45 in/hr for a 10-year, 5-minute storm (Hydrologic Analysis and Design) and a runoff coefficient of 0.9, the peak flow was estimated as:

$$Q = C \cdot I \cdot A = 0.0002491661 \cdot 0.9 \cdot 2.45 \cdot 4070 = 2.23 \text{ cfs}$$

To safely convey stormwater runoff, we need to determine the diameter of the pipe with a slope of 1.5%. Using Manning's equation with a roughness coefficient of $n = 0.013$, the pipe has to withstand a flow of approximately 2.23 cfs.

$$Q = \frac{1.486AR^{2/3}S^{1/2}}{n}$$

$$n = 0.013$$

$$S = 0.015$$

$$A = \frac{\pi D^2}{4} - \text{Area}$$

$$R = \frac{D}{4} - \text{Hydraulic radius (A/P)}$$

By several iterations (Table 4.3) of calculations there was identified that a pipe with 0.8 ft diameter gives us flow equal to 2.41 cfs. The closest diameter that meets the size of a real pipe is 10 inches. Therefore we will be using a 10 inch diameter PVC pipe.

Table 4.3 Iteration to design pipe diameter.

D, ft	A, ft ²	P, ft	R, ft	n	s	Q, cfs
1	0.7853981634	3.141592654	0.25	0.013	0.015	4.363526728
0.9	0.6361725124	2.827433388	0.225	0.013	0.015	3.294713577
0.8	0.5026548246	2.513274123	0.2	0.013	0.015	2.406638938
0.7	0.3848451001	2.199114858	0.175	0.013	0.015	1.685643566
0.67	0.3525652355	2.104867078	0.1675	0.013	0.015	1.499813002

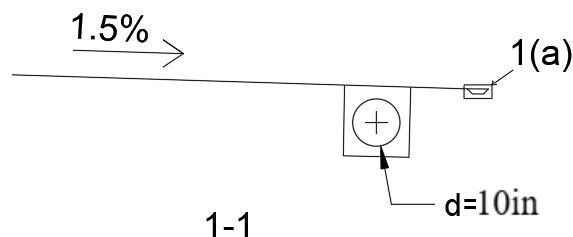


Figure 4.7. Cross-Section view of pipe.

1.6.2. Gutter (open channel) design

The highest flow that has to be carried by gutters is the sum of Area 2, 4 and 5 which is:

$$Q = 0.137 + 0.305 + 0.305 = 0.747 \text{ cfs}$$

So we need to design the gutter's size according to that flow. By several iterations as illustrated on the table below there was chosen a gutter with width $b=0.75\text{ft}$ and height of flow $h = 0.25\text{ft}$, but for design we will make it 0.3ft .

$$Q = \frac{1.486AR^{2/3}S^{1/2}}{n}$$

$$A = by + Zy^2 - \text{Area}$$

$$Z = 1 - \text{side slope (H/V)}$$

$$P = b + 2y * \sqrt{(1 + Z^2)} - \text{Perimeter}$$

$$R = \frac{A}{P} - \text{Hydraulic radius}$$

$$n = 0.016 \text{ (according to local Manual)}$$

$$s = 0.015 - \text{slope}$$

Table 4.4. Iteration for design of gutters.

b, ft	y, ft	Z	A, ft ²	P, ft	R, ft	n	s	Q, cfs
0.75	0.62	1	0.8494	2.504	0.339	0.016	0.015	4.700
0.75	0.5	1	0.625	2.164	0.289	0.016	0.015	3.106
0.75	0.1	1	0.085	1.033	0.082	0.016	0.015	0.183
0.75	0.4	1	0.46	1.881	0.245	0.016	0.015	2.046
0.75	0.2	1	0.19	1.316	0.144	0.016	0.015	0.595
0.75	0.3	1	0.315	1.599	0.197	0.016	0.015	1.213
0.75	0.25	1	0.25	1.457	0.172	0.016	0.015	0.878

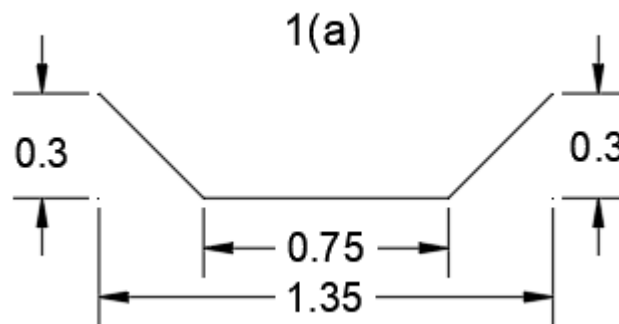


Figure 4.8. Cross-section view of gutter.

1.7. Integration and sustainability.

The outlet of the system connects to Seattle's public storm sewer at 8th Avenue. Gutter flows are collected by inlets and connected to the city's pipelines. System layout and slopes

are coordinated with city-provided GIS elevation data to ensure compatibility and legal compliance.

- Minimized earthwork through balanced cut and fill
- Low-flow-friendly slope designs to reduce erosion
- Paved gutters lined with concrete for durability and low maintenance
- Code-compliant stormwater management reduces urban runoff impacts
- System supports long-term serviceability and water quality

1.8. Conclusion

This report outlines the complete environmental and hydrologic analysis and design for a 15-story hotel project in Seattle. The storm sewer system includes effective grading, time of concentration assessment, peak flow calculation, open channel design, and storm pipe sizing. The system was designed for current and projected rainfall conditions using the most reliable tools and codes. The final result is a robust, efficient, and environmentally responsible stormwater infrastructure that integrates seamlessly into Seattle's existing drainage network.

5. Construction Management Area

5.1 Project Charter

Table 5.1. Project Charter

General Project Information	
Project Title	Design of a 12-story Hotel “Skyline Palace” in Seattle, Washington, USA
Project description	There will be designed and constructed a 12-story Hotel in Seattle to provide temporary accommodation to rest for visitors
Start date	Finish date
September 1st, 2025	March 30th, 2028
Project members	Assem Nurtaza Meruyert Kerimbay Kazybek Bestan Aruzhan Serikova Zhanserik Shadiyar Rolan Rysbek
Project Scope	
Project Deliverables	• A 12-story high-rise hotel: The project aims to build a luxury hotel catering to tourists, offering modern amenities and services that contribute to the local tourism industry. • A parking lot: This will accommodate the hotel’s guests and employees, ensuring smooth accessibility and convenience for visitors. • Basement: The basement will house storage facilities, the building’s mechanical and electrical systems, and a staff area for operational functions.
Goals	• Providing travelers with temporary lodging: Creating a comfortable, modern, and luxurious space for travelers to stay during their visits, whether for business or leisure. • Contributing to the development of the local architecture: Designing a building that blends with the city’s landscape while offering a modern architectural statement that draws visitors. • Development of the local economy: The hotel will generate jobs both during and after construction, providing employment opportunities in construction, hospitality, and services. • Attracting tourists: The hotel aims to become a destination in itself, with services such as guided tours, event spaces,

	and local cultural experiences.		
Possible risks	• Environmental Risks: Potential delays due to weather conditions, or environmental impact assessments that could alter the scope of construction. • Regulatory Risks: Changes in local laws or zoning regulations that may cause delays in construction or lead to design modifications. • Technical Issues: Challenges in building technology implementation, such as the integration of sustainable energy solutions or modern infrastructure systems.		
Assumptions/ Dependencies	• The hotel construction expenses were estimated to be paid off in the first 8.5 years: We assume that the project's financing and profitability model, including guest occupancy rates and operational costs, will meet projected expectations. • Dependencies on local contractors: The timely availability of qualified contractors and suppliers is essential for the project to stay on schedule.		
Estimated Budget	\$24612733		
Stages	Phase	Status	Finish date
	Pre-construction	In progress	December 30th, 2025
	Site work	To be done	May 28th, 2026
	Main construction	To be done	August 3rd, 2027
	Finishing	To be done	March 30th, 2028

5.2 Feasibility Study

The purpose of this feasibility study is to assess the viability of the proposed construction project, ensuring that the investment is economically, technically, and environmentally feasible. The project involves the construction of the hotel, which is designed to provide visitors with temporary lodging to rest, located in Seattle, Washington, USA.

5.3 Risk Management and Safety Assessment

Table 5.2. General Risk Management

Risk category	Description	Probability	Significance	Mitigation plan
Financial risks	Excessive use of materials	medium	medium	The usage of detailed project plans, checking its accuracy, and implementation of the waste reduction practices
	Significant cut of project financing	low	high	Exploring alternative resources by finding new investors, evaluating the remaining budget, and redirecting it
	Fluctuation in material and labor cost	high	high	Improving the budget management, analyzing the market change, and working with suppliers that have fixed-prices
Safety risks	Material Handling	high	medium	Implementing safety training sessions for construction workers and providing them with proving them with safety equipment
	Electrical hazards	medium	high	Identifying all areas with electrical equipment and checking its safety, conducting electrical safety measurements
	Weather-related dangers	medium	high	Designing the building that will be adaptable to climate change, and providing emergency training sessions
Scheduling risks	Delays in construction	high	high	Implementing the weather monitoring systems, which would allow construction workers to predict and prepare for the weather changes
	Labor shortages	high	high	Automation of the major part of construction by using drones would allow the reduction of the reliance on labor.
	Construction errors	high	high	Recruiting highly skilled workers for the project construction, planning the project with an error-detecting program

Quality risks	The inaccurate usage of construction materials	medium	medium	Improving the partnership with suppliers that provide high-quality construction materials
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Table 5.3. Detailed Construction Safety Assessment

Identified Hazard	Likelihood (1-5)	Severity (1-5)	Risk Score
Heights & Fall Risks	4	2	8
Slips, Trips, & Falls	3	4	12
Moving Equipment Hazards	3	5	15
Fire & Explosion Risks	3	3	9
Structural Integrity Failures	2	4	8
Insufficient PPE Use	3	3	9
Extreme Temperature Exposure	3	3	9
Electrical Shock Hazards	2	4	8
Excessive Noise Levels	2	3	6
Entrapment & Pinch Points	2	3	6
Hazardous Material Handling	3	2	6
Repetitive Strain & Vibration	3	2	6

5.4 Work Breakdown Structure

Table 5.4. Work Breakdown Structure distribution

Construction of a 12-story Hotel		
Elements	sub-elements	Process description
1. Pre-construction		
1.1 Planning	Site Selection, Building Design, Feasibility study	Scheduling the step-by-step construction process, choosing the suitable location for a building, and estimating the budget and profitability of the building.
1.2 Documentation	Land permission, Construction permission	Receiving the permission license for the land and building a hotel that would meet all the requirements
1.3 Financing	Budget estimation, Budget management, Project Financing	Finding investors or an investment company that would finance the project, and control the accurate management of the budget
1.4 Procurement	Recruitment, Material sourcing, Equipment procurement	Recruiting qualified workforce groups, purchasing necessary material elements, including columns, excavators, trucks, and technical equipment
2. Site Work		
2.1 Site Preparation	Excavation, Site Cleaning	Creating a stable foundation, and removing all building elements that would interfere with the building construction
2.2 Foundation	Piling, Concrete foundation, Water protection	Implementing the site investigation, selecting the appropriate pile, creating the concrete mixture, and setting up the drainage system on the lower levels of the building
2.3 Utility Installation	Water pile connection, MEP	Installing the PVC piles, connecting piles with the main water supply, and connecting electrical wires to the electrical grid
3. Main construction		
3.1 Structural Frame	Columns, Beams, Slabs	Steel reinforcement bars are moved to the formwork and held until it reaches a significant strength level, and the formwork is removed at the end of the process
3.2 Walls and Roofing	Brick masonry, Roof covering	Laying bricks into a staggered pattern with mortar, and preparing necessary roof materials

3.3 Doors, and Windows	-	Placing doors and windows
4. Finishing		
4.1 Exterior finishing	Coating, Painting	Covering the wall surface with paintings and a coating layer that prevents water penetration
4.2 Interior finishing	Soundproofing, Flooring, Decorating, Cleaning, Inspecting	Installing, acoustic panels, decorating rooms, and making final corrections

5.5 Quality Management and Procurement Planning



Quality Control Checklist

PROJECT INFORMATION:

LOCATION: SEATTLE, WASHINGTON, USA

DATE: 18.04.2025

PROJECT NAME: SKYLINE PALACE

BUILDING TYPE: 12-STORY HOTEL

PROJECT MEMBERS:

KAZYBEK BESTAN

MERUYERT KERIMBAY

ASSEM NURTAZA

ROLAN RYSBEK

ARUZHAN SERIKOVA

ZHANSEK SHADIYAR

No	ACTIVITIES	☒	FEEDBACK
1	Site Preparation & Safety	☐	
2	Permit & Legal Requirements	☐	
3	Foundation Work	☐	
4	Structural Steel & Framing	☐	
5	Concrete Work	☐	
6	Roofing	☐	
7	Exterior Cladding	☐	
8	Plumbing Installation	☐	
9	Electrical Installation	☐	
10	HVAC Systems	☐	
11	Insulation & Soundproofing	☐	
12	Drywall & Interior Finish	☐	
13	Flooring Installation	☐	
14	Painting & Surface Finishes	☐	
15	Windows & Doors	☐	
16	Staircases & Elevators	☐	
17	Fire Safety Systems	☐	
18	Site Cleanliness & Waste Management	☐	
19	Final Inspections	☐	
20	Commissioning & Handover	☐	

Figure 5.1 General Quality Checklist

5.6 Green Building Certification

The green building certification plan for a 12-story hotel in Seattle is designed to attain LEED Gold certification, highlighting a commitment to sustainability and energy efficiency. This project will pursue LEED v4 for Building Design and Construction, aiming to accumulate 60-79 points required for the Gold level certification.

For the Location and Transportation criteria, the project will facilitate access to public transportation and provide electric vehicle charging stations. The Sustainable Sites plan includes strategies to reduce heat island effects through the use of reflective materials and implementing stormwater management systems like rain gardens. Water Efficiency will be enhanced by installing low-flow water fixtures inside the building and utilizing drought-resistant plants in the landscape to decrease irrigation needs.

In terms of Energy and Atmosphere, the project will prioritize energy optimization by using high-efficiency systems and adding on-site renewable energy sources such as solar panels. Sustainable and recycled materials will be used wherever possible, while a robust construction waste management plan will aim to reduce waste sent to landfills, addressing the Materials and Resources category.

To ensure high-quality indoor environments, the project will focus on improving Indoor Environmental Quality by increasing ventilation rates, using materials with low levels of volatile organic compounds (VOCs), and maximizing the use of natural daylight. Additionally, the project will incorporate innovative solutions, such as biophilic design, and pursue exemplary performance in water use reduction under the Innovation and Design category. For Regional Priority Credits, the hotel will concentrate on energy efficiency and effective stormwater management, tailored to Seattle's local conditions. Achieving LEED Gold certification will offer numerous benefits, including reduced operational costs, enhanced indoor air quality, alignment with Seattle's sustainability initiatives, and improved marketability of the hotel as a leader in environmental responsibility and eco-friendly hospitality.

5.7 Gantt Chart

The Gantt Chart was created by the use of the platform "Primavera p6". The overall duration of the Construction process was expected to take 674 days, where Saturdays, Sundays, and National Holidays in the Calendar between September 1st of 2025 and March 30th of 2027 were taken into consideration and excluded from the duration process of the construction.

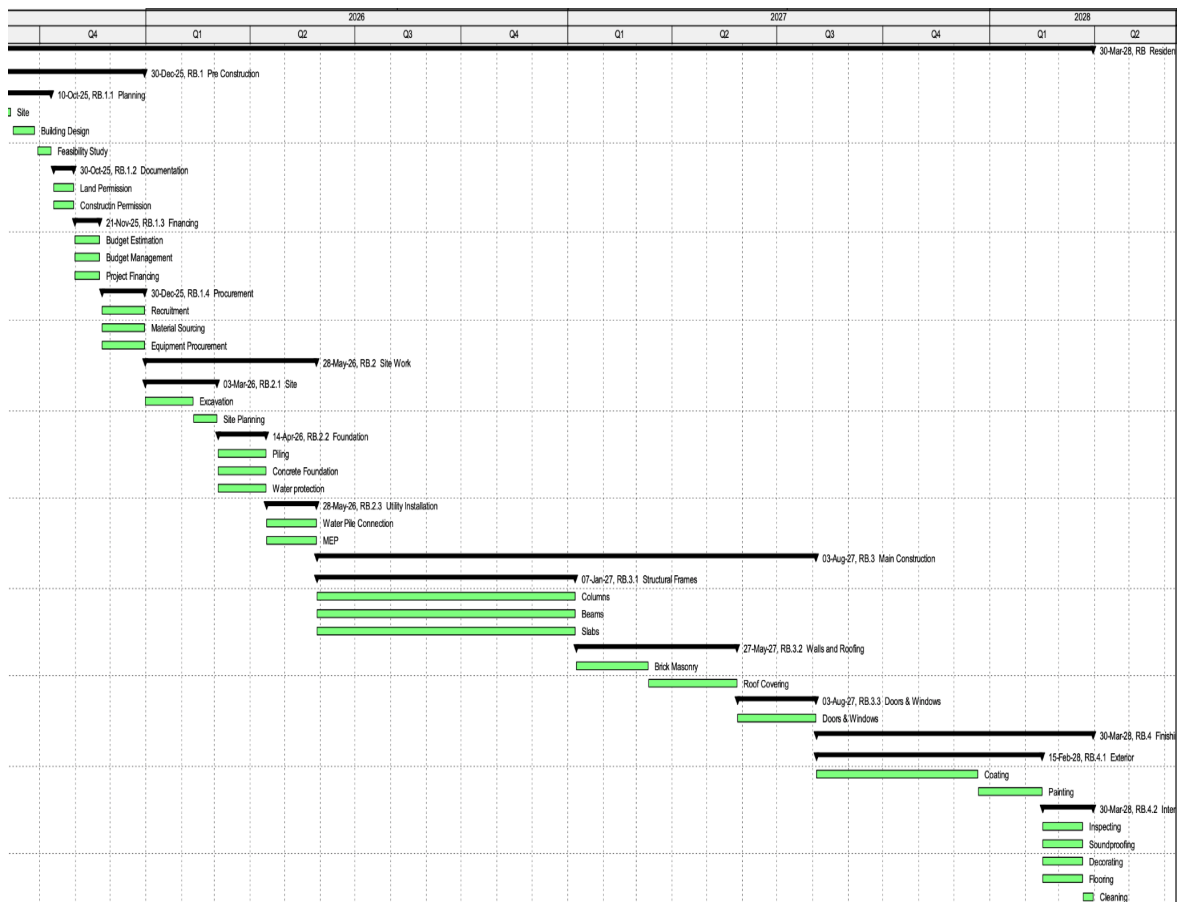


Figure 5.2. Gantt Chart (part 1)

2		RB.1 Pre Construction	87	87	0%	01-Sep-25	30-Dec-25
3		RB.1.1 Planning	30	30	0%	01-Sep-25	10-Oct-25
4		P1250 Site	5	5	0%	01-Sep-2	05-Sep-25
5		P1040 Building Design	15	15	0%	08-Sep-2	26-Sep-25
6		P1140 Feasibility Study	10	10	0%	29-Sep-2	10-Oct-25
7		RB.1.2 Documentation	14	14	0%	13-Oct-25	30-Oct-25
8		P1170 Land Permission	14	14	0%	13-Oct-25	30-Oct-25
9		P1090 Constructin Permission	14	14	0%	13-Oct-25	30-Oct-25
10		RB.1.3 Financing	16	16	0%	31-Oct-25	21-Nov-25
11		P1020 Budget Estimation	16	16	0%	31-Oct-25	21-Nov-25
12		P1030 Budget Management	16	16	0%	31-Oct-25	21-Nov-25
13		P1220 Project Financing	16	16	0%	31-Oct-25	21-Nov-25
14		RB.1.4 Procurement	27	27	0%	24-Nov-25	30-Dec-25
15		P1230 Recruitment	27	27	0%	24-Nov-2	30-Dec-25
16		P1180 Material Sourcing	27	27	0%	24-Nov-2	30-Dec-25
17		P1120 Equipment Procurement	27	27	0%	24-Nov-2	30-Dec-25
18		RB.2 Site Work	107	107	0%	31-Dec-25	28-May-26
19		RB.2.1 Site	45	45	0%	31-Dec-25	03-Mar-26
20		P1130 Excavation	30	30	0%	31-Dec-2	10-Feb-26
21		P1260 Site Planning	15	15	0%	11-Feb-25	03-Mar-26
22		RB.2.2 Foundation	30	30	0%	04-Mar-26	14-Apr-26
23		P1210 Piling	30	30	0%	04-Mar-25	14-Apr-26
24		P1080 Concrete Foundation	30	30	0%	04-Mar-25	14-Apr-26
25		P1300 Water protection	30	30	0%	04-Mar-25	14-Apr-26
26		RB.2.3 Utility Installation	32	32	0%	15-Apr-26	28-May-26
27		P1290 Water Pile Connection	32	32	0%	15-Apr-25	28-May-26
28		P1190 MEP	32	32	0%	15-Apr-25	28-May-26
29		RB.3 Main Construction	308	308	0%	29-May-26	03-Aug-27
30		RB.3.1 Structural Frames	160	160	0%	29-May-26	07-Jan-27
31		P1070 Columns	160	160	0%	29-May-2	07-Jan-27
32		P1000 Beams	160	160	0%	29-May-2	07-Jan-27
33		P1270 Slabs	160	160	0%	29-May-2	07-Jan-27
34		RB.3.2 Walls and Roofing	100	100	0%	08-Jan-27	27-May-27
35		P1010 Brick Masonry	45	45	0%	08-Jan-25	11-Mar-27
36		P1240 Roof Covering	55	55	0%	12-Mar-2	27-May-27
37		RB.3.3 Doors & Windows	48	48	0%	28-May-27	03-Aug-27
38		P1110 Doors & Windows	48	48	0%	28-May-2	03-Aug-27
39		RB.4 Finishing	172	172	0%	04-Aug-27	30-Mar-28
40		RB.4.1 Exterior	140	140	0%	04-Aug-27	15-Feb-28
41		P1060 Coating	100	100	0%	04-Aug-2	21-Dec-27
42		P1200 Painting	40	40	0%	22-Dec-2	15-Feb-28
43		RB.4.2 Interior Finishing	32	32	0%	16-Feb-28	30-Mar-28
44		P1160 Inspecting	25	25	0%	16-Feb-25	21-Mar-28
45		P1280 Soundproofing	25	25	0%	16-Feb-25	21-Mar-28
46		P1100 Decorating	25	25	0%	16-Feb-25	21-Mar-28
47		P1150 Flooring	25	25	0%	16-Feb-25	21-Mar-28
48		P1050 Cleaning	7	7	0%	22-Mar-25	30-Mar-28

Figure 5.3. Gantt Chart (part 2)

Work Breakdown Structure

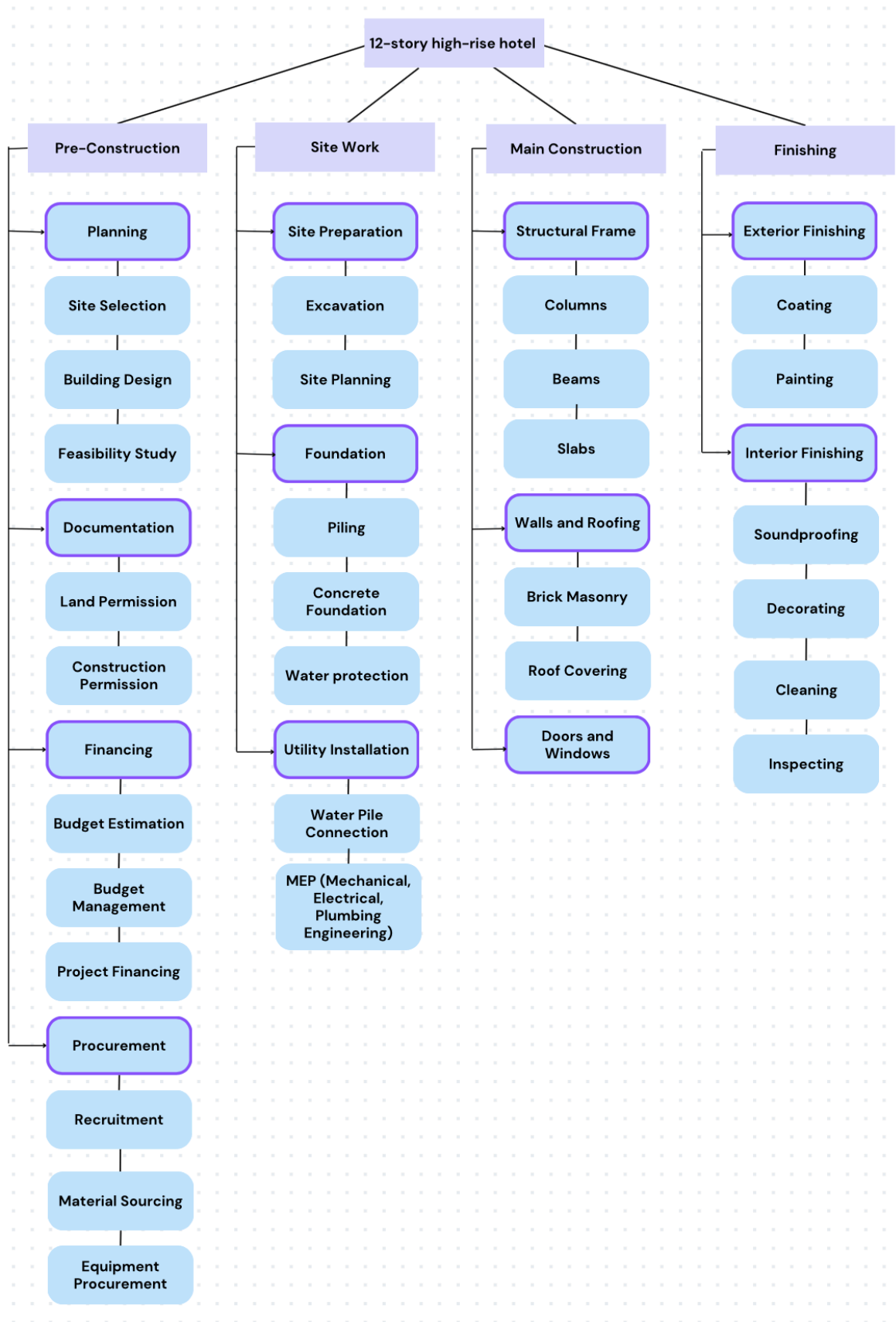


Figure 5.4. Work Breakdown Structure

5.8 RSMeans Cost Estimation

The approximate cost of the construction was estimated by the implementation of the RSMeans Online calculator:

Model: Hotel, 8-24 Story with Brick Veneer / Reinforced Concrete

Default Building Image
📷 🗑️



\$17,381,460.36	SEATTLE, WA	12	Yes
Building Cost	Location	Stories (Ea.)	Basement
\$1,076.52	16,146	10.50	\$3,683,887.00
Cost per S.F.	Floor Area	Story Height	Additive Cost

Figure 5.5. The cost estimation in RSMeans

Table 5.5. The detailed cost estimation in RSMeans

		Quantity	% of Total	Cost per S.F.	Cost
A	Substructure		3.29%	\$26.70	\$431,162.74
A1010	Standard Foundations			\$0.46	\$7,456.01
	Pile caps, 6 piles, 8' - 6" x 5' - 6" x 40", 80 ton capacity, 19" column size, 936 K column	1.14		\$0.16	\$2,529.86
	Pile caps, 8 piles, 8' - 6" x 7' - 9" x 44", 80 ton capacity, 22" column size, 1243 K column	1.61		\$0.31	\$4,926.15
A1020	Special Foundations			\$14.79	\$238,862.76
	Steel H piles, 100' long, 800K load, end bearing, 5 pile cluster	1.14		\$2.42	\$39,146.75
	Steel H piles, 100' long, 1200K load, end bearing, 8 pile cluster	1.61		\$5.45	\$87,976.33
	Grade beam, 30' span, 52" deep, 14" wide, 12 KLF load	520		\$6.92	\$111,739.68
A1030	Slab on Grade			\$0.51	\$8,299.58
	Slab on grade, 4" thick, non industrial, reinforced	1345.5		\$0.51	\$8,299.58
A2010	Basement Excavation			\$0.34	\$5,481.03
	Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on site storage	1345.5		\$0.34	\$5,481.03
A2020	Basement Walls			\$10.59	\$171,063.36

	Foundation wall, CIP, 12' wall height, pumped, .591 CY/LF, 28.79 PLF, 16" thick	520		\$10.59	\$171,063.36
B	Shell		50.23%	\$408.07	\$6,588,741.50
B1010	Floor Construction			\$230.64	\$3,723,846.76
	Cast-in-place concrete column, 26" square, tied, 1000K load, 12' story height, 667 lbs/LF, 4000PSI	1083.09		\$14.65	\$236,461.52
	Cast-in-place concrete column, 12", square, tied, minimum reinforcing, 150K load, 10'-14' story height, 135 lbs/LF, 4000PSI	681.2		\$3.02	\$48,754.30
	Cast-in-place concrete column, 16", square, tied, minimum reinforcing, 300K load, 10'-14' story height, 240 lbs/LF, 4000PSI	681.2		\$4.15	\$66,935.73
	Cast-in-place concrete column, 20", square, tied, minimum reinforcing, 500K load, 10'-14' story height, 375 lbs/LF, 4000PSI	681.2		\$5.93	\$95,696.34
	Concrete I beam, precast, 18" x 36", 790 PLF, 25' span, 6.44 KLF superimposed load	8777.6		\$184.66	\$2,981,542.25
	Flat slab, concrete, with drop panels, 6" slab/2.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 PSF total load	1345.5		\$1.28	\$20,600.28
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 30 PSF superimposed load, 120 PSF total load	14800.5		\$15.49	\$250,069.99
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 75 PSF superimposed load, 165 PSF total load	1345.5		\$1.47	\$23,786.35
B2010	Exterior Walls			\$145.65	\$2,351,683.28
	Brick wall, composite double wythe, standard face/CMU back-up, 8" thick, perlite core fill, 3" XPS	54912		\$145.65	\$2,351,683.28
B2020	Exterior Windows			\$28.89	\$466,468.49
	Windows, aluminum, awning, insulated glass, 4'-5" x 5'-3"	596.86		\$28.89	\$466,468.49
B2030	Exterior Doors			\$0.33	\$5,286.91
	Door, aluminum & glass, without transom, narrow stile, with panic hardware, 3'-0" x 7'-0" opening	0.35		\$0.09	\$1,411.31
	Door, aluminum & glass, without transom, narrow stile, double door, hardware, 6'-0" x 7'-0" opening	0.43		\$0.18	\$2,829.95
	Door, steel 18 gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening	0.35		\$0.06	\$1,045.65
B3010	Roof Coverings			\$2.10	\$33,970.90
	Roofing, single ply membrane, EPDM, 60 mils, loosely laid, stone ballast	1345.5		\$0.15	\$2,410.19
	Insulation, rigid, roof deck, extruded polystyrene, 40 PSI compressive strength, 4" thick, R20	1345.5		\$0.37	\$6,054.68

	Roof edges, aluminum, duranodic, .050" thick, 6" face	520		\$0.97	\$15,620.33
	Flashing, aluminum, no backing sides, .019"	520		\$0.19	\$3,100.19
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick	520		\$0.42	\$6,785.51
B3020	Roof Openings			\$0.46	\$7,485.16
	Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs	6		\$0.46	\$7,485.16
C	Interiors		5.67%	\$46.03	\$743,217.94
C1010	Partitions			\$11.69	\$188,814.18
	Concrete block (CMU) partition, light weight, hollow, 6" thick, no finish	1435.2		\$0.95	\$15,375.15
	Metal partition, 5/8" fire rated gypsum board face, no base, 3'-5/8" @ 24" OC framing, same opposite face, sound attenuation insulation	12916.8		\$4.84	\$78,115.64
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"	54912		\$3.59	\$57,934.36
	Add for the following: taping and finishing	54912		\$2.32	\$37,389.03
C1020	Interior Doors			\$14.36	\$231,878.81
	Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"	179.4		\$14.36	\$231,878.81
C2010	Stair Construction			\$2.31	\$37,294.30
	Stairs, steel, pan tread for conc in-fill, picket rail, 16 risers w/ landing	2.22		\$2.31	\$37,294.30
C3010	Wall Finishes			\$7.38	\$119,231.01
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	22963.2		\$1.30	\$20,970.45
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	54912		\$3.11	\$50,146.74
C3020	Floor Finishes			\$5.99	\$96,763.01
	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 35 oz	12916.8		\$4.42	\$71,373.07
	Vinyl, composition tile, maximum	1614.6		\$0.32	\$5,179.43
	Tile, ceramic natural clay	1614.6		\$1.25	\$20,210.51
C3030	Ceiling Finishes			\$4.29	\$69,236.63
	Gypsum board ceilings, 5/8" fire rated gypsum board, painted and textured finish, 1-5/8" metal stud furring, 24" OC support	16146		\$4.29	\$69,236.63
D	Services		12.74%	\$103.50	\$1,671,074.11
D1010	Elevators and Lifts			\$6.97	\$112,562.38
	Traction geared freight, 4000 lb., 15 floors, 10' story height, 200FPM	0.03		\$1.31	\$21,158.44
	Traction, geared passenger, 3500 lb, 15 floors, 10' story height, 2 car group, 350 FPM	0.17		\$5.66	\$91,403.94
D2010	Plumbing Fixtures			\$20.39	\$329,221.86
	Water closet, vitreous china, bowl only with flush valve, wall hung	40.36		\$8.80	\$142,145.35

	Urinal, vitreous china, wall hung	0.89		\$0.09	\$1,435.34
	Lavatory w/trim, vanity top, PE on CI, 20" x 18"	40.36		\$3.74	\$60,439.52
	Kitchen sink w/trim, countertop, stainless steel, 33" x 22" double bowl	0.28		\$0.04	\$708.10
	Service sink w/trim, PE on CI, wall hung w/rim guard, 22" x 18"	1.07		\$0.26	\$4,252.32
	Bathtub, recessed, PE on CI, mat bottom, 5' long	40.36		\$7.21	\$116,409.63
	Shower, stall, baked enamel, terrazzo receptor, 36" square	0.89		\$0.17	\$2,706.16
	Water cooler, electric, wall hung, wheelchair type, 7.5 GPH	0.53		\$0.07	\$1,125.44
D2020	Domestic Water Distribution			\$0.21	\$3,465.99
	Gas fired water heater, commercial, 100< F rise, 500 MBH input, 480 GPH	0.12		\$0.21	\$3,465.99
D2040	Rain Water Drainage			\$0.25	\$4,058.71
	Roof drain, CI, soil, single hub, 5" diam, 10' high	0.28		\$0.06	\$895.60
	Roof drain, CI, soil, single hub, 5" diam, for each additional foot add	40.18		\$0.20	\$3,163.11
D3010	Energy Supply			\$3.05	\$49,298.99
	Commercial building heating system, fin tube radiation, forced hot water, 1mil SF, 10 mil CF, total 5 floors	17760.6		\$3.05	\$49,298.99
D3030	Cooling Generating Systems			\$14.99	\$241,955.88
	Packaged chiller, water cooled, with fan coil unit, medical centers, 60,000 SF, 140.00 ton	16146		\$14.99	\$241,955.88
D4010	Sprinklers			\$4.51	\$72,811.85
	Wet pipe sprinkler systems, steel, light hazard, 1 floor, 50,000 SF	11302.2		\$2.23	\$35,999.20
	Wet pipe sprinkler systems, steel, light hazard, each additional floor, 50,000 SF	15015.78		\$2.22	\$35,904.98
	Standard High Rise Accessory Package 16 story	0.03		\$0.06	\$907.67
D4020	Standpipes			\$6.63	\$107,040.23
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor	0.10		\$0.11	\$1,821.71
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, additional floors	15.06		\$4.31	\$69,642.27
	Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM	1		\$1.96	\$31,688.50
	Fire pump, electric, for jockey pump system, add	1		\$0.24	\$3,887.75
D5010	Electrical Service/Distribution			\$28.97	\$467,669.22
	Underground service installation, includes excavation, backfill, and compaction, 100' length, 4' depth, 3 phase, 4 wire, 277/480 volts, 2000 A	2		\$7.06	\$114,012.00
	Feeder installation 600 V, including RGS conduit and XHHW wire, 60 A	100		\$0.14	\$2,210.22
	Feeder installation 600 V,	100		\$0.32	\$5,244.00

	including RGS conduit and XHHW wire, 200 A				
	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A	400		\$13.43	\$216,770.40
	Switchgear installation, incl switchboard, panels & circuit breaker, 277/480 V, 2000 A	2		\$8.02	\$129,432.60
D5020	Lighting and Branch Wiring			\$12.21	\$197,201.79
	Receptacles incl plate, box, conduit, wire, 10 per 1000 SF, 1.2 W per SF, with transformer	16468.92		\$4.34	\$70,093.37
	Wall switches, 5.0 per 1000 SF	16146		\$1.46	\$23,536.99
	Miscellaneous power, to .5 watts	16146		\$0.17	\$2,711.56
	Central air conditioning power, 4 watts	19698.12		\$0.86	\$13,855.86
	Motor installation, three phase, 460 V, 15 HP motor size	10		\$1.80	\$29,065.05
	Motor feeder systems, three phase, feed to 200 V 5 HP, 230 V 7.5 HP, 460 V 15 HP, 575 V 20 HP	500		\$0.39	\$6,363.62
	Motor connections, three phase, 200/230/460/575 V, up to 5 HP	1		\$0.01	\$149.54
	Motor connections, three phase, 200/230/460/575 V, up to 100 HP	1		\$0.04	\$647.28
	Fluorescent fixtures recess mounted in ceiling, 0.8 watt per SF, 20 FC, 5 fixtures @32 watt per 1000 SF	16146		\$3.14	\$50,778.52
D5030	Communications and Security			\$4.95	\$79,966.54
	Communication and alarm systems, fire detection, addressable, 100 detectors, includes outlets, boxes, conduit and wire	0.41		\$2.22	\$35,799.28
	Fire alarm command center, addressable with voice, excl. wire & conduit	0.10		\$0.09	\$1,448.21
	Communication and alarm systems, includes outlets, boxes, conduit and wire, intercom systems, 100 stations	0.12		\$1.18	\$19,027.70
	Communication and alarm systems, includes outlets, boxes, conduit and wire, master TV antenna systems, 100 outlets	0.09		\$0.92	\$14,882.23
	Internet wiring, 2 data/voice outlets per 1000 S.F.	12.10		\$0.55	\$8,809.12
D5090	Other Electrical Systems			\$0.36	\$5,820.67
	Generator sets, w/battery, charger, muffler and transfer switch, diesel engine with fuel tank, 500 kW	24.29		\$0.36	\$5,820.67
E	Equipment & Furnishings		28.08%	\$228.16	\$3,683,887.00
E1090	Other Equipment			\$228.16	\$3,683,887.00
E1090D10101502000	2.00-Traction gearless elevators, passenger, 3000 lb, 10 floors, 200 FPM	2.00		\$52.88	\$853,776.00
E1090D10101503800	2.00-Traction gearless elevators, passenger, 5000 lb, 10 floors, 200 FPM	2.00		\$54.00	\$871,920.00
E10901013101	2.00-Directory boards, plastic,	2.00		\$0.20	\$3,269.50

00100	glass covered, 36" x 48"				
E10901013101 02900	2.00-Directory boards, building directory, aluminum black felt panels, 2 door, 48" x 60"	2.00		\$0.27	\$4,426.40
E109011217319 3700	10.00-Folders, blankets and sheets, king-size with automatic stacker	10.00		\$47.98	\$774,620.00
E109011217326 6200	10.00-Laundry equipment, combination washer/extractor, 50 lb. capacity	10.00		\$9.72	\$156,936.00
E10901324165 01600	2.00-Saunas, prefabricated, cedar on cedar, 10' x 12' x 7' h, incl. heater & controls	2.00		\$1.93	\$31,186.00
E10901254162 00200	4.00-Restaurant furniture, bar, built-in, back bar	4.00		\$0.09	\$1,448.64
E109011217323 4500	10.00-Ironers, institutional, single roll, 110"	10.00		\$25.48	\$411,454.00
E109011217326 6350	10.00-Laundry equipment, combination washer/extractor, 125 lb. capacity	10.00		\$24.30	\$392,340.00
E10902652131 00500	24.00-Emergency lighting units, lead battery operated, twin sealed beam light, 25 W, 6 V each	24.00		\$0.54	\$8,743.68
E10902741331 00600	30.00-TV systems, VHF reception & distribution, 100 outlets	30.00		\$0.76	\$12,254.40
E109028461127 5400	24.00-Detection system, smoke detector, duct type, excl. wires & conduit	24.00		\$0.87	\$14,124.24
E109027511910 0600	24.00-Sound system, trumpet, excl rough-in wires, cables & conduits	24.00		\$0.75	\$12,188.16
E10902741331 00400	30.00-TV systems, VHF reception & distribution, 30 outlets	30.00		\$0.69	\$11,095.20
E10902652131 00700	24.00-Emergency lighting units, nickel cadmium battery operated, twin sealed beam light, 25 W, 6 V each	24.00		\$0.90	\$14,572.80
E10902741331 00200	30.00-TV systems, VHF reception & distribution, 12 outlets	30.00		\$0.50	\$8,015.10
E109028461127 5200	24.00-Detection system, heat detector, smoke detector, ceiling type, excl. wires & conduit	24.00		\$0.36	\$5,891.04
E109027511910 0400	24.00-Sound system, speaker, ceiling or wall, excl rough-in wires, cables & conduits	24.00		\$0.39	\$6,359.04
E10902823131 02000	24.00-Closed circuit television system (CCTV), surveillance, one station (camera & monitor)	24.00		\$2.21	\$35,612.40
E109027511910 1600	24.00-Sound system, amplifier, 250 W, excl rough-in wires, cables & conduits	24.00		\$3.32	\$53,654.40
F	Special Construction		0%		
G	Building Sitework		0%		

The total amount of the construction cost was divided into Material Costs, Architectural Costs, Workforce and Technical Costs:

Table 5.6. Structural materials cost

#	Quantity,	h, m	b, m	L, m	Gross volume, m ³	Steel volume, m ³	Steel weight, kg	Concrete unit cost per m ² , \$	Steel unit cost per kg, \$	Roofing cost, \$	Total concrete gross volume cost, \$	Total steel gross volume cost, \$
Floors 1-3												
Beam (major)	93	0.6	0.3	6	1.08	0.0108	84.78	\$180	\$0.6	-	\$18079	\$4730
Beam (minor)	40	0.2	0.1	6	0.12	0.0012	9.42	\$180	\$0.6	-	\$864	\$226
Column 1	60	0.60	0.60	4	1.44	0.0144	133.04	\$180	\$0.6	-	\$15552	\$4789
Slab	3	30.6	48.6	0.15	223	2.23	17505	\$180	\$0.6	-	\$120420	\$31509
Floors 4-7												
Beam (major)	93	0.6	0.3	6	1.08	0.0108	84.78	\$180	\$0.6	-	\$18079	\$4730
Beam (minor)	40	0.2	0.1	6	0.12	0.0012	9.42	\$180	\$0.6	-	\$864	\$226
Column 2	60	0.50	0.50	3	0.75	0.0075	58.875	\$180	\$0.6	-	\$8100	\$2119
Slab	4	30.6	48.6	0.15	223	2.23	17505	\$180	\$0.6	-	\$160560	\$42012
Floors 8-11												

Beam (major)	93	0.6	0.3	6	1.08	0.0108	84.78	\$180	\$0.6	-	\$18079	\$4730
Beam (minor)	40	0.2	0.1	6	0.12	0.0012	9.42	\$180	\$0.6	-	\$864	\$226
Column 3	60	0.40	0.40	3	0.48	0.0048	37.68	\$180	\$0.6	-	\$3456	\$904.32
Slab	4	30.6	48.6	0.15	223	2.23	17505	\$180	\$0.6	-	\$160560	\$42012
Floor 12												
Beam (major)	93	0.6	0.3	6	1.08	0.0108	84.78	\$180	\$0.6	-	\$18079	\$4730
Beam (minor)	40	0.2	0.1	6	0.12	0.0012	9.42	\$180	\$0.6	-	\$864	\$226
Column 4	60	0.25	0.25	3	0.1875	0.001875	14.72	\$180	\$0.6	-	\$2025	\$530
Slab	1	30.6	48.6	0.15	223	2.23	17505	\$180	\$0.6	-	\$40140	\$10503
Roof												
Beam (major)	16	0.6	0.3	6	1.08	0.0108	84.78	\$180	\$0.6	-	\$13824	\$814
Column 5	16	0.25	0.25	3	0.1875	0.001875	14.72	\$180	\$0.6	-	\$540	\$141
Roof level												
Slab	1	30.6	48.6	0.15	223	2.23	17505	\$180	\$0.6	-	\$40140	\$10503
Fibr	1	30.6	48.6							\$240	Total	\$356

ous glass						cost for area, \$	918
Wate rpro of mem bran e	1	30.6	48.6		\$75	Total cost for area, \$	\$111 537
Thre e ply read y roofi ng	1	30.6	48.6		\$140	Total cost for area, \$	\$208 202
Total structural cost						\$1447280	

Table 5.7. Architectural Costs

	Qu anti ty for floors 1-3	Qu anti ty for floors 4-12	Thi ckn ess, m	Surface area, m ²	Area, m ² for floors 1-3	Area, m ² for floors 4-12	Material unit cost per m ² , \$	Mat eria l cost , \$	Mat eria l cost , \$	Tot al mat eria l cost , \$
Floor layers										
Har dw ood flo or ring	3	9	0.022		216	774	\$42	\$27216	\$292572	\$319788
Cer ami c	3	9	0.02		214	126	\$15	\$9630	\$17010	\$26640

tile										
Linoleum	1	9	0.006		282	0	\$18	\$5076	\$0	\$5076
Marble	3	9	0.02		332	313	\$70	\$69720	\$197190	\$266910
Granite stone	1	9	0.02		485	0	\$60	\$29100	\$0	\$29100
Rubber	1	9	0.02		293	0	\$35	\$10255	\$0	\$10255
Ceiling layers										
Acoustic fiber	3	9	0.02		1487	1487	\$25	\$11525	\$334575	\$446100
Mechanical duct allowance	3	9	0.15		1487	1487	\$15	\$66915	\$200745	\$267660
Suspended steel channel system	3	9	0.2		1487	1487	\$10	\$44610	\$133830	\$178440
Gy	3	9	0.0		1487	1487	\$15	\$66	\$20	\$26

psu m boa rd			3					915	074 5	766 0
Interior walls										
Wal lpa per	1	1	0.0 05	680	828		\$10	\$68 00	\$82 80	\$15 080
Gy psu m boa rd	1	1	0.0 3	680	828		\$15	\$10 200	\$12 420	\$22 620
Rig id ins ulat ion	1	1	0.0 12	680	828		\$40	\$27 200	\$33 120	\$60 320
Soli d CM U wal l	1	1	0.1 02	680	828		\$55	\$37 400	\$45 540	\$82 940
Rig id ins ulat ion	1	1	0.0 12	680	828		\$40	\$27 200	\$33 120	\$60 320
Gy	1	1	0.0	680	828		\$15	\$10	\$12	\$22

psu m boa rd			3					200	420	620
Wal lpa per	1	1	0.0 05	680	828		\$10	\$68 00	\$82 80	\$15 080
Exterior walls										
Wal lpa per	1	1	0.0 05	640	576		\$10	\$64 00	\$57 60	\$12 160
Gy psu m boa rd	1	1	0.0 3	640	576		\$15	\$96 00	\$86 40	\$18 240
Rig id ins ulat ion	1	1	0.0 24	640	576		\$40	\$25 600	\$23 040	\$48 640
Soli d CM U wal l	1	1	0.2 54	640	576		\$55	\$35 200	\$31 680	\$66 880
Rig	1	1	0.0 24	640	576		\$40	\$25 600	\$23 040	\$48 640

id ins ulat ion										
Vap or con trol lay er	1	1	0	640	576		\$2	\$12 80	\$11 52	\$24 32
Rei nfo rcin g net	1	1	0.0 06	640	576		\$2.5	\$16 00	\$14 40	\$30 40
Pla ster	1	1	0.0 1	640	576		\$12	\$76 80	\$69 12	\$14 592
Rai nsc ree n cla ddi ng pan els	1	1	0.0 16	640	576		\$60	\$38 400	\$34 560	\$72 960
Total cost								\$2384193		

5.9 Construction Cost Estimations

Table 5.8. Workforce and Technical Costs

Labor type	Quantity, n	Equipment	Duration	Unit price per day, \$	Cost, \$
Earthwork	2	Excavator	54	\$1200	\$129600
	2	Grader, roller	54	\$600	\$64800
	2	Loader	54	\$610	\$65880
	2	Bulldozer	54	\$320	\$34560
	2	Tipper	54	\$440	\$47520
Foundation	2	Pile driving rig	45	\$1000	\$90000
	2	Pile driver	45	\$750	67500
	2	Crane	270	\$2300	\$1242000
	3	Concrete mixer, pump	270	\$320	\$259200
Structure	3	Concrete mixing truck	270	\$320	\$259200
	3	Truck-mounted concrete pump	270	\$700	\$567000
Total Cost					\$2827260

Additional expenses were calculated as follows:

The parking lot and technical equipment required approximately \$2000000, while the interior design of the 12-story hotel with the basement was estimated to cost:

$$\text{number of rooms} \times \text{average room area} \times \text{high end luxury design cost} = 54 \times 129 \times \$1500 =$$

The total additional cost will be \$10449000 + \$2000000 = \$12449000

Table 5.9. Initial Construction Expenses

Construction part	Required budget
Structural materials cost	\$1447280
Architectural Costs	\$2384193
Workforce and Technical Costs	\$2827260
Additional expenses	\$12449000
Total construction	\$19107733

Table 5.10. Final Cost Estimation

Component	Cost
Subtotal Construction Cost	\$ 19107733
Land Acquisition	\$25000
Equipment Rental	\$1300000
Utility Connection	\$180000
Permits	\$4000000
Subtotal Construction Cost/Initial Investment	\$24612733

5.10 Revenue calculation

Table 5.11. Updated revenue data

Period of time	Revenue
Year 1	\$2500000
Year 2	\$2625000
Year 3	\$2756250
Year 4	\$2894063
Year 5	\$3038766
Year 6	\$3190704
Year 7	\$3350239
Year 8	\$3517750
Year 9	\$3693638
The total amount of Revenue	\$27566410

The annual revenue was expected to grow by 5 % every year

The approximate payback period is calculated to be roughly 8.5 years

5.11 Site Planning

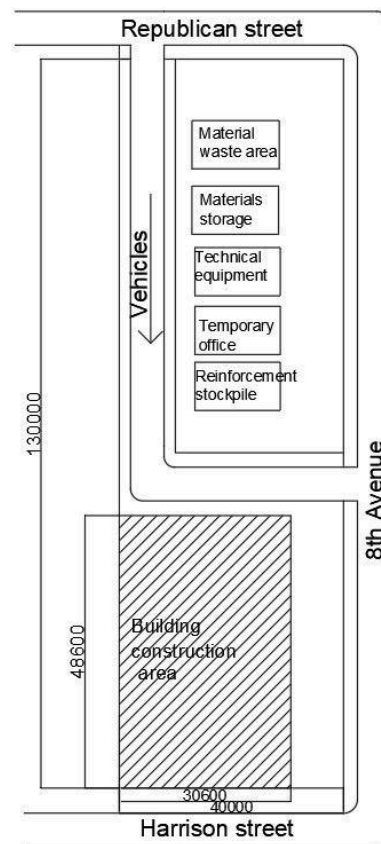


Figure 5.6. Site planning area

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Appendix

Appendix

Appendix A

Included here are wind load forces with torsional effect considerations.

Level	Transverse				Longitudinal			
2	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.4615	12.82	11.3628	1	-2.2237	14.92	12.6959
	B	-0.8339	12.82	11.9905	2	-1.6678	14.92	13.2519
	C	-0.2780	12.82	12.5464	3	-1.1118	14.92	13.8078
	D	0.2780	12.82	13.1023	4	-0.5559	14.92	14.3637
	E	0.8339	12.82	13.6582	5	0.0000	14.92	14.9196
	F	1.3898	12.82	14.2142	6	0.5559	14.92	15.4755
					7	1.1118	14.92	16.0315
					8	1.6678	14.92	16.5874
					9	2.2237	14.92	17.1433

Level	Transverse				Longitudinal			
3	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.5539	12.07	10.5192	1	-2.4862	13.96	11.4777
	B	-0.9323	12.07	11.1408	2	-1.8647	13.96	12.0993
	C	-0.3108	12.07	11.7623	3	-1.2431	13.96	12.7208
	D	0.3108	12.07	12.3839	4	-0.6216	13.96	13.3424
	E	0.9323	12.07	13.0054	5	0.0000	13.96	13.9639
	F	1.5539	12.07	13.6270	6	0.6216	13.96	14.5855
					7	1.2431	13.96	15.2070
					8	1.8647	13.96	15.8286

		9	2.4862	13.96	16.4502
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Level	Transverse				Longitudinal			
4	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.6122	11.08	9.4641	1	-2.5795	12.76	10.1766
	B	-0.9673	11.08	10.1090	2	-1.9346	12.76	10.8215
	C	-0.3224	11.08	10.7539	3	-1.2897	12.76	11.4663
	D	0.3224	11.08	11.3987	4	-0.6449	12.76	12.1112
	E	0.9673	11.08	12.0436	5	0.0000	12.76	12.7561
	F	1.6122	11.08	12.6884	6	0.6449	12.76	13.4009
					7	1.2897	12.76	14.0458
					8	1.9346	12.76	14.6907
					9	2.5795	12.76	15.3355

Level	Transverse				Longitudinal			
5	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.6623	11.42	9.7610	1	-2.6597	13.11	10.4527
	B	-0.9974	11.42	10.4259	2	-1.9948	13.11	11.1176
	C	-0.3325	11.42	11.0908	3	-1.3298	13.11	11.7826
	D	0.3325	11.42	11.7557	4	-0.6649	13.11	12.4475
	E	0.9974	11.42	12.4207	5	0.0000	13.11	13.1124
	F	1.6623	11.42	13.0856	6	0.6649	13.11	13.7773
					7	1.3298	13.11	14.4422
					8	1.9948	13.11	15.1072
					9	2.6597	13.11	15.7721

Level	Transverse				Longitudinal			
6	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.7066	11.73	10.0259	1	-2.7306	13.43	10.6994
	B	-1.0240	11.73	10.7085	2	-2.0479	13.43	11.3820
	C	-0.3413	11.73	11.3912	3	-1.3653	13.43	12.0647
	D	0.3413	11.73	12.0738	4	-0.6826	13.43	12.7473
	E	1.0240	11.73	12.7564	5	0.0000	13.43	13.4299
	F	1.7066	11.73	13.4391	6	0.6826	13.43	14.1126
					7	1.3653	13.43	14.7952
					8	2.0479	13.43	15.4778
					9	2.7306	13.43	16.1605

Level	Transverse				Longitudinal			
7	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.7465	12.01	10.2661	1	-2.7943	13.72	10.9232
	B	-1.0479	12.01	10.9646	2	-2.0957	13.72	11.6218
	C	-0.3493	12.01	11.6632	3	-1.3972	13.72	12.3203
	D	0.3493	12.01	12.3618	4	-0.6986	13.72	13.0189
	E	1.0479	12.01	13.0604	5	0.0000	13.72	13.7175
	F	1.7465	12.01	13.7590	6	0.6986	13.72	14.4161
					7	1.3972	13.72	15.1147
					8	2.0957	13.72	15.8132
					9	2.7943	13.72	16.5118

Level	Transverse				Longitudinal			
8	Frame	Ftorsion	Fdirect	Ftotal	Frame	Ftorsion	Fdirect	Ftotal

		(kN)	(kN)	(kN)		(kN)	(kN)	(kN)
	A	-1.7828	12.27	10.4864	1	-2.8525	13.98	11.1287
	B	-1.0697	12.27	11.1995	2	-2.1394	13.98	11.8418
	C	-0.3566	12.27	11.9127	3	-1.4262	13.98	12.5549
	D	0.3566	12.27	12.6258	4	-0.7131	13.98	13.2680
	E	1.0697	12.27	13.3389	5	0.0000	13.98	13.9811
	F	1.7828	12.27	14.0520	6	0.7131	13.98	14.6943
					7	1.4262	13.98	15.4074
					8	2.1394	13.98	16.1205
					9	2.8525	13.98	16.8336

Level	Transverse				Longitudinal			
9	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.8163	12.51	10.6905	1	-2.9061	14.23	11.3190
	B	-1.0898	12.51	11.4170	2	-2.1796	14.23	12.0456
	C	-0.3633	12.51	12.1436	3	-1.4531	14.23	12.7721
	D	0.3633	12.51	12.8701	4	-0.7265	14.23	13.4986
	E	1.0898	12.51	13.5966	5	0.0000	14.23	14.2252
	F	1.8163	12.51	14.3232	6	0.7265	14.23	14.9517
					7	1.4531	14.23	15.6782
					8	2.1796	14.23	16.4047
					9	2.9061	14.23	17.1313

Level	Transverse				Longitudinal			
10	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	343.63	-1.8475	12.73	1	-2.9560	14.45	11.4967

	B	343.63	-1.1085	12.73	2	-2.2170	14.45	12.2357
	C	343.63	-0.3695	12.73	3	-1.4780	14.45	12.9747
	D	343.63	0.3695	12.73	4	-0.7390	14.45	13.7137
	E	343.63	1.1085	12.73	5	0.0000	14.45	14.4527
	F	343.63	1.8475	12.73	6	0.7390	14.45	15.1917
					7	1.4780	14.45	15.9307
					8	2.2170	14.45	16.6697
					9	2.9560	14.45	17.4087

Level	Transverse				Longitudinal			
11	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)
	A	349.05	-1.8766	12.94	1	-3.0026	14.67	11.6637
	B	349.05	-1.1260	12.94	2	-2.2520	14.67	12.4143
	C	349.05	-0.3753	12.94	3	-1.5013	14.67	13.1650
	D	349.05	0.3753	12.94	4	-0.7507	14.67	13.9156
	E	349.05	1.1260	12.94	5	0.0000	14.67	14.6663
	F	349.05	1.8766	12.94	6	0.7507	14.67	15.4169
					7	1.5013	14.67	16.1676
					8	2.2520	14.67	16.9182
					9	3.0026	14.67	17.6689

Level	Transverse				Longitudinal			
12	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.9041	13.13	11.2285	1	-3.0465	14.87	11.8212
	B	-1.1424	13.13	11.9901	2	-2.2849	14.87	12.5828
	C	-0.3808	13.13	12.7517	3	-1.5233	14.87	13.3445

	D	0.3808	13.13	13.5134	4	-0.7616	14.87	14.1061
	E	1.1424	13.13	14.2750	5	0.0000	14.87	14.8677
	F	1.9041	13.13	15.0366	6	0.7616	14.87	15.6293
					7	1.5233	14.87	16.3910
					8	2.2849	14.87	17.1526
					9	3.0465	14.87	17.9142

Level	Transverse				Longitudinal			
13	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-3.4785	6.61	3.1354	1	-5.5656	7.48	1.9172
	B	-2.0871	6.61	4.5268	2	-4.1742	7.48	3.3086
	C				3			
	D				4	-1.3914	7.48	6.0914
	E	2.0871	6.61	8.7011	5	0.0000	7.48	7.4828
	F	3.4785	6.61	10.0925	6	-5.5656	7.48	1.9172
					7	-4.1742	7.48	3.3086
					8			
					9	-1.3914	7.48	6.0914

Appendix B

Included here are seismic load forces with torsional effect considerations.

Level	Transverse				Longitudinal			
2	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-0.7809	11.3	10.5333	1	-2.3292	17.0	14.6421
	B	-0.4685	11.3	10.8457	2	-1.7469	17.0	15.2244
	C	-0.1562	11.3	11.1580	3	-1.1646	17.0	15.8067

	D	0.1562	11.3	11.4704	4	-0.5823	17.0	16.3890
	E	0.4685	11.3	11.7827	5	0.0000	17.0	16.9713
	F	0.7809	11.3	12.0951	6	0.5823	17.0	17.5536
					7	1.1646	17.0	18.1359
					8	1.7469	17.0	18.7182
					9	2.3292	17.0	19.3005

Level	Transverse				Longitudinal			
3	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.3810	20.0	18.6285	1	-4.1193	30.0	25.8950
	B	-0.8286	20.0	19.1809	2	-3.0895	30.0	26.9248
	C	-0.2762	20.0	19.7333	3	-2.0597	30.0	27.9546
	D	0.2762	20.0	20.2857	4	-1.0298	30.0	28.9844
	E	0.8286	20.0	20.8381	5	0.0000	30.0	30.0143
	F	1.3810	20.0	21.3905	6	1.0298	30.0	31.0441
					7	2.0597	30.0	32.0739
					8	3.0895	30.0	33.1038
					9	4.1193	30.0	34.1336

Level	Transverse				Longitudinal			
4	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-1.8510	26.8	24.9681	1	-5.5212	40.2	34.7075
	B	-1.1106	26.8	25.7085	2	-4.1409	40.2	36.0878
	C	-0.3702	26.8	26.4490	3	-2.7606	40.2	37.4681
	D	0.3702	26.8	27.1894	4	-1.3803	40.2	38.8484
	E	1.1106	26.8	27.9298	5	0.0000	40.2	40.2287

	F	1.8510	26.8	28.6702	6	1.3803	40.2	41.6090
					7	2.7606	40.2	42.9893
					8	4.1409	40.2	44.3696
					9	5.5212	40.2	45.7499

Level	Transverse				Longitudinal			
5	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-2.4879	36.0	33.5592	1	-7.4209	54.1	46.6497
	B	-1.4927	36.0	34.5543	2	-5.5657	54.1	48.5049
	C	-0.4976	36.0	35.5495	3	-3.7105	54.1	50.3601
	D	0.4976	36.0	36.5446	4	-1.8552	54.1	52.2154
	E	1.4927	36.0	37.5398	5	0.0000	54.1	54.0706
	F	2.4879	36.0	38.5350	6	1.8552	54.1	55.9258
					7	3.7105	54.1	57.7811
					8	5.5657	54.1	59.6363
					9	7.4209	54.1	61.4915

Level	Transverse				Longitudinal			
6	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-3.1899	46.2	43.0285	1	-9.5149	69.3	59.8128
	B	-1.9140	46.2	44.3045	2	-7.1362	69.3	62.1915
	C	-0.6380	46.2	45.5805	3	-4.7574	69.3	64.5703
	D	0.6380	46.2	46.8565	4	-2.3787	69.3	66.9490
	E	1.9140	46.2	48.1324	5	0.0000	69.3	69.3277
	F	3.1899	46.2	49.4084	6	2.3787	69.3	71.7064
					7	4.7574	69.3	74.0852

		8	7.1362	69.3	76.4639
		9	9.5149	69.3	78.8426

Level	Transverse				Longitudinal			
7	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-3.9527	57.3	53.3169	1	-11.7900	85.9	74.1144
	B	-2.3716	57.3	54.8980	2	-8.8425	85.9	77.0619
	C	-0.7905	57.3	56.4790	3	-5.8950	85.9	80.0094
	D	0.7905	57.3	58.0601	4	-2.9475	85.9	82.9569
	E	2.3716	57.3	59.6412	5	0.0000	85.9	85.9044
	F	3.9527	57.3	61.2222	6	2.9475	85.9	88.8519
					7	5.8950	85.9	91.7993
					8	8.8425	85.9	94.7468
					9	11.7900	85.9	97.6943

Level	Transverse				Longitudinal			
8	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-4.6531	67.4	62.7646	1	-13.8791	101.1	87.2474
	B	-2.7918	67.4	64.6258	2	-10.4093	101.1	90.7172
	C	-0.9306	67.4	66.4871	3	-6.9396	101.1	94.1870
	D	0.9306	67.4	68.3483	4	-3.4698	101.1	97.6567
	E	2.7918	67.4	70.2095	5	0.0000	101.1	101.1265
	F	4.6531	67.4	72.0707	6	3.4698	101.1	104.5963
					7	6.9396	101.1	108.0661
					8	10.4093	101.1	111.5359
					9	13.8791	101.1	115.0056

Level	Transverse				Longitudinal			
9	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-5.5053	79.8	74.2602	1	-16.4211	119.6	103.2271
	B	-3.3032	79.8	76.4623	2	-12.3159	119.6	107.3323
	C	-1.1011	79.8	78.6644	3	-8.2106	119.6	111.4376
	D	1.1011	79.8	80.8665	4	-4.1053	119.6	115.5429
	E	3.3032	79.8	83.0686	5	0.0000	119.6	119.6482
	F	5.5053	79.8	85.2708	6	4.1053	119.6	123.7535
					7	8.2106	119.6	127.8588
					8	12.3159	119.6	131.9641
					9	16.4211	119.6	136.0693

Level	Transverse				Longitudinal			
10	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-6.4079	92.8	86.4359	1	-19.1135	139.3	120.1521
	B	-3.8448	92.8	88.9990	2	-14.3352	139.3	124.9305
	C	-1.2816	92.8	91.5622	3	-9.5568	139.3	129.7089
	D	1.2816	92.8	94.1254	4	-4.7784	139.3	134.4873
	E	3.8448	92.8	96.6886	5	0.0000	139.3	139.2657
	F	6.4079	92.8	99.2517	6	4.7784	139.3	144.0441
					7	9.5568	139.3	148.8225
					8	14.3352	139.3	153.6008
					9	19.1135	139.3	158.3792

Level	Transverse				Longitudinal			
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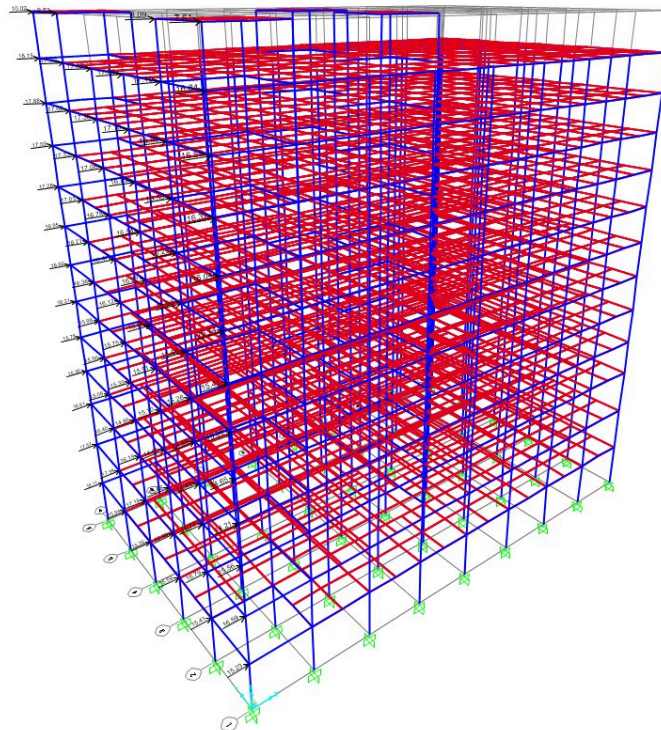
11	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-7.3589	106.6	99.2633	1	-21.9501	159.9	137.9832
	B	-4.4153	106.6	102.2069	2	-16.4626	159.9	143.4707
	C	-1.4718	106.6	105.1504	3	-10.9750	159.9	148.9583
	D	1.4718	106.6	108.0940	4	-5.4875	159.9	154.4458
	E	4.4153	106.6	111.0375	5	0.0000	159.9	159.9333
	F	7.3589	106.6	113.9811	6	5.4875	159.9	165.4208
					7	10.9750	159.9	170.9083
					8	16.4626	159.9	176.3959
					9	21.9501	159.9	181.8834

Level	Transverse				Longitudinal			
12	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
	A	-8.1480	118.1	109.9079	1	-24.3039	177.1	152.7799
	B	-4.8888	118.1	113.1671	2	-18.2279	177.1	158.8559
	C	-1.6296	118.1	116.4263	3	-12.1520	177.1	164.9319
	D	1.6296	118.1	119.6855	4	-6.0760	177.1	171.0079
	E	4.8888	118.1	122.9447	5	0.0000	177.1	177.0838
	F	8.1480	118.1	126.2039	6	6.0760	177.1	183.1598
					7	12.1520	177.1	189.2358
					8	18.2279	177.1	195.3118
					9	24.3039	177.1	201.3877

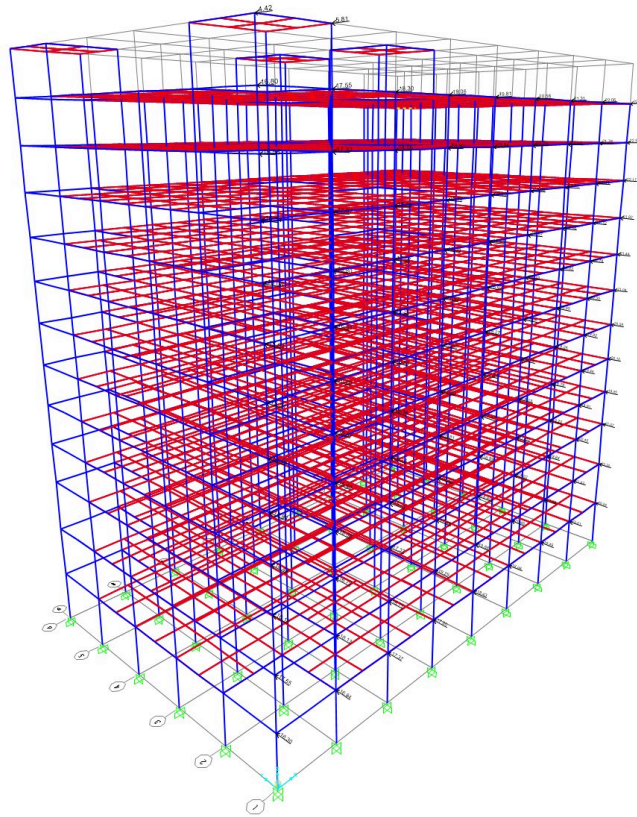
Level	Transverse				Longitudinal			
13	Frame	Ftorsion	Fdirect	Ftotal	Frame	Ftorsion	Fdirect	Ftotal

		(kN)	(kN)	(kN)		(kN)	(kN)	(kN)
	A	-25.6016	205.9	180.2730	1	-76.3643	308.8	232.4476
	B	-15.3609	205.9	190.5136	2	-57.2732	308.8	251.5387
	C	-5.1203	205.9	200.7543	3			
	D	5.1203	205.9	210.9949	4	-19.0911	308.8	289.7208
	E	15.3609	205.9	221.2355	5	0.0000	308.8	308.8118
	F	25.6016	205.9	231.4761	6	-76.3643	308.8	232.4476
					7	-57.2732	308.8	251.5387
					8			
					9	-19.0911	308.8	289.7208

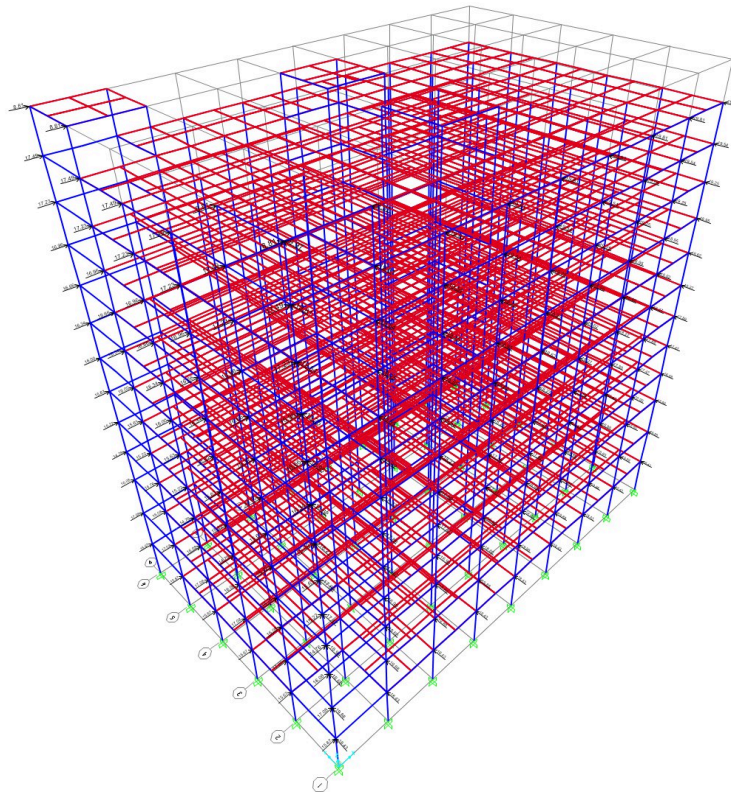
Appendix C



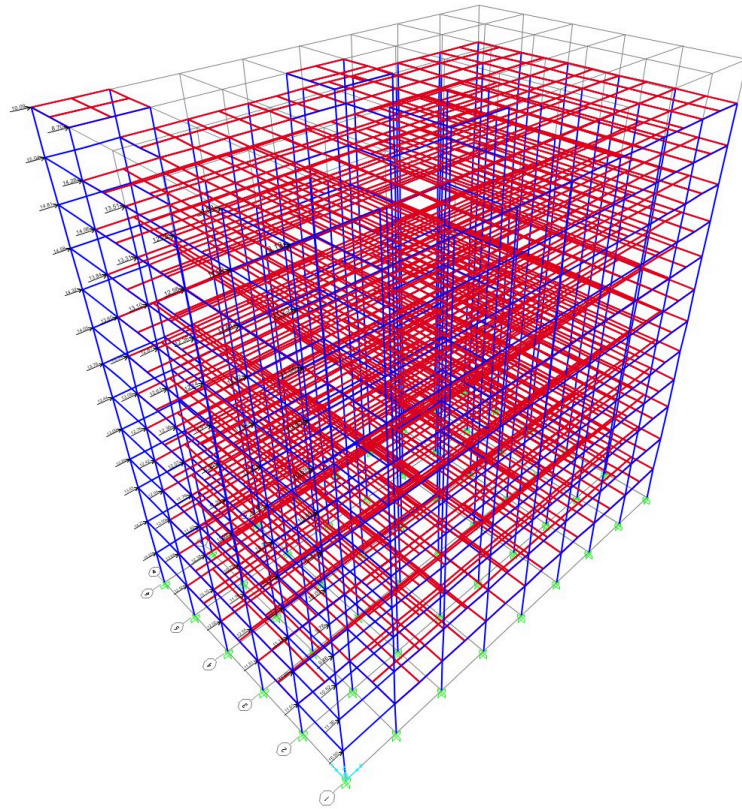
Wind load Case 2 Transverse



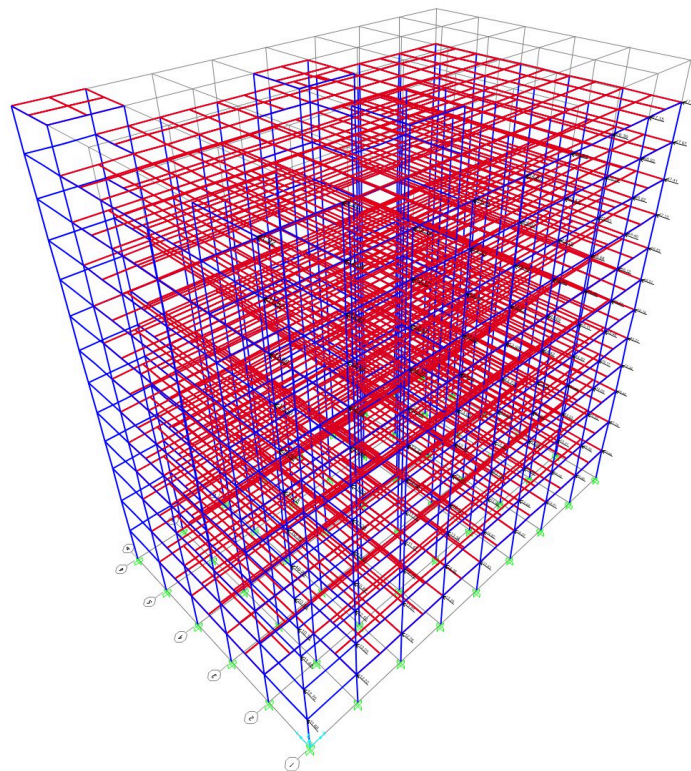
Wind load Case 2 Longitudinal



Wind load Case 3

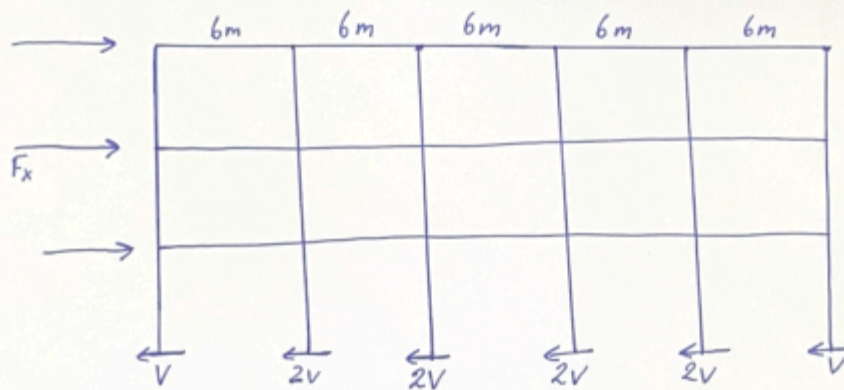


Wind load Case 4 Transverse



Wind load Case 4 Longitudinal

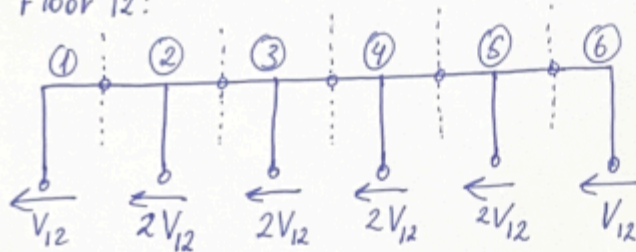
Appendix D



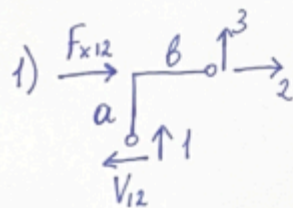
$$\sum F_x = 10V$$

$$V = \frac{\sum F_x}{10}$$

Floor 12:



$$V_{12} = \frac{F_{x12}}{10}$$



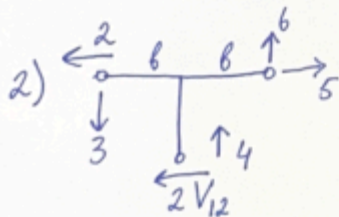
$$\sum F_x = 0 \Rightarrow F_{x12} + 2 = V_{12}$$

$$2 = V_{12} - F_{x12}$$

$$\sum M = 0 \Rightarrow 1 \cdot b = -V_{12} \cdot a$$

$$1 = -\frac{V_{12} \cdot a}{b}$$

$$F_y = 0 \Rightarrow 1 + 3 = 0 \quad 3 = -1$$

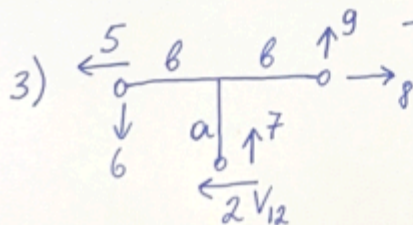


$$\sum F_x = 0 \Rightarrow 5 = 2 + 2V_{12}$$

$$\sum H = 0 \Rightarrow 3 \cdot 2b = 4b + 2V_{12} \cdot a$$

$$4 = \frac{3 \cdot 2b - 2V_{12} \cdot a}{b}$$

$$\sum F_y = 0 \Rightarrow 6 + 4 = 3 \quad 6 = 3 - 4$$

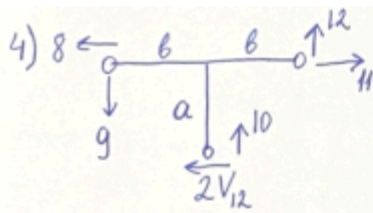


$$\sum F_x = 0 \Rightarrow 8 = 5 + 2V_{12}$$

$$\sum M = 0 \Rightarrow 6 \cdot 2b = 7 \cdot b + 2V_{12} \cdot a$$

$$7 = \frac{6 \cdot 2b - 2V_{12} \cdot a}{b}$$

$$\sum F_y = 0 \Rightarrow 9 + 7 = 6 \quad 9 = 6 - 7$$

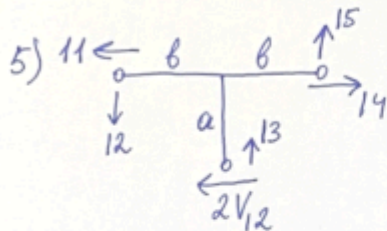


$$\sum F_x = 0 \Rightarrow 11 = 8 + 2V_{12}$$

$$\sum M = 0 \Rightarrow 9 \cdot 2b = 10 \cdot b + 2V_{12} \cdot a$$

$$10 = \frac{9 \cdot 2b - 2V_{12} \cdot a}{b}$$

$$F_y = 0 \quad 12 + 10 = 9 \quad 12 = 9 - 10$$

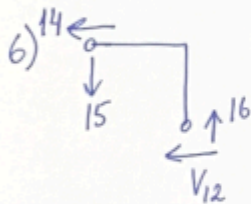


$$\sum F_x = 0 \Rightarrow 14 = 11 + 2V_{12}$$

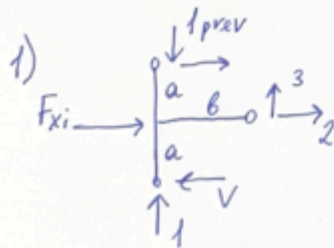
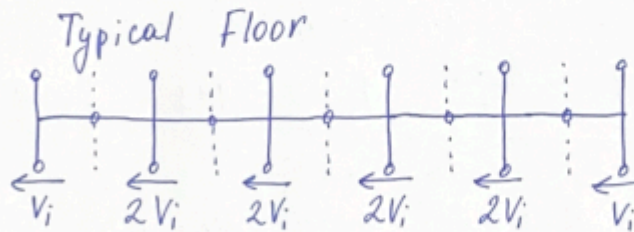
$$\sum M = 0 \Rightarrow 12 \cdot 2b = 13 \cdot b + 2V_{12} \cdot a$$

$$13 = \frac{12 \cdot 2b - 2V_{12} \cdot a}{b}$$

$$15 + 13 = 12 \quad 15 = 12 - 13$$



$$\sum F_y = 0 \Rightarrow 16 = 15$$



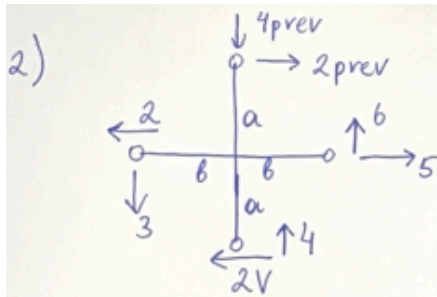
$$\sum F_x = 0 \Rightarrow V_{prev} \leftarrow F_{xi} + 2 = V$$

$$2 = V - V_{prev} - F_{xi}$$

$$\sum M = 0 \Rightarrow V_{prev} \cdot a + V \cdot a + 1 \cdot b = 1_{prev} \cdot b$$

$$1 = \frac{1_{prev} \cdot b - (V_{prev} + V_i) \cdot a}{b}$$

$$\sum F_y = 0 \Rightarrow 3 + 1 = 1_{prev} \quad 3 = 1_{prev} - 1$$



$$\sum F_x = 0 \quad 2 + 2V = 2V_{prev} + 5$$

$$5 = 2 + 2(V - V_{prev})$$

$$\sum M = 0 \Rightarrow 3 \cdot 2b + 4_{prev} \cdot b = (2V_{prev} + 2V)a + 4b$$

$$4 = \frac{(3 \cdot 2 + 4_{prev}) \cdot b - (2V_{prev} + 2V)a}{b}$$

$$\sum F_y = 0 \Rightarrow 4 + b = 3 + 4_{prev} \quad b = 3 + 4_{prev} - 4$$

3) $8 = 5 + 2(V - V_{prev})$

$$7 = \frac{(6 \cdot 2 + 7_{prev}) \cdot b - (2V_{prev} + 2V)a}{b}$$

$$9 = 6 + 7_{prev} - 7$$

4) $11 = 8 + 2(V - V_{prev})$

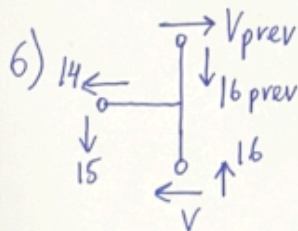
$$10 = \frac{(9 \cdot 2 + 10_{prev}) \cdot b - (2V_{prev} + 2V)a}{b}$$

$$12 = 9 + 10_{prev} - 10$$

5) $14 = 11 + 2(V - V_{prev})$

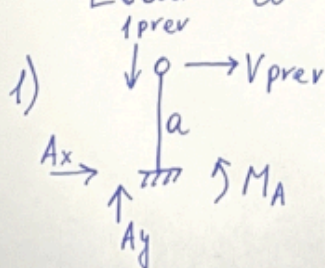
$$13 = \frac{(12 \cdot 2 + 13_{prev}) \cdot b - (2V_{prev} + 2V)a}{b}$$

$$15 = 12 + 13_{prev} - 13$$



$$\sum F_y = 0 \Rightarrow 15 + 16_{prev} = 10$$

Load to the ground:



$$\sum F_x = 0$$

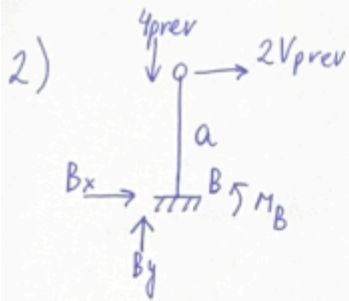
$$A_x = -V_{prev}$$

$$\sum F_y = 0$$

$$A_y = 1_{prev}$$

$$\sum M @ A = 0$$

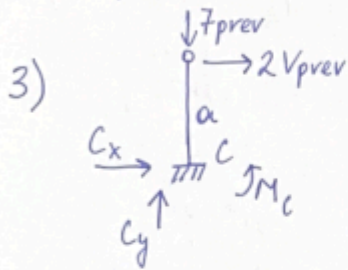
$$M_A = V_{prev} \cdot a$$



$$B_x = -2 V_{prev}$$

$$B_y = 4 p_{prev}$$

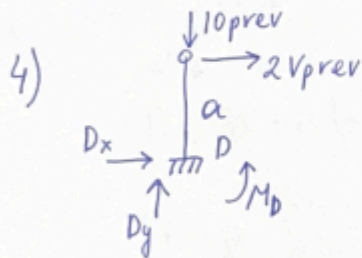
$$M_B = 2 V_{prev} \cdot a$$



$$C_x = -2 V_{prev}$$

$$C_y = 7 p_{prev}$$

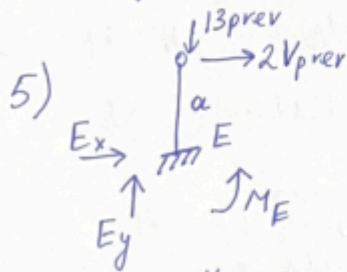
$$M_C = 2 V_{prev} \cdot a$$



$$D_x = -2 V_{prev}$$

$$D_y = 10 p_{prev}$$

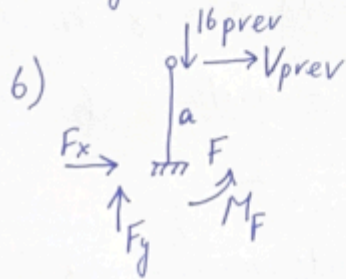
$$M_D = 2 V_{prev} \cdot a$$



$$E_x = -2 V_{prev}$$

$$E_y = 13 p_{prev}$$

$$M_E = 2 V_{prev} \cdot a$$

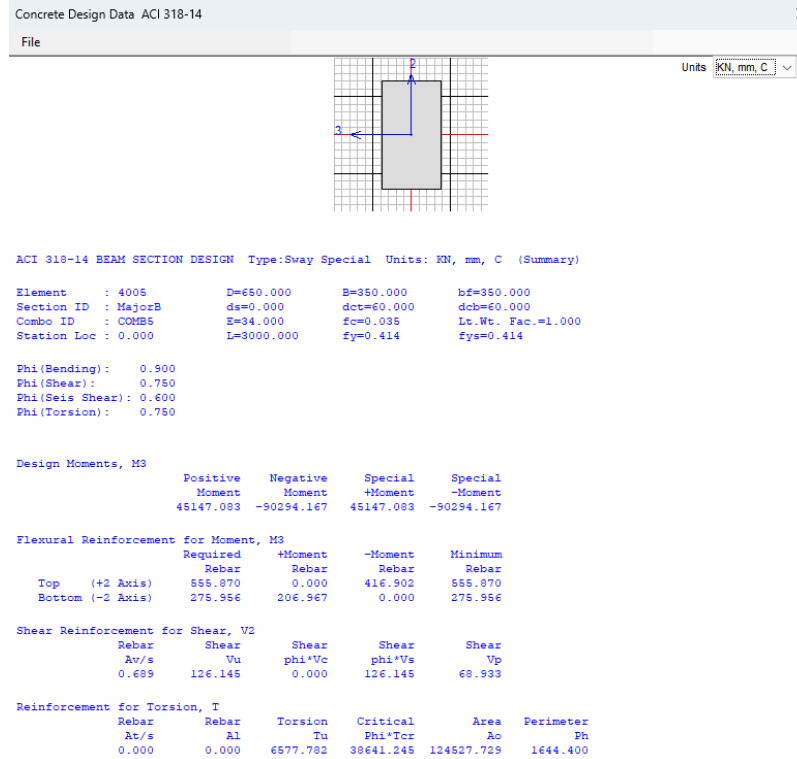


$$F_x = -V_{prev}$$

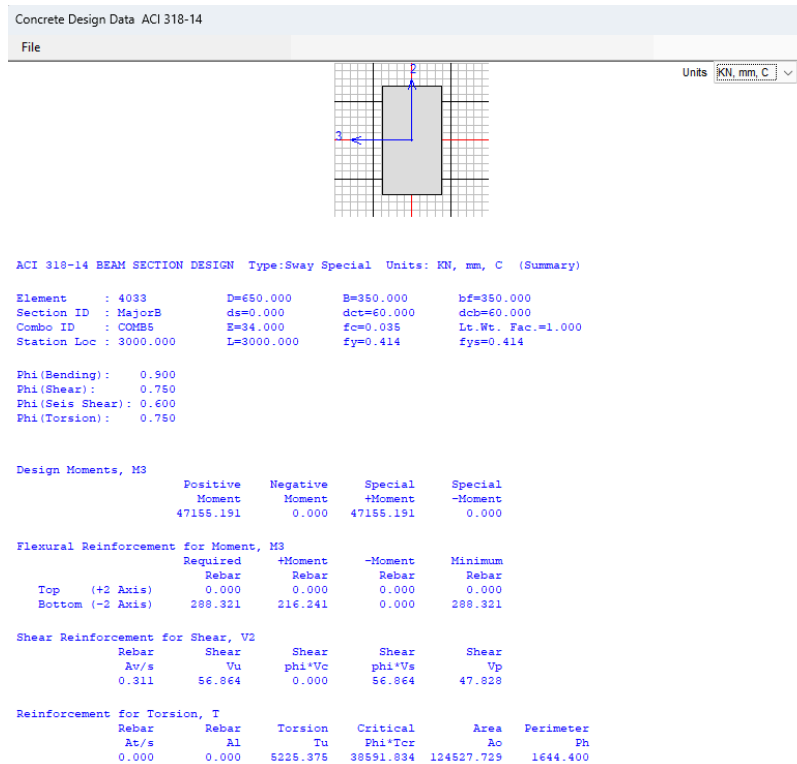
$$F_y = 16 p_{prev}$$

$$M_F = V_{prev} \cdot a$$

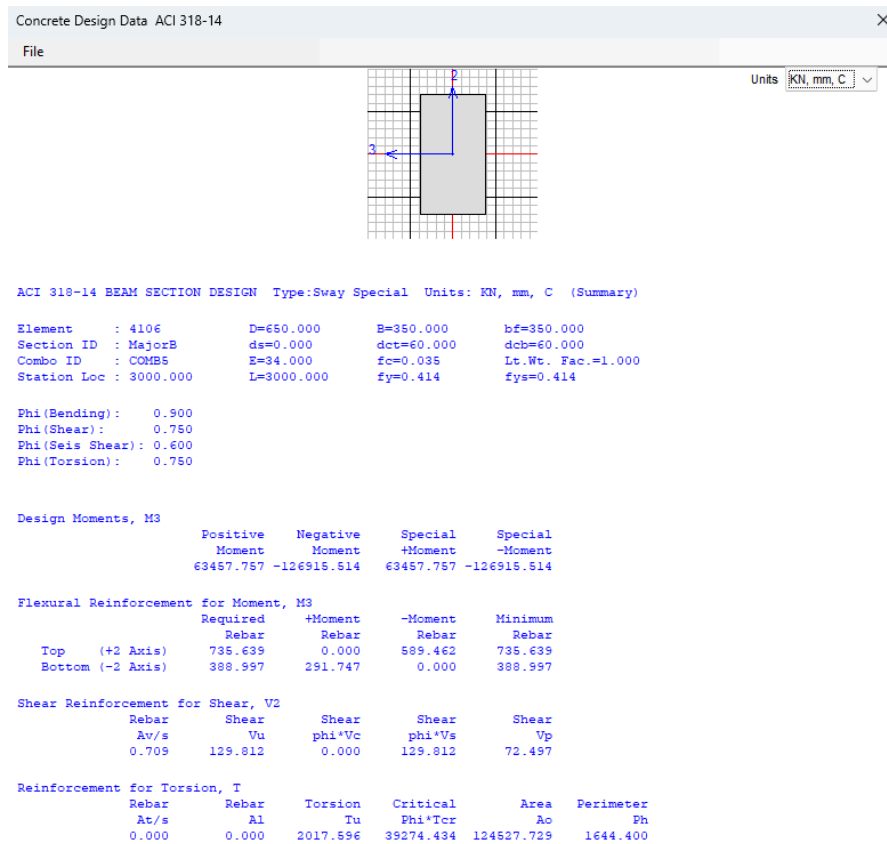
Appendix E



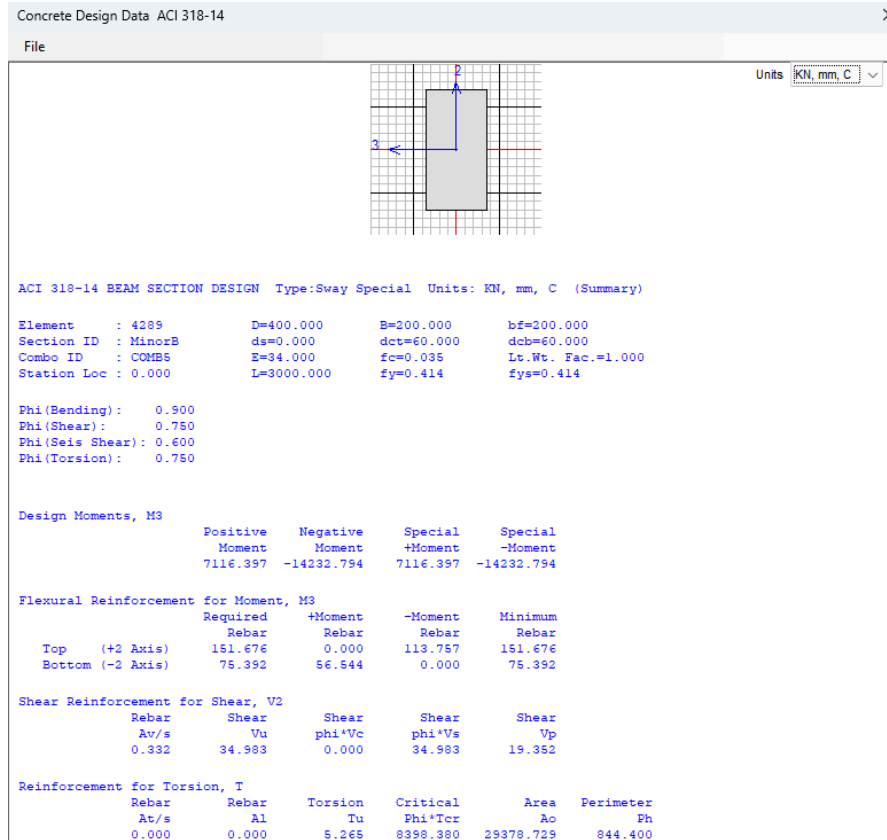
Left end critical moment for Major Beam 12th floor



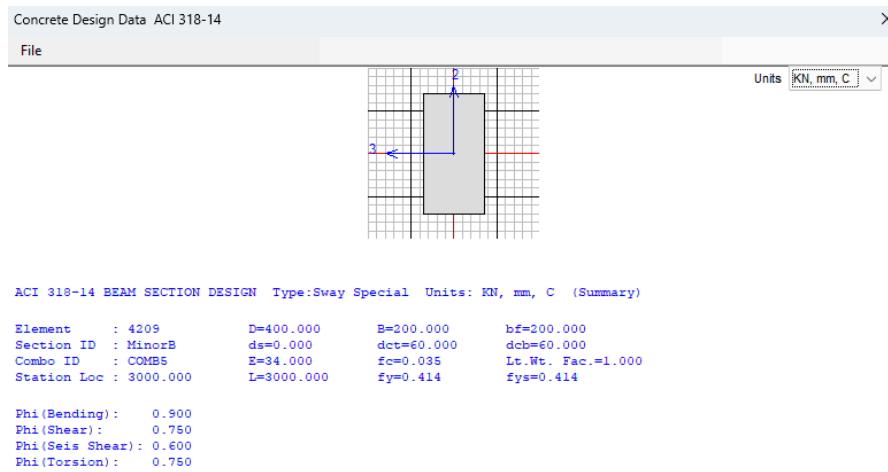
Mid critical moment for Major Beam 12th floor



Right end critical moment for Major Beam 12th floor



Left end critical moment for Minor Beam 12th floor



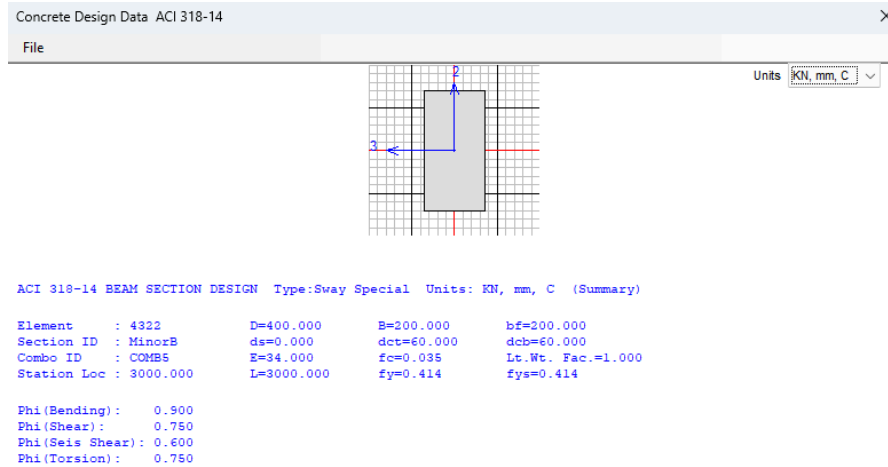
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	6736.129	0.000	6736.129	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	71.341	53.506	0.000	71.341

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.228	24.049	0.000	24.049
				20.952

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tr
	0.000	0.000	200.660	7751.449
				29378.729
				844.400

Mid critical moment for Minor Beam 12th floor



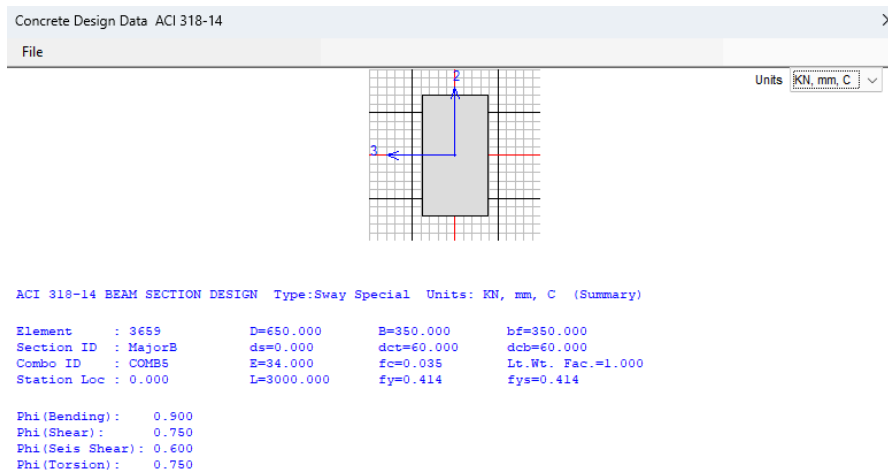
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	10218.046	-20436.091	10218.046	-20436.091

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	218.926	0.000	164.195	218.926
Bottom (-2 Axis)	108.529	81.396	0.000	108.529

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.332	35.049	0.000	35.049
				19.412

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tr
	0.000	0.000	167.430	8488.159
				29378.729
				844.400

Right end critical moment for Minor Beam 12th floor



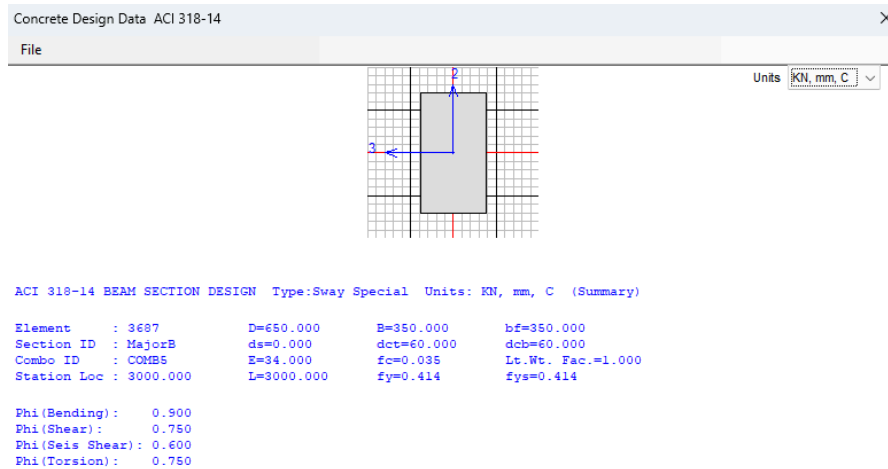
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	92226.636	-184453.272	92226.636	-184453.272

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	864.880	0.000	864.880	735.639
Bottom (-2 Axis)	567.942	425.956	0.000	567.942

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.380	221.755	152.162	69.593

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	8836.362	38294.101

Left end critical moment for Major Beam 11th floor



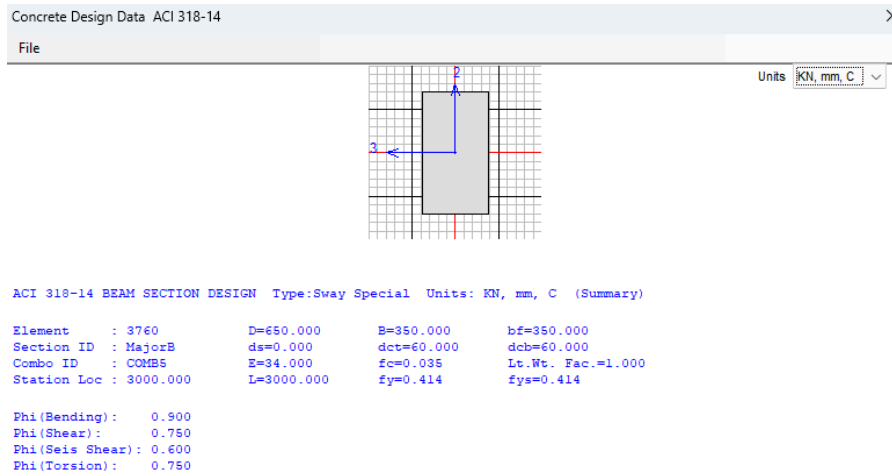
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	115647.154	0.000	115647.154	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	714.859	536.144	0.000	714.859

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.312	115.500	152.162	57.061

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	10451.162	38237.477

Mid critical moment for Major Beam 11th floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	124026.258	-248052.516	124026.258	-248052.516

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1175.766	0.000	1175.766	735.639
Bottom (-2 Axis)	735.639	575.772	0.000	735.639

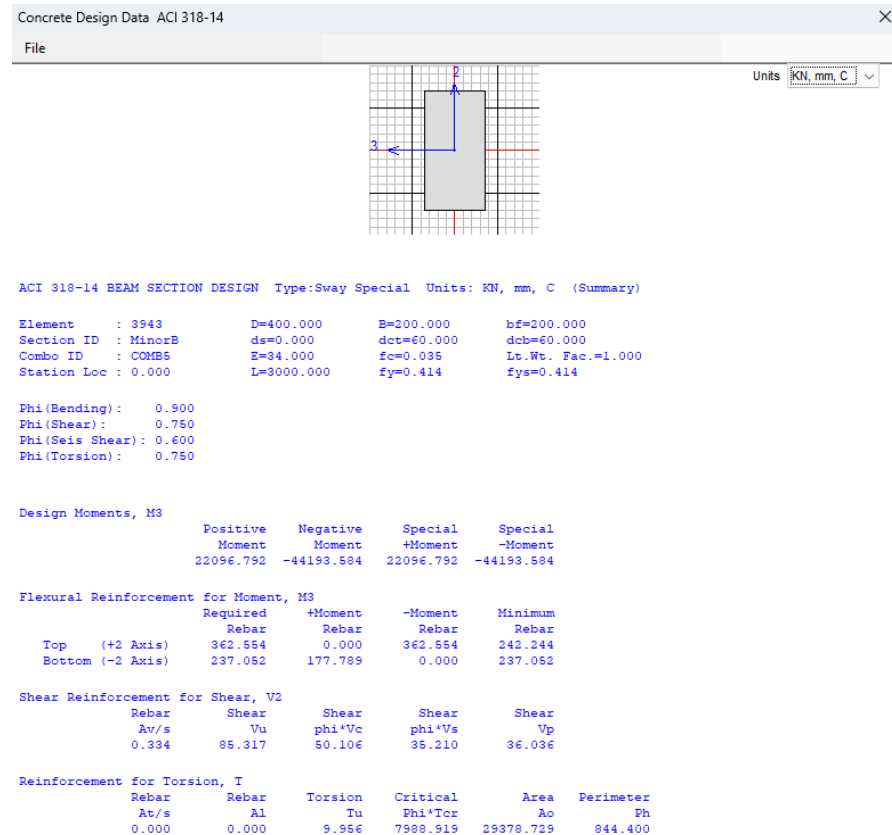
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.528	248.817	152.162	96.655	116.644

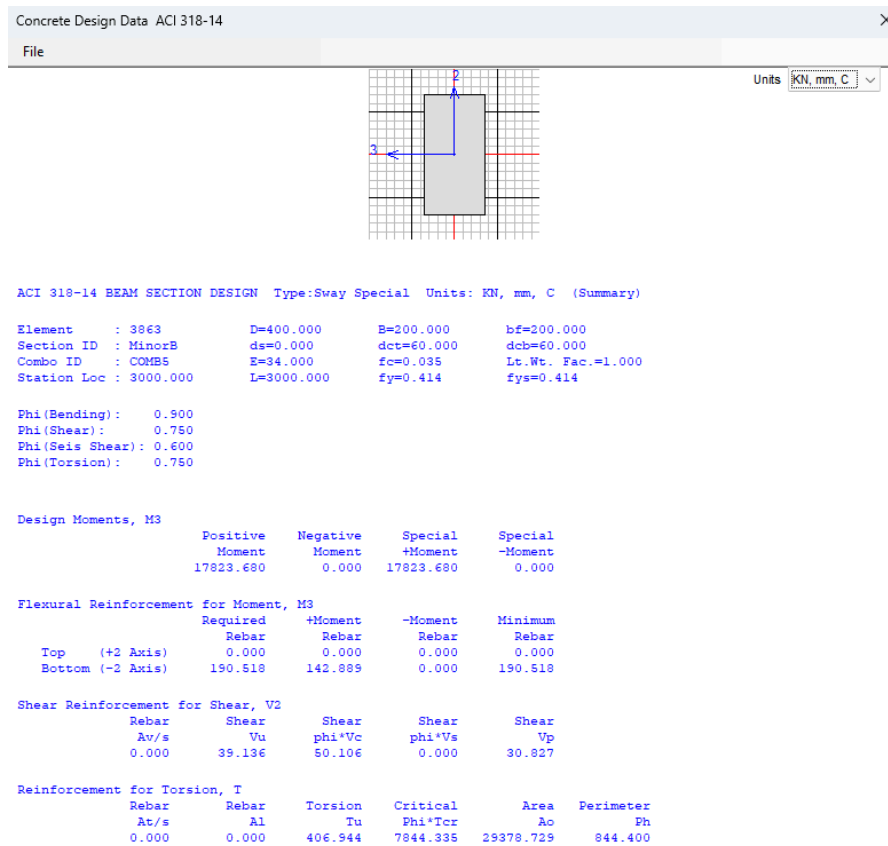
Reinforcement for Torsion, T

	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tr	Area Ao	Perimeter Ph
	0.000	0.000	1669.050	38407.155	124527.729	1644.400

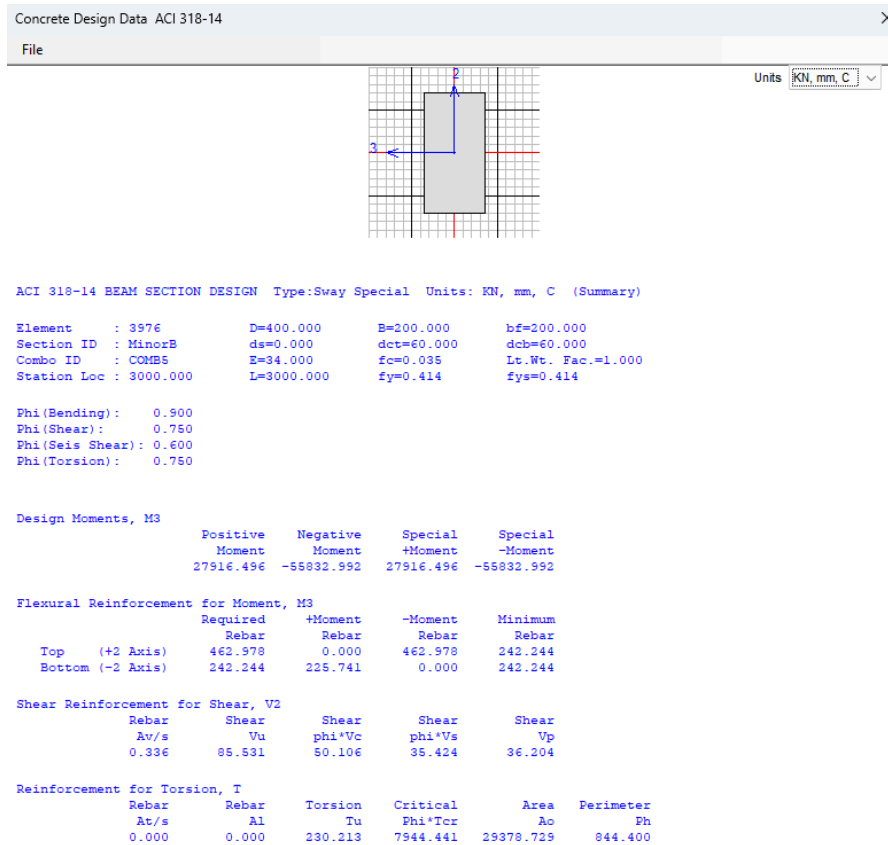
Right end critical moment for Major Beam 11th floor



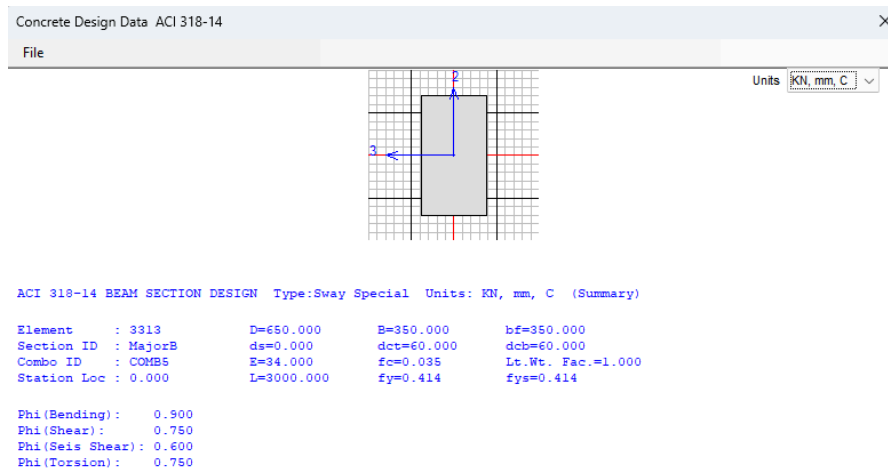
Left end critical moment for Minor Beam 11th floor



Mid critical moment for Minor Beam 11th floor



Right end critical moment for Minor Beam 11th floor



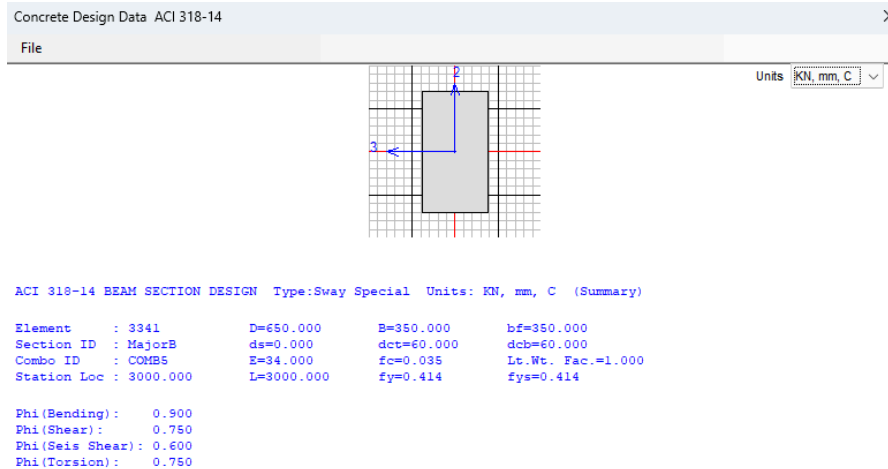
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	94129.443	-188258.886	94129.443	-188258.886

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	883.288	0.000	883.288	735.639
Bottom (-2 Axis)	579.836	434.877	0.000	579.836

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.390	223.464	152.162	71.302

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	10702.453	37956.391

Left end critical moment for Major Beam 10th floor



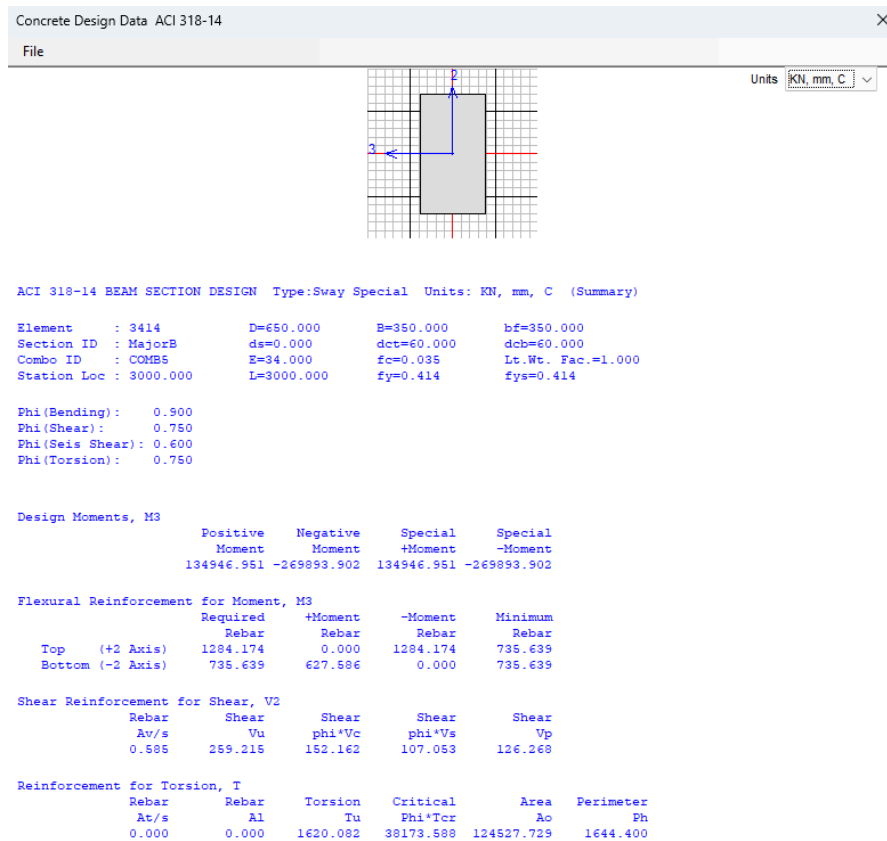
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	113958.816	0.000	113958.816	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	704.230	528.172	0.000	704.230

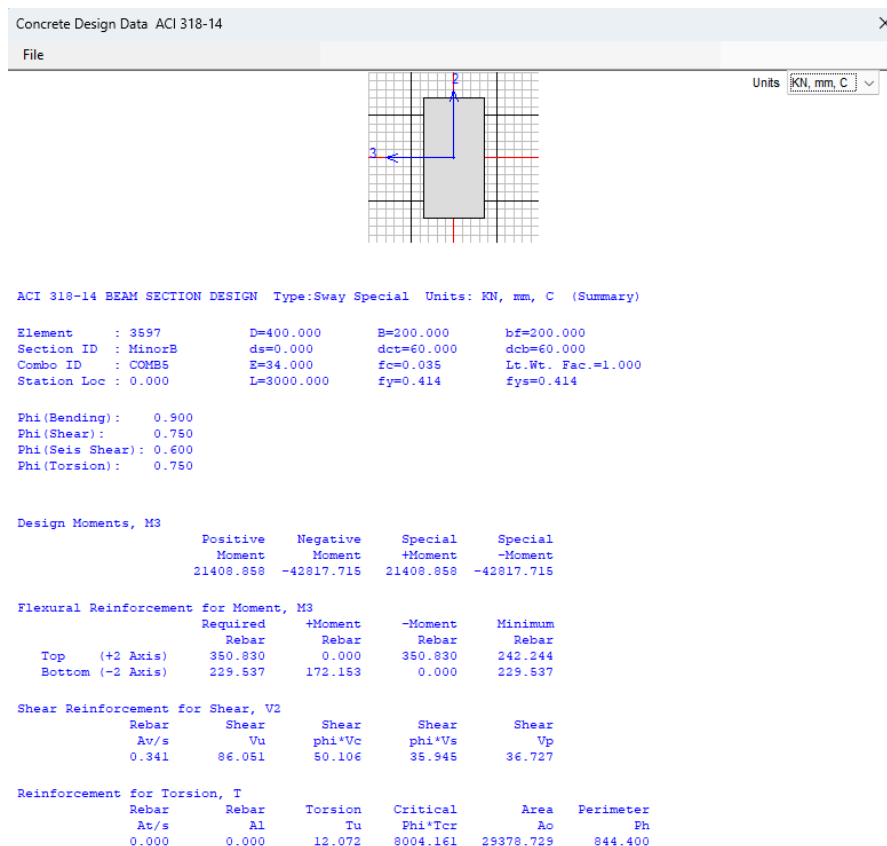
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.312	115.211	152.162	57.061

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	11617.423	37965.080

Mid critical moment for Major Beam 10th floor



Right end critical moment for Major Beam 10th floor



Left end critical moment for Minor Beam 10th floor

Concrete Design Data ACI 318-14

File

Units

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

Element	: 3517	D=400.000	B=200.000	bf=200.000
Section ID	: MinorB	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: COMB5	E=34.000	fc=0.035	Lt.Wt. Fac.=1.000
Station Loc	: 3000.000	L=3000.000	fy=0.414	fys=0.414

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

Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	17541.823	0.000	17541.823	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	187.461	140.596	0.000	187.461

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.000	39.114	50.106	0.000
				30.696

Reinforcement for Torsion, T					
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao
	0.000	0.000	486.645	7838.506	29378.729
					844.400

Mid critical moment for Minor Beam 10th floor

Concrete Design Data ACI 318-14

File

Units

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

Element	: 3630	D=400.000	B=200.000	bf=200.000
Section ID	: MinorB	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: COMB5	E=34.000	fc=0.035	Lt.Wt. Fac.=1.000
Station Loc	: 3000.000	L=3000.000	fy=0.414	fys=0.414

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

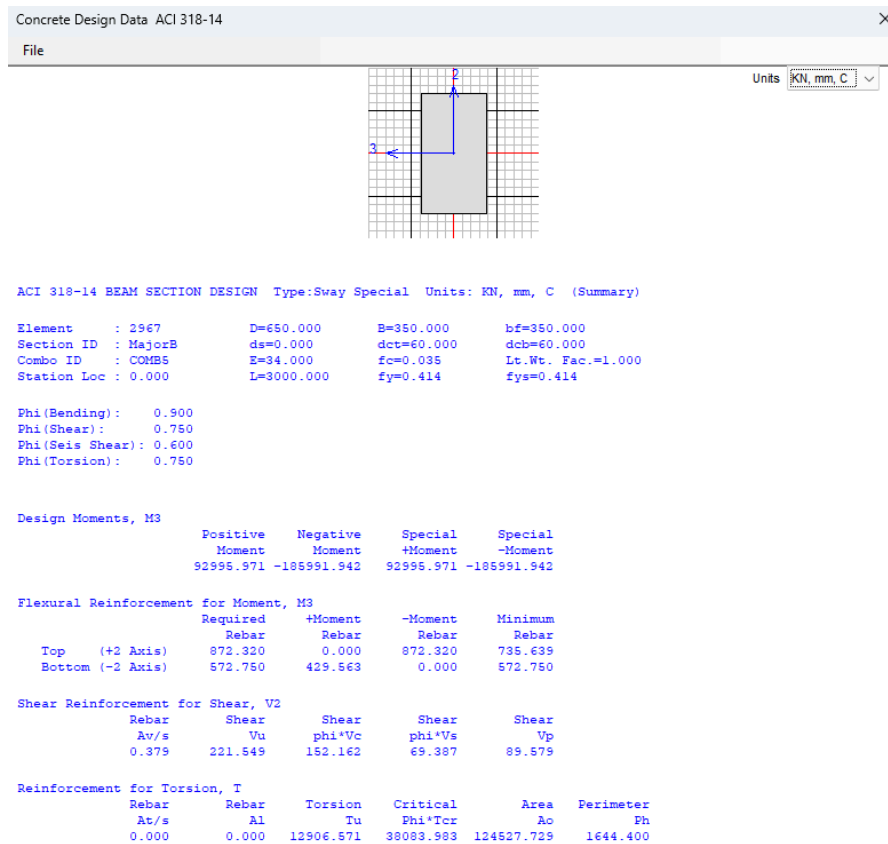
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	28579.948	-57159.895	28579.948	-57159.895

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	474.571	0.000	474.571	242.244
Bottom (-2 Axis)	242.244	231.239	0.000	242.244

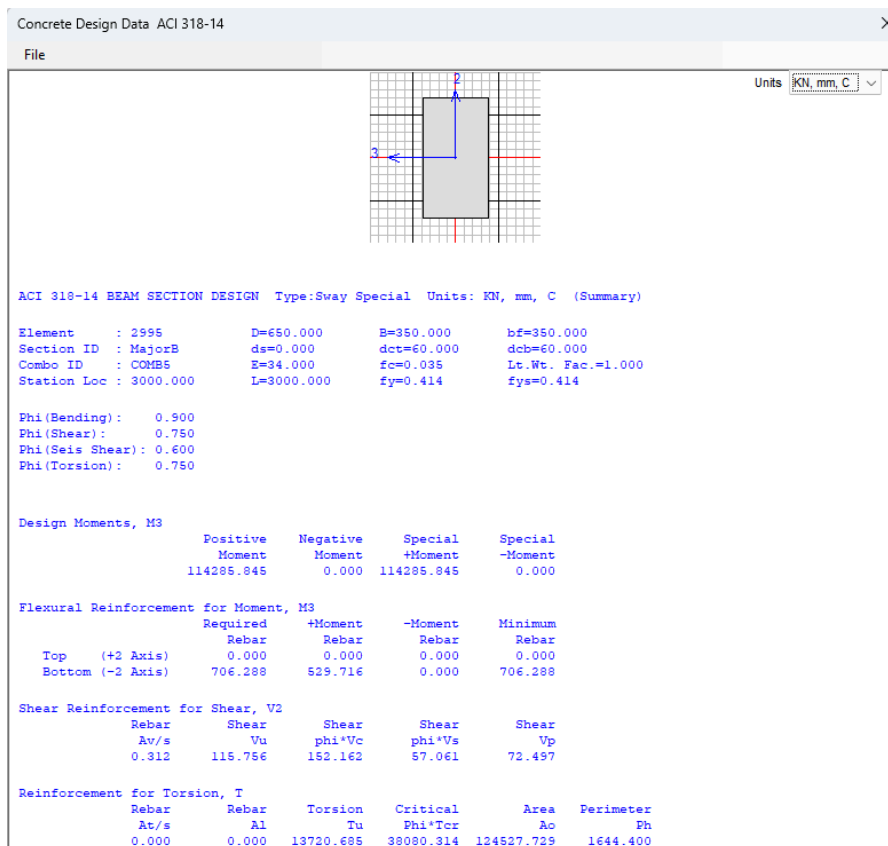
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.342	86.233	50.106	36.127
				36.882

Reinforcement for Torsion, T					
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao
	0.000	0.000	196.757	7992.191	29378.729
					844.400

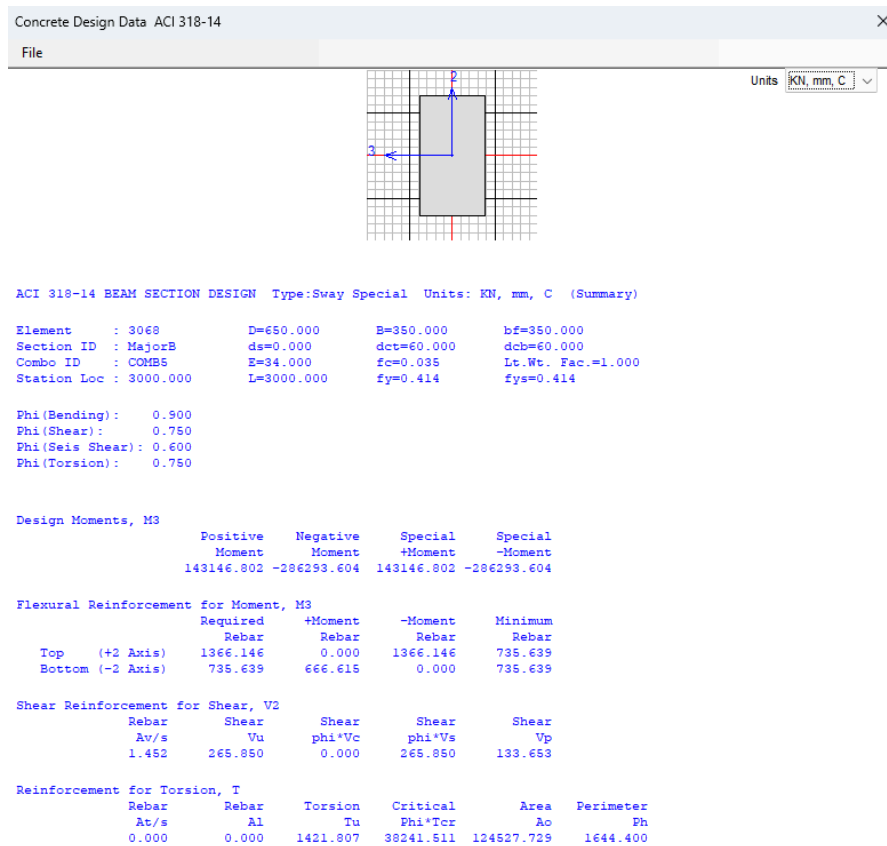
Right end critical moment for Minor Beam 10th floor



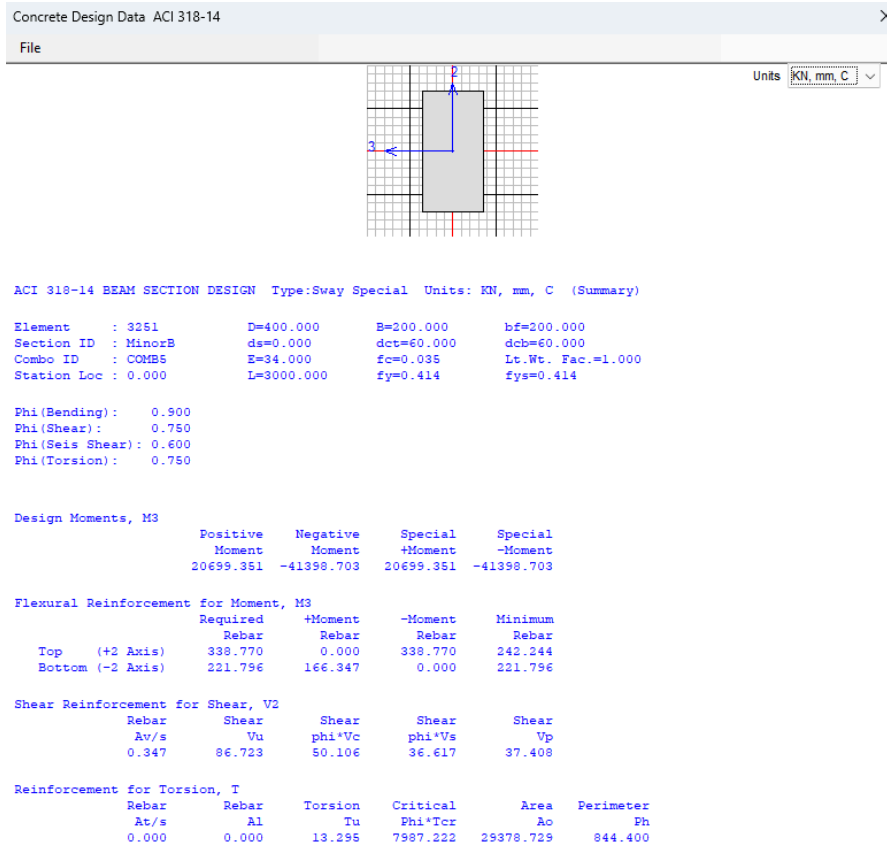
Left end critical moment for Major Beam 9th floor



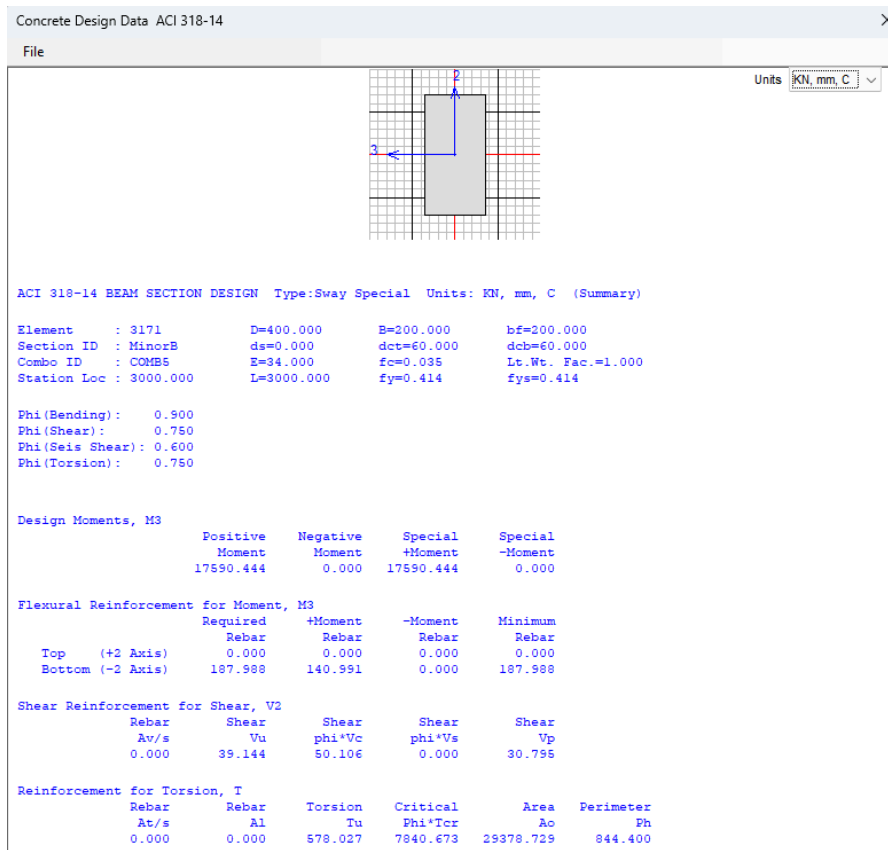
Mid critical moment for Major Beam 9th floor



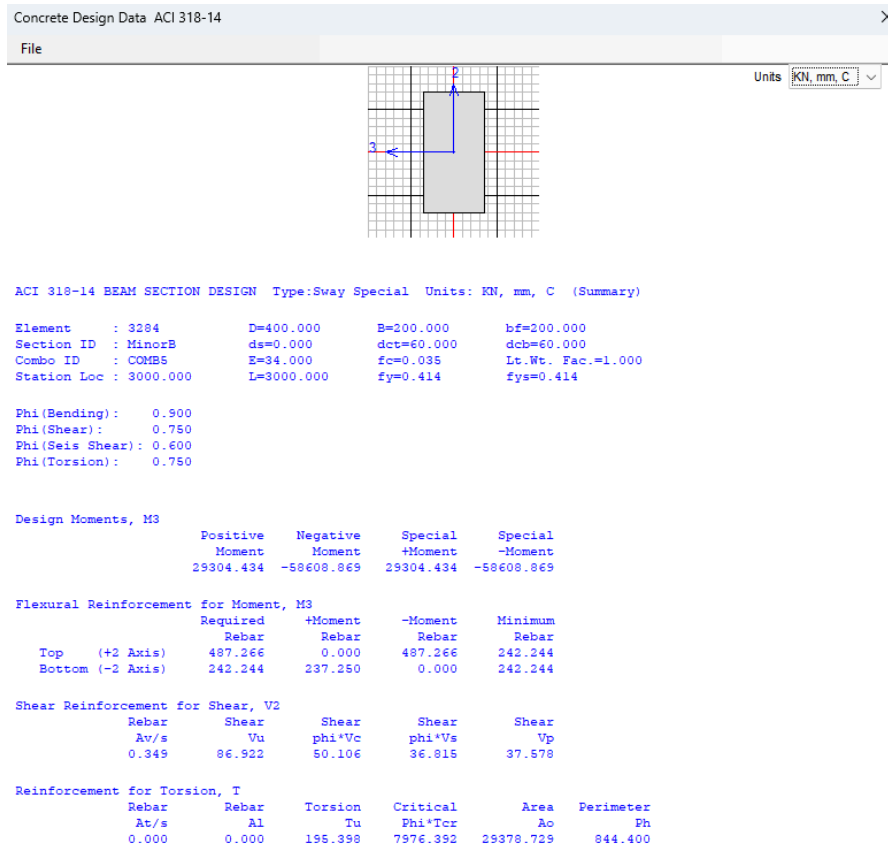
Right end critical moment for Major Beam 9th floor



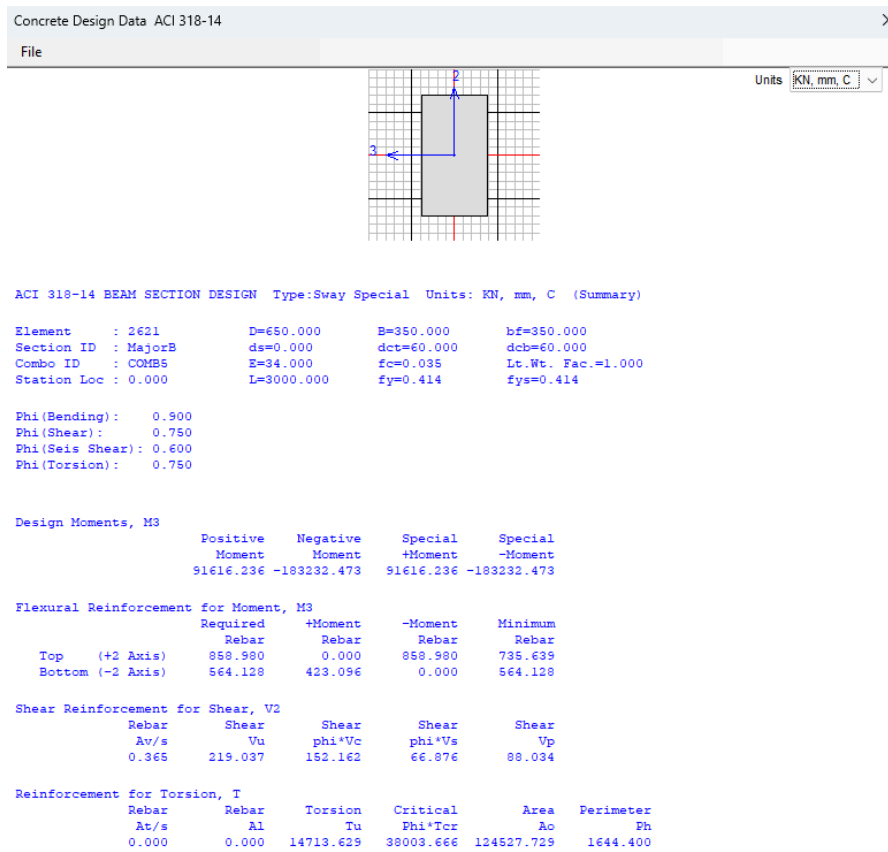
Left end critical moment for Minor Beam 9th floor



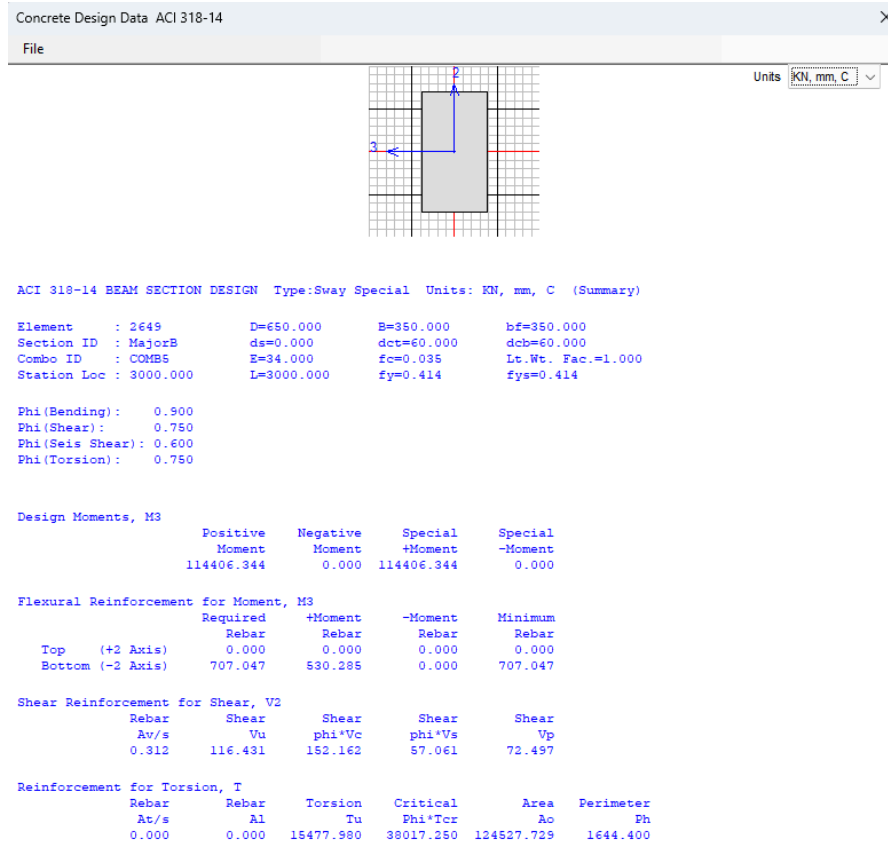
Mid critical moment for Minor Beam 9th floor



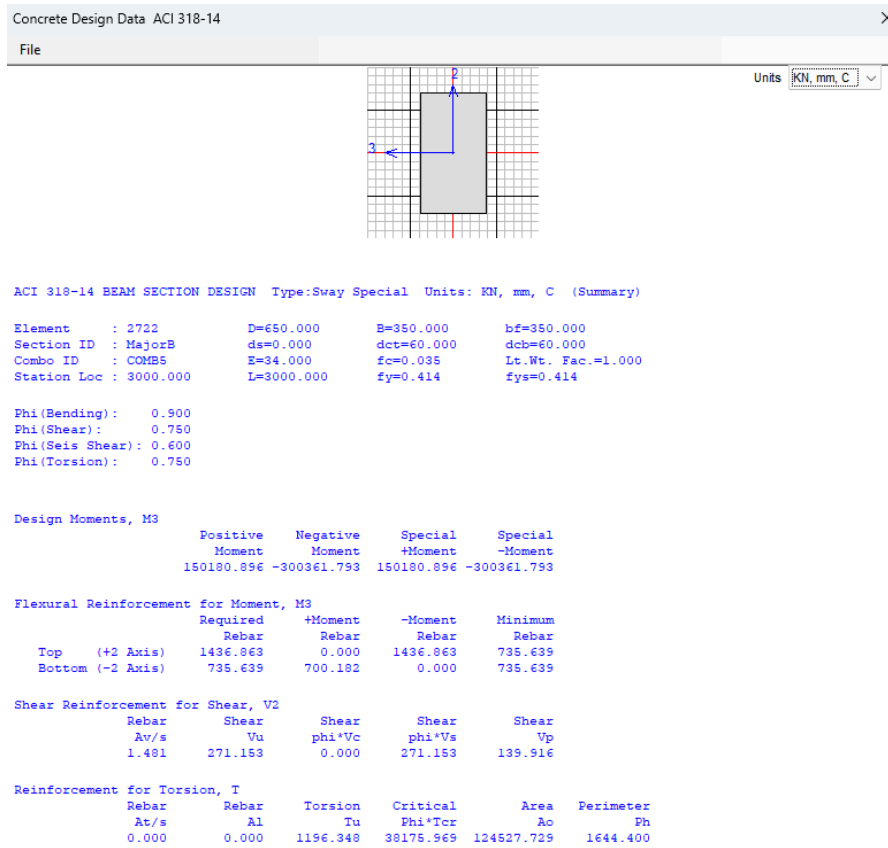
Right end critical moment for Minor Beam 9th floor



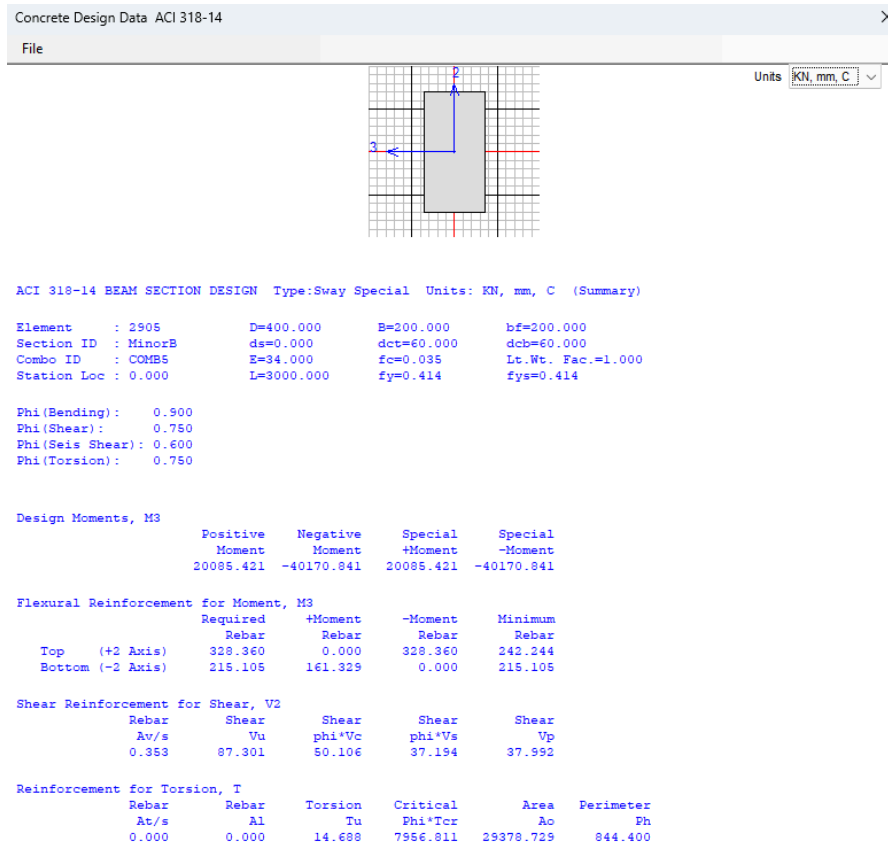
Left end critical moment for Major Beam 8th floor



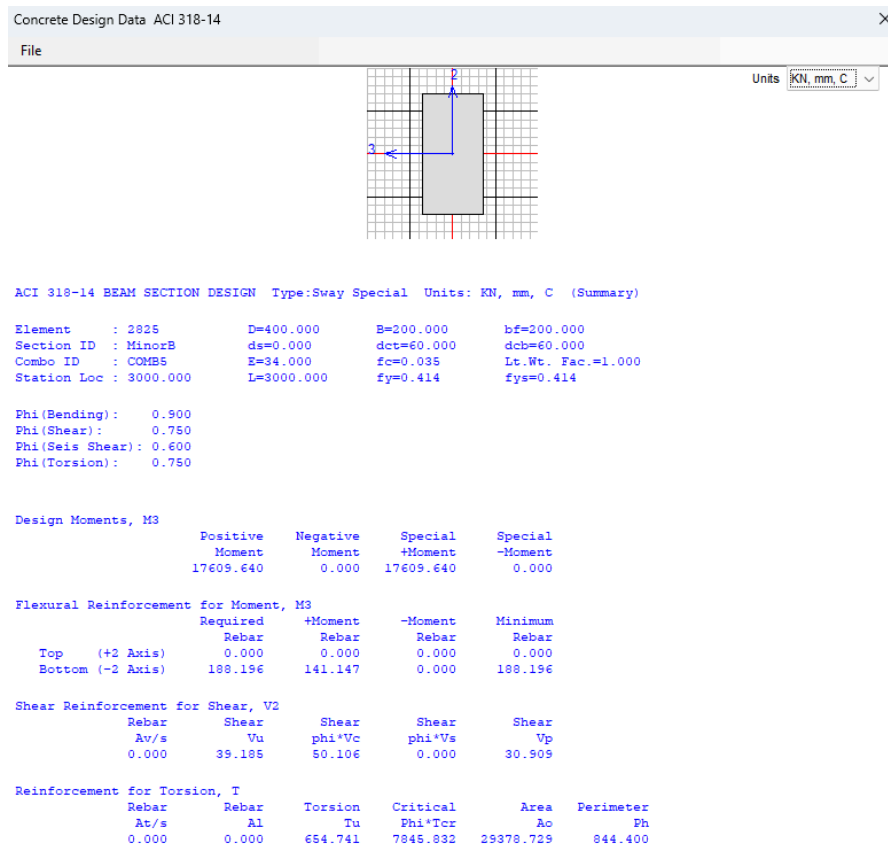
Mid critical moment for Major Beam 8th floor



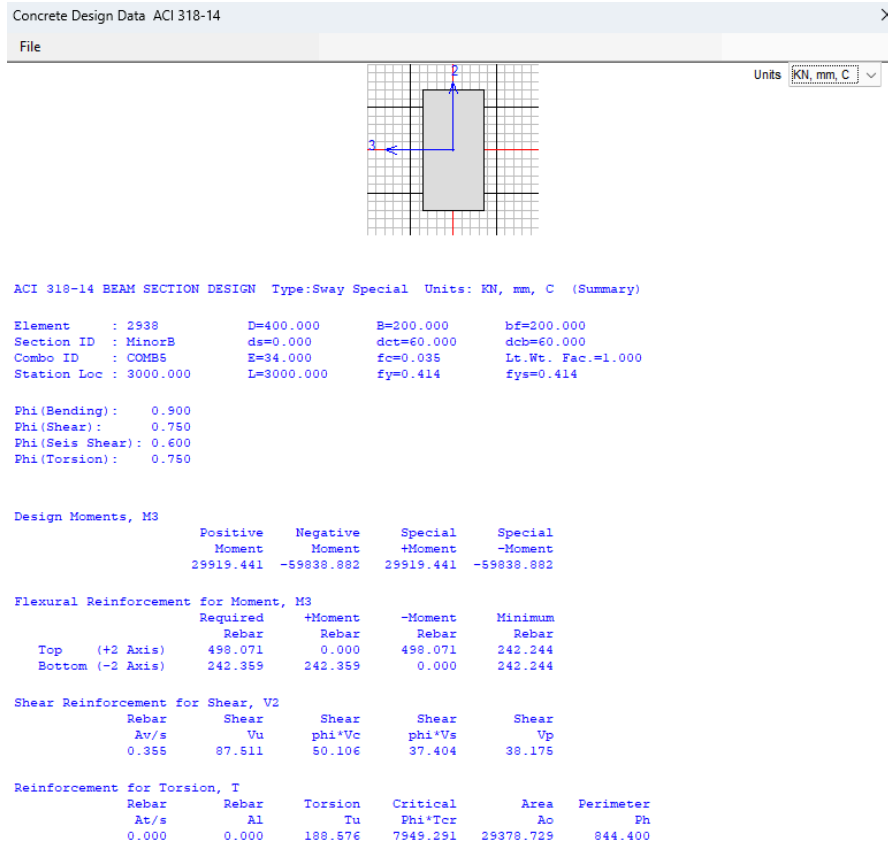
Right end critical moment for Major Beam 8th floor



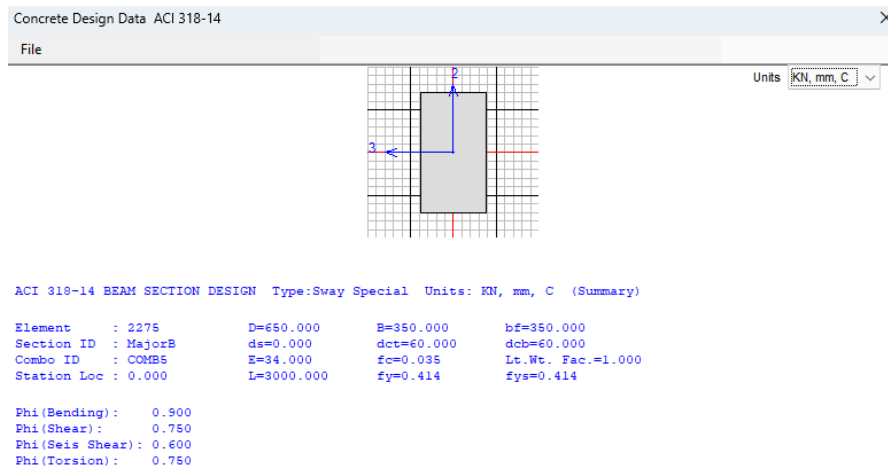
Left end critical moment for Minor Beam 8th floor



Mid critical moment for Minor Beam 8th floor



Right end critical moment for Minor Beam 8th floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	91717.552	-183435.103	91717.552	-183435.103

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	859.959	0.000	859.959	735.639
Bottom (-2 Axis)	564.761	423.571	0.000	564.761

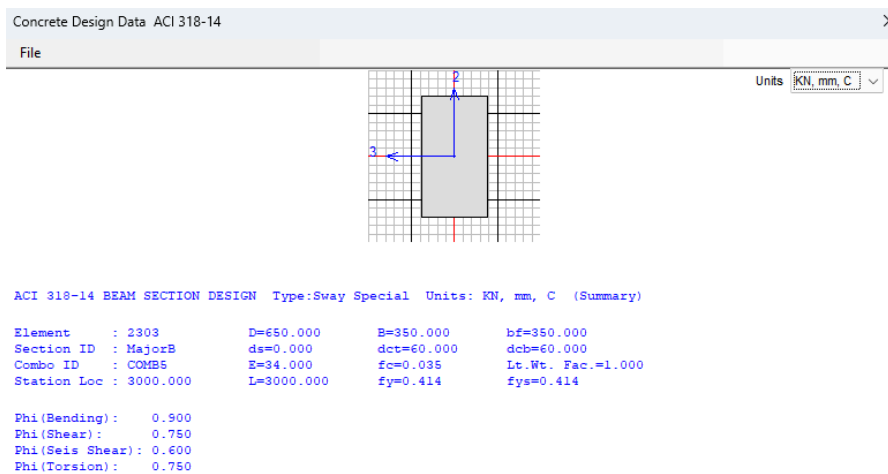
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.364	218.765	152.162	66.603	87.989

Reinforcement for Torsion, T

Rebar	Rebar	Torsion	Critical	Area	Perimeter
At/s	Al	Tu	Phi*Tcr	Ac	Ph
0.000	0.000	16343.158	38188.462	124527.729	1644.400

Left end critical moment for Major Beam 7th floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	113421.247	0.000	113421.247	0.000

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	700.847	525.635	0.000	700.847

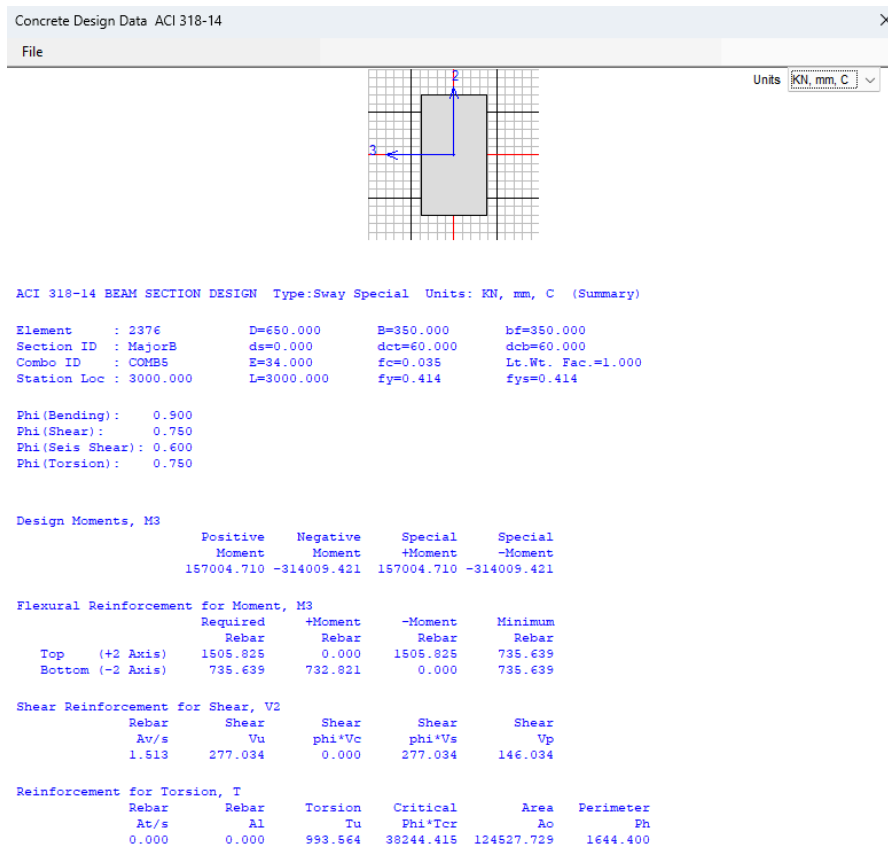
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.312	116.673	152.162	57.061	72.497

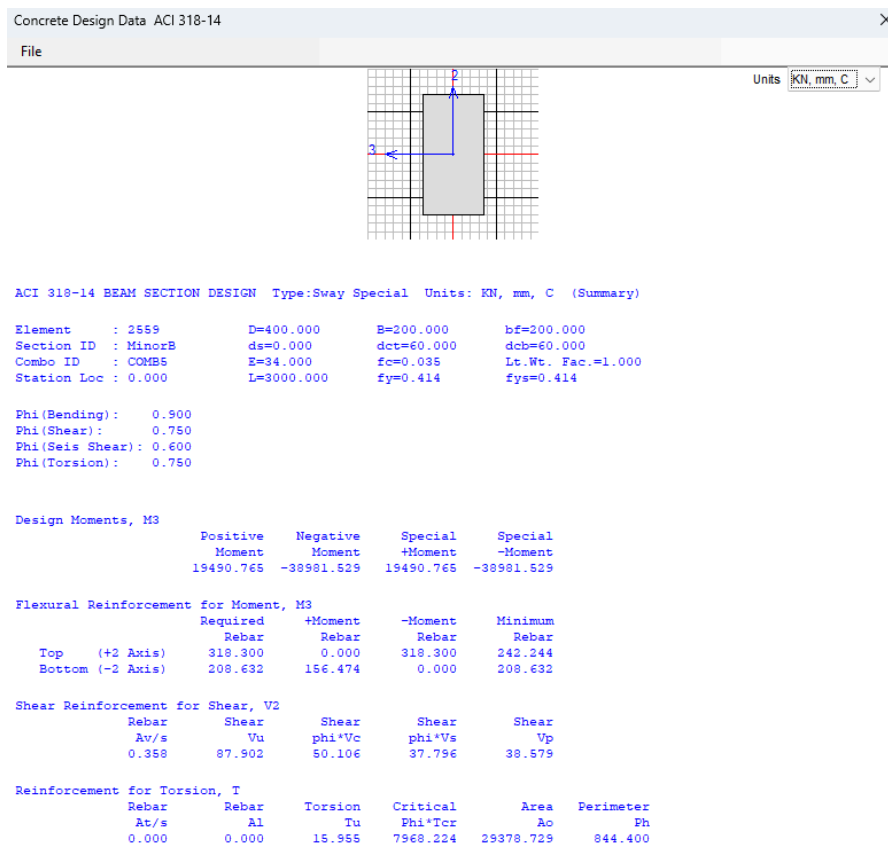
Reinforcement for Torsion, T

Rebar	Rebar	Torsion	Critical	Area	Perimeter
At/s	Al	Tu	Phi*Tcr	Ac	Ph
0.000	0.000	16716.451	38165.924	124527.729	1644.400

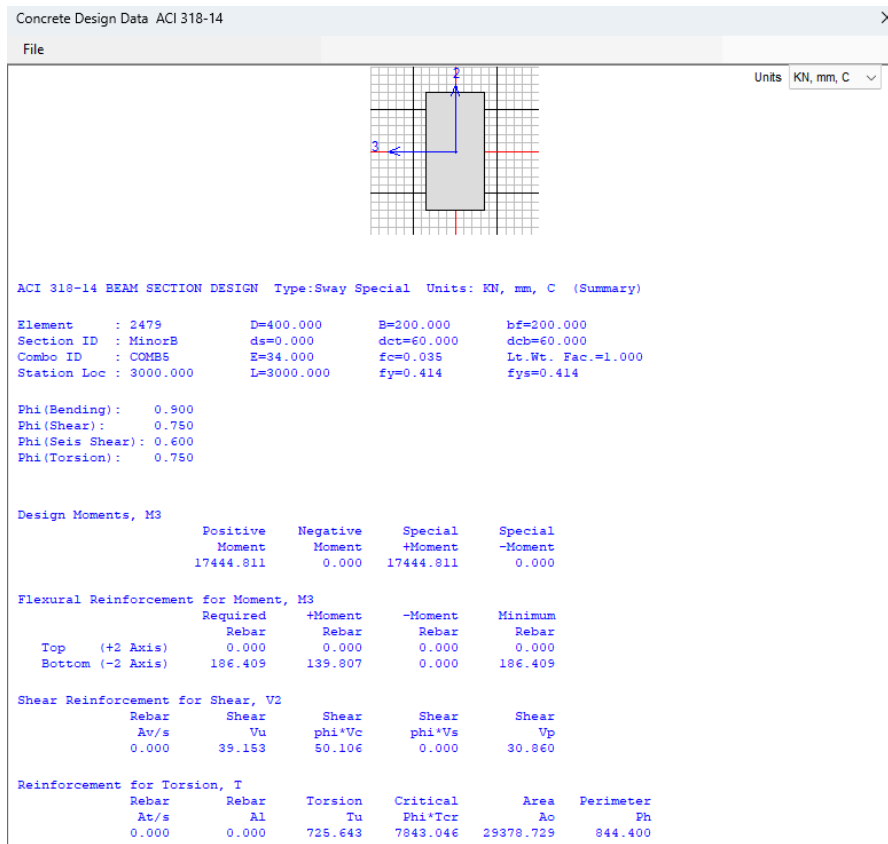
Mid critical moment for Major Beam 7th floor



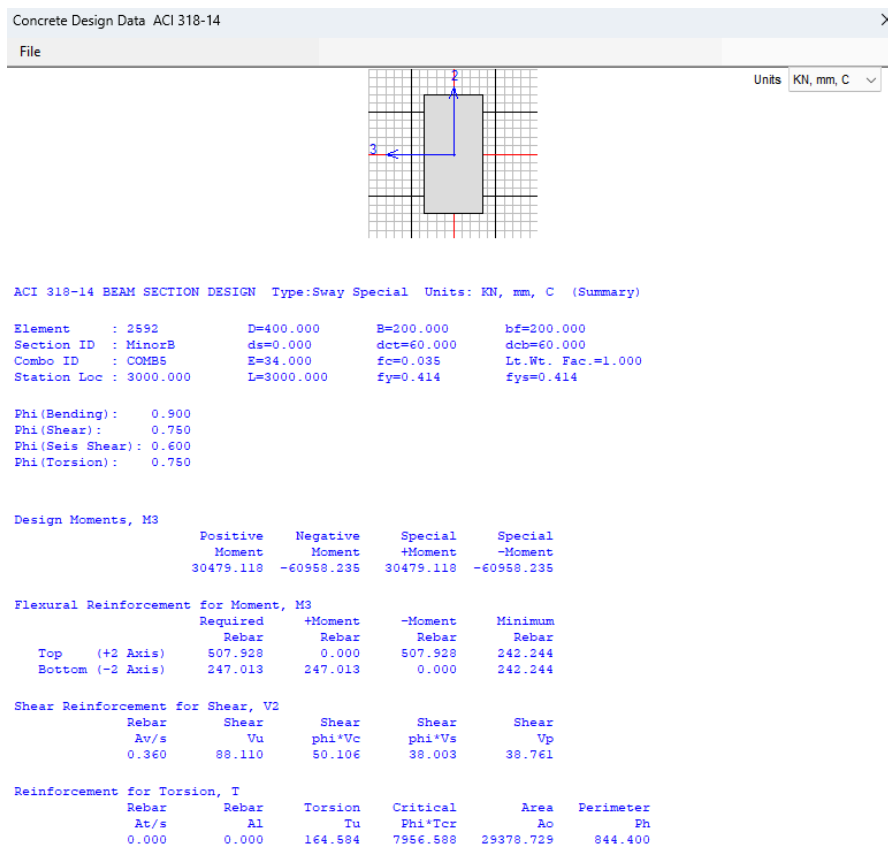
Right end critical moment for Major Beam 7th floor



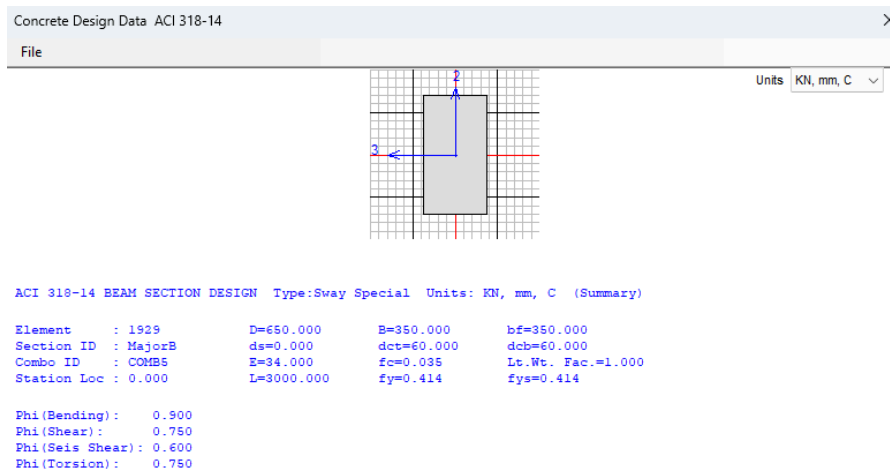
Left end critical moment for Minor Beam 7th floor



Mid critical moment for Minor Beam 7th floor



Right end critical moment for Minor Beam 7th floor



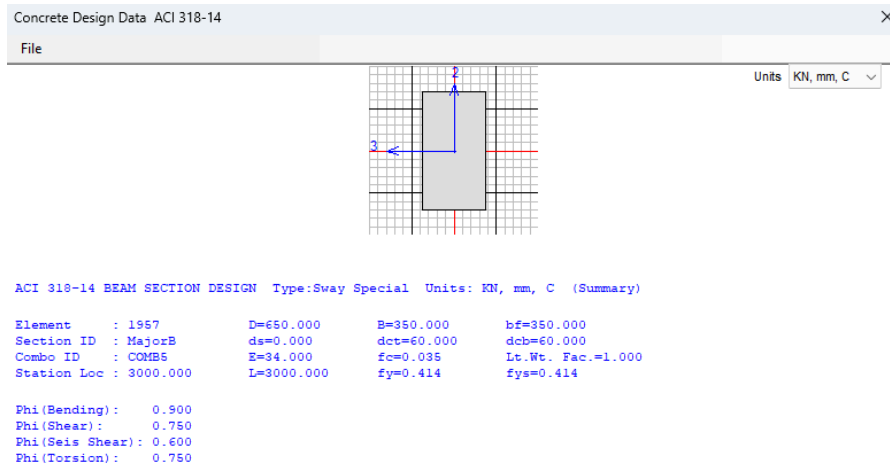
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	91289.211	-182578.422	91289.211	-182578.422

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	855.820	0.000	855.820	735.639
Bottom (-2 Axis)	562.085	421.564	0.000	562.085

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs Vp
	0.358	217.737	152.162	65.576

Reinforcement for Torsion, T					
Rebar	Rebar	Torsion	Critical	Area	Perimeter
At/s	Al	Tu	Phi*Tr	Ac	Ph
0.000	0.000	17555.084	38024.855	124527.729	1644.400

Left end critical moment for Major Beam 6th floor



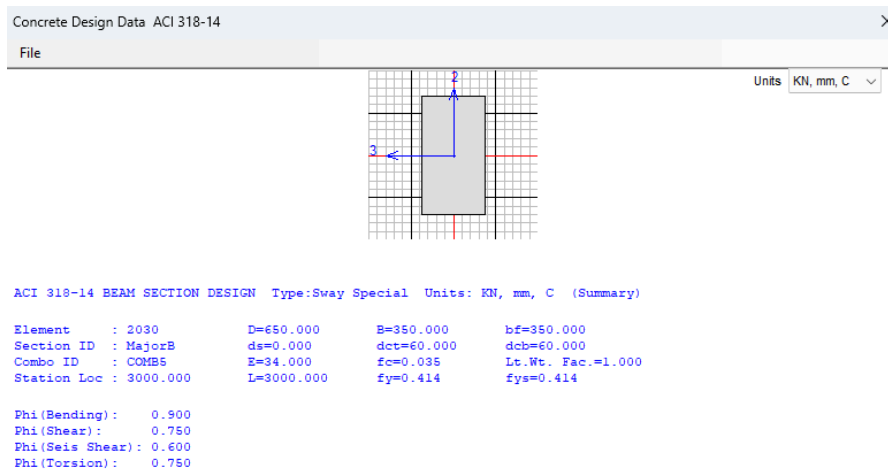
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	112624.318	0.000	112624.318	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	695.833	521.875	0.000	695.833

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs Vp
	0.312	117.071	152.162	57.061

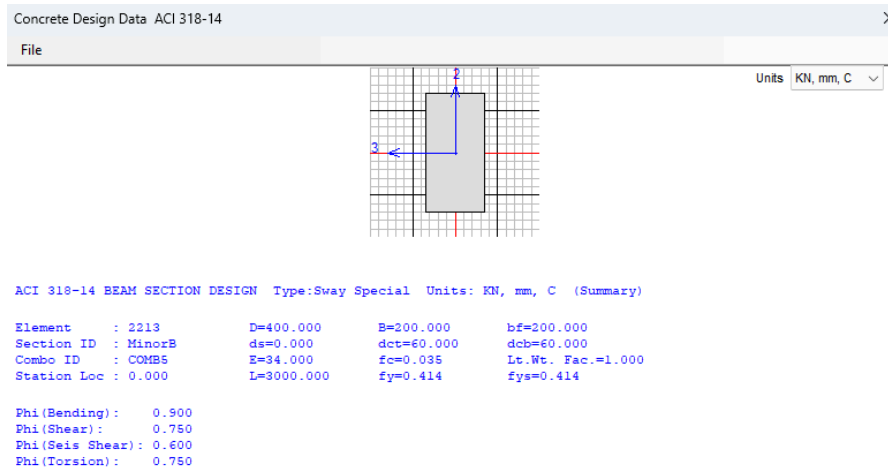
Reinforcement for Torsion, T					
Rebar	Rebar	Torsion	Critical	Area	Perimeter
At/s	Al	Tu	Phi*Tr	Ac	Ph
0.000	0.000	17628.815	38043.943	124527.729	1644.400

Mid critical moment for Major Beam 6th floor



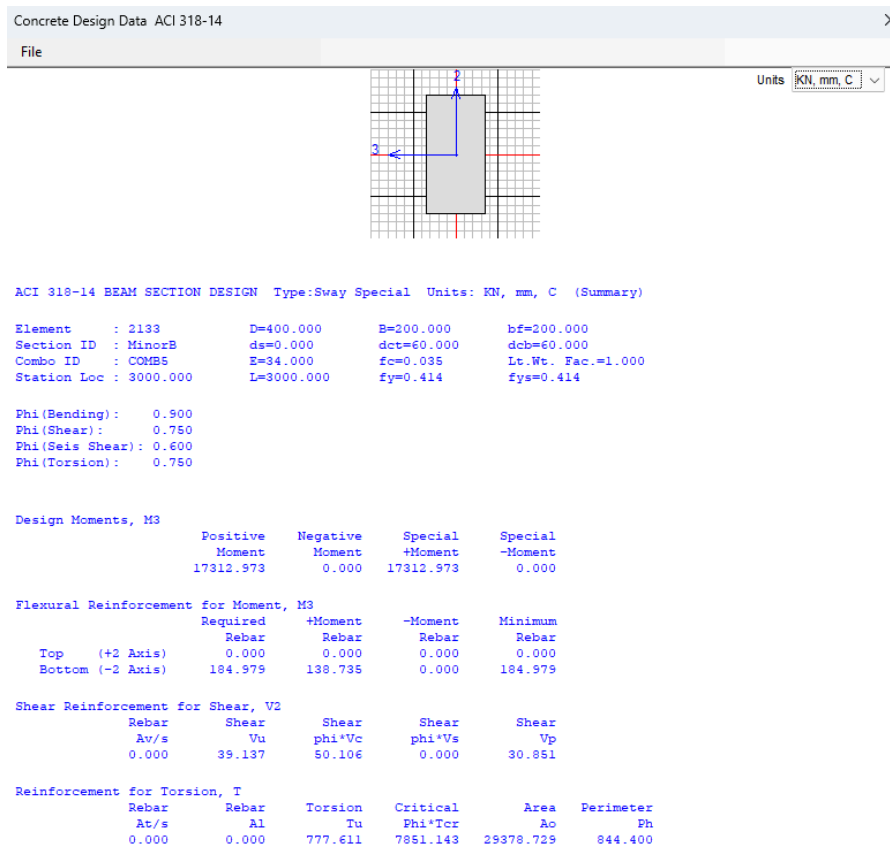
Design Moments, M3					
	Positive Moment	Negative Moment	Special +Moment	Special -Moment	
	161559.387	-323118.774	161559.387	-323118.774	
Flexural Reinforcement for Moment, M3					
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar	
Top (+2 Axis)	1552.053	0.000	1552.053	735.639	
Bottom (-2 Axis)	754.649	754.649	0.000	735.639	
Shear Reinforcement for Shear, V2					
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	1.533	280.570	0.000	280.570	150.092
Reinforcement for Torsion, T					
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tr	Area Ao
	0.000	0.000	784.321	38155.882	124527.729
					Perimeter Ph
					1644.400

Right end critical moment for Major Beam 6th floor

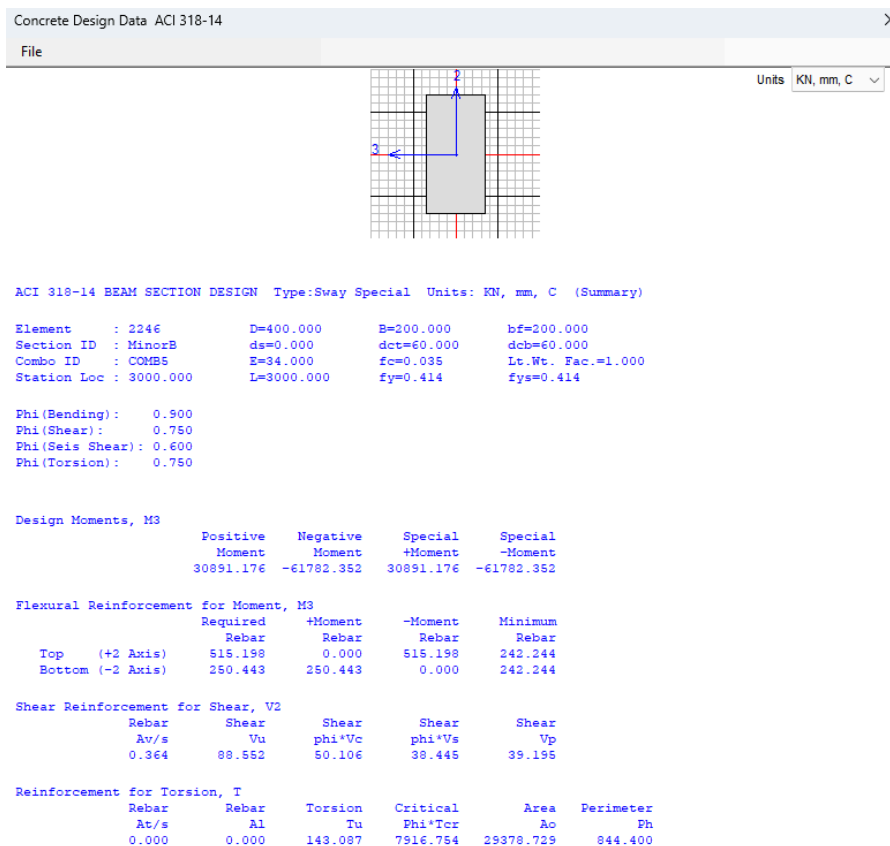


Design Moments, M3					
	Positive Moment	Negative Moment	Special +Moment	Special -Moment	
	19043.471	-38086.941	19043.471	-38086.941	
Flexural Reinforcement for Moment, M3					
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar	
Top (+2 Axis)	310.747	0.000	310.747	242.244	
Bottom (-2 Axis)	203.767	152.825	0.000	203.767	
Shear Reinforcement for Shear, V2					
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.363	88.348	50.106	38.241	39.014
Reinforcement for Torsion, T					
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tr	Area Ao
	0.000	0.000	16.913	7920.827	29378.729
					Perimeter Ph
					844.400

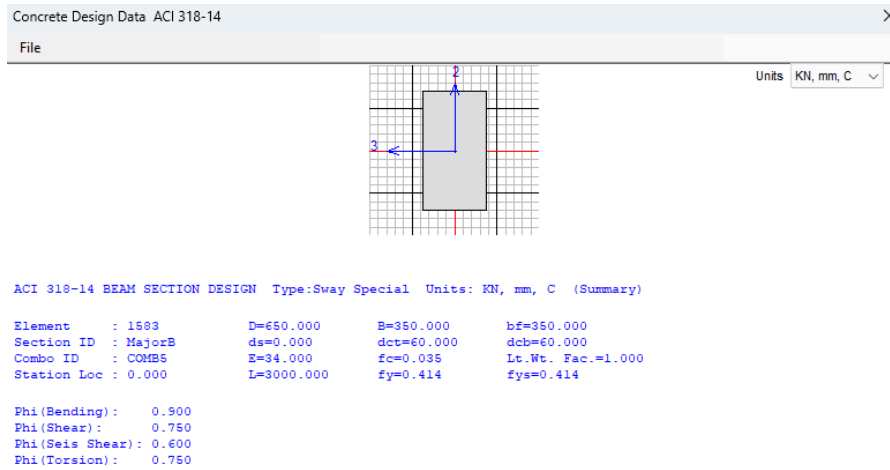
Left end critical moment for Minor Beam 6th floor



Mid critical moment for Minor Beam 6th floor



Right end critical moment for Minor Beam 6th floor



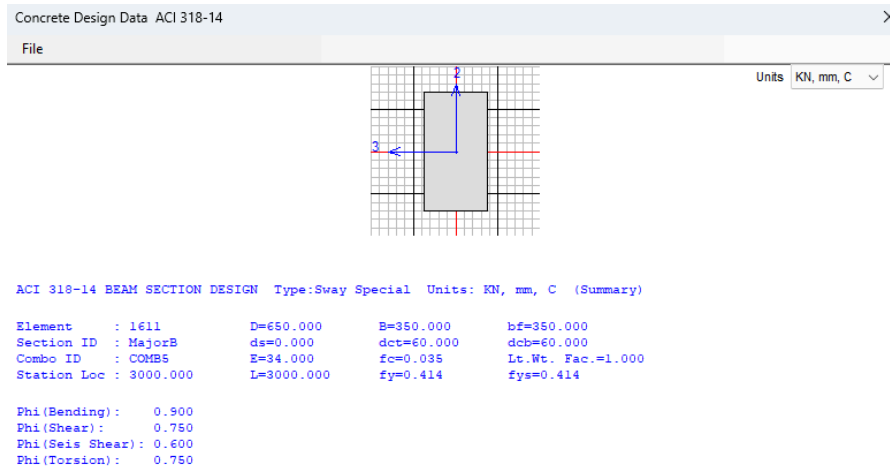
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	88862.520	-177725.039	88862.520	-177725.039

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	832.395	0.000	832.395	735.639
Bottom (-2 Axis)	546.931	410.198	0.000	546.931

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.336	213.635	152.162	61.474
				84.959

Reinforcement for Torsion, T						
Rebar	Rebar	Torsion	Critical	Area	Perimeter	
At/s	Al	Tu	Phi*Tr	Ac	Ph	
0.000	0.000	18648.314	38073.653	124527.729	1644.400	

Left end critical moment for Major Beam 5th floor



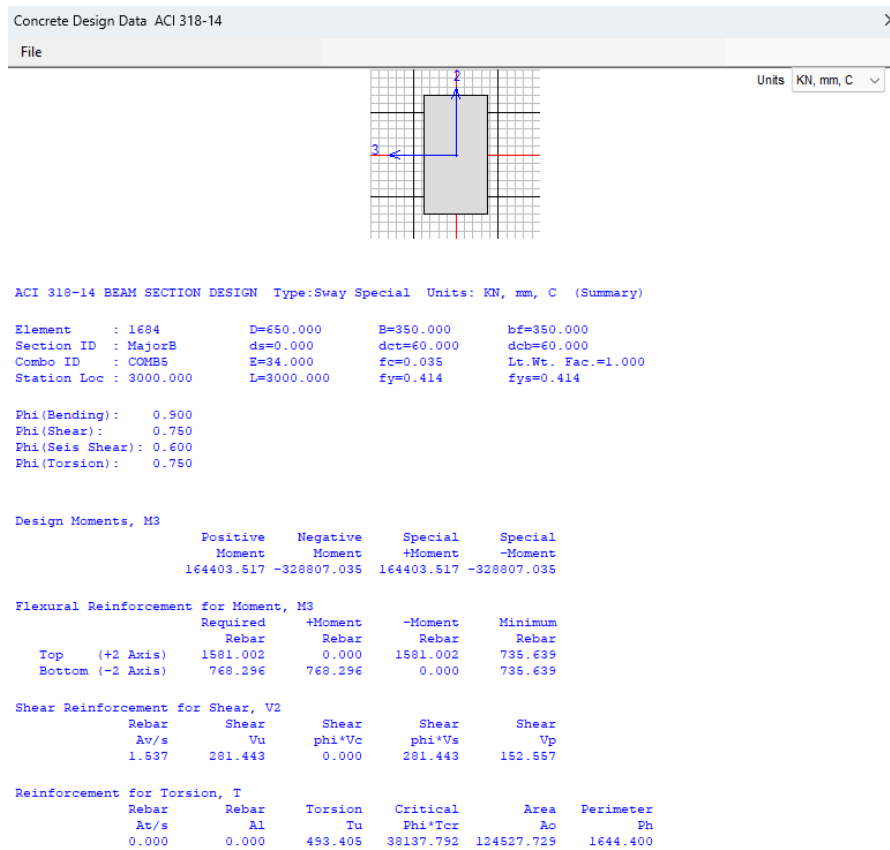
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	112820.472	0.000	112820.472	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	697.067	522.800	0.000	697.067

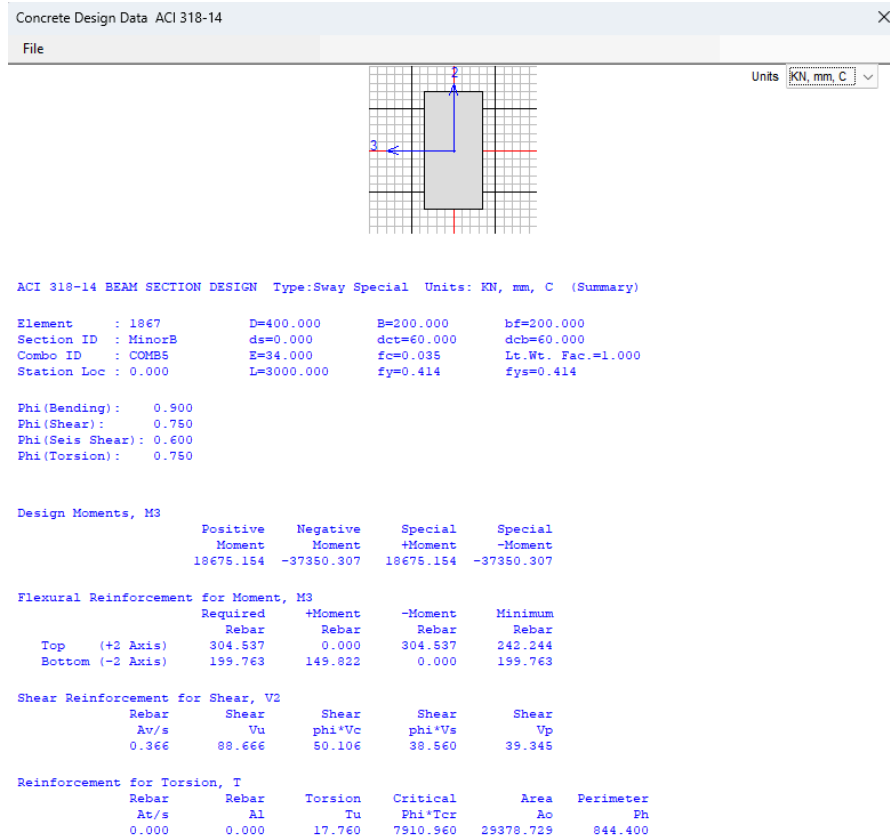
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.312	118.065	152.162	57.061
				72.497

Reinforcement for Torsion, T						
Rebar	Rebar	Torsion	Critical	Area	Perimeter	
At/s	Al	Tu	Phi*Tr	Ac	Ph	
0.000	0.000	18748.457	38084.714	124527.729	1644.400	

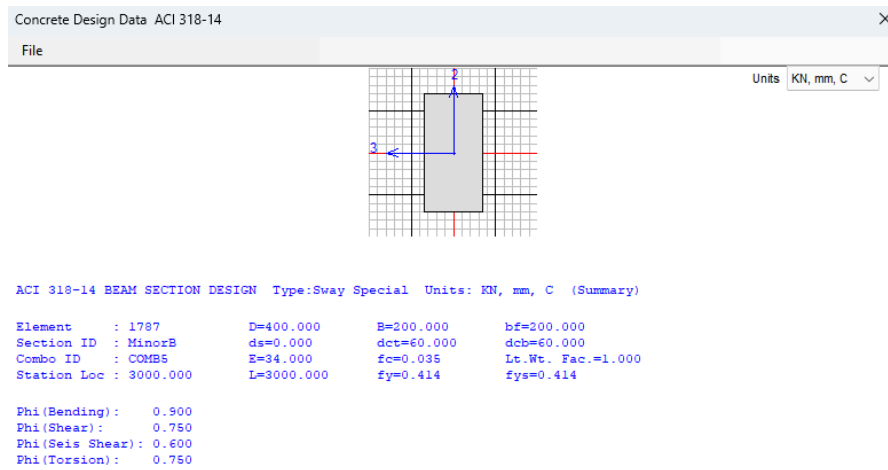
Mid critical moment for Major Beam 5th floor



Right end critical moment for Major Beam 5th floor



Left end critical moment for Minor Beam 5th floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	17350.368	0.000	17350.368	0.000

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	185.385	139.039	0.000	185.385

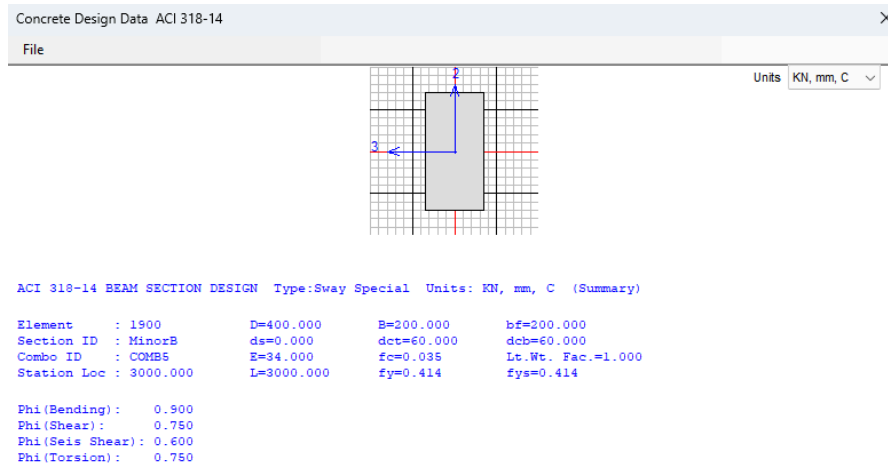
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.000	39.200	50.106	0.000	31.026

Reinforcement for Torsion, T

Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	820.744	7852.426	29378.729	844.400

Mid critical moment for Minor Beam 5th floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	31244.773	-62489.547	31244.773	-62489.547

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	521.447	0.000	521.447	242.244
Bottom (-2 Axis)	253.388	253.388	0.000	242.244

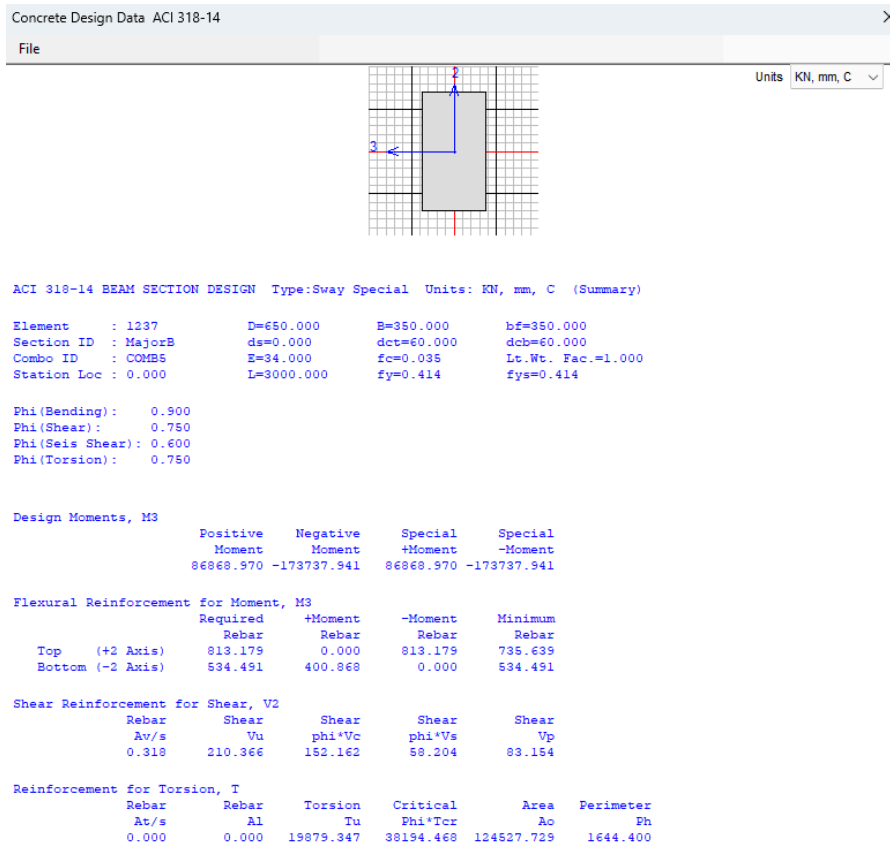
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.368	88.876	50.106	38.769	39.533

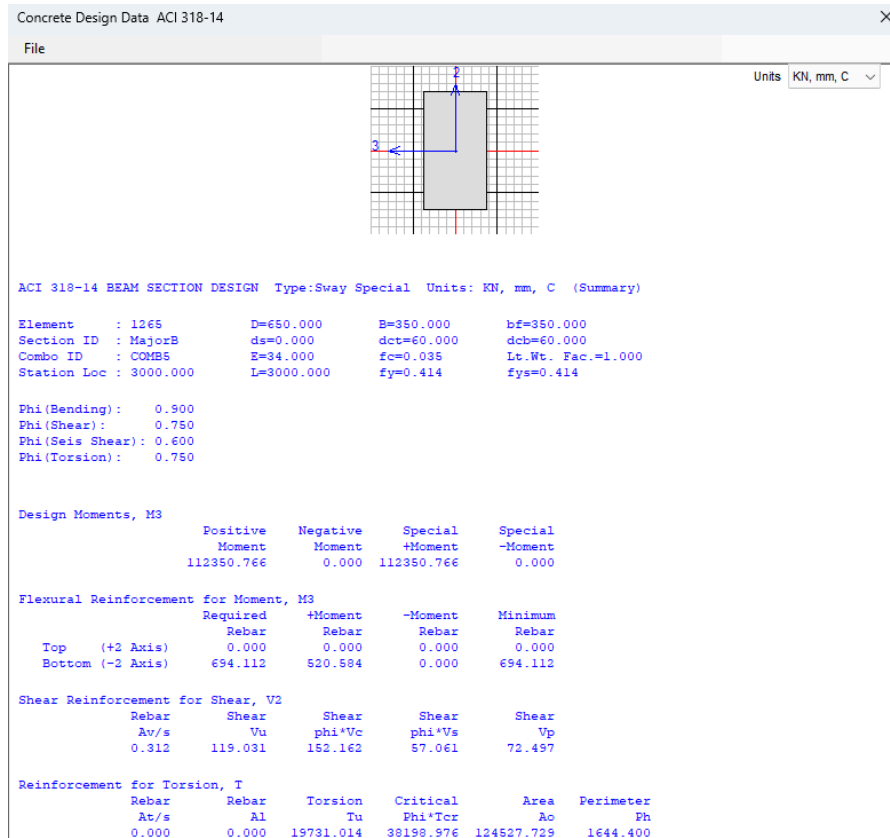
Reinforcement for Torsion, T

Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	135.490	7906.424	29378.729	844.400

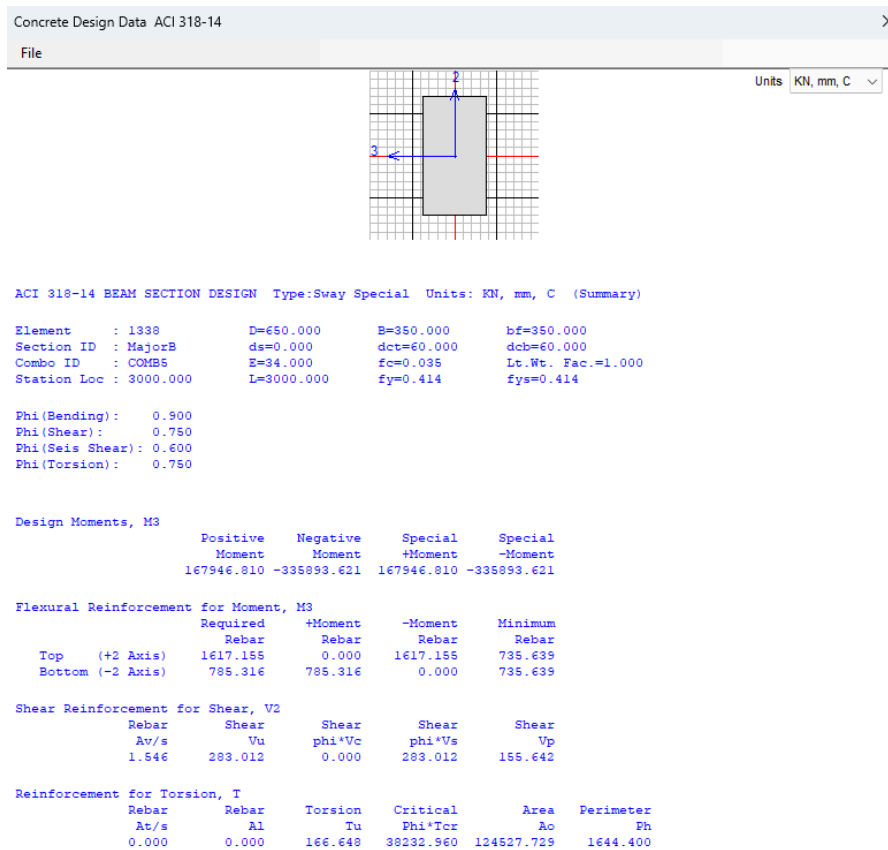
Right end critical moment for Minor Beam 5th floor



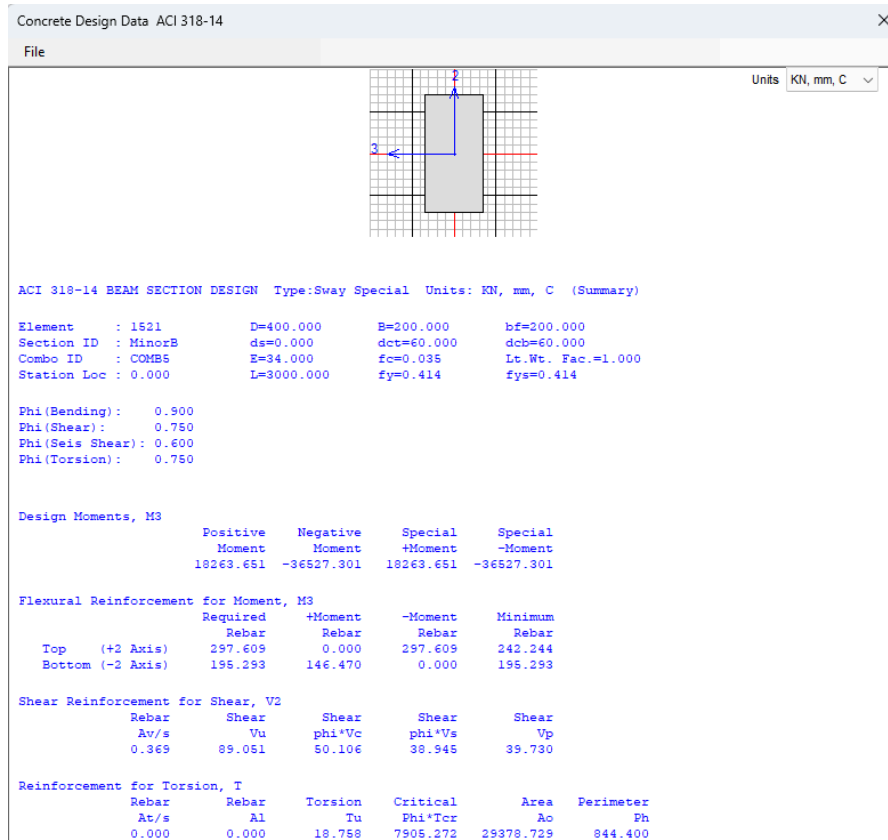
Left end critical moment for Major Beam 4th floor



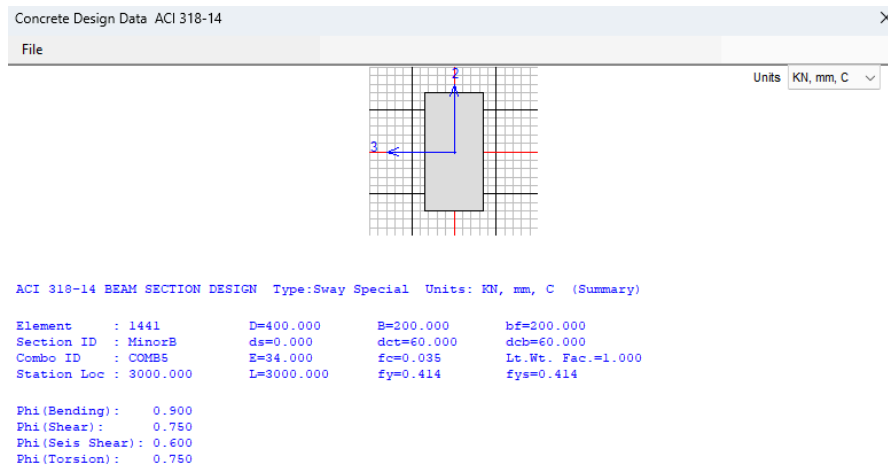
Mid critical moment for Major Beam 4th floor



Right end critical moment for Major Beam 4th floor



Left end critical moment for Minor Beam 4th floor



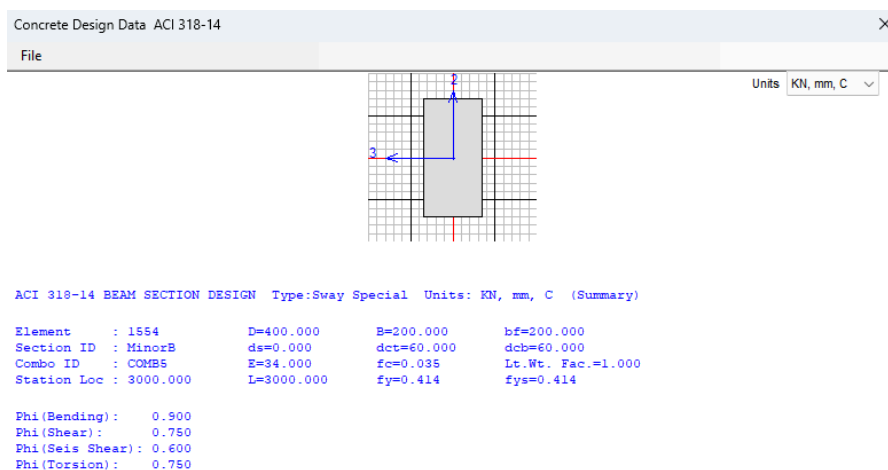
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	17276.248	0.000	17276.248	0.000

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	184.581	138.436	0.000	184.581

Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.000	39.199	50.106	0.000

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	866.908	7852.436

Mid critical moment for Minor Beam 4th floor



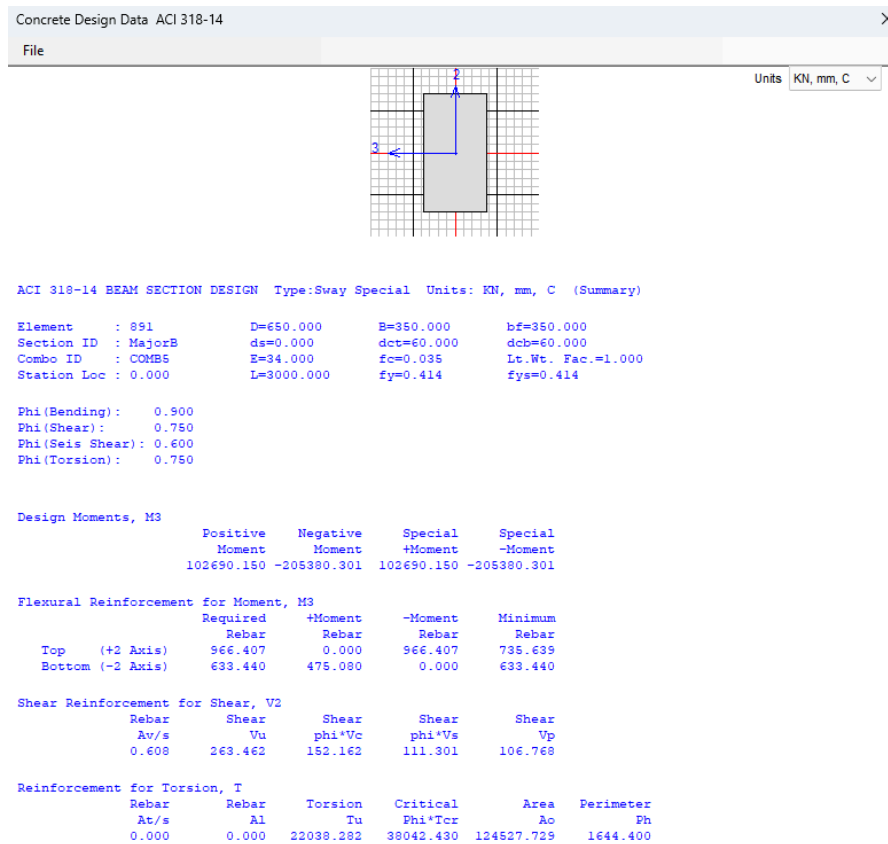
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	31623.866	-63247.733	31623.866	-63247.733

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	528.157	0.000	528.157	242.244
Bottom (-2 Axis)	256.547	256.547	0.000	242.244

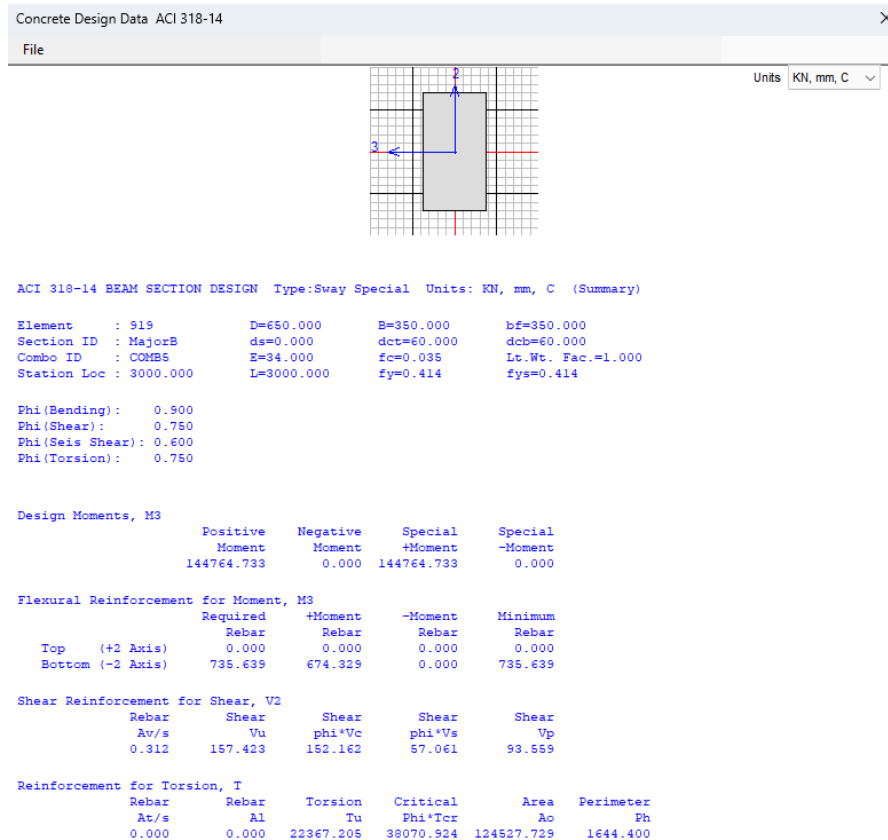
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.371	89.259	50.106	39.153

Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	116.688	7900.662

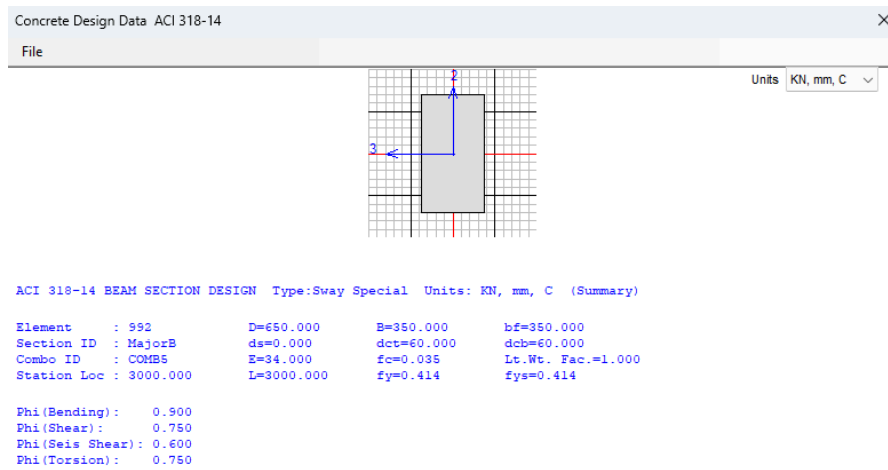
Right end critical moment for Minor Beam 4th floor



Left end critical moment for Major Beam 3rd floor



Mid critical moment for Major Beam 3rd floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	193321.870	-386643.739	193321.870	-386643.739

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1879.011	0.000	1879.011	735.639
Bottom (-2 Axis)	907.816	907.816	0.000	735.639

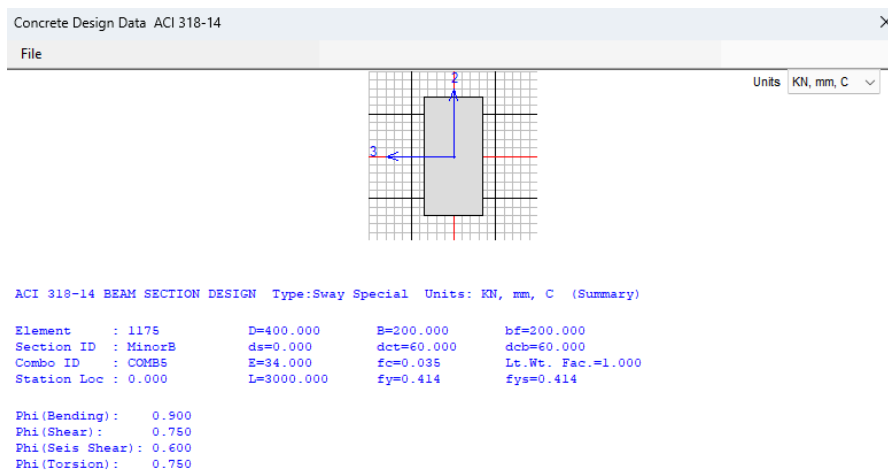
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	1.830	335.073	0.000	335.073	178.267

Reinforcement for Torsion, T

Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*Tr	Area Ao	Perimeter Ph
0.000	0.000	437.174	38165.038	124527.729	1644.400

Right end critical moment for Major Beam 3rd floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	25122.451	-50244.903	25122.451	-50244.903

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	414.483	0.000	414.483	242.244
Bottom (-2 Axis)	242.244	202.658	0.000	242.244

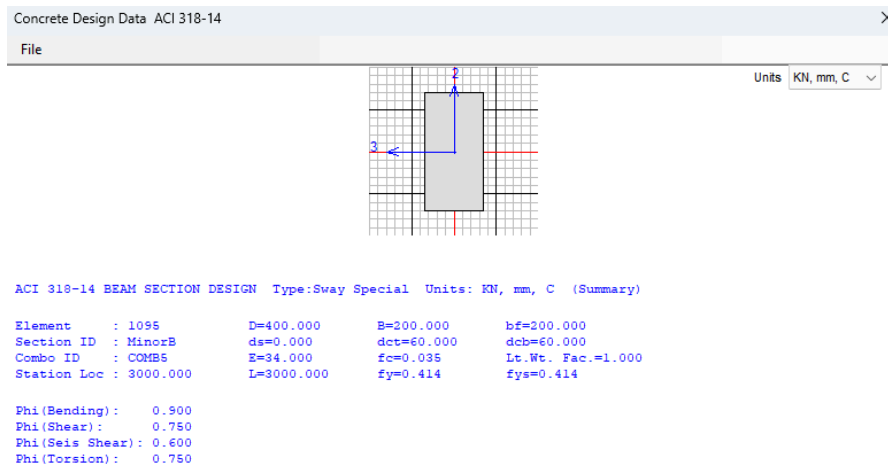
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.613	114.794	50.106	64.687	50.572

Reinforcement for Torsion, T

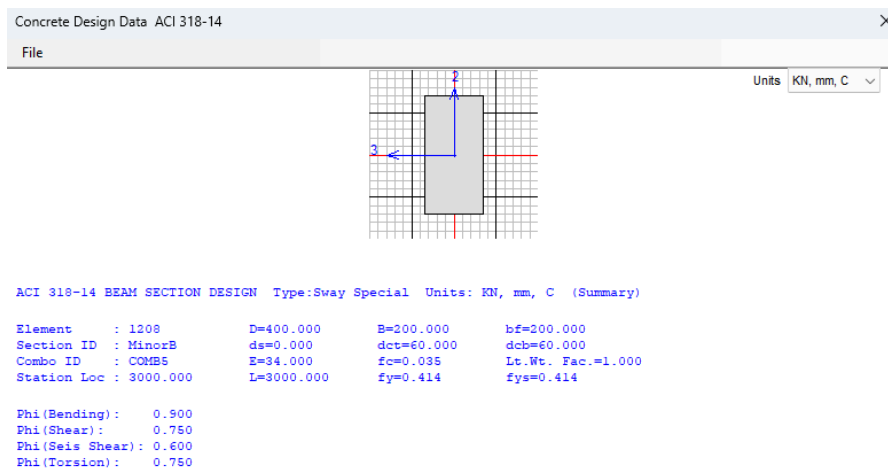
Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*Tr	Area Ao	Perimeter Ph
0.000	0.000	20.648	7871.603	29378.729	844.400

Left end critical moment for Minor Beam 3rd floor



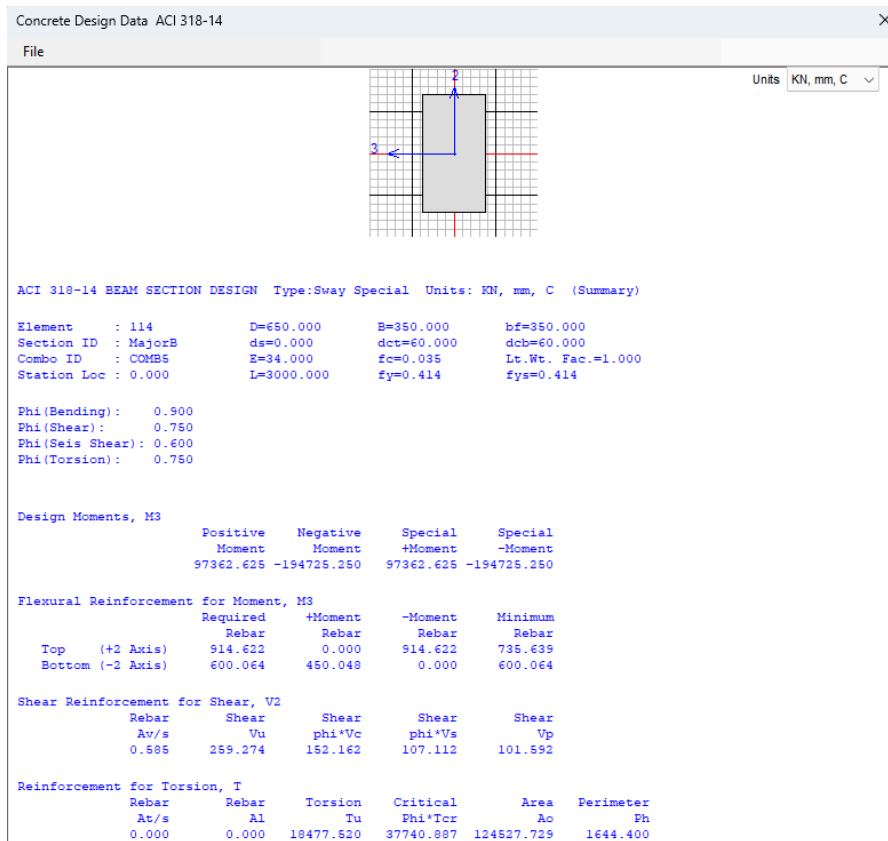
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	22526.573	0.000	22526.573	0.000
Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	0.000	0.000	0.000	0.000
Bottom (-2 Axis)	241.751	181.314	0.000	241.751
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.032	53.481	50.106	3.374
Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	951.483	7858.491
				Area Ao
				29378.729
				Perimeter Ph
				844.400

Mid critical moment for Minor Beam 3rd floor

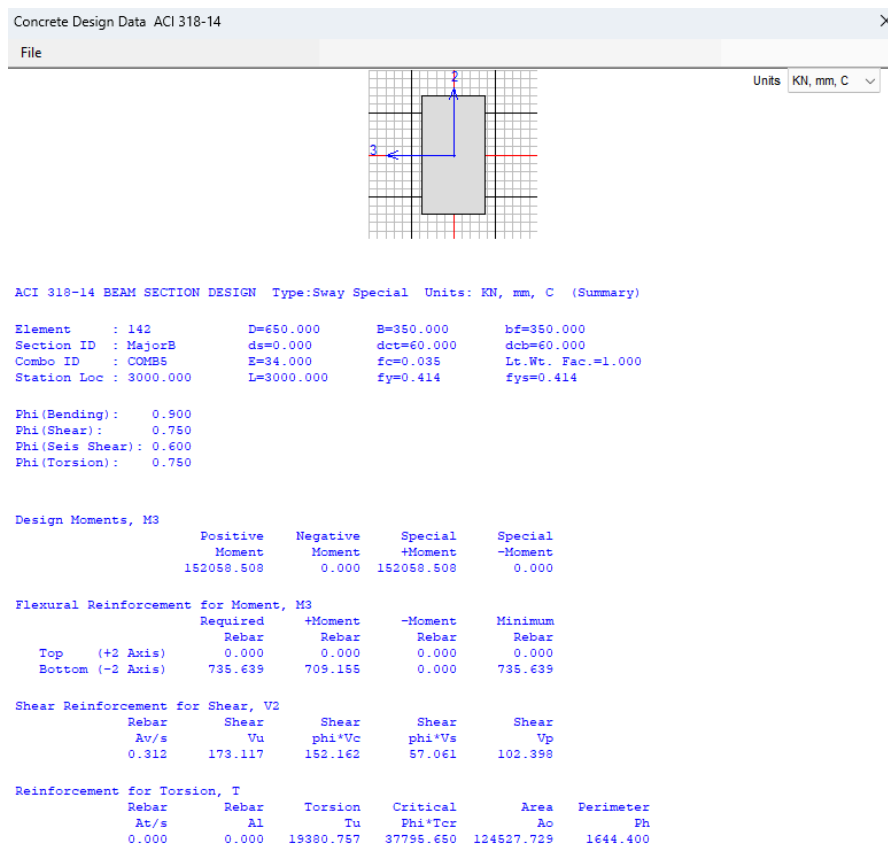


Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	39874.978	-75749.956	39874.978	-75749.956
Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	676.837	0.000	676.837	242.244
Bottom (-2 Axis)	325.855	325.855	0.000	242.244
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.614	114.904	50.106	64.797
Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr
	0.000	0.000	128.695	7872.706
				Area Ao
				29378.729
				Perimeter Ph
				844.400

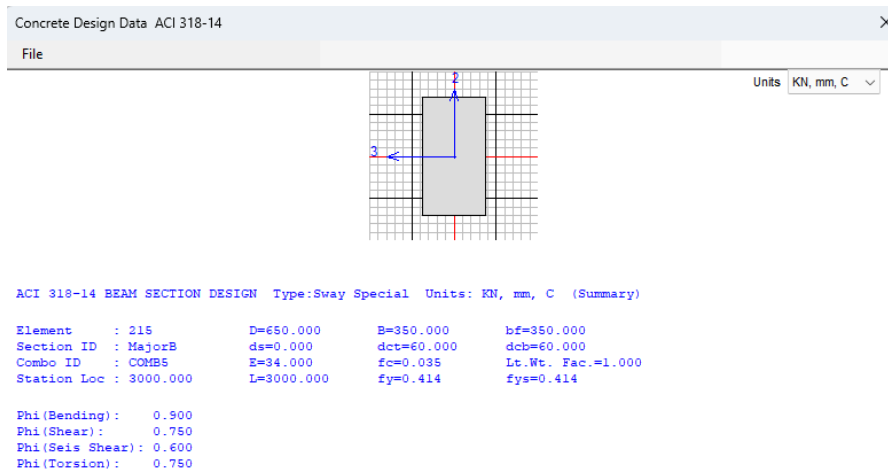
Right end critical moment for Minor Beam 3rd floor



Left end critical moment for Major Beam 1st floor



Mid critical moment for Major Beam 1st floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	174687.885	-349375.771	174687.885	-349375.771

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1696.211	0.000	1696.211	735.639
Bottom (-2 Axis)	817.755	817.755	0.000	735.639

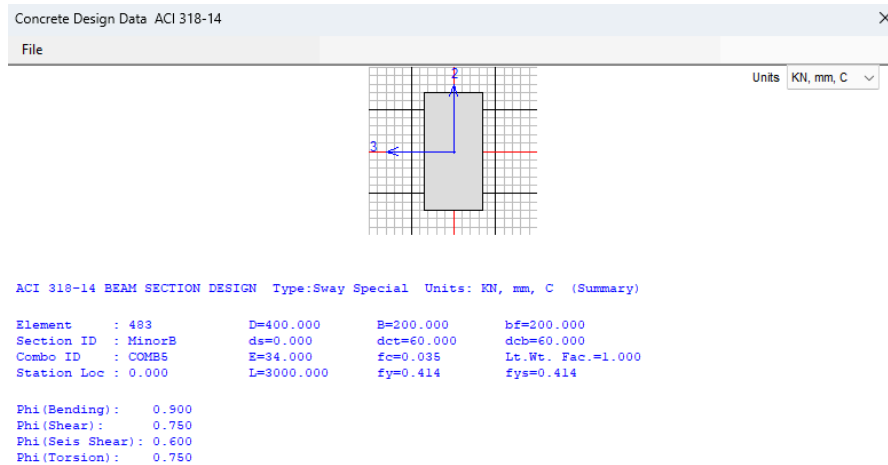
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	1.743	319.157	0.000	319.157	161.444

Reinforcement for Torsion, T

	Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*TCr	Area Ao	Perimeter Ph
	0.000	0.000	1195.202	38007.314	124527.729	1644.400

Right end critical moment for Major Beam 1st floor



Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	27923.215	-55846.431	27923.215	-55846.431

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	463.095	0.000	463.095	242.244
Bottom (-2 Axis)	242.244	225.797	0.000	242.244

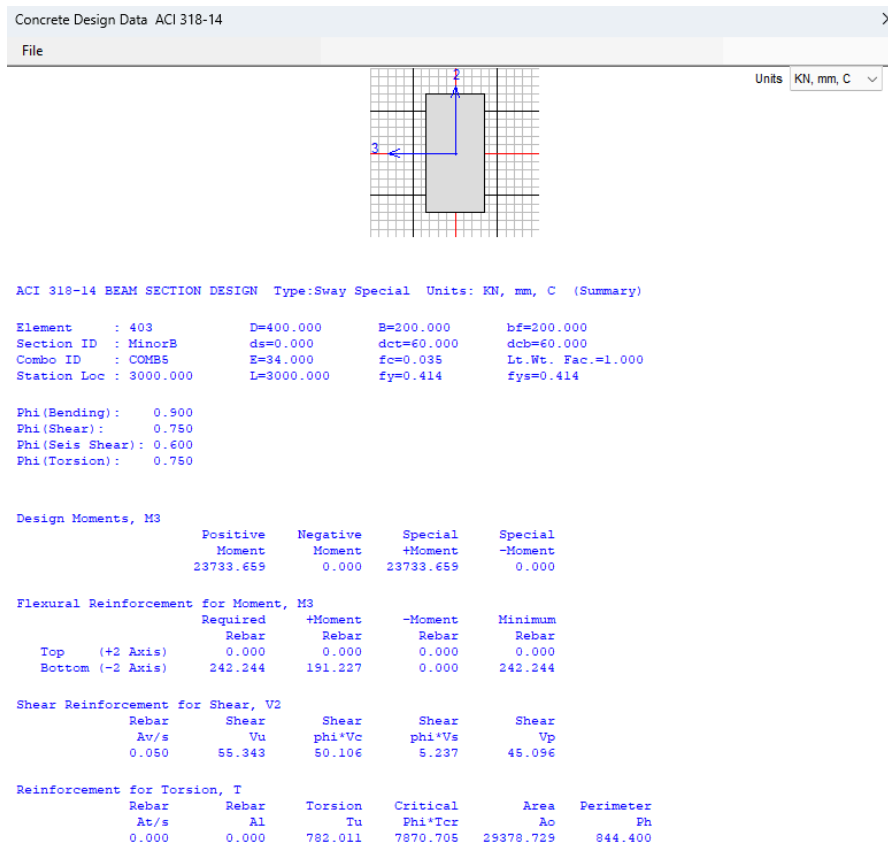
Shear Reinforcement for Shear, V2

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.646	118.271	50.106	68.164	50.911

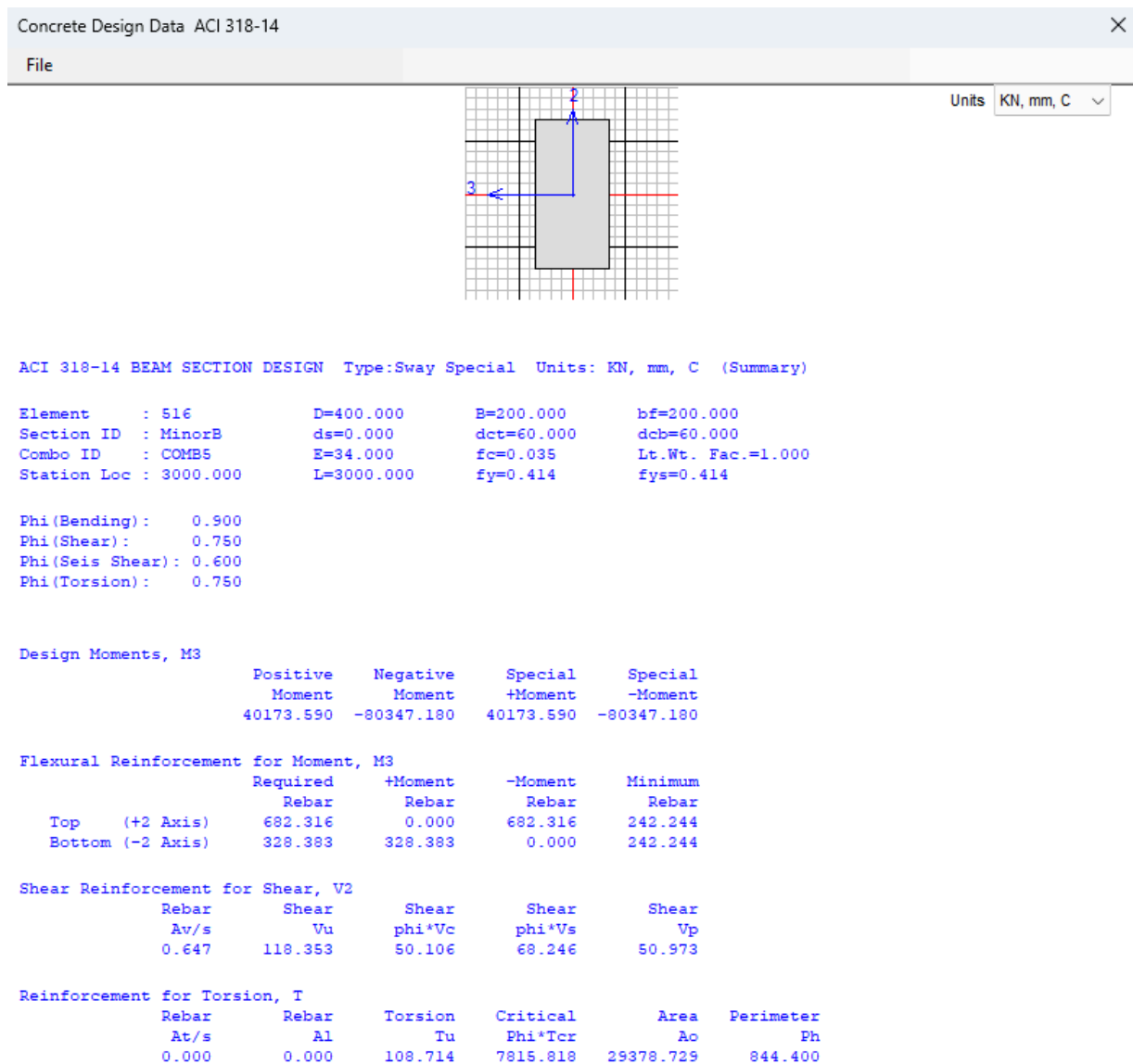
Reinforcement for Torsion, T

	Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*TCr	Area Ao	Perimeter Ph
	0.000	0.000	17.101	7804.183	29378.729	844.400

Left end critical moment for Minor Beam 1st floor



Mid critical moment for Minor Beam 1st floor



Right end critical moment for Minor Beam 1st floor