NAZARBAYEV
UNIVERSITY

SCHOOL OF ENGINEERING AND DIGITAL SCIENCES
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING
ENG 400: CAPSTONE II

DESIGN OF A 12-STORY HIGH-RISE RESIDENTIAL BUILDING IN SAN FRANCISCO, CALIFORNIA, USA

Capstone Project II

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Declaration

We declare that this report, titled "Design of a 12-story High-rise Residential Building in San Francisco, California, USA" is the product of our own project work, with all quotations and citations appropriately referenced. We further confirm that it has not been submitted previously or simultaneously for any other degree at Nazarbayev University.

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Acknowledgements

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Additionally, we extend our heartfelt gratitude to the entire faculty of the Civil Engineering and Environmental Department for their unwavering support during our coursework across all semesters and throughout the capstone project. Their expertise, mentorship, and commitment to excellence have been a source of inspiration, and we are profoundly grateful for the knowledge and skills they have imparted to us.

Abstract

This paper presents the design and construction proposal for a 12-story residential building located at 1335 6th St, San Francisco, California, USA. The location offers a blend of residential, commercial, and mixed-use developments. Urbanization and a shortage in housing within the city have accelerated the need for innovative residential construction solutions, necessitating a comprehensive approach that incorporates architectural, structural, material, and geotechnical considerations.

The building's architectural design complies with the standards outlined in the International Building Code (IBC). The structural design follows the guidelines of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE), with SAP 2000 software employed to perform structural analyses and simulate the building's response to gravity and lateral forces.

Reinforced concrete has been chosen as the primary construction material due to its availability, versatility, and cost-efficiency. A comparative analysis of foundation options, supported by geotechnical studies, identified a pile foundation as the most suitable solution for supporting the superstructure.

The stormwater management design includes sustainable drainage through the use of permeable surfaces and green infrastructure to reduce runoff and risks of flooding. Full construction management strategies include estimating costs, scheduling, risk assessment, and follow-through with LEED standard to provide a safe, environmentally responsible execution process. This development will realise an integrated approach to urban residential construction through innovative design, sustainability, following international and local building codes.

Keywords: architectural design, structural design, geotechnical analysis, stormwater water design, construction management.

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1. Introduction

This paper presents the design and construction proposal for a 12-story residential building located in San Francisco, California, USA. This report presents the findings of a comprehensive literature review, the design assumptions, and the calculations for the proposed building. It encompasses the results and calculations for the architectural, structural, geotechnical, and environmental aspects, as well as the construction management details associated with the building design. Table 1.1 provides an overview of the project disciplines, which include structural, architectural, geotechnical, environmental, and construction management components.

Table 1.1. Job distribution

Part	Weight Percentage	Responsible person
Structural and Architectural	40%	Aliyar Kadyrov Ramir Gumarov Aldiyar Sagidolla
Geotechnical	35%	Ayaulym Baktiyarkyzy Zhaniya Sagyndykova
Environmental	15%	Bek Yergeldinov
Construction Management	10%	Arsen Yelubayev

To fulfill the project requirements, the International Building Code, ASCE-7, and ACI 318-19 will be adhered to for the architectural and structural components. The structural analysis of the building will evaluate gravity and lateral loads, including dead, live, snow, wind, and seismic loads. Additionally, the residential building is designed to meet LEED (Leadership in Energy and Environmental Design) standards for sustainable design. The geotechnical analysis will assess various foundation types to identify the most suitable option for a 12-story residential structure. In the construction management section, the project scope will be defined, and a detailed schedule will be developed, while the environmental component will perform the stormwater design.

This project aims to create a modern, high-performance residential building that prioritizes safety, sustainability, and functionality. The report will provide a comprehensive analysis to ensure the building is structurally sound, environmentally responsible, and efficiently managed throughout its lifecycle. The main objectives include:

- To conduct a thorough study to assess the feasibility of the proposed development;
- To develop an architectural design that complies with IBC standards, balancing aesthetic appeal and functionality;
- To evaluate geotechnical data to determine the most suitable foundation and basement design;
- To perform the stormwater design for the building.
- To formulate a construction management plan for the building project.
- To adhere to LEED standards to ensure a sustainable design.

1.1 Ethical Considerations in Report Preparation

The ethical preparation of a report involves committing to principles like accuracy, transparency, and accountability. Presented information must be valid and appropriately cited. The report's integrity can be undermined by misrepresentation and plagiarism, along with omission of critical details. Well prepared report should present findings in an unbiased way, which reflects the restraints and limitations of the project.

Confidentiality is also a part of ethical reporting. Sensitive or proprietary information must be treated with care and only used for its purpose. As an example, some engineering designs or feedback from locals should not be disclosed without consent. Respecting privacy and ensuring that all data usage complies with ethical and legal standards creates trust between the stakeholders

Language and structure also play a significant ethical role in reporting. The use of clear and inclusive language enables the audience to comprehend the findings. Maintaining technical accuracy while simplifying complex concepts makes it easier for all the parties involved to understand. Some of the parties might include regulatory agencies, professionals, and members of the public. In our case, it is usually the professors of the Civil and Environmental Engineering department, that is why mostly the report consists of technical language that is heavily loaded with engineering terms and concepts. However that is a justified shortcoming when it comes to the language of the report.

Construction of the proposed 12-story high-rise residential building in San Francisco, California, involves meeting ethical and professional responsibilities. San Francisco expects a construction project to ensure public safety and environmental awareness

Ethical Considerations

It is vital to decrease the environmental impact of San Francisco. Environmental sustainability should be ensured, with the use of eco-friendly or local building materials. Meeting the California Green Building Standards would mean the project is in line with all state environmental goals in regard to the sustainable design. Due to the seismic hazard in San Francisco, structural resilience should be ensured. The project is supposed to comply with some strict seismic safety provisions as set by the IBC and the San Francisco Building Code. Notable ethical obligations include offering of detailed geotechnical studies, formulation of sophisticated engineering techniques which can ensure safety during seismic events. A great involvement of people within the community is required for the project. Communicating with the community in a transparent way, where all the foreseen nuisance caused by noise, jams, and construction schedule invokes trust and reduces the worries of people. Pro-activities in informing and involving residents help to keep good relations with the neighborhood. The other essential ethical responsibility relates to worker safety. OSHA standards need to be met by all construction teams. They also should be prepared for emergency situations, especially in a high seismic environment.

Professional Considerations

Professional responsibilities within this project will be done with strict adherence to all zoning laws, environmental regulations, and building codes of the city of San Francisco. Among others, these requirements involve keeping the building within the height restrictions, integrating greenery into space, and making the facility energy-efficient, meeting the city's urban and environmental planning objectives. ISO standards, seismic safety requirements, and certified quality materials shall be availed. Ongoing inspections, while the process of building is going on, should be done in correspondence with local authorities, aiming at avoiding non-conformities and finding out about problems as early as possible.

One of the key aims of the project will be sustainability. This includes creating a stormwater design system that requires minimum energy by following the natural terrain of the site. Additionally, a membrane filtration system under the building is also considered at this point, but it is not clear how feasible that is going to be in the next semester.

2. Architectural Design

2.1 Overview

The construction project is a 12-story high-rise residential building with the first story being rented out for commercial use and the remaining 11 stories are residential. The address of the construction project is 1335 6th St, San Francisco, California, USA. The residential building is to be located in the Mission Bay area of San Francisco near parks and transportation lines. The residential complex will offer 110 places to reside as well as 8 commercial units for renting. The site of the high-rise building will also include a spacious parking and playground for children. Figure 1 represents the rendered site 3D view.

Figure 2.1.1. Rendered Site 3D View

2.2 Site selection and analysis

The site at 1335 6th St, San Francisco, CA 94158, USA is located in the Mission Bay neighborhood that offers a blend of residential, commercial, and mixed-use developments. The local climate is characterized by a temperate maritime pattern, with mild, wet winters and cool, dry summers. Average temperatures range from 8°C (46°F) in the coldest months to 20°C (68°F) in the warmest. Below the graph of average maximum and average minimum temperature by each month for 2022 and 2023 years in San Francisco:

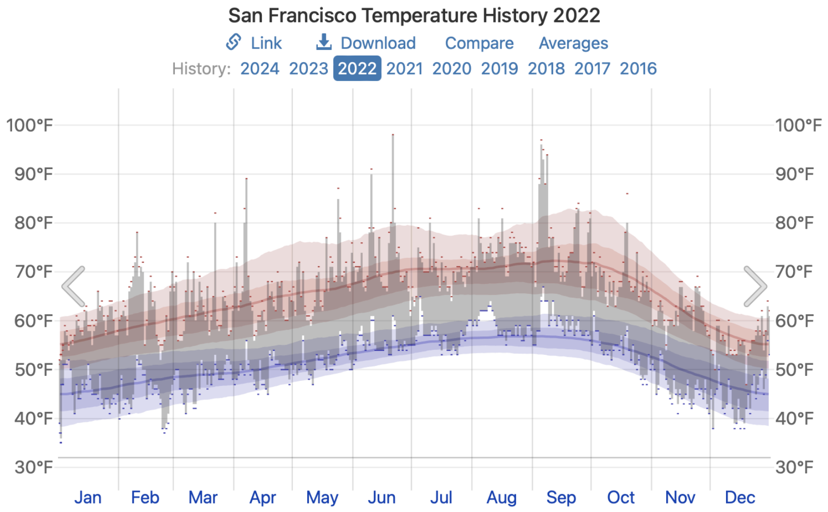


Figure 2.2.2. San Francisco temperature history in 2022 (WeatherSpark)

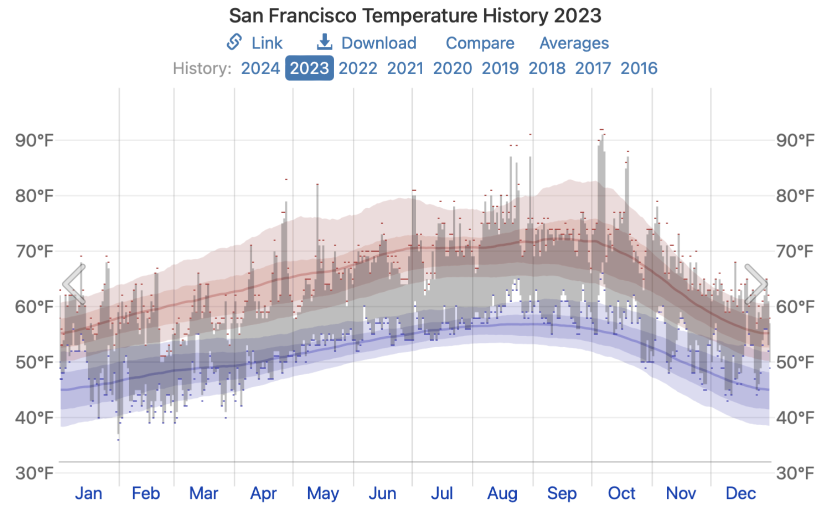
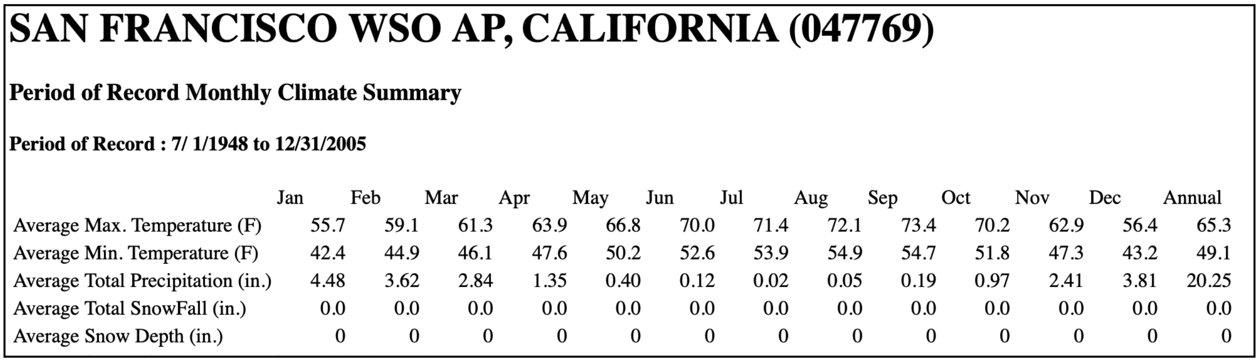


Figure 2.2.3. San Francisco temperature history in 2023 (WeatherSpark)

The city also experiences relatively high annual humidity levels of about 75%, impacting indoor comfort during the cooler months. Annual rainfall according to the West Region Climate

Center averages around 20 inches, primarily between November and March, while snowfall is virtually nonexistent. Fog is common in summer, affecting visibility and outdoor comfort.



SAN FRANCISCO WSO AP, CALIFORNIA (047769)

Period of Record Monthly Climate Summary

Period of Record : 7/ 1/1948 to 12/31/2005

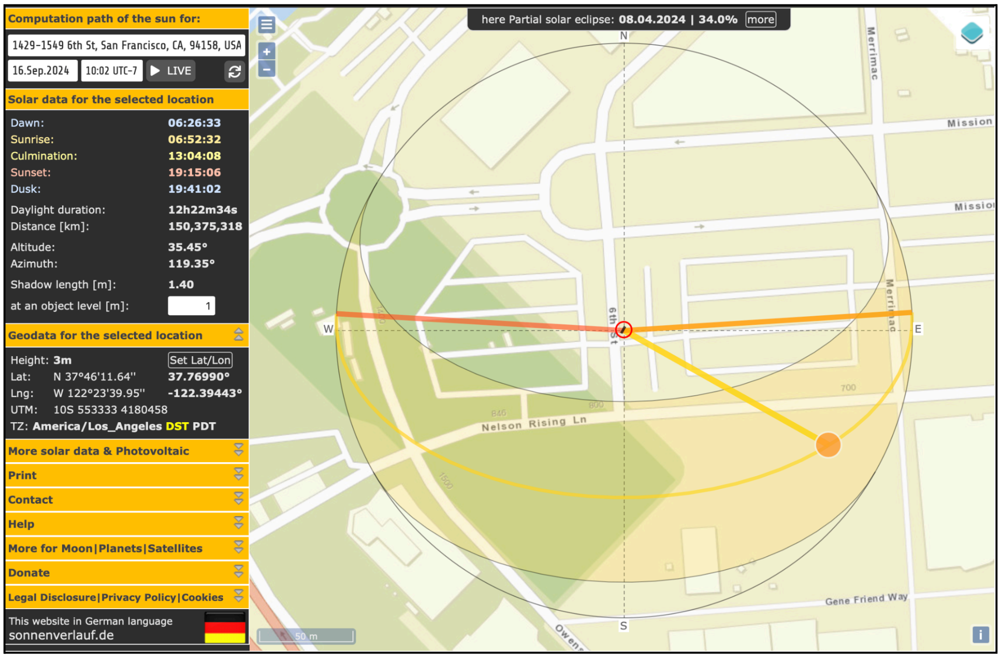
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	55.7	59.1	61.3	63.9	66.8	70.0	71.4	72.1	73.4	70.2	62.9	56.4	65.3
Average Min. Temperature (F)	42.4	44.9	46.1	47.6	50.2	52.6	53.9	54.9	54.7	51.8	47.3	43.2	49.1
Average Total Precipitation (in.)	4.48	3.62	2.84	1.35	0.40	0.12	0.02	0.05	0.19	0.97	2.41	3.81	20.25
Average Total SnowFall (in.)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Figure 2.2.4. San Francisco monthly climate change

In the Mission Bay area, where the lack of tall buildings allows for unobstructed sunlight and strong coastal breezes, our building design emphasizes sustainability and energy efficiency through climate-responsive strategies. Selecting a material with a high thermal mass, such as concrete, plays a critical role in stabilizing indoor temperatures. Concrete's ability to absorb heat during the day and slowly release it at night helps reduce the need for artificial heating and cooling, making it an ideal choice for the local climate. The building's facades will be strategically oriented to optimize solar gain, with recessed windows and integrated shading elements on the south and west sides to mitigate excessive heating during peak solar hours. These shades will not only prevent overheating in the summer, but will also allow low-angle sunlight to warm the interiors in cooler months, improving energy efficiency. Natural ventilation will also be a key feature, using operable windows and strategically placed vents to take advantage of prevailing winds from the west. The design will include rooftop wind catchers that channel cool breezes into the building, promoting cross-ventilation and reducing reliance on mechanical cooling systems. This approach will improve indoor air quality and create a comfortable living environment that naturally responds to outdoor conditions. Additionally, green roofing systems will be used to improve insulation, reduce the urban heat island effect, and manage stormwater runoff. Not only will these green spaces provide aesthetic value, they will also contribute to the overall thermal performance of the building, helping to keep spaces cool during the warmer months. The design approach is not only about sustainable construction, but also about creating spaces that work harmoniously with the unique microclimate of Mission Bay.

Historically, Mission Bay, where the site is located, has undergone significant transformation from a former industrial and rail yard area into a modern, mixed-use neighborhood. The site itself does not possess notable historic structures, as much of the area has been developed or repurposed in the last two decades to accommodate new commercial, residential, and research facilities, particularly associated with the nearby UCSF Medical Center (San Francisco General Plan, 2023).

The site is oriented approximately south, with primary facades facing south, and big windows on each side. This orientation results in substantial morning and afternoon sun exposure, necessitating shading solutions on the west facade to reduce heat gain in the afternoon. Optimizing window placement, using shading devices, and employing reflective surfaces can enhance energy efficiency and comfort while maximizing natural light and minimizing glare. San Francisco experiences the longest days, with an average of about **14 hours and 46 minutes** of daylight. (Suncalc.org, 2023)



Computation path of the sun for:
1429-1549 6th St, San Francisco, CA, 94158, USA
16.Sep.2024 10:02 UTC-7

Solar data for the selected location
Dawn: 06:26:33
Sunrise: 06:52:32
Culmination: 13:04:08
Sunset: 19:15:06
Dusk: 19:41:02
Daylight duration: 12h22m34s
Distance [km]: 150,375,318
Altitude: 35.45°
Azimuth: 119.35°
Shadow length [m]: 1.40
at an object level [m]: 1

Geodata for the selected location
Height: 3m
Lat: N 37°46'11.64'' 37.76990°
Lng: W 122°23'39.95'' -122.39443°
UTM: 10S 553333 4180458
TZ: America/Los_Angeles DST PDT

here Partial solar eclipse: 08.04.2024 | 34.0%

This website in German language
sonnenverlauf.de

Figure 2.2.5. Sun direction

Solar access is an important consideration for this site. Due to its orientation, the building receives sample sunlight from the east in the morning and from the west in the afternoon. Incorporating design elements that optimize solar access, such as strategic window placement and shading devices, will help maximize natural light and minimize energy consumption for lighting and heating.

Cool breezes predominantly come from the west-southwest, driven by ocean air funneled through the Golden Gate. These winds are strongest in the afternoons, particularly in summer, with average speeds of 15-20 km/h, occasionally reaching 30-40 km/h. The open layout of Mission Bay, with fewer high-rise buildings compared to downtown, allows better access to these breezes. The design should incorporate operable windows, wind catchers, and strategically placed openings to enhance cross-ventilation, reducing reliance on mechanical cooling systems (San Francisco Climate Data, USGS, 2023).

Figure 2.2.6. The wind direction on the site

The site offers potential views of the San Francisco Bay, nearby parks, and the city skyline. To maximize these views, large windows and terraces should be positioned to face east toward the Bay and west towards the downtown skyline. Thoughtful placement of these elements can enhance the appeal of residential or commercial spaces (San Francisco Planning Department, 2023).

Figure 2.2.7. View to downtown area from the site (Google earth)

A site survey reveals that despite the fact that San Francisco mostly has the areas with 15-20% slopes, the site is relatively flat, with a slope of between 0-2%, minimizing challenges related to grading and foundation construction (USGS). The site is not overshadowed by nearby buildings, there are no nearby high rise buildings, which is good for natural light from the outside. The soil type for the site, as determined by a geotechnical report, consists of compacted urban fill overlying native soil, which is typical for downtown San Francisco and suitable for high-rise construction.

Figure 2.2.8. Site shadowing view

Above picture is the neighborhood of the selected site. We can see that nearby buildings are not so tall and there are no problems with the overshadowing.

The site at **1335 6th St, San Francisco, CA 94158** is well-connected to various modes of transportation, making it highly accessible. It is located near the T Third Street Muni Metro line, which provides convenient access to the city's broader Muni network and connects directly to major hubs such as Downtown and Union Square. Additionally, several bus routes serve the area, offering further connectivity across San Francisco. The nearby Caltrain station provides commuter rail service to the Peninsula and South Bay, enhancing regional access. The site is also pedestrian-friendly, with well-maintained sidewalks, and features dedicated bicycle lanes, promoting sustainable transportation options (San Francisco Municipal Transportation Agency, 2023).

Below the map of public transportation routes going through our site:

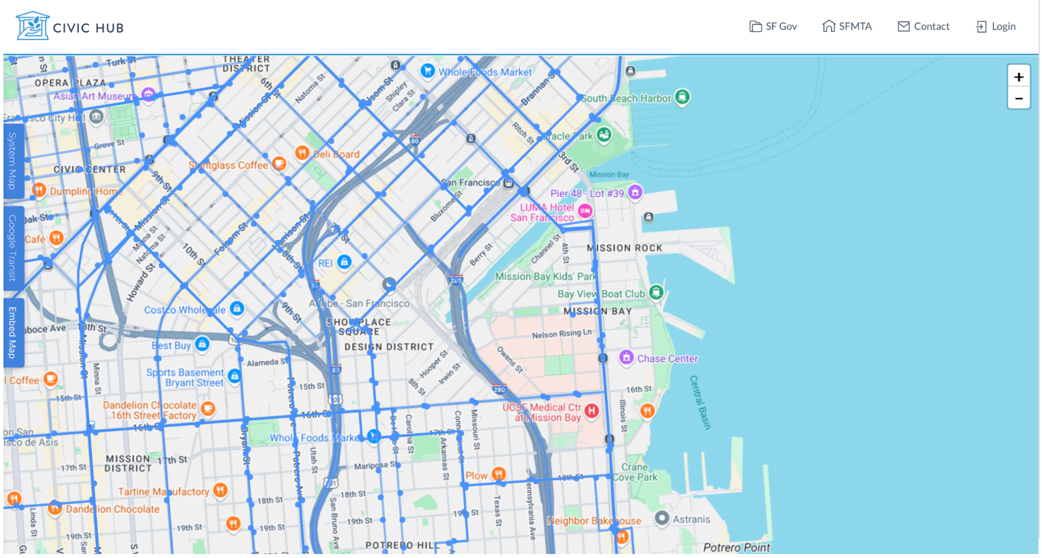


Figure 2.2.9. Public transportation map

Below the San Francisco Transit Map, the public transportation routes of the whole city and there we can see that the Mission Bay area has direct connection to the downtown of city which has access to almost everywhere:

Figure 2.2.10. The train transportation map of SF

The site is classified as a Non-Very High Fire Hazard Severity Zone (Non-VHFHSZ) by the California Department of Forestry and Fire Protection. This classification indicates a low risk of bushfires due to its urban location, far from dense vegetation or wildfire-prone areas. Despite this low risk, standard fire safety measures, such as using non-combustible materials and ensuring adequate emergency access, should still be incorporated into the design. (Cal Fire, 2023).

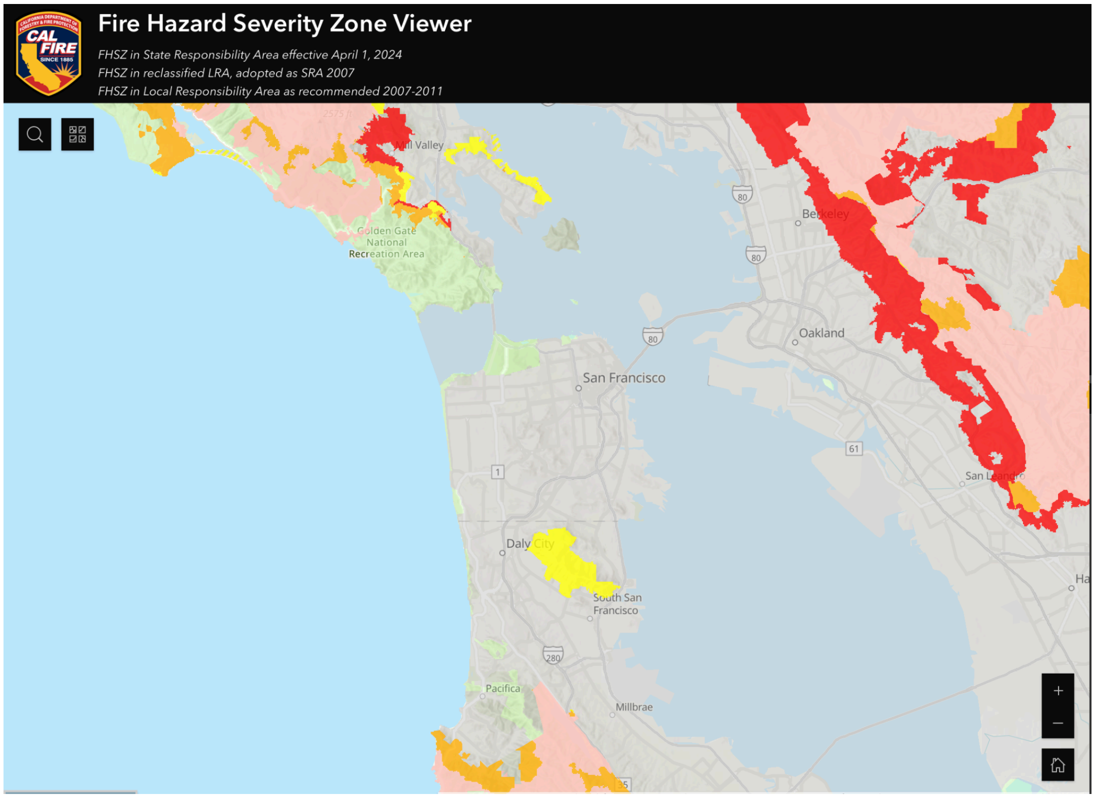
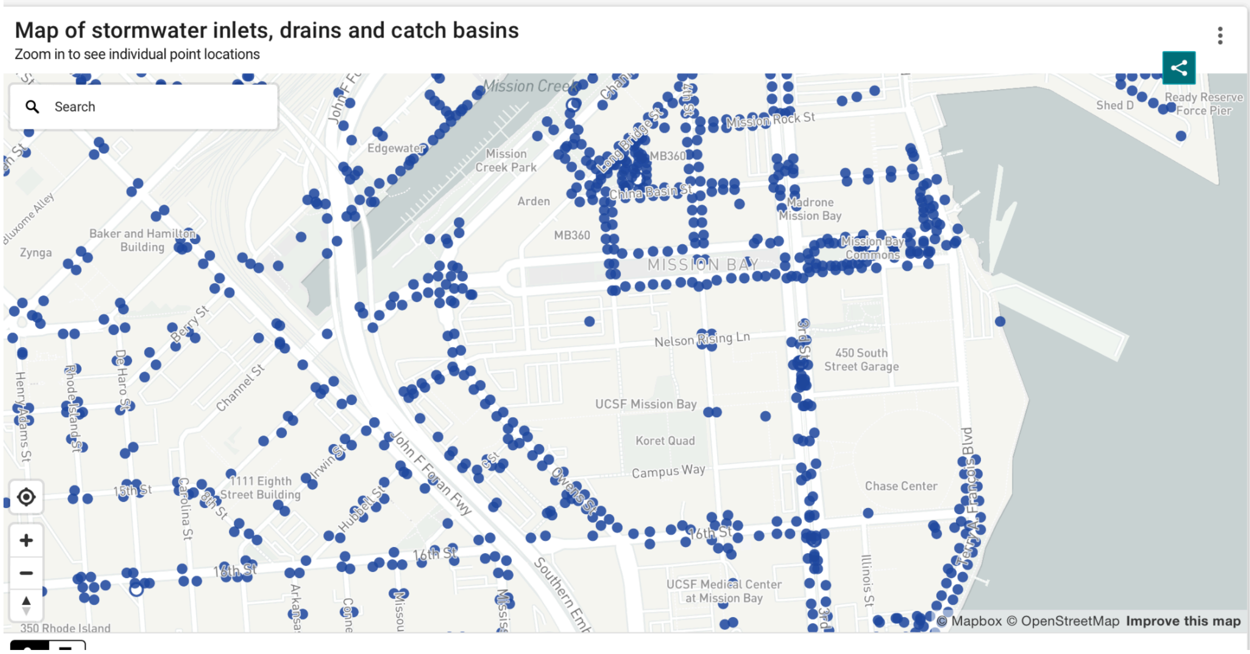


Figure 2.2.11. The Cal fire map

The site at **1335 6th St, San Francisco, CA 94158** benefits from modern stormwater management infrastructure designed to handle runoff effectively in the Mission Bay area. The neighborhood features a combination of permeable surfaces, bioswales, and green infrastructure that help manage stormwater. Given the relatively flat terrain and proximity to the Bay, development at this site should incorporate sustainable drainage solutions like permeable paving, green roofs, and rain gardens to reduce runoff and mitigate potential flooding risks, especially during heavy rain events (San Francisco Public Utilities Commission, 2023).



Map of stormwater inlets, drains and catch basins

Zoom in to see individual point locations

Figure 2.2.12. Map of inlets in Mission Bay area

Map of stormwater inlets, drains and catch basins

Zoom in to see individual point locations

Figure 2.2.13. Drop inlet in selected site

The site is well-serviced by the city's comprehensive utility network. **Power and gas** are supplied by Pacific Gas and Electric (PG&E), ensuring reliable energy access for any development. **Water** is provided by the San Francisco Public Utilities Commission (SFPUC), sourced from the Hetch Hetchy Reservoir and local water supplies, while the **sewer system** is integrated into the city's extensive wastewater infrastructure. The area also has robust **telecommunications services**, including high-speed internet and phone connectivity, supporting both residential and commercial use (San Francisco Public Utilities Commission and PG&E, 2023).

2.3 Design Concept

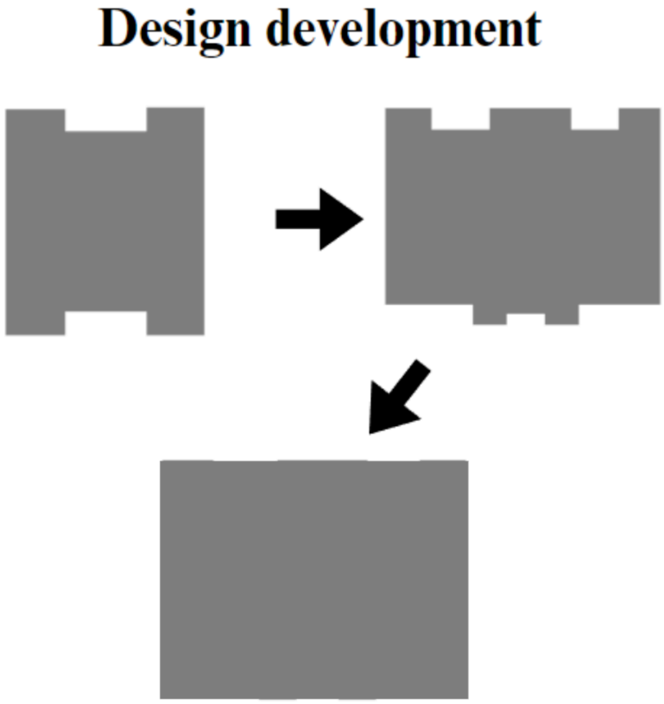


Figure 2.3.1 First and last versions of the shape of our building.

At the beginning, our team tried to design a building that was distinctive, visually engaging, and welcoming, intentionally steering away from common architectural styles. However, after thorough discussions, we prioritized functionality, selecting a building shape that is not only aesthetically pleasing but also reliable and easy to change the layout of apartments.

Additionally, we integrated balconies into each section to enhance appeal and usability. The current design, as illustrated in the figure, reflects these considerations.

Besides that, before choosing a location for our project, we surely wanted it to contain some features, such as: free transport access, prime location, enough area for kids playground and sport zone, as well as for spacious commercial zone.

2.4 Geometry design and site layout

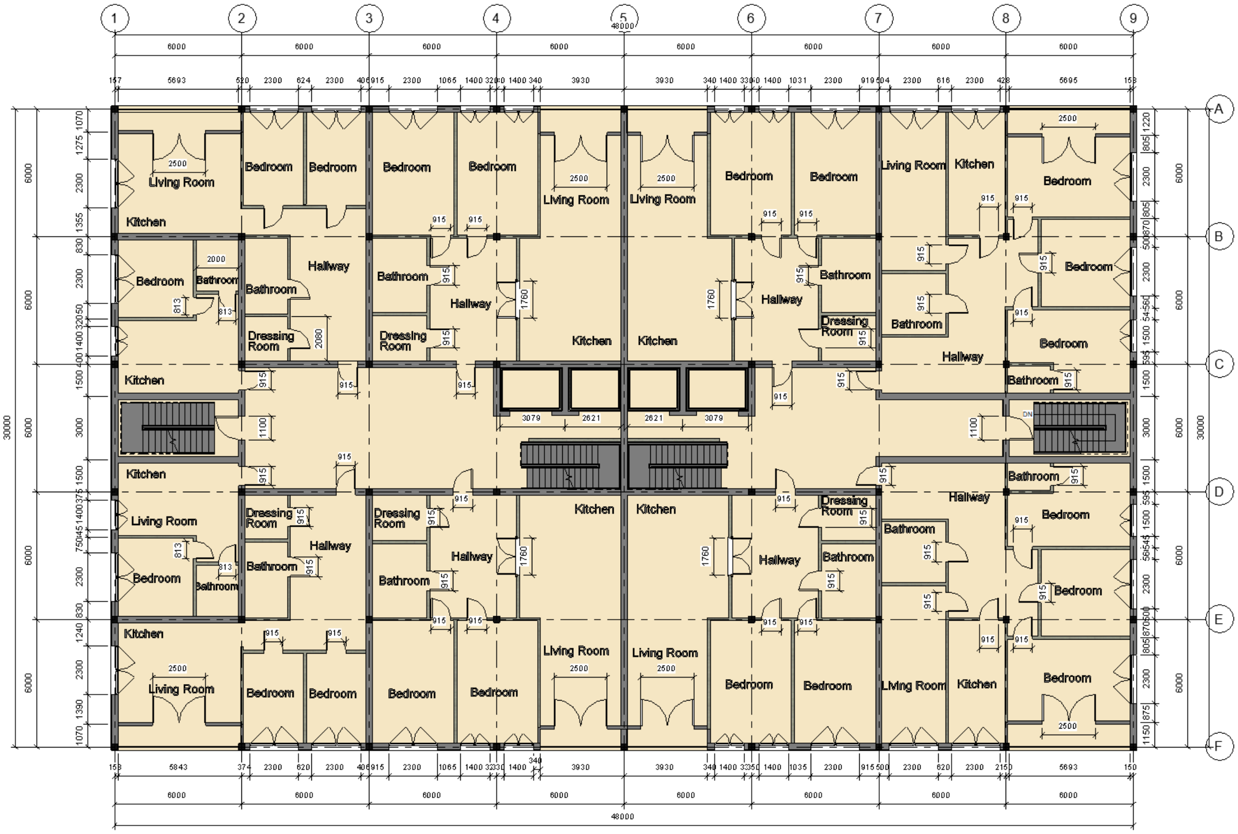


Figure 2.4.1. Floor plan for Levels 2-12.

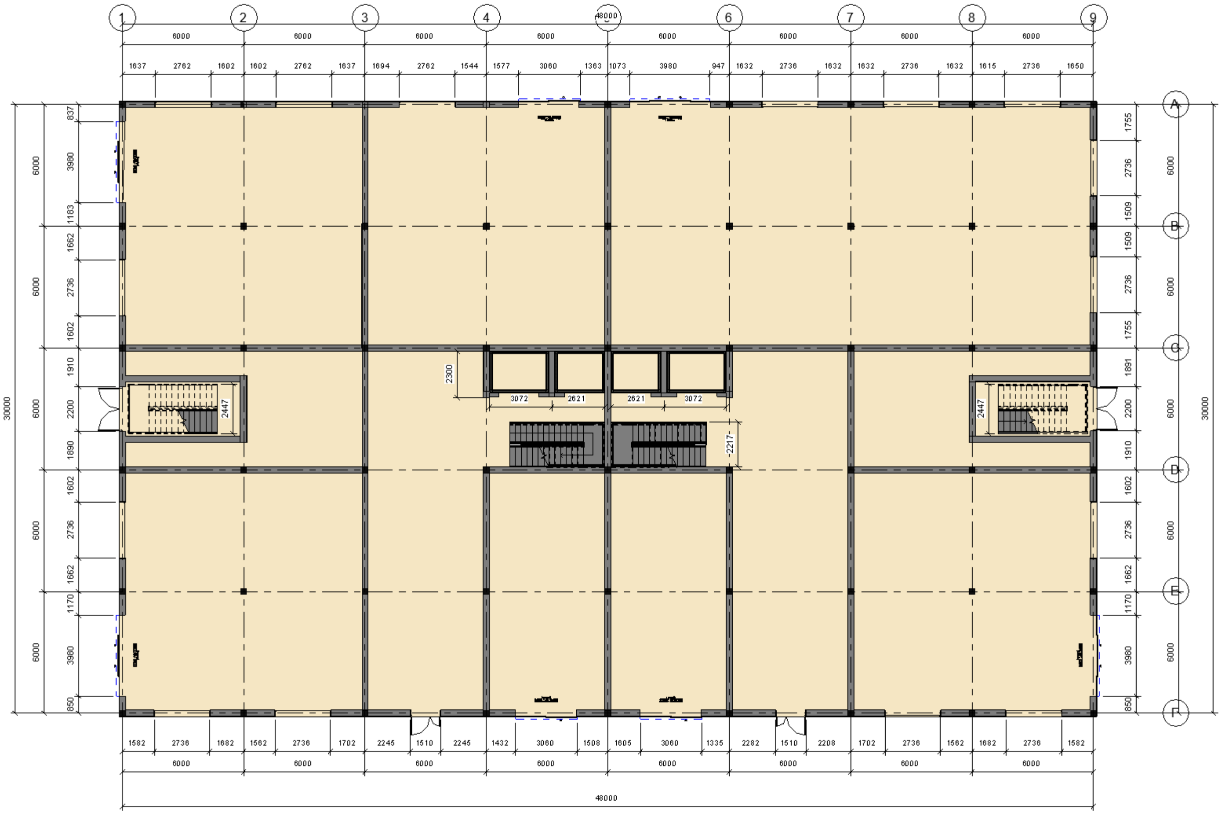


Figure-2.4.2. Floor plan for Level-1

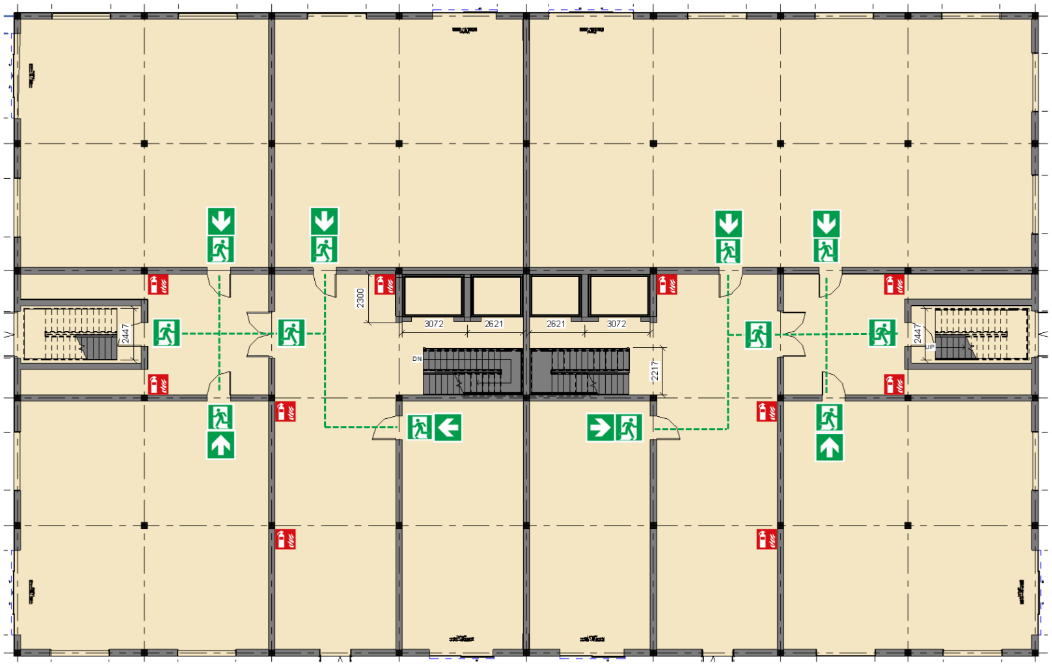


Figure 2.4.3. Emergency Plan, First Floor

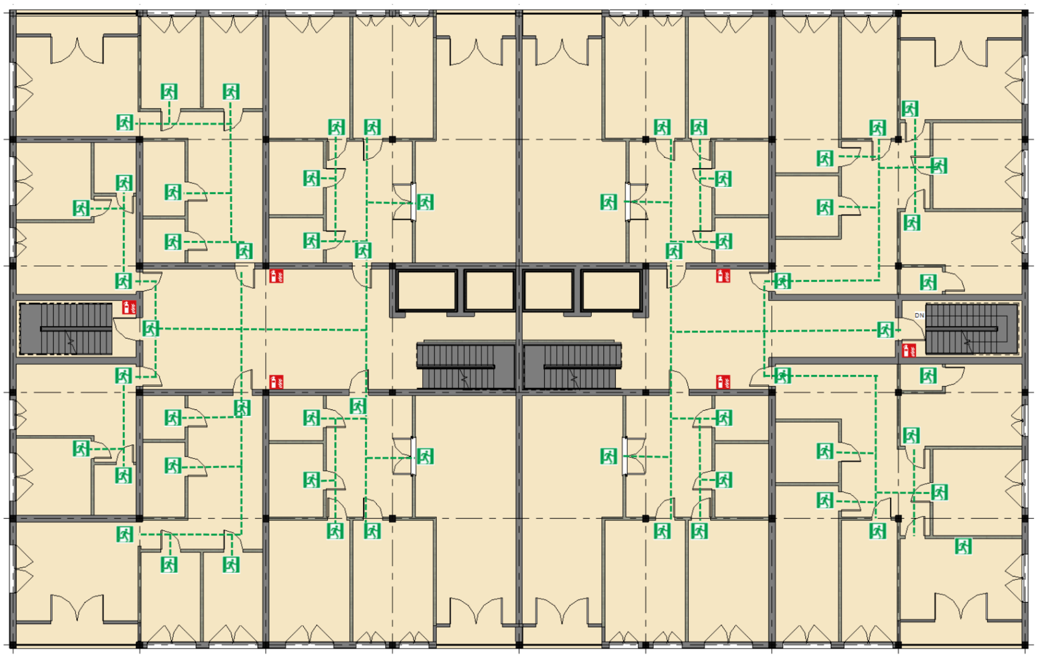


Figure 2.4.4. Emergency Plan, 2-12 Floors

In discussing the floor plan, it is important to acknowledge that we were provided with a substantial area to design, which presented both opportunities and challenges. One of the key considerations was how to maximize efficiency in the arrangement of apartments and their respective rooms. While the current iteration reflects a thoughtful approach, I anticipate that both minor and significant revisions may be made as the project progresses. Overall, our team is satisfied with the outcome, as the plan accommodates a diverse range of residents, including smaller units suitable for students or young families, as well as more spacious apartments designed for larger households.

In the end, we designed 10 flats with a range of area from 60 to 160 square meters approximately. They are wide enough to accommodate big households. The windows in our house are planned to be floor-to-ceiling, which will provide natural sunlight to save electricity, and will please the habitants with its aesthetics. Doors are standardly wide. The house will have 2 elevators, where one of them is a freight elevator. The stairs are wide enough for all the residents to safely leave the building in case of emergency.

According to the San Francisco Planning Code, there is no minimum required number of parking spaces for any type of facility since December 11, 2018 (American Legal Publishing, 2024). This was done in an attempt to reduce the number of individual transportation and decrease the level of dependency on cars, so that people would use more public transportation.

Despite that however, there is still a recommended **number of parking spaces** for residential building types. Thus, according to the San Francisco Building Code, 1 apartment unit requires up to 1.5 parking spaces, and the same is applied to the commercial areas, where at most 1.5 parking spaces can be provided for 1 commercial unit (American Legal Publishing, 2024). As was mentioned before, there will be 104 apartments and 7 commercial spaces in our high-rise building. Therefore, following the given expression:

$$N_{parking} = (104 + 7) * 1.5 = 166.5 \approx 167 \, parking\ spaces$$

Thus, the external parking lot will include 167 parking spaces for all the future residents and tenants of the commercial units. Figure 5 below depicts the 2D view of the site plan, demonstrating the parking area, the entry and exit ways, playground area and the building itself.

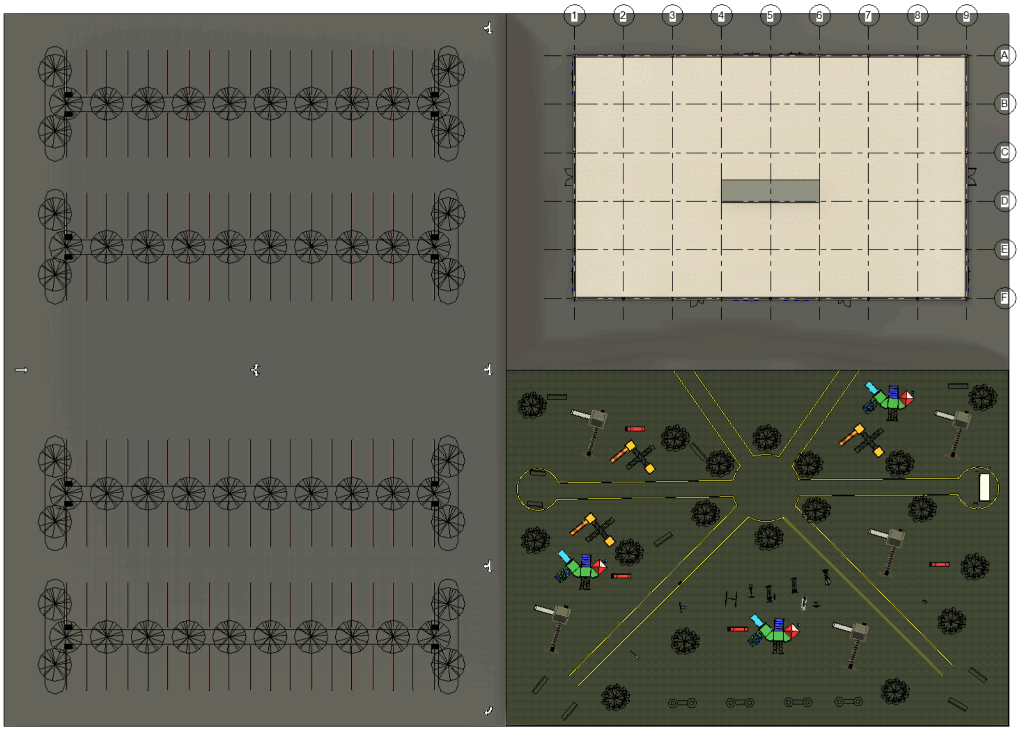


Figure 2.4.5. 2D Site Plan View

2.5 Leadership in Energy and Environmental Design (LEED) for Green Building Design, Construction, Operations and Performance

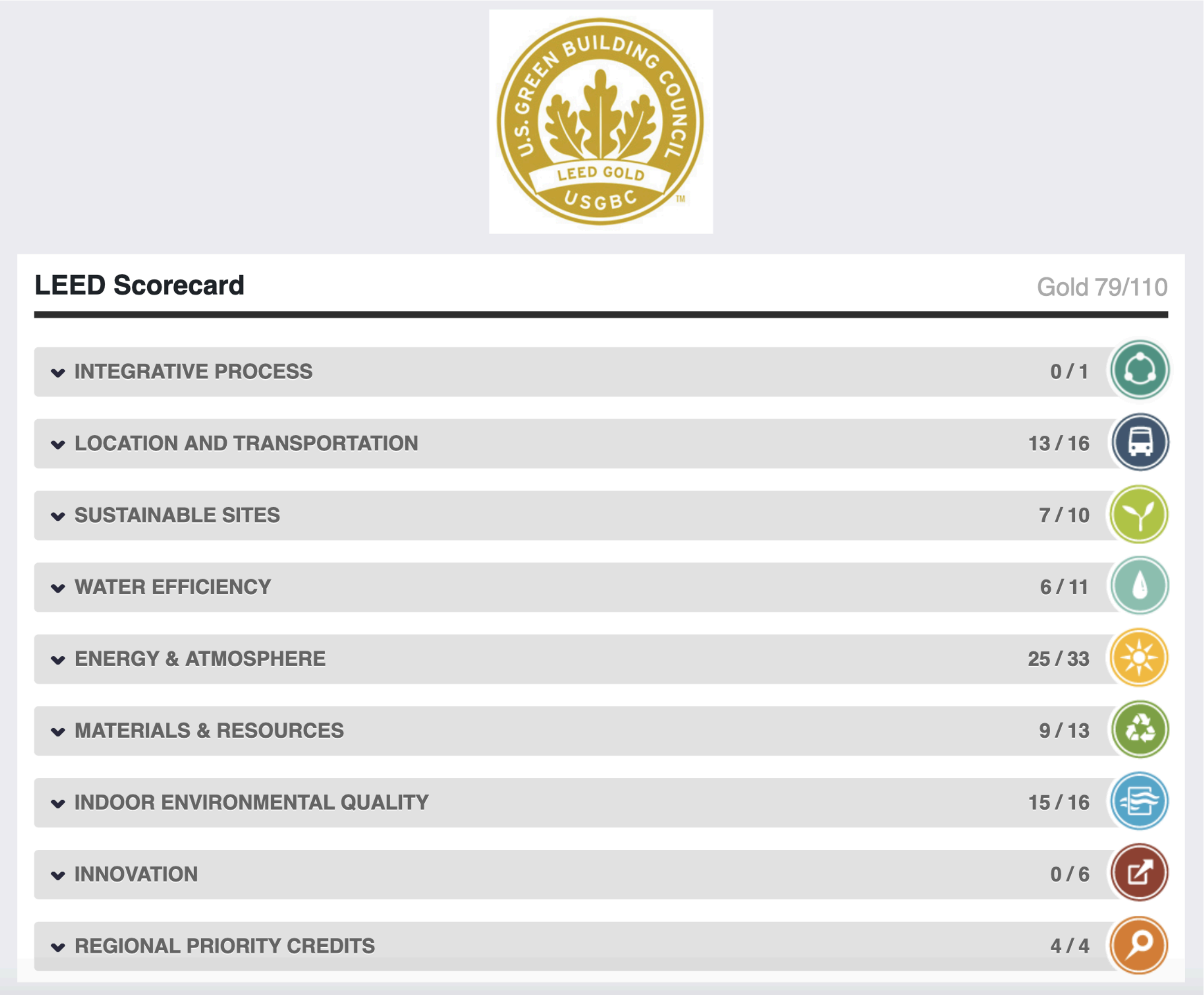
LEED, which stands for Leadership in Energy and Environmental Design is the most widely used green building certification system, developed by the U.S. Green Building Council (USGBC) to recognize best-in-class sustainable construction practices. It highlights top-tier sustainable construction methods and provides a framework for designing, building, and managing high-performance structures. These methods aim to minimize environmental harm while enhancing the health of occupants and boosting operational efficiency.

The rating system evaluates projects across following components:

- Energy efficiency (optimized HVAC systems, renewable energy integration)
- Water conservation (low-flow fixtures, rainwater harvesting)
- Sustainable material selection (recycled content, locally sourced materials)
- Indoor environmental quality (improved ventilation, low-VOC materials)
- Site sustainability (urban heat island mitigation, access to public transit)

LEED certification is divided into 4 levels based on their effectiveness. These levels are Certified (40-49), Silver (50-59), Gold (60-79), Platinum (80+).

Below is the certificate our project obtained based on the LEED metrics. As displayed, “Bay Symphony” project got 79 points and was awarded a Gold level of LEED scorecard.



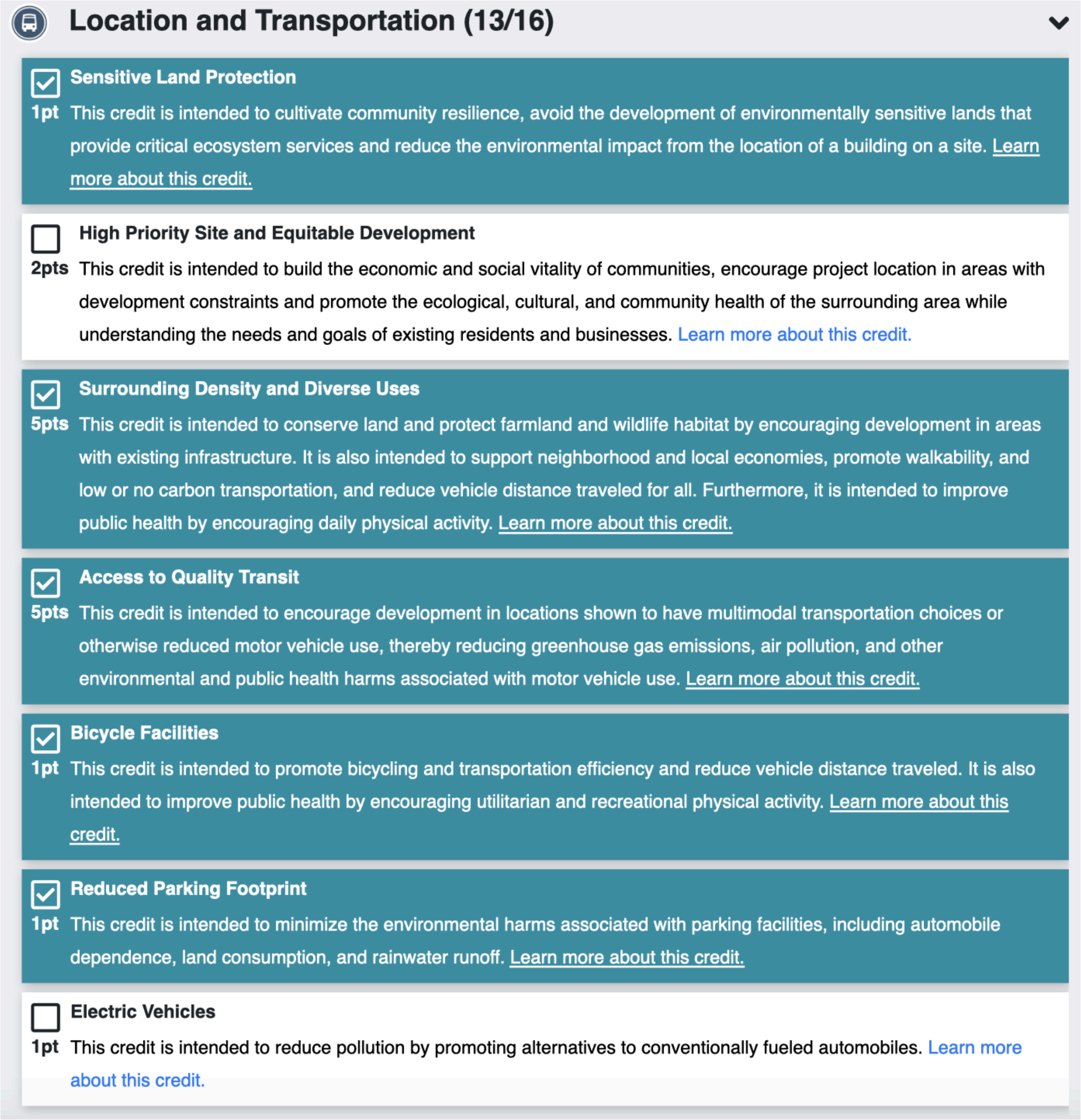
LEED Scorecard

Gold 79/110

INTEGRATIVE PROCESS	0 / 1
LOCATION AND TRANSPORTATION	13 / 16
SUSTAINABLE SITES	7 / 10
WATER EFFICIENCY	6 / 11
ENERGY & ATMOSPHERE	25 / 33
MATERIALS & RESOURCES	9 / 13
INDOOR ENVIRONMENTAL QUALITY	15 / 16
INNOVATION	0 / 6
REGIONAL PRIORITY CREDITS	4 / 4

Figure 2.5.1. LEED Scorecard

Location and Transportation. To achieve one point in Sensitive Land Protection we provided documentation that our building area was previously developed land as it served the purpose of a parking lot. 5 points for Surrounding Density and Diverse Uses were given because of the impact our mixed-use building has on an already developed area with existing infrastructure. Access to quality transit is achieved due to the geolocation of the site. The site will have both a special bike station for convenience, and areas for physical activity and sports.



Location and Transportation (13/16)

☑ **Sensitive Land Protection**
1pt This credit is intended to cultivate community resilience, avoid the development of environmentally sensitive lands that provide critical ecosystem services and reduce the environmental impact from the location of a building on a site. Learn more about this credit.

☐ **High Priority Site and Equitable Development**
2pts This credit is intended to build the economic and social vitality of communities, encourage project location in areas with development constraints and promote the ecological, cultural, and community health of the surrounding area while understanding the needs and goals of existing residents and businesses. Learn more about this credit.

☑ **Surrounding Density and Diverse Uses**
5pts This credit is intended to conserve land and protect farmland and wildlife habitat by encouraging development in areas with existing infrastructure. It is also intended to support neighborhood and local economies, promote walkability, and low or no carbon transportation, and reduce vehicle distance traveled for all. Furthermore, it is intended to improve public health by encouraging daily physical activity. Learn more about this credit.

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

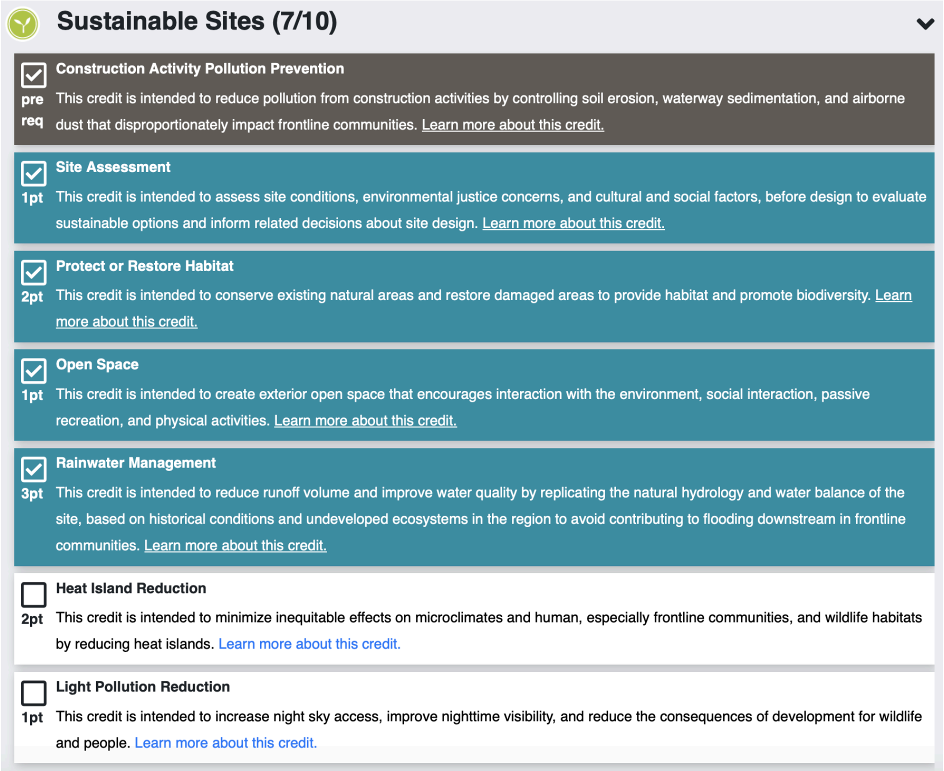
☑ **Bicycle Facilities**
1pt This credit is intended to promote bicycling and transportation efficiency and reduce vehicle distance traveled. It is also intended to improve public health by encouraging utilitarian and recreational physical activity. Learn more about this credit.

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

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

Figure 2.5.2. L&T Score

Sustainable Sites. Comprehensive site analysis was conducted and documented in the Site Analysis section. Special areas with planted trees were performed in order to establish vegetation. The main goal of site planning was to create accessible and convenient outdoor space for every resident. Rainwater management considered the natural hydrology of the site along with historical data and made correct steps to effectively manage rainwater.



Sustainable Sites (7/10)

☑ pre req **Construction Activity Pollution Prevention**
This credit is intended to reduce pollution from construction activities by controlling soil erosion, waterway sedimentation, and airborne dust that disproportionately impact frontline communities. Learn more about this credit.

☑ 1pt **Site Assessment**
This credit is intended to assess site conditions, environmental justice concerns, and cultural and social factors, before design to evaluate sustainable options and inform related decisions about site design. Learn more about this credit.

☑ 2pt **Protect or Restore Habitat**
This credit is intended to conserve existing natural areas and restore damaged areas to provide habitat and promote biodiversity. Learn more about this credit.

☑ 1pt **Open Space**
This credit is intended to create exterior open space that encourages interaction with the environment, social interaction, passive recreation, and physical activities. Learn more about this credit.

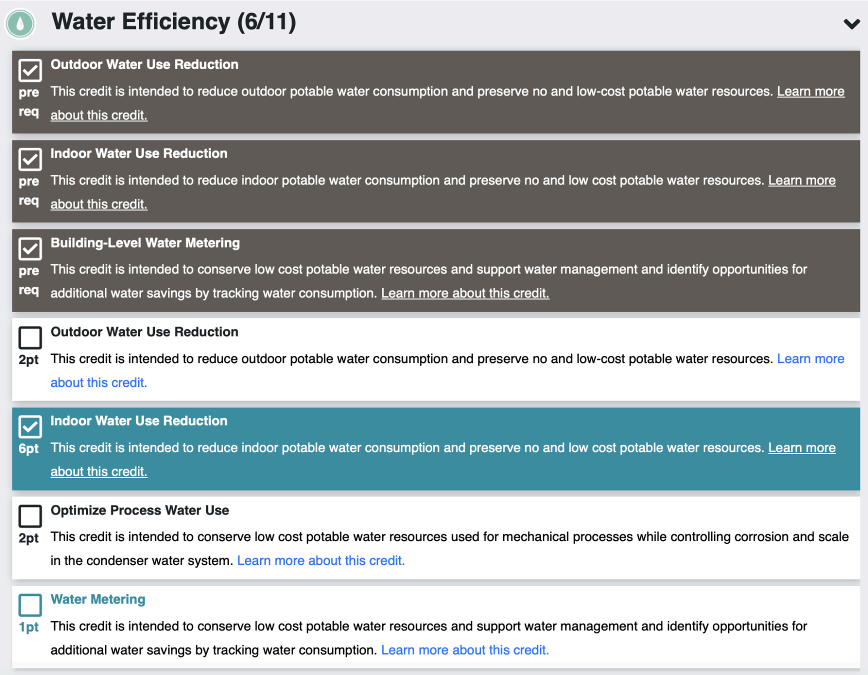
☑ 3pt **Rainwater Management**
This credit is intended to reduce runoff volume and improve water quality by replicating the natural hydrology and water balance of the site, based on historical conditions and undeveloped ecosystems in the region to avoid contributing to flooding downstream in frontline communities. Learn more about this credit.

☐ 2pt **Heat Island Reduction**
This credit is intended to minimize inequitable effects on microclimates and human, especially frontline communities, and wildlife habitats by reducing heat islands. Learn more about this credit.

☐ 1pt **Light Pollution Reduction**
This credit is intended to increase night sky access, improve nighttime visibility, and reduce the consequences of development for wildlife and people. Learn more about this credit.

Figure 2.5.3. Sustainable Sites Score

Water Efficiency. Install high-efficiency plumbing fixtures and appliances to achieve at least 35% indoor water use reduction compared to baseline calculations.



Water Efficiency (6/11)

☑ pre req **Outdoor Water Use Reduction**
This credit is intended to reduce outdoor potable water consumption and preserve no and low-cost potable water resources. Learn more about this credit.

☑ pre req **Indoor Water Use Reduction**
This credit is intended to reduce indoor potable water consumption and preserve no and low cost potable water resources. Learn more about this credit.

☑ pre req **Building-Level Water Metering**
This credit is intended to conserve low cost potable water resources and support water management and identify opportunities for additional water savings by tracking water consumption. Learn more about this credit.

☐ 2pt **Outdoor Water Use Reduction**
This credit is intended to reduce outdoor potable water consumption and preserve no and low-cost potable water resources. Learn more about this credit.

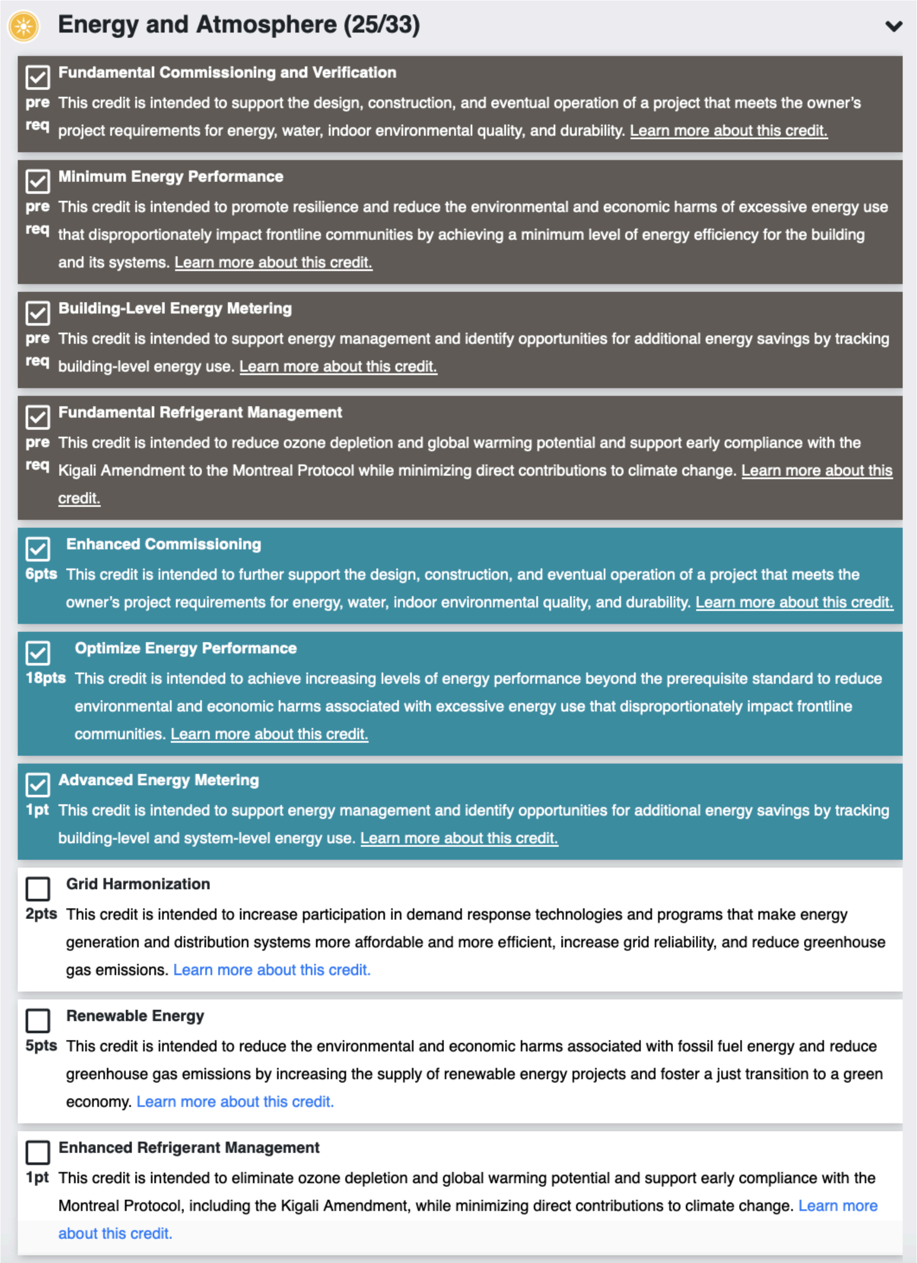
☑ 6pt **Indoor Water Use Reduction**
This credit is intended to reduce indoor potable water consumption and preserve no and low cost potable water resources. Learn more about this credit.

☐ 2pt **Optimize Process Water Use**
This credit is intended to conserve low cost potable water resources used for mechanical processes while controlling corrosion and scale in the condenser water system. Learn more about this credit.

☐ 1pt **Water Metering**
This credit is intended to conserve low cost potable water resources and support water management and identify opportunities for additional water savings by tracking water consumption. Learn more about this credit.

Figure 2.5.4. Water Efficiency Score

Energy and Atmosphere. Enhanced conditioning was achieved through reviewing contractor submittals, training building staff, and performing monitoring-based commissioning. To optimize energy performance, multiple strategies were used. Namely, efficient HVAC and lighting systems, demand control ventilation, and smart building controls. Advanced energy metering was achieved using modern meters that records data at hourly intervals and stores data for a minimum of 24 months.


Energy and Atmosphere (25/33)

☑ **Fundamental Commissioning and Verification**
pre req This credit is intended to support the design, construction, and eventual operation of a project that meets the owner's project requirements for energy, water, indoor environmental quality, and durability. Learn more about this credit.

☑ **Minimum Energy Performance**
pre req This credit is intended to promote resilience and reduce the environmental and economic harms of excessive energy use that disproportionately impact frontline communities by achieving a minimum level of energy efficiency for the building and its systems. Learn more about this credit.

☑ **Building-Level Energy Metering**
pre req This credit is intended to support energy management and identify opportunities for additional energy savings by tracking building-level energy use. Learn more about this credit.

☑ **Fundamental Refrigerant Management**
pre req This credit is intended to reduce ozone depletion and global warming potential and support early compliance with the Kigali Amendment to the Montreal Protocol while minimizing direct contributions to climate change. Learn more about this credit.

☑ **Enhanced Commissioning**
6pts This credit is intended to further support the design, construction, and eventual operation of a project that meets the owner's project requirements for energy, water, indoor environmental quality, and durability. Learn more about this credit.

☑ **Optimize Energy Performance**
18pts This credit is intended to achieve increasing levels of energy performance beyond the prerequisite standard to reduce environmental and economic harms associated with excessive energy use that disproportionately impact frontline communities. Learn more about this credit.

☑ **Advanced Energy Metering**
1pt This credit is intended to support energy management and identify opportunities for additional energy savings by tracking building-level and system-level energy use. Learn more about this credit.

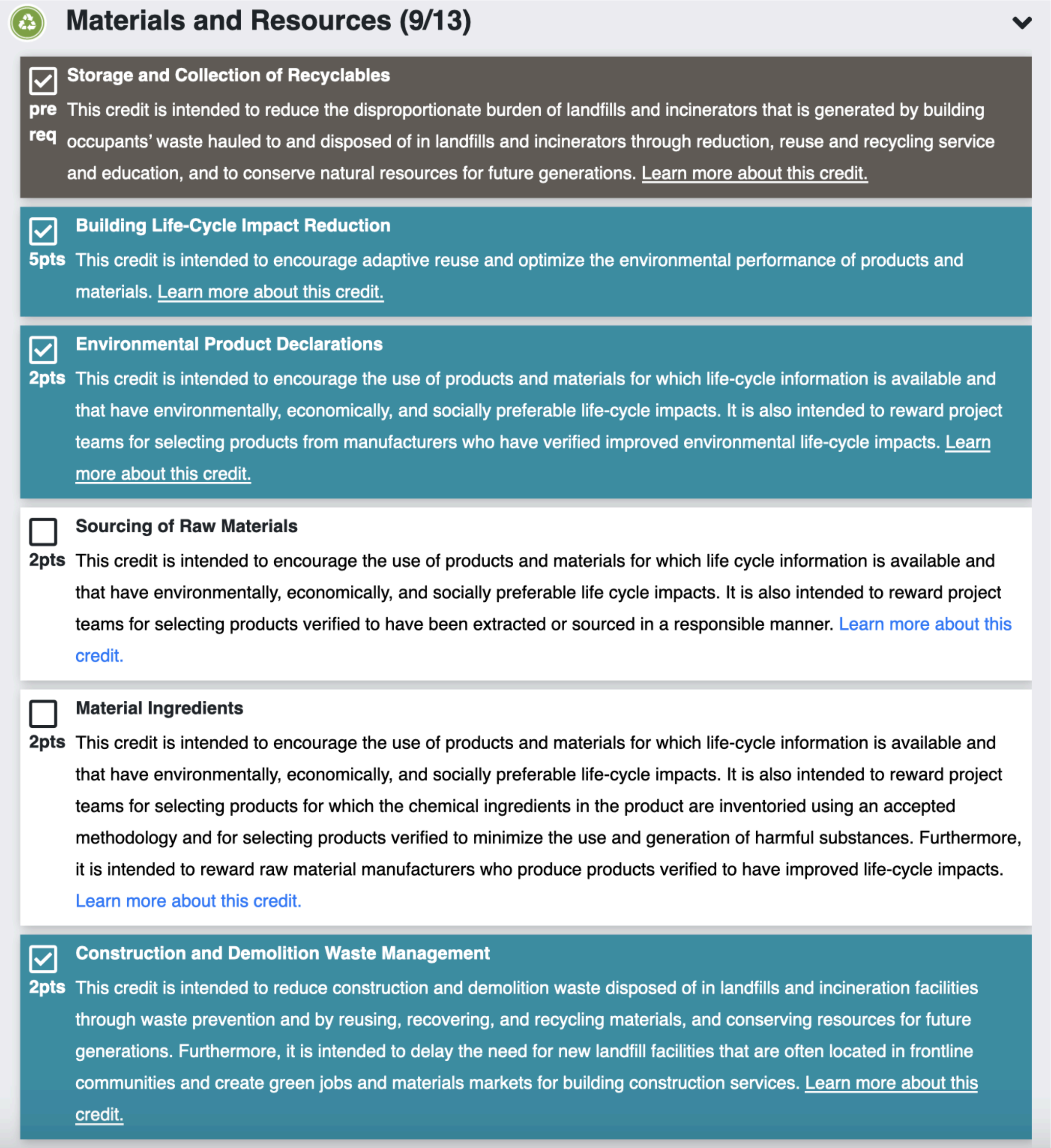
☐ **Grid Harmonization**
2pts This credit is intended to increase participation in demand response technologies and programs that make energy generation and distribution systems more affordable and more efficient, increase grid reliability, and reduce greenhouse gas emissions. Learn more about this credit.

☐ **Renewable Energy**
5pts This credit is intended to reduce the environmental and economic harms associated with fossil fuel energy and reduce greenhouse gas emissions by increasing the supply of renewable energy projects and foster a just transition to a green economy. Learn more about this credit.

☐ **Enhanced Refrigerant Management**
1pt This credit is intended to eliminate ozone depletion and global warming potential and support early compliance with the Montreal Protocol, including the Kigali Amendment, while minimizing direct contributions to climate change. Learn more about this credit.

Figure 2.5.5. Figure E&A Score

Materials and Resources. In order to reduce the impact whole-building life cycle assessment was conducted that displayed at least 10% reduction in ozone depletion, acidification, smog formation, eutrophication. The focus was on selecting materials and products that have industry wide verified EPDs. Construction and demolition waste management was recycled or kept out of landfills.



Materials and Resources (9/13)

☑ pre req **Storage and Collection of Recyclables**
This credit is intended to reduce the disproportionate burden of landfills and incinerators that is generated by building occupants' waste hauled to and disposed of in landfills and incinerators through reduction, reuse and recycling service and education, and to conserve natural resources for future generations. Learn more about this credit.

☑ 5pts **Building Life-Cycle Impact Reduction**
This credit is intended to encourage adaptive reuse and optimize the environmental performance of products and materials. Learn more about this credit.

☑ 2pts **Environmental Product Declarations**
This credit is intended to encourage the use of products and materials for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts. It is also intended to reward project teams for selecting products from manufacturers who have verified improved environmental life-cycle impacts. Learn more about this credit.

☐ 2pts **Sourcing of Raw Materials**
This credit is intended to encourage the use of products and materials for which life cycle information is available and that have environmentally, economically, and socially preferable life cycle impacts. It is also intended to reward project teams for selecting products verified to have been extracted or sourced in a responsible manner. Learn more about this credit.

☐ 2pts **Material Ingredients**
This credit is intended to encourage the use of products and materials for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts. It is also intended to reward project teams for selecting products for which the chemical ingredients in the product are inventoried using an accepted methodology and for selecting products verified to minimize the use and generation of harmful substances. Furthermore, it is intended to reward raw material manufacturers who produce products verified to have improved life-cycle impacts. Learn more about this credit.

☑ 2pts **Construction and Demolition Waste Management**
This credit is intended to reduce construction and demolition waste disposed of in landfills and incineration facilities through waste prevention and by reusing, recovering, and recycling materials, and conserving resources for future generations. Furthermore, it is intended to delay the need for new landfill facilities that are often located in frontline communities and create green jobs and materials markets for building construction services. Learn more about this credit.

Figure 2.5.6. Figure M&R Score

Indoor Environmental Quality. Smoke-free policy and comprehensive air quality measures using low-VOC materials and construction management protocols were established, also maximizing occupant comfort through 75% daylit spaces, LED lighting controls, and thermal comfort design, verified by post-construction air testing.

Indoor Environmental Quality (15/16)

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

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

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

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

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

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

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

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

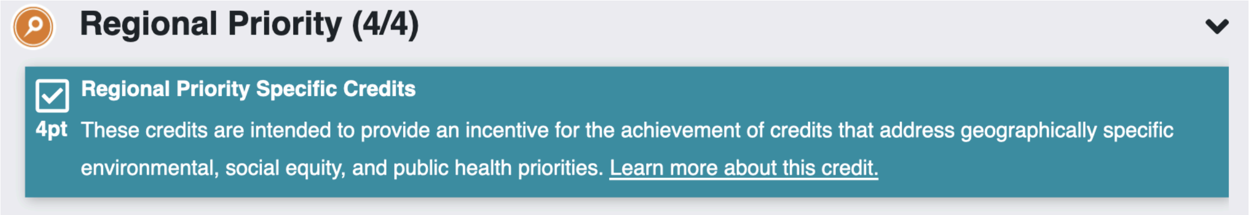
☑ **Daylight**
3pts This credit is intended to connect building occupants with the outdoors, reinforce circadian rhythms, and reduce the use of electrical lighting by introducing daylight into the space. Learn more about this credit.

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

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

Figure 2.5.7. Figure Indoor Environmental Quality Score

Regional Priority. Our building aims to provide accessible housing in a city by addressing social equity and urban sustainability goals.



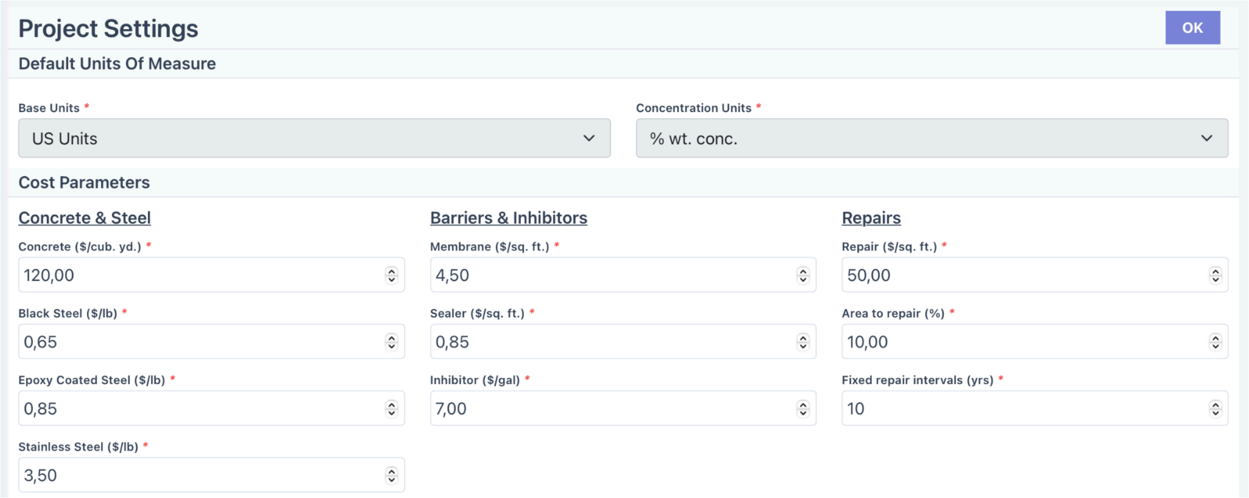
Regional Priority (4/4)

☑ **Regional Priority Specific Credits**

4pt These credits are intended to provide an incentive for the achievement of credits that address geographically specific environmental, social equity, and public health priorities. Learn more about this credit.

Figure 2.5.8 Regional Priority

Life 365 is a concrete service life prediction software tool that helps estimate the service life of concrete structures exposed to chloride environments. Cost parameters were taken based on an average pricing of San Francisco.



Project Settings

Default Units Of Measure

Base Units *: US Units

Concentration Units *: % wt. conc.

Cost Parameters

Concrete & Steel

Concrete ($/cub. yd.) *: 120,00

Black Steel ($/lb) *: 0,65

Epoxy Coated Steel ($/lb) *: 0,85

Stainless Steel ($/lb) *: 3,50

Barriers & Inhibitors

Membrane ($/sq. ft.) *: 4,50

Sealer ($/sq. ft.) *: 0,85

Inhibitor ($/gal) *: 7,00

Repairs

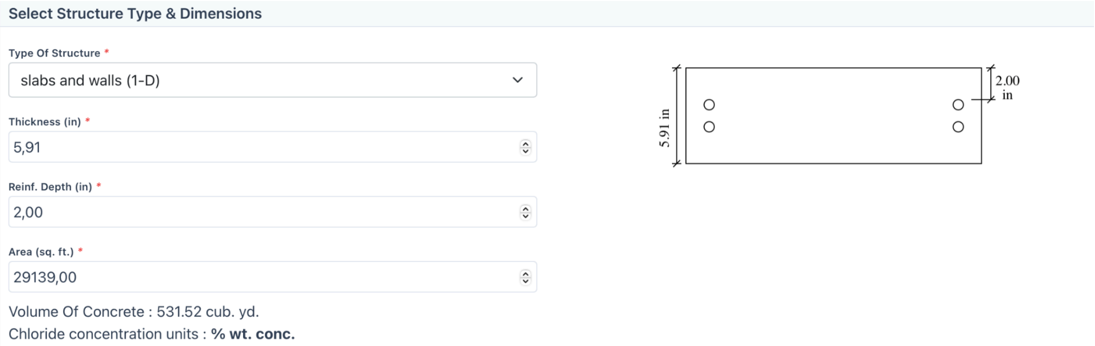
Repair ($/sq. ft.) *: 50,00

Area to repair (%) *: 10,00

Fixed repair intervals (yrs) *: 10

Figure 2.5.9. Life-365 Cost Parameters

As a Type of Structure “Slabs and Walls (1-D)” was chosen:



Select Structure Type & Dimensions

Type Of Structure *: slabs and walls (1-D)

Thickness (in) *: 5,91

Reinf. Depth (in) *: 2,00

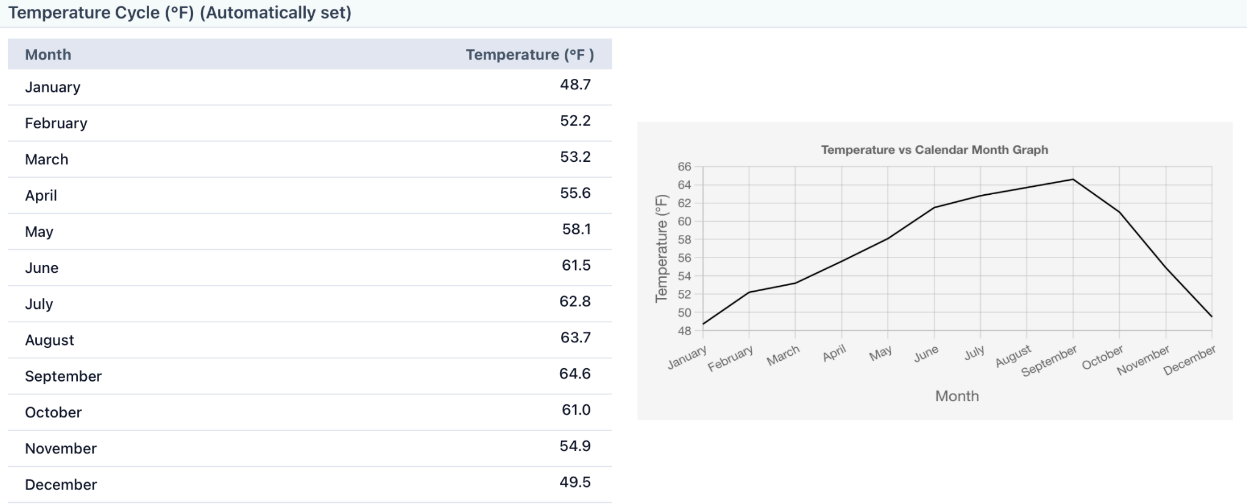
Area (sq. ft.) *: 29139,00

Volume Of Concrete : 531.52 cub. yd.

Chloride concentration units : **% wt. conc.**

Figure 2.5.10. Life-365 Cost Parameters

For the climate, built-in system settings for California, San Francisco were used



Temperature Cycle (°F) (Automatically set)

Month	Temperature (°F)
January	48.7
February	52.2
March	53.2
April	55.6
May	58.1
June	61.5
July	62.8
August	63.7
September	64.6
October	61.0
November	54.9
December	49.5

Figure 2.5.11. Life-365 Exposure Parameters

Concrete Mixtures for Three Cases:

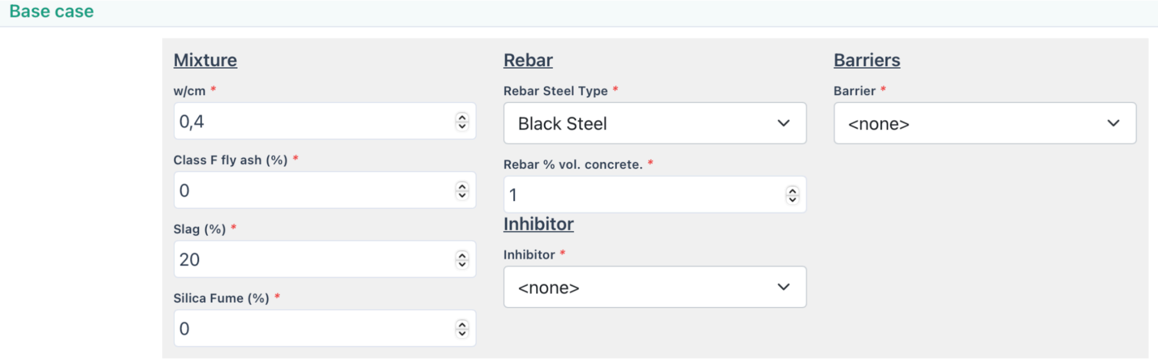
w/cm ratio was calculated to be 0.4 in order to match the concrete strength of 40 MPa.

$$w/cm = \frac{W_{water}}{W_{cement} + W_{slag}}$$

Abrams Law: $f_c = \frac{K_1}{K_2^{\,w/cm}}$; $K_1 = 96.5\,MPa,\ K_2 = 7$

$$f_c = \frac{96.5}{7^{0.40}} = 40.7\,MPa$$

With 20% slag, early strength might be slightly lower since reactivity will be low, but long-term strength will satisfy concrete strength of 40 MPa.



Base case

Mixture

w/cm *: 0,4

Class F fly ash (%) *: 0

Slag (%) *: 20

Silica Fume (%) *: 0

Rebar

Rebar Steel Type *: Black Steel

Rebar % vol. concrete. *: 1

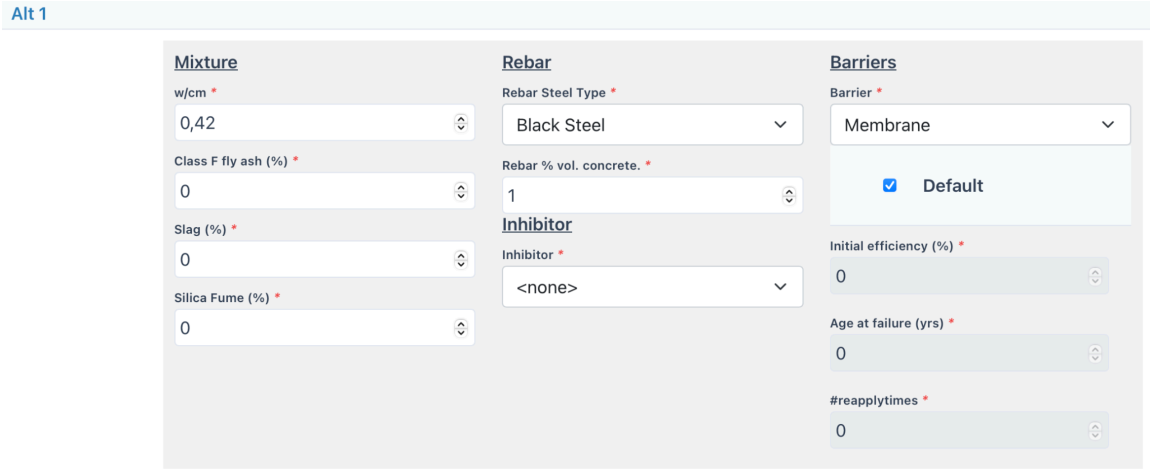
Inhibitor

Inhibitor *: <none>

Barriers

Barrier *: <none>

Figure 2.5.12. Base Case Mixture


Alt 1

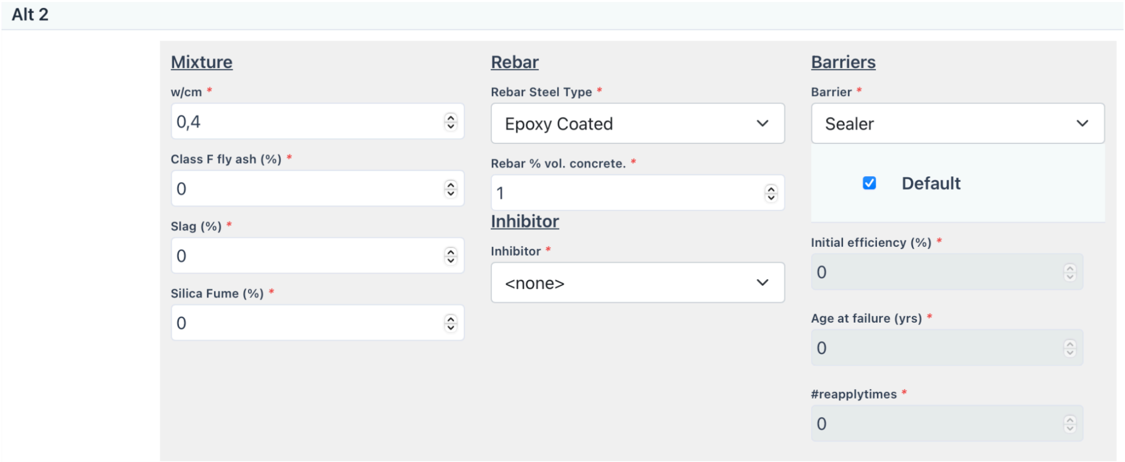
Mixture
w/cm * 0,42
Class F fly ash (%) * 0
Slag (%) * 0
Silica Fume (%) * 0

Rebar
Rebar Steel Type * Black Steel
Rebar % vol. concrete. * 1

Inhibitor
Inhibitor * <none>

Barriers
Barrier * Membrane
☑ Default
Initial efficiency (%) * 0
Age at failure (yrs) * 0
#reapplytimes * 0

Figure 2.5.13 Alternative 1 Mixture


Alt 2

Mixture
w/cm * 0,4
Class F fly ash (%) * 0
Slag (%) * 0
Silica Fume (%) * 0

Rebar
Rebar Steel Type * Epoxy Coated
Rebar % vol. concrete. * 1

Inhibitor
Inhibitor * <none>

Barriers
Barrier * Sealer
☑ Default
Initial efficiency (%) * 0
Age at failure (yrs) * 0
#reapplytimes * 0

Figure 2.5.14. Alternative 2 Mixture

Finally, obtained results are below:


Life Cycle Costs

Name	Construction Cost	Barrier Cost	Repair Cost	Life Cycle Cost
Base case	$109,030	$0	$1,072,626	$1,181,655
Alt 1	$109,030	$0	$1,327,368	$1,436,397
Alt 2	$122,952	$0	$1,322,168	$1,445,119

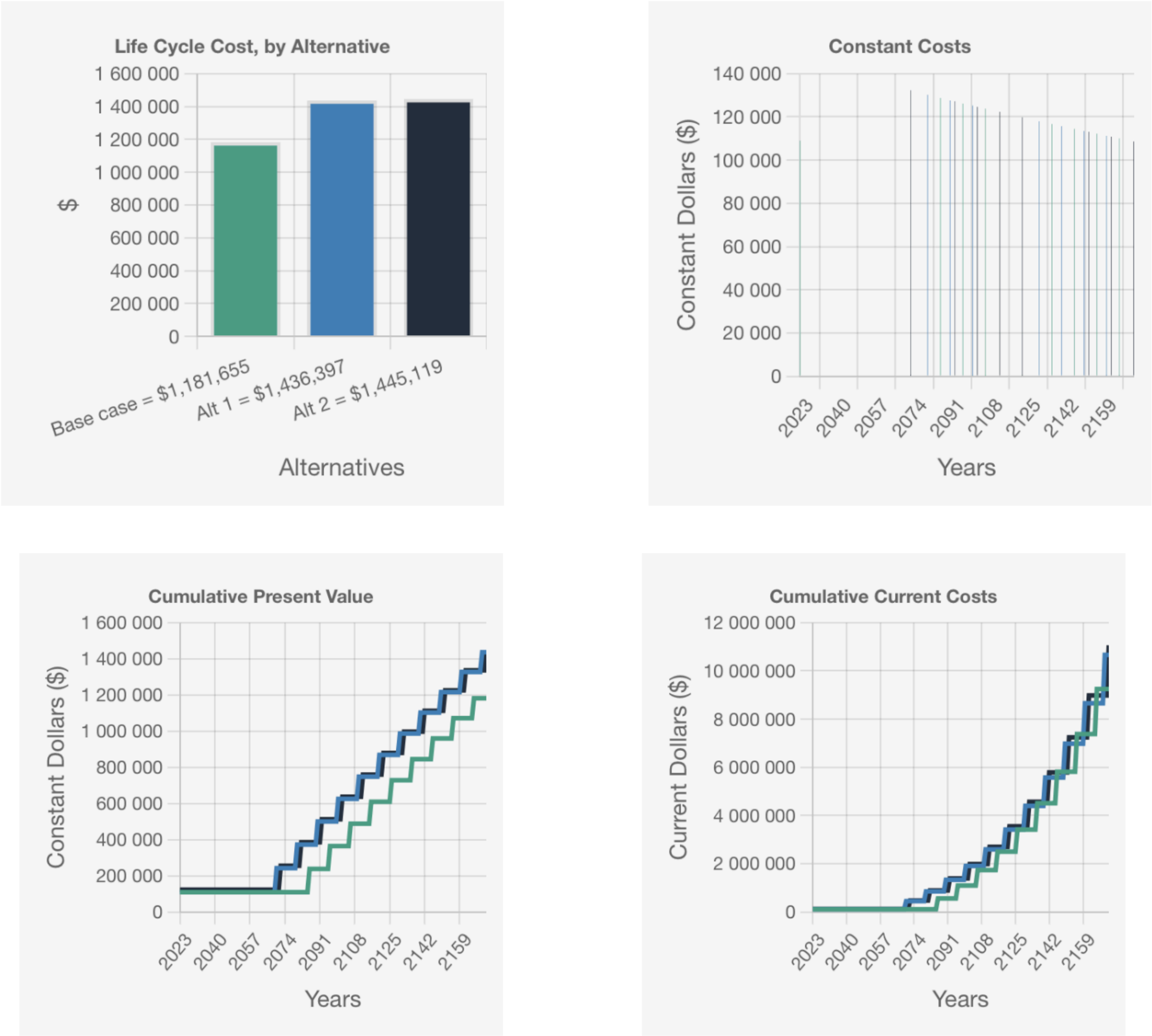


Figure 2.5.15. Life-365 LCC Results

2.6. Literature Review of Design Codes

According to the San Francisco Building Code the height limit for a high-rise building in the Mission Bay area varies from 135 to 240 feet, meaning that our building falls into the given range as it is 41 meter high (American Legal Publishing, 2024). Additionally, according to the Mission Bay zoning map, the site we chose is to be utilized for the construction of residential blocks, therefore, the aforementioned limit applies to our building (OCII, 2024). Figure 6 depicts the Mission Bay zoning map.

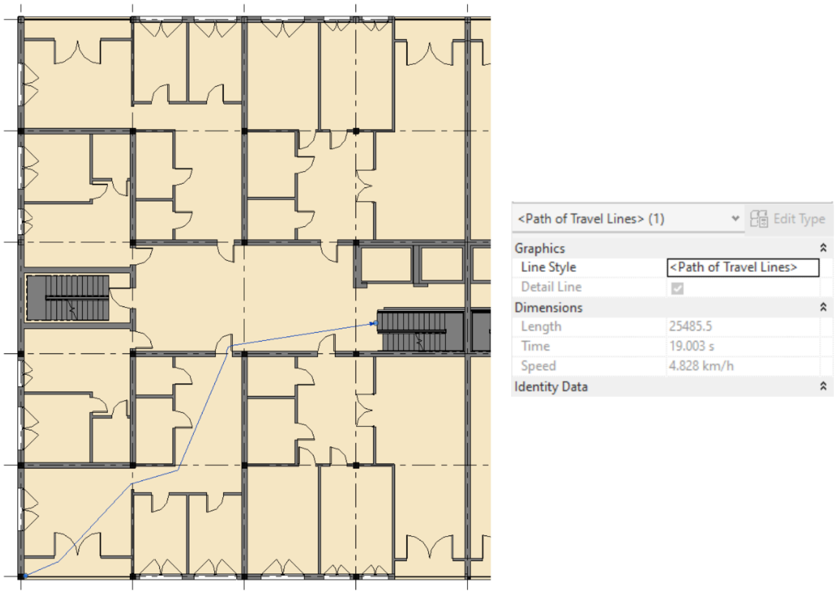
Figure 2.6.1 Mission Bay Zoning Map

To satisfy all the structural and architectural design requirements International Building Code 2024 (or IBC-2024) was closely followed. The main requirements list includes the following aspects:

a. Maximum distance from a room to the entrance of the building

According to IBC-2024, the maximum exit travel distance is to be calculated by taking into account the occupancy classification of the building and the presence of the automatic sprinkler system. Based on the definitions from Section 302.1, the occupancy type for this residential building is R-2, and thus according to Section 1017.2 the maximum travel distance from the furthest room to the entrance of the building should not exceed 250 feet or 76.2 meters, given the automatic sprinkler system is installed.

Maximum travel distance of the left entrance is 25.49 meters:



<Path of Travel Lines> (1)

Graphics

Line Style <Path of Travel Lines>

Detail Line ☑

Dimensions

Length 25485.5

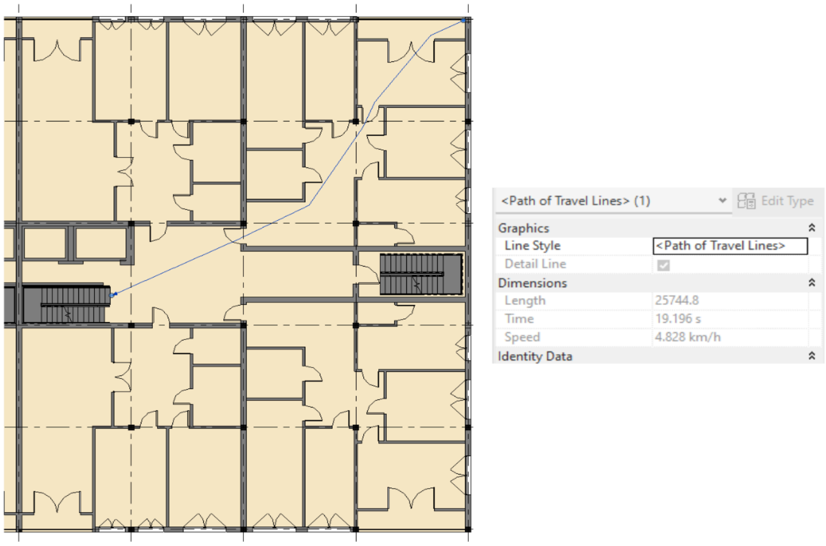
Time 19.003 s

Speed 4.828 km/h

Identity Data

Figure 2.6.2 Maximum Travel Distance, Left Entrance

Maximum travel distance of the right entrance is 25.74 meters:



<Path of Travel Lines> (1)

Graphics

Line Style <Path of Travel Lines>

Detail Line ☑

Dimensions

Length 25744.8

Time 19.196 s

Speed 4.828 km/h

Identity Data

Figure 2.6.3 Maximum Travel Distance, Right Entrance

The project's floor plan complies with the established code, with the maximum exit travel distance of 25.74 meters.

b. Number of elevators and stairs

To determine the minimum number of the exits, the occupancy load must be computed. Implementing the Maximum Floor Area Allowances per Occupant table from Section 1004.5, for residential spaces the occupant load factor is 200 gross square feet per occupant. To identify the maximum occupancy load , the area of largest private dwellings must be used, and thus a 154 square meter apartment is taken. For further calculations, the following equations are utilized:

$$1\, square\ foot\ =\ 0.0929\, m^2$$

$$A_{per\ occupant}\ =\ 200\ *\ 0.0929\ =\ 18.58\, m^2$$

$$Occupancy\ load\ per\ room\ =\frac{154}{18.58}=\ 8.29$$

$$Occupancy\ load\ per\ story\ =\ 8.29\ *\ 4\ =\ 33.15$$

Thus, from the Section 1006.3.3 Minimum Number of Exits per Story table, the occupancy load per story value falls into the range of 1-500, meaning the minimum number of exits must be equal to 2. As for the maximum occupancy load per space, 8.29 is less than 20 specified in the Spaces with One Exit or Exit Access Doorway table from Section 1006.2.1, and so one entrance door per apartment is allowed given that the common path is less or equal to 125 feet or 38.1 meters. The maximum common path for the project is 22.3 meters which satisfies the aforementioned criterion.

Thus, based on the calculations and conclusions above, the minimum number of exits for one story is 2. Therefore, it was decided to provide one passenger and one service elevator along with one general-use and one emergency staircase.

Regarding the dimensions for stairways from Section 1011:

- Minimum width of the strairways cannot be less than 1118 mm or less than 914 mm in case the stairway serving an occupant load less than 50
- Headroom clearance cannot be less than 2032 mm
- Stair riser’s maximum and minimum heights are 178 and 102 mm respectively

- Rectangular tread depth’s minimum value is 279 mm
- Length of landing cannot be less than the stairways it serves
- Minimum landing width is 1219 mm or the length of the landing, whichever is less
- General use stairway extend to the building’s roof as it is stated in Section 1011.12
- Handrail height shall not be less than 864 mm and not exceed the value of 965 mm (for us 900 mm)

Stair dimensions:

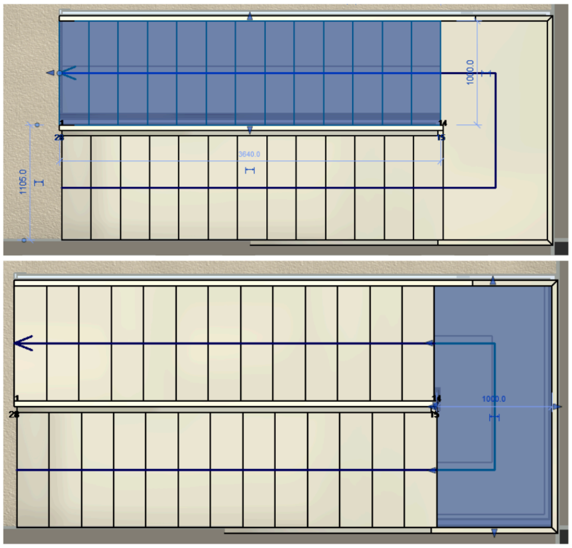
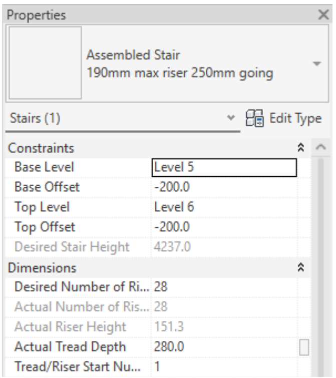


Figure 2.6.4 Stair Dimensions

Stair dimensions including riser and tread depth:



Properties

Assembled Stair
190mm max riser 250mm going

Stairs (1)

Constraints	
Base Level	Level 5
Base Offset	-200.0
Top Level	Level 6
Top Offset	-200.0
Desired Stair Height	4237.0
Dimensions	
Desired Number of Ri...	28
Actual Number of Ris...	28
Actual Riser Height	151.3
Actual Tread Depth	280.0
Tread/Riser Start Nu...	1

Figure 2.6.5 Stair Dimensions, Riser & Tread Depth

For elevators from Section 3001:

- Elevator car must accommodate an ambulance stretcher (610 mm by 2134 mm)

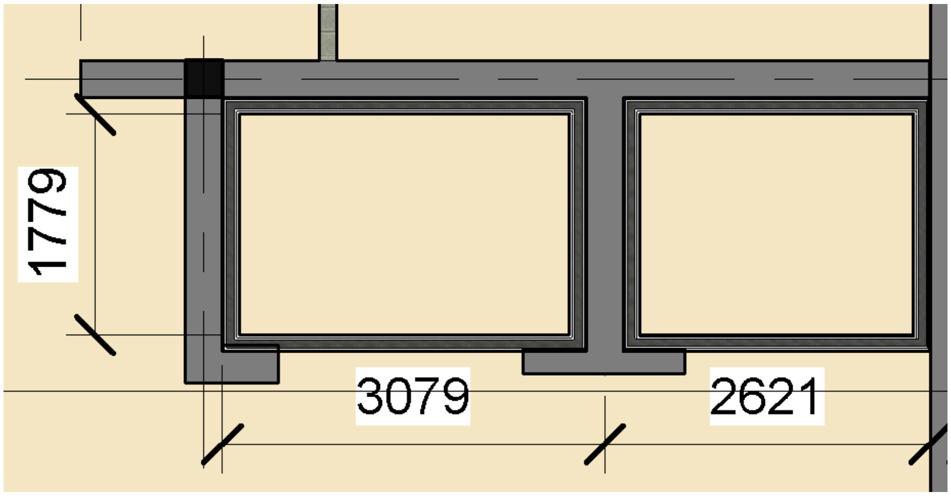


Figure 2.6.6. Elevator Dimensions

c. Areas for the residential building's rooms

From Section 602.2, the residential building is a Type I construction project as it is primarily made out of noncombustible materials. Based on the Allowable Area Factor table from Section 506.2, the R-2, Type I constructions are unlimited in building area. Therefore, the building area for this project is unlimited according to the code.

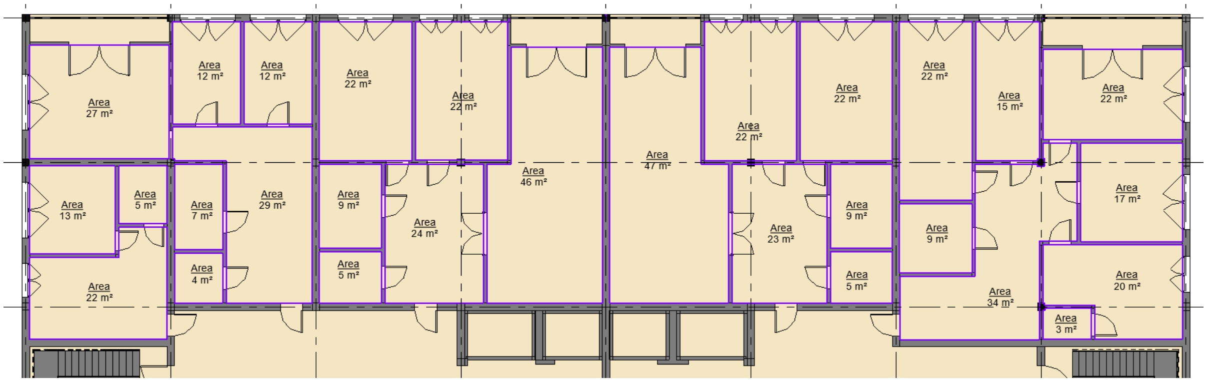


Figure 2.6.7. Room Areas

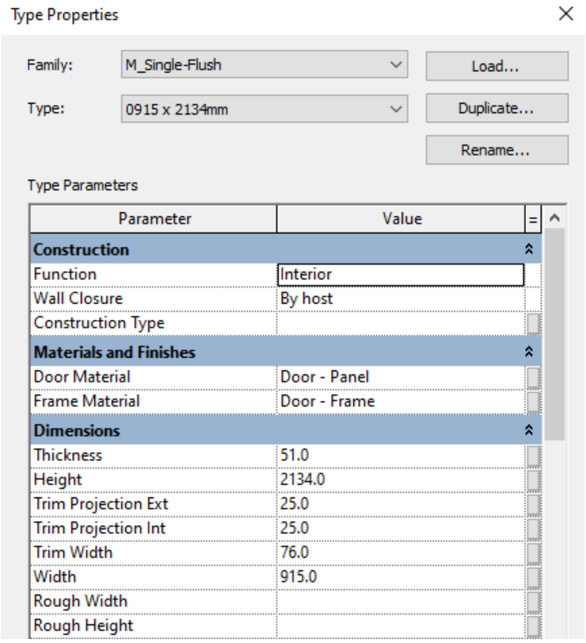
d. Number of doors and door’s dimensions

According to Section 1004.5, the number of exit doors should be calculated based on the occupancy load of the space. See section b for the calculations of the minimum number of exits per story.

Regarding the door dimensions, Section 1010.1.1 clearly states the following minimum requirements for the door dimensions:

- Minimum door opening width: 813 mm
- Minimum door width: 914 mm
- Minimum door height: 2032 mm
- Minimum spacing between doors in a series: 1219 mm

On the basis of these criteria, the door with the clear opening width of 915 mm and the height of 2134 mm was chosen. The door height and spacing requirements are satisfied as well.



Family: M_Single-Flush

Type: 0915 x 2134mm

Type Parameters

Parameter	Value
Construction	
Function	Interior
Wall Closure	By host
Construction Type	
Materials and Finishes	
Door Material	Door - Panel
Frame Material	Door - Frame
Dimensions	
Thickness	51.0
Height	2134.0
Trim Projection Ext	25.0
Trim Projection Int	25.0
Trim Width	76.0
Width	915.0
Rough Width	
Rough Height	

Figure 2.6.8. Main Door Dimensions

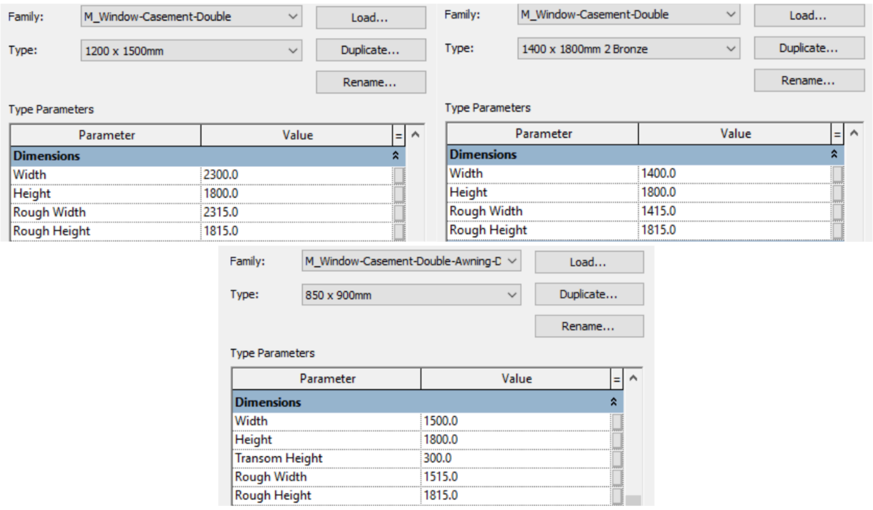
e. Window dimensions

Window area for each room depends on the room's function but the main guideline from Section 1204.2 is that every habitable room in the residential building must have a source of natural light, with at least 8% of floor area reserved for the window. For instance, for a 13 square meter room, the window must be at least 1.04 square meter to achieve the natural light guidelines. This requirement was directly followed so that each habitable room has a window that constitutes more than 8% of the total room area.

From Sections 1031.3.1, 1031.3.2, 1031.3.3 the minimum dimensioning requirements for a window are as follows:

- Minimum net clear opening area: 0.53 m^2
- Minimum net clear width: 508 mm
- Minimum net clear height: 610 mm
- Maximum height from floor: 1118 mm

All the established requirements are met for each window type chosen in the project. Window dimensions vary for each room, with most of them having substantial size to capture more natural daylight, thus completely satisfying all the criteria.



Family: M_Window-Casement-Double
Type: 1200 x 1500mm

Type Parameters

Parameter	Value
Dimensions	
Width	2300.0
Height	1800.0
Rough Width	2315.0
Rough Height	1815.0

Family: M_Window-Casement-Double
Type: 1400 x 1800mm 2 Bronze

Type Parameters

Parameter	Value
Dimensions	
Width	1400.0
Height	1800.0
Rough Width	1415.0
Rough Height	1815.0

Family: M_Window-Casement-Double-Awning-D
Type: 850 x 900mm

Type Parameters

Parameter	Value
Dimensions	
Width	1500.0
Height	1800.0
Transom Height	300.0
Rough Width	1515.0
Rough Height	1815.0

Figure 2.6.9. Room Areas

f. Corridor dimensions

For the corridor guidelines, Section 1020.3 suggests the minimum requirements for the corridor width:

- General minimum corridor width: 1.12 m
- Minimum corridor width within an apartment: 0.91 m

These minimum dimensions are taken into account for the project and the corridor width is more than the given values at all spaces in the residential building.

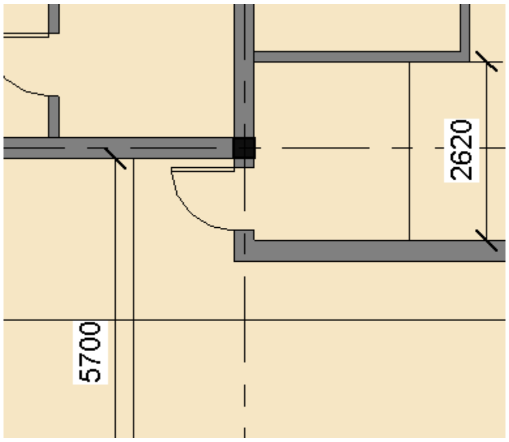


Figure 2.6.10. General Corridor and the Narrowest Corridor Widths

g. Allowable clear building height

As for the allowable building height, it is completely dependent on the presence and type of the sprinkler system. For this project, the National Fire Protection Association 13 system or NFPA 13 is to be implemented. Therefore, the building height is unlimited according to Section 504.3. The same principle is applied for the minimum number of stories from Section 504.4 as the NFPA 13 system is utilized, therefore the number of floors is unlimited as well.

h. Traffic flow for parking

Traffic flow was analyzed for the site analysis section, and is to be designed according to the flow of traffic outside the construction site.

i. Roof slope for drainage

For drainage purposes, a slope of 2% is to be designed on the rooftop, so that the stormwater is collected to the stormwater drainage system as stated in Section 1507.10.1.

Given the flat roof of the building, the gravity-driven scupper drainage system integrated into parapet walls has been selected as the optimal hydraulic solution. Figure X.X. showcases the sketch of such construction.

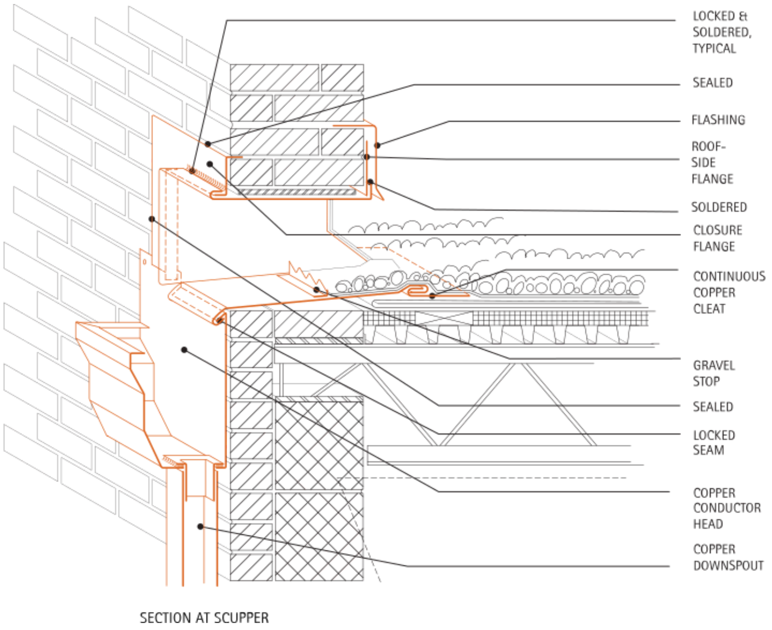


Figure 2.6.11. Scupper Roof Drainage System (Copper Development Association, n.d)

j. Parking ramp design

No parking is planned for the basement level as the chosen site provides enough space for the residential building, parking, and park with the playground for children.

2.7 Design of Non-structural Elements

Autoclaved aerated concrete was selected for the partition wall design due to its excellent thermal insulation, structural strength, light weight, flexibility, and high fire resistance (PCA, n.d.). In addition, the partition walls feature appropriate insulation and cement plaster.

For the exterior walls, concrete masonry units were chosen for their durability. The external cladding, previously mentioned, will be made of fiber cement. The partition and exterior walls are shown in Figures X and X, respectively. The total thicknesses of interior and exterior walls were 262 mm and 406 mm respectively.

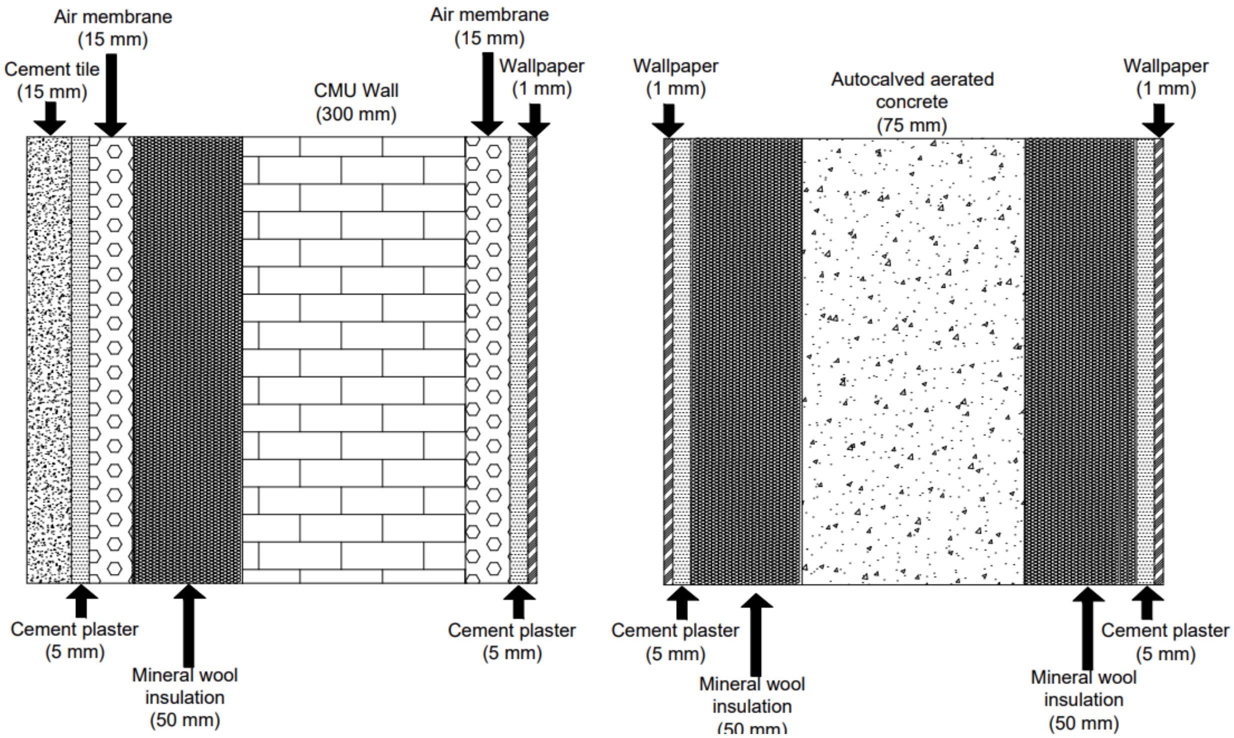


Figure 2.7.1. Exterior and Interior (inside apartments) walls compositions

For a typical floor we used RC slab, as well as acoustic fiber with a single-ply sheet. In addition, concrete screed and hardwood was used for the upper surface, and gypsum board on the lower side of the slab. The total thickness of floor was 297 mm, in which 150 mm was RC slab itself.

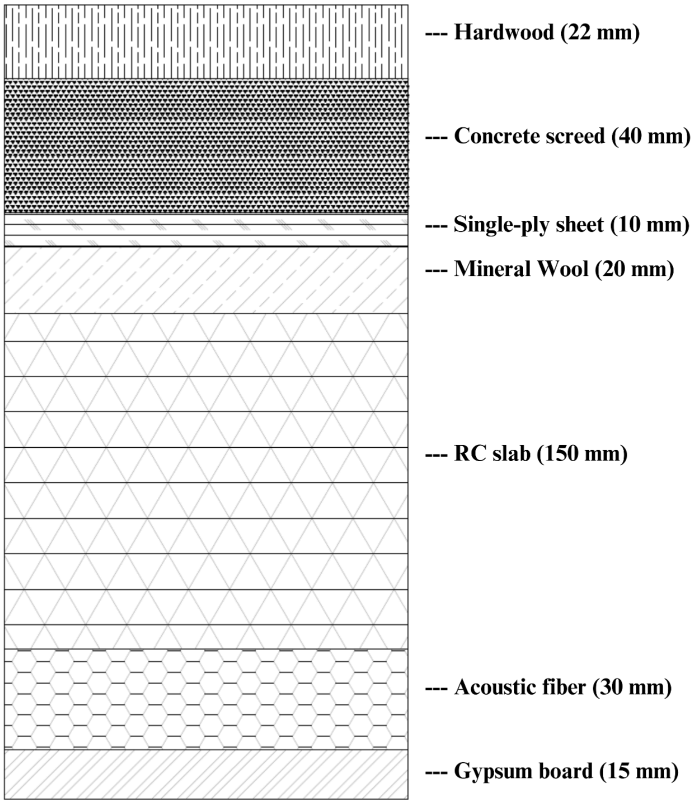


Figure 2.7.2 Composition of typical floor

3. Structural Design

3.1 Material Selection

The scope of Structural Design for this project includes designing a high-rise residential building in a high seismic zone.

The main structural material that was chosen for the building is reinforced concrete. A thorough analysis was conducted between reinforced concrete and steel to select the main material. Several scientific sources were analyzed to get the proper information about characteristics, and then the data was organized into a tabular format. The data in the table is according to Kodur et al. (2007), Thapa et al. (2020), and Mehta & Monteiro (2006).

Table 3.1. The comparison of Reinforced concrete and steel

Criteria	**Reinforced Concrete**	**Steel**
Strength	High-compressive stress	High stress-to-weight ratio
Durability	Excellent long-term durability	Susceptible to corrosion
Fire Resistance	Inherent fire resistance	Loses strength at high temperature
Labor	More widely available skills	Requires specialized welders
Cost	Lower initial cost	Higher initial cost
Seismic Performance	Less ductile for seismic conditions	Better resistance under seismic conditions
Thermal Properties	Good thermal mass	High thermal expansion
Quality Control	Factory controlled quality	On-site quality control

The reason for our selection of reinforced concrete mainly lies in the geotechnical characteristics of our site. Since our site is located near San Francisco Bay, the seismic performance achieved by the high ductility of reinforced concrete is more suitable for this region, and its durability can withstand the marine climate. Additionally, its high thermal mass aligns with the mild climate of the area. Since the site is located in a busy financial district, fire resistance is an important characteristic, and the sound-dampening feature of reinforced concrete is another benefit.

3.2 Analytical Model Development

The SAP 2000 software has been used extensively for both structural design and analysis purposes. All the properties given to the SAP 2000 model including the frame section materials, concrete and rebar steel characteristics, along with the model boundary conditions and geometry is to be discussed in this section.

3.2.1 Model Geometry

As was stated previously, the entire structure was created in SAP 2000. The building model includes beam and column sections, and two-way slab design. Figure X below demonstrates the model geometry in the SAP 2000 software.

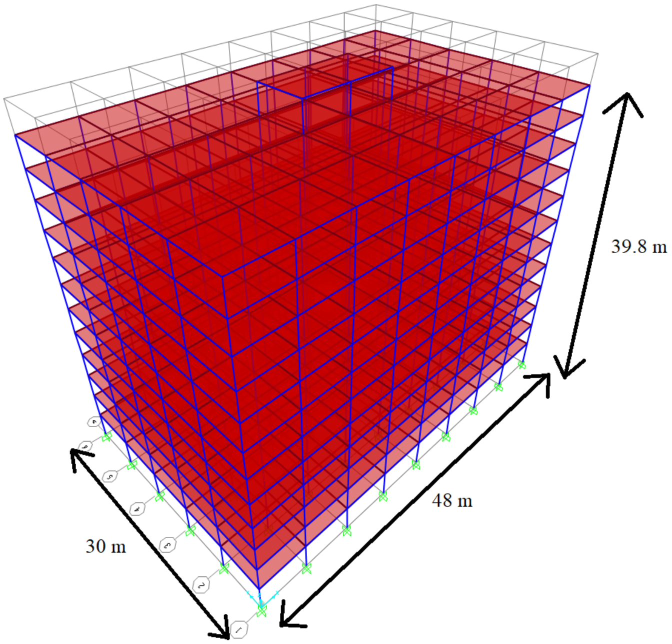


Figure 3.1. Building Model Geometry

3.2.2 Material Properties for SAP 2000

As was already established, material properties implemented in hand calculations match the material properties in the software.

3.2.2.1. Reinforced Concrete

Table 3.2.2.1 below depicts the concrete properties utilized in the software.

Table 3.2. Properties of reinforced concrete

Structural Member	***f'_c*, MPa**	***ρ*, kN/m^3**	***E_c*, MPa**	***v***	***G_c*, MPa**
Slabs, columns, beams	40	25	29725	0.2	12386

3.2.2.2. Steel Rebar

Below you can find Table 3.3.2.2 where the main characteristics of the reinforcing steel are summarized.

Table 3.3. Properties of steel rebar

***E*, MPa**	***f_y*, MPa**	***v***	***ρ*, kN/m^3**
200000	517.11	0.3	76.97

3.2.3. Frame Sections

For the frame sections of columns and beams, the following cracking moments of inertia listed in Table 3.4. were implemented in the software.

Table 3.4. Cracking moment of inertia for each structural member

Structural Member	**Cracking moment of inertia factors**
Beams	0.35
Slabs	0.7
Columns	0.25
Stairs	0.25

3.2.4. Element Types and Connections

All slabs are shell elements with two-way configuration, whereas the frame elements were used for modelling the columns and beams. The connection between the joints was assumed to be rigid.

3.2.5. Boundary Connections

As the first level columns are on the ground level, their horizontal and vertical movement is restricted, while the rotation movement is not, to allow the structure to be displaced when the later loads are applied.

3.3 Analysis and Design of Gravity Load Resisting System (GLRS)

3.3.1. Dead load calculations

Calculations of non-structural dead loads were performed for both roof and floor loads in accordance with the guidelines provided in ASCE 7-16 Chapter 3 (American Society of Civil Engineers, 2017). Two types of flooring finishes applied to apartments and corridors on the first floor were used to calculate floor dead loads. However, the most important floor finishing load was chosen for the structural component design. The following tables show the calculations for dead load from floor finishing on first floor/corridors and apartments, and roof.

Table 3.5 Dead load calculations for typical floors.

Function	Material Type	Thickness, mm	Load, kPa
Cover	Hardwood	22	0.19
Leveling	Concrete screed	40	0.92
Water-proofing	Single-ply sheet	10	0.03
Insulation	Mineral Wool	20	0.044
Structural Slab	RC slab	150	3.75
Acoustic Fiber		30	0.05
Mechanical Duct Allowance			0.19
Gypsum Board		15	0.12

Total		287	5.294

Table 3.6 Dead load calculations for 1st floor/corridors

Function	**Material Type**	**Thickness, mm**	**Load, kPa**
Cover	Ceramic tiles	19	0.77
	Mortar bed	13	
Leveling	Concrete screed	40	0.92
Water-proofing	Single-ply sheet	10	0.03
Insulation	Mineral Wool	20	0.044
Structural Slab	RC slab	150	5
Acoustic Fiber		30	0.05
Mechanical Duct Allowance			0.19
Gypsum Board		15	0.12
Total		297	5.874

Table 3.7. Dead load calculations for roof

Function	**Material Type**	**Thickness, mm**	**Load, kPa**
Cover	Three-ply ready roofing	10	0.05
Insulation	Polysterene foam	50	0.02
Water-proofing	Bituminous, gravel-covered	5	0.26
Leveling	Stone concrete	30	0.69
Structural Slab	RC slab	150	3.75

Acoustic Fiber	30	0.05
Mechanical Duct Allowance		0.19
Gypsum Board	15	0.12
Total	290	5.13

Additionally, the loads from partition walls, exterior walls, and roof parapets were calculated. It was assumed that the partition wall weight is uniformly distributed across the floor area, while the loads from exterior walls and roof parapets were applied exclusively to edge beams.

Table 3.8. Dead load calculations for Exterior Wall

Material type	**Material type**	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m3)**	**Density (kg/m3)**	**Weight (kg)**
Exterior finish	Cement tile	15	3300	6000	0.297	1800	534.6
Cement plaster	Cement plaster	5	3300	6000	0.099	2,040.00	201.96
Air infiltration barrier	Bituminous waterproofing membrane	15	3300	6000	0.297	1,270.00	377.19
Insulation	Mineral wool insulation	50	3300	6000	0.99	100	99
Autoclaved aerated concrete block	CMU Wall	300	3300	6000	5.94	1920	11404.8

Air infiltration barrier	Bituminous waterproofing membrane	15	3300	6000	0.297	1,270.00	377.19
Cement plaster	Cement plaster	5	3300	6000	0.099	2,040.00	201.96
Interior Finish	Wallpaper	5	3300	6000	0.099	0.1	0.0099
	Total	410					13197

Table 3.9. Calculations of the weight and load for interior walls inside the apartments.

Material type	**Material type**	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m3)**	**Density (kg/m3)**	**Weight (kg)**
Interior Finish	Wallpaper	1	3300	330584	1.09	0.10	0.109
Cement plaster	Cement Plaster	5	3300	330584	5.45	2040	11127.457
Insulation	Mineral wool	25	3300	330584	27.27	100	2727.318
Autoclaved aerated concrete block	Autoclaved aerated concrete	75	3300	330584	81.82	580	47455.333
Insulation	Mineral wool	25	3300	330584	27.27	100	2727.318
Cement plaster	Cement Plaster	5	3300	330584	5.45	2040	11127.457

Interior Finish	Wallpaper	1	3300	330584	1.09	0.1	0.109
	Total	137			149.46		75165

Table 3.10. Calculations of the weight and load for interior walls on 1st floor.

Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)
Cement plaster	Cement plaster	5	3500	170100	2.98	2040	6072.57
Insulation	Mineral wool	50	3500	170100	29.77	100	2976.75
Autoclaved aerated concrete block	Autoclaved aerated concrete	150	3500	170100	89.30	580	51795.45
Insulation	Mineral wool	50	3500	170100	29.77	100	2976.75
Cement plaster	Cement plaster	5	3500	170100	2.98	2,040	6072.57
	Total	260			154.79		69894

Table 3.11. Calculations of the weight and load for interior walls between apartments.

Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)
Interior Finish	Wallpaper	1	3300	6000	0.0198	0.10	0.00198
Cement	Cement	5	3300	6000	0.099	2,040.00	201.96

plaster	plaster						
Insulation	Mineral wool	50	3300	6000	0.99	100.00	99
Autoclaved aerated concrete block	Autoclaved aerated concrete	150	3300	6000	2.97	580.00	1722.6
Insulation	Mineral wool	50	3300	6000	0.99	100	99
Cement plaster	Cement plaster	5	3300	6000	0.099	2,040.00	201.96
Interior Finish	Wallpaper	1	3300	6000	0.0198	0.1	0.00198
	Total	262			5.1876		2325
Distributed load, kN/m							7.60

Table 3.12. Calculations of the weight and load for parapetes.

Material type	**Material type**	**Thickness (mm)**	**Height (mm)**	**Span (mm)**	**Volume (m3)**	**Density (kg/m3)**	**Weight (kg)**
Autoclaved aerated concrete block	Autoclaved aerated concrete	300	1500	6000	2.7	580	1566
Distributed load, kN/m							2.56041

3.3.2. Live load calculations

The minimum uniformly distributed loads for the residential building, as specified by ASCE 7-16, have been determined and are presented below. (American Society of Civil Engineers, 2017):

- Public rooms: $L_0 = 4.79\ kN/m^2$
- Private rooms and corridors serving them: $L_0 = 1.92\ kN/m^2$
- Corridors serving public rooms: $L_0 = 4.79\ kN/m^2$
- Ordinary flat roofs: $L_0 = 0.96\ kN/m^2$
- Stairs and exit ways: $L_0 = 4.79\ kN/m^2$

According to ASCE 7-16, live load reduction should be applied to all uniformly distributed loads, except for public rooms. The reduction for floor live loads should be calculated using the following formula:

$$L = L_0(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}) \tag{3.1}$$

Live load element factors, K_{ll}, will be as following:

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

Table-3.13. Floor live load calculations.

Space			**Space (m)**	**Span (m)**	**At (m2)**	**KLL**	**Reduction**	**L0 (kN/m2)**	**L (kN/m2)**
Typical floor	Private rooms and	Exterior columns	3	6	18	4	0.789	1.92	1.51
		Interior	6	6	36	4	0.631	1.92	1.21

	corridors serving them	columns							
		Corner columns	3	3	9	4	1	1.92	1.92
		Edge beams	3	6	9	2	1	1.92	1.92
		Interior beams	6	6	36	2	0.789	1.92	1.514
	Stairs and exit ways	Interior columns	6	6	36	4	0.631	4.79	3.02
		Exterior columns	3	6	18	4	0.789	4.79	3.78
		Interior beams	6	6	36	2	0.789	4.79	3.777
		Edge beams	3	6	9	2	1	4.79	4.79
First floor		Exterior columns	3	6	18	4	0.789	4.79	3.78
		Interior columns	6	6	36	4	0.631	4.79	3.02
		Corner columns	3	3	9	4	1	4.79	4.79
		Edge beams	6	6	18	2	1	4.79	4.79
		Interior beams	6	6	36	2	0.789	4.79	3.777

To calculate Roof live load, we utilized the following formula:

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

The reduction factor R_1 can be determined as follows:

$$R_1 = \begin{array}{ll} 1 & \text{for } A_T \leq 18.58\, m^2 \\ 1.2 - 0.011A_T & \text{for } 18.58\, m^2 < A_T < 55.74\, m^2 \\ 0.6 & \text{for } A_T \geq 55.74\, m^2 \end{array}$$

For ordinary R2 flat roofs the reduction factor R_2 flat roofs is 1.

Table-3.14. Roof live loads calculations

Column type	**Space (m)**	**Span (m)**	**At (m2)**	**KLL**	**R1**	**R2**	**L0 (kN/m2)**	**L (kN/m2)**
Exterior columns	3	6	18	4	1	1	0.96	0.96
Interior columns	6	6	36	4	0.804	1	0.96	0.77
Corner columns	3	3	9	4	1	1	0.96	0.96
Edge beams	3	6	9	2	1	1	0.96	0.96
Interior beams	3	6	18	2	1	1	0.96	0.96

Table 3.15. Loads Summary

Dead Load		**Live Load**	
Floor Type	**Load, kPa**	**Floor Type**	**Load, kPa**
Typical Floor Load	5.294	Typical Floor Load	1.92
First Floor Load	5.874	First Floor Load	4.79
Roof Load	5.13	Roof Load	0.96

3.3.3 Snow loads calculations

It has been revealed that there are no snow loads to be calculated, since our location, San Francisco, California, is not exposed to any. Therefore, snow loads were not calculated for our project (American Society of Civil Engineers, 2022).

3.3.4 Estimation of member sizes

3.3.4.1. Determination of structural layout

The spacing between columns was established at 6 meters to create a larger interior area for the residential building. Additionally, the column spacing should be within the range of 2.5 to 7.5 meters, making 6 meters a suitable choice (Pooja, 2022).

Three potential GLRS (Grid Layout for Reinforced Slabs) layouts were analyzed, and the most suitable one was selected based on material cost, which was determined by calculating the volume of structural members. The first layout involves a one-way slab system with major beams and a two-way slab. The second layout features a one-way slab with major beams and one minor beam. The third layout includes a two-way slab with major beams and two minor beams, as illustrated in Figure 1.

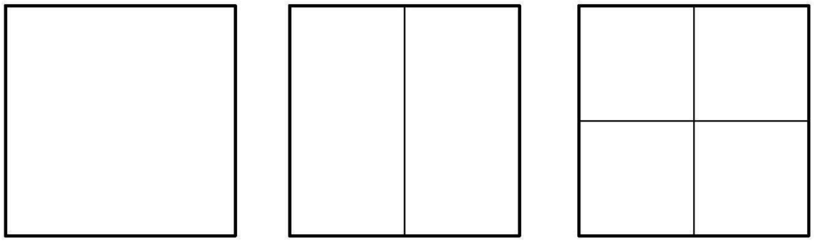


Figure 3.2. GLRS layout variants

For further analysis, the preliminary sizing of members was conducted based on the following guidelines from ACI 318-19 Chapters 7-10 and heuristic methods (American Concrete Institute, 2022):

- The thickness of major beams under a two-way slab without any interior beams is equal to 10% of its span.

- The width of major beams under a two-way slab without any interior beams is equal $b = \frac{h}{1.5}$.
- The thickness of major beams under a slab with interior beams is equal to 8% of its span.
- The width of major beams under a slab with interior beams is equal $b = \frac{h}{2}$.
- The thickness of simply supported minor beams is equal to $h = \frac{l}{16}$.
- The thickness of simply supported one-way slab is equal to $h = \frac{l}{20}$.

Option 1: Two-way slab

The lengths of the slab and of the beam were taken as 6 m for the conventional design. As previously mentioned, the thickness of the main beam (h) is 10% of its length, while its width (b) is given by $b = \frac{h}{2}$.

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

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

To determine the slab thickness, the moment of inertia and elastic modulus of both the beam and slab must be calculated. C40 grade concrete was selected for both components, as it is commonly used for structural members supporting heavier loads (Base Concrete, 2023). The calculations were as follows:

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

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

$$E_b = E_s = 4700\sqrt{f_c} \tag{3.3}$$

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

$$I_{b,\, cr} = \frac{1}{12} \cdot b \cdot h^3 \cdot 0.35 = \frac{1}{12} \cdot 0.3 \cdot 0.6^3 \cdot 0.35 = 1.89 \times 10^{-3}\, m^4$$

$$I_{s,\, cr} = \frac{1}{12} \cdot L \cdot h^3 \cdot 0.25 = \frac{1}{12} \cdot 6 \cdot 0.2^3 \cdot 0.25 = 1 \times 10^{-3}\, m^4$$

Once the members' moments of inertia have been determined, the stiffness ratio should then be calculated:

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

$$\alpha_{fm} = \frac{29725.41 \cdot 1.89 \times 10^{-3}}{29725.41 \cdot 1 \times 10^{-3}} = 1.89$$

$$\alpha_{fm} = 1.89 < 2$$

The minimum slab thickness should be determined using the following formula:

$$h = \frac{l_n (0.8 + f_y / 200{,}000)}{36 + 9\beta} \geq 5\ in. \tag{3.5}$$

where

$\beta = L_{yn} / L_{xn} = 1$

And the clear span is:

$$l_n = L - \frac{b_{major}}{2} - \frac{b_{major}}{2} = 6 - 0.15 - 0.15 = 5.7\, m$$

$$h = \frac{5.7(0.8 + 75000/200{,}000)}{36 + 5 * 1 * (1.89 - 0.2)} = 0.15067\, m > 0.127\, m\ (5\ in.)$$

For the conventional slab design, a thickness of h=0.15 m should be used. Consequently, the volume can be calculated as follows:

$$V(slab) = 6 \cdot 6 \cdot 0.15 = 5.4\, m^3$$

$$V(beam) = 0.6 \cdot 0.3 \cdot 6 \cdot 4 = 4.32\, m^3$$

$$V_{total,\, option\, 1} = 5.4 + 4.32 = 9.72\, m^3$$

***Option 2: Two-way slab with minor beam*s**

The same procedure was applied to this option. The thickness of the major beam was assumed to be 10% of its length, while the thickness of the simply supported minor beam was taken as $\frac{l}{16}$. The slab thickness was assumed to be 0.15 m.

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

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

$$h_{minor} = L/16 = 6/16 = 0.375\, m$$

$$b_{minor} = 0.375/2 = 0.188\, m$$

Then,we calculate the moment of inertia and stiffness ratio.

$$I_{b,\, major} = \frac{1}{12} * 0.3 * 0.6^3 * 0.35 = 1.189 \times 10^{-3}\, m^4$$

$$I_{b,\, minor} = \frac{1}{12} * 0.188 * 0.375^3 * 0.35 = 2.88 \times 10^{-4}\, m^4$$

$$I_s = \frac{1}{12} * 3 * 0.2^3 * 0.25 = 5 \times 10^{-4}\, m^4$$

$$\alpha_{f,\, major} = 3.78 \quad \text{and} \quad \alpha_{f,\, minor} = 0.57678$$

$$\alpha_{fm} = \frac{\alpha_{f,\, major} + \alpha_{f,\, minor}}{2} = 2.178 > 2$$

Next, the clear span of slabs with beams running in the long direction was measured, and subsequently, the ratio of the clear span in the long direction to that in the short direction of the slab was determined:

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

$$l_n = L - \frac{b_{major}}{2} - \frac{b_{minor}}{2} = 3 - 0.3 - 0.188 = 2.512\, m$$

Based on the calculated values, the minimum thickness of slab was found using the equation 3.5:

$$h = \frac{2.512(0.8+75000/200{,}000)}{36+9} = 0.0719m < 0.09\, m\, (3.5\, in.)$$

The minimum slab thickness is required to be 0.09 m. Thus, the slab thickness was rounded to 0.1 m. Following this, the volumes of the structural members were calculated.

$$V(slab) = 3 \cdot 3 \cdot 0.1 = 0.9\, m^3$$

$$V(major\ beam) = 0.6 \cdot 0.3 \cdot 6 = 1.08\, m^3$$

$$V(minor\ beam) = 0.375 \cdot 0.188 \cdot 6 = 0.421875\, m^3$$

$$V_{total,\, option\, 2} = 0.9 * 4 + 1.08 * 4 + 0.422 * 2 = 8.764\, m^3$$

Option 3: One-way slab with minor beam

In this final scenario, the dimensions of the major and minor beams were calculated using the same method as in the previous option.

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

$$h_{minor} = 0.375\,m;\, b_{minor} = 0.188\,m$$

$$h_{one\ way\ slab} = \frac{l}{20}$$

$$h_{slab} = \frac{L}{20} = \frac{3}{20} = 0.15\,m$$

Accordingly, the volumes of members can be calculated as:

$$V(slab) = 3 \cdot 6 \cdot 0.15 \cdot 2 = 5.4\,m^3$$

$$V(major\ beam) = 6 * 0.6 * 0.3 = 4.32\,m^3$$

$$V(minor\ beam) = 6 * 0.375 * 0.1875 = 0.422\,m^3$$

$$V_{total,\ option\ 3} = 5.4 + 4.32 + 0.422 = 10.14\,m^3$$

Ultimately, we ended up with three different options with varying dimensions. Despite the fact that a two-way slab with minor beams is the most economical structural layout, a two-way slab without minor beams was chosen as it requires less construction time and it is easier to manage in the project. Minor beams require more formwork, prolonging the overall process to a significant extent. Besides,

Table 3.16. Final calculations of three cases for layout

Option	Member	L (m)	h (m)	b (m)	Vt (m^3)
Two way slab	Slab	6.000	0.150	6.000	9.72
	Major beam	6.000	0.600	0.300	
Two way slab with minor beams	Slab	3.000	0.100	3.000	8.77
	Major	6.000	0.600	0.300	

	beam				
	Minor beam	6.000	0.375	0.188	
One way with minors	Slab	6.000	0.150	3.000	10.141875
	Major beam	6.000	0.600	0.300	
	Minor beam	6.000	0.375	0.188	

Table-3.17 Further information about Two Way slab case

Option	**Member**	**L (m)**	**h (m)**	**b (m)**	**V (m^3)**	**n**	**Vt (m^3)**	**Self-Weight, kN**	**Self-weight, kg**
Two way slab	Slab	6.000	0.150	6.000	7.200	1	9.72	135	13761.5
	Major beam	6.000	0.600	0.300	1.080	4		27	2752.5

Two way slab method of layout was chosen. You can see the layout plan below on Figure-2.

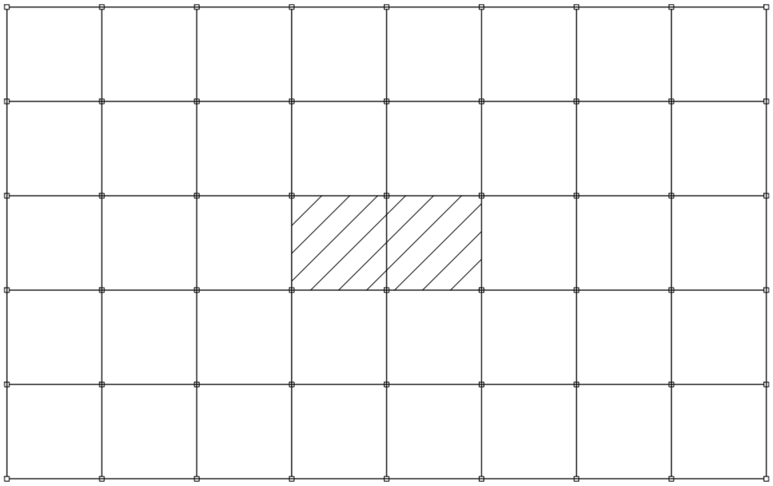


Figure 3.3. Structural layout of the building

3.3.4.2. Calculation of size dimensions of columns

Next we have to calculate the size dimensions of columns for every level of the building. It can be performed using the self weight of structural members and dead and live loads that were calculated before.

Major beam self-weight:

$$SW_{major\ beam} = \rho_{RC} \cdot V_{beam}(major) = 25\ kN/m^3 \cdot (0.3 \cdot 0.6 \cdot 6)m^3 = 27\ kN$$

Load combination was applied to estimate the dimensions of columns

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

Self weight of major beam transferred to interior column:

$$D_{major\ beam} = \frac{(4 \cdot 27)\ kN}{36\ m^2} = 3\ kN/m^2$$

$$D_{floor} = D_{major\ beam} + D_{partition\ walls} + D_{finishing,floor} = 3 + 0.964 + 5.294 = 9.258\ kN/m^2$$

$$D_{roof} = 5.13\ kN/m^2$$

$L = 3.02\ kN/m^2$ (critical live load on interior column)

$L_r = 0.96\ kN/m^2$ (critical roof live load on interior column)

$$W_u = 11(1.2 * 9.258 + 1.6 * 3.02) + 1.2 * 5.13 + 0.5 * 0.96 = 181.99\ kN/m^2$$

The proportions of the columns for the basement level were also established since the building has a basement. The first level and basement are 3.5 meters high, while the floor height above the first floor is 3.3 meters. Our building is mainly built from reinforced concrete, so the ACI 318-19 code (American Concrete Institute, 2022) was utilized to determine the dimensions. Additionally, the iteration approach was used to determine the column diameters for each story, taking into consideration the weight of the columns above the first floor.

$$P_u = w_u \cdot A_T + w_{columns\ above\ first\ floor} = 181.99\ kN/m^2 \cdot 36\ m^2 + 370\ kN = 6921.77\ kN$$

$$\phi P_n = 0.8\phi(0.85 f_c' A_c + f_y A_s)$$

Reinforcement ratio was assumed as 1%, hence, $A_s = 0.01 A_c$, $\phi = 0.65$ (based on ACI 318-19), $f_c' = 40\ MPa$, $f_y = 517.1\ MPa$.

$$\phi P_n = 0.8 \cdot 0.65 \cdot (0.85 \cdot 40000A_c + 420000 \cdot (0.01A_c)) = 20368.9A_c$$

$$\phi P_n \geq P_u$$

$$20368.9A_c \geq 6921.77\ kN$$

$$A_c \geq 0.533\ m^2$$

Our interior column measurements for one level will be 0.73 m if we assume that we have square columns of size $a = b \geq \sqrt{0.533}$. Since $a = b \geq \sqrt{0.533}$, we will take an increment and establish that the column size on the 1st floor is going to be 0.75 X 0.75. Table 14 shows the results of comparable calculations that were performed as an example for the first floor's exterior and corner columns. These sizes were consistently allocated to each column at each level since the inside columns had the biggest dimensions.

Table-3.18. Column size estimations for exterior, interior, and corner columns on the 1st floor

Column Type	**Wu (kN/m^2)**	**Pu (kN)**	**Pn (kN)**	**Ac (m^2)**	**a (m)**	**a=b (m)**	**Ag**	**Sum of area**
Interior column	237.815	9350.7	20368.9	0.45906	0.678	0.750	0.563	3.525
Exterior column	198.177	5498.8	20368.9	0.26996	0.520	0.550	0.303	1.515
Corner column	208.077	2841	20368.9	0.13948	0.373	0.400	0.160	0.785

Table-3.19. Column Size Distribution

Floors	**Square Column width, m**
0-1	0.75
2-3	0.7
4-5	0.65
6-7	0.6
8	0.55

9	0.5
10	0.45
11	0.4
12	0.35

As can be observed from Table 3X, the smallest column dimension is 0.35 m, which complies with the established ACI 318-19 requirements stating that the minimum column dimension shall not be less than 12 in or 305 mm. Additionally, the ratio of the smallest dimension to the dimension perpendicular to it shall not be less than 0.4 (ACI, 2019). All of these requirements are satisfied for the column size distribution for this residential complex.

Table-3.20. Calculation of exterior columns

Level	**Wu (kN/m^2)**	**Pu (kN)**	**Pn (kN)**	**Ac (m^2)**	**a (m)**	**a=b (m)**	**Ag**	**Sum of area**
0	215.590	6002.768	20368.92	0.2947	0.543	0.550	0.303	1.818
1	198.177	5498.761	20368.92	0.270	0.520	0.550	0.303	1.515
2	180.764	4999.676	20368.92	0.2455	0.495	0.500	0.250	1.265
3	163.351	4505.289	20368.92	0.2212	0.470	0.450	0.203	1.063
4	145.938	4010.901	20368.92	0.1969	0.444	0.450	0.203	0.860
5	128.526	3516.274	20368.92	0.1726	0.415	0.450	0.203	0.658
6	111.113	3078.081	20368.92	0.1511	0.389	0.400	0.160	0.498
7	93.700	2538.708	20368.92	0.1246	0.353	0.350	0.123	0.938
8	76.287	2202.169	20368.92	0.1081	0.329	0.350	0.123	0.253
9	58.874	1567.697	20368.92	0.0770	0.277	0.300	0.090	1.778
10	41.462	1083.643	20368.92	0.0532	0.231	0.300	0.090	0.073
11	24.049	602.012	20368.92	0.0296	0.172	0.250	0.063	0.010
12	6.636	125.515	20368.92	0.0062	0.078	0.100	0.010	

Table-3.21. Calculation of interior columns

Level	Wu (kN/m^2)	Pu (kN)	Pn (kN)	Ac (m^2)	a (m)	a=b (m)	Ag	Sum of area
0	254.328	10039.596	20368.92	0.4929	0.702	0.750	0.563	4.088
1	237.815	9350.655	20368.92	0.4591	0.678	0.750	0.563	3.525
2	221.302	8668.883	20368.92	0.4256	0.652	0.700	0.490	3.035
3	204.790	7986.730	20368.92	0.3921	0.626	0.700	0.490	2.545
4	188.277	7311.252	20368.92	0.3589	0.599	0.650	0.423	2.123
5	171.764	6635.420	20368.92	0.3258	0.571	0.650	0.423	1.700
6	155.251	5965.444	20368.92	0.2929	0.541	0.600	0.360	1.340
7	138.738	5301.153	20368.92	0.2603	0.510	0.550	0.303	1.038
8	122.226	4636.566	20368.92	0.2276	0.477	0.550	0.303	0.735
9	105.713	3976.902	20368.92	0.1952	0.442	0.500	0.250	0.485
10	89.200	3321.695	20368.92	0.1631	0.404	0.450	0.203	0.283
11	72.687	2670.479	20368.92	0.13111	0.362	0.400	0.160	0.123
12	56.174	2022.278	20368.92	0.0993	0.315	0.350	0.123	

Table-3.22. Calculations for corner columns

Level	Wu (kN/m^2)	Pu (kN)	Pn (kN)	Ac (m^2)	a (m)	a=b (m)	Ag	Sum of area
0	226.390	3102.013	20368.92	0.1523	0.390	0.400	0.160	0.945
1	208.077	2841.030	20368.92	0.1395	0.373	0.400	0.160	0.785
2	189.764	2583.755	20368.92	0.1269	0.356	0.350	0.123	0.663

3	171.451	2326.480	20368.92	0.1142	0.338	0.350	0.123	0.540
4	153.138	2069.022	20368.92	0.1016	0.319	0.350	0.123	0.418
5	134.826	1814.777	20368.92	0.0891	0.298	0.300	0.090	0.328
6	116.513	1560.377	20368.92	0.0766	0.277	0.300	0.090	0.238
7	98.200	1308.696	20368.92	0.0643	0.253	0.250	0.063	0.175
8	79.887	1056.888	20368.92	0.0519	0.228	0.250	0.063	0.113
9	61.574	807.304	20368.92	0.0396	0.199	0.200	0.040	0.073
10	43.262	557.622	20368.92	0.0274	0.165	0.200	0.040	0.033
11	24.949	309.600	20368.92	0.0152	0.123	0.150	0.023	0.010
12	6.636	62.758	20368.92	0.0031	0.056	0.100	0.010	

3.3.5. Assigning Gravity Forces to the SAP 2000 model

For Load Assignment Procedure Figure 3.4 below was created to clearly demonstrate the chosen frame. Please note that in all the calculations, Figure 3.4 below was referred to when frame numbers were mentioned. However, due to SAP 2000 technical limitations, the grid line names are swapped.

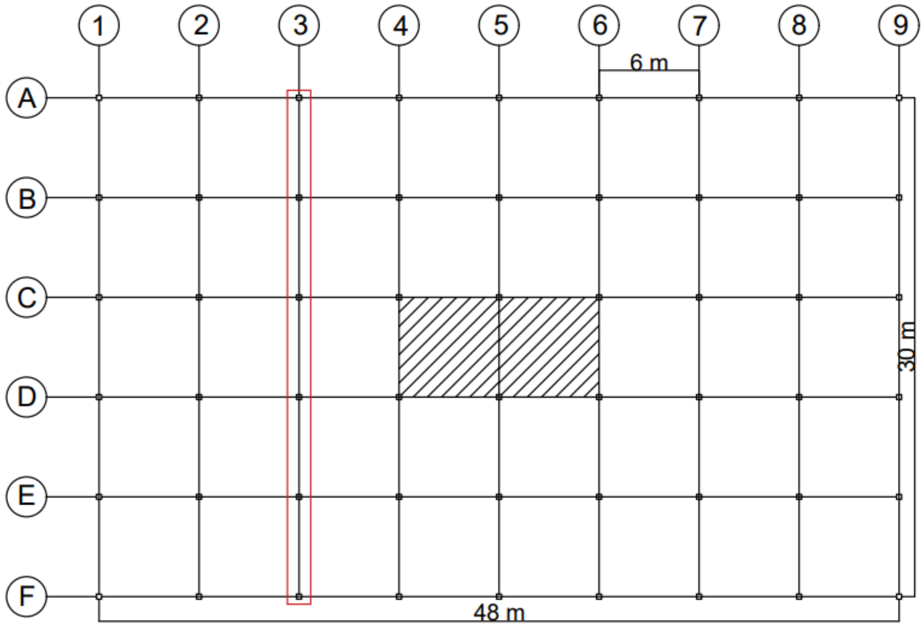
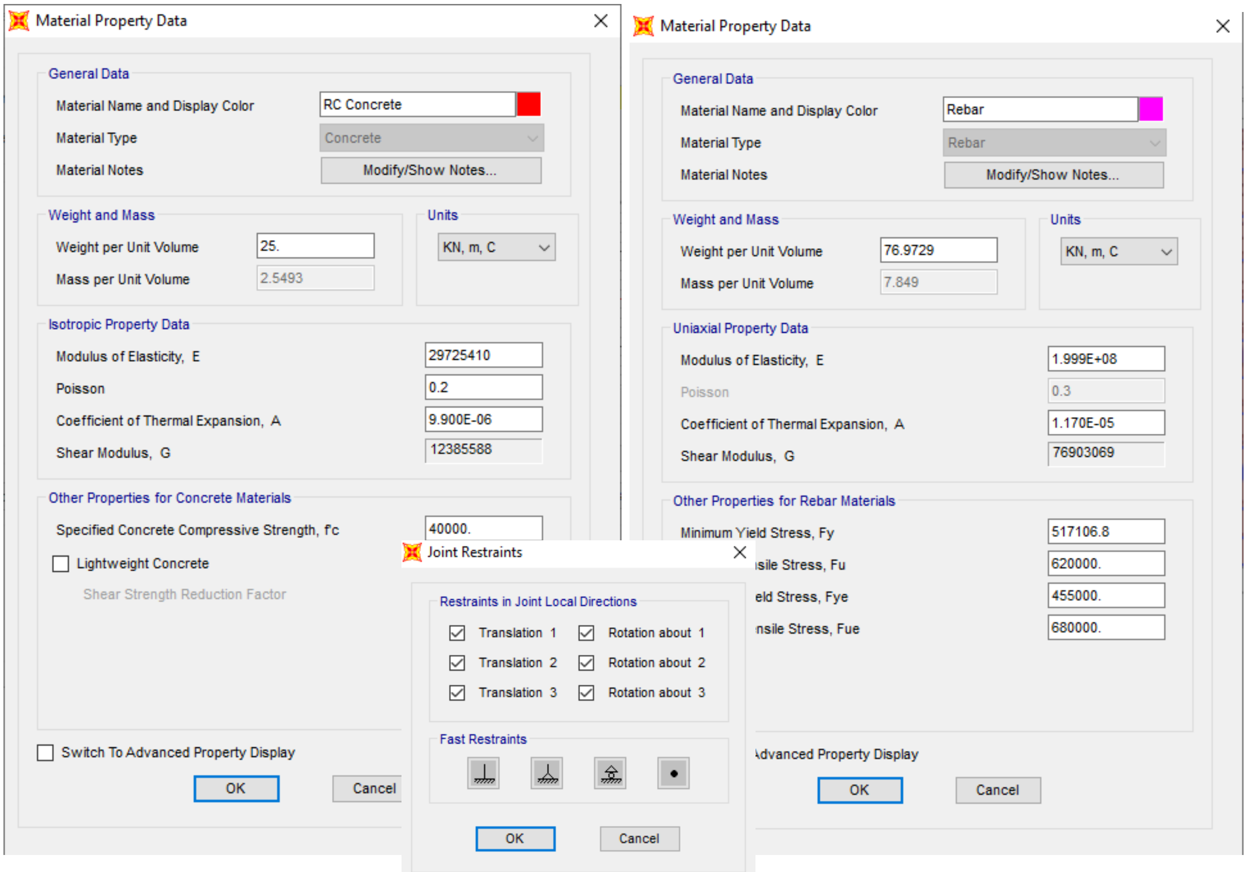


Figure 3.4. Chosen Frame for Load Assignment

Figure 3.5 below depicts the material property data for reinforced concrete and steel rebar entered into SAP 2000 and joint restraints parameters. All the parameters are exactly the same as discussed in earlier sections of the project.



Material Property Data

General Data
Material Name and Display Color: RC Concrete
Material Type: Concrete
Material Notes: Modify/Show Notes...

Weight and Mass
Weight per Unit Volume: 25.
Mass per Unit Volume: 2.5493
Units: KN, m, C

Isotropic Property Data
Modulus of Elasticity, E: 29725410
Poisson: 0.2
Coefficient of Thermal Expansion, A: 9.900E-06
Shear Modulus, G: 12385588

Other Properties for Concrete Materials
Specified Concrete Compressive Strength, f'c: 40000.
☐ Lightweight Concrete
Shear Strength Reduction Factor

☐ Switch To Advanced Property Display

Material Property Data

General Data
Material Name and Display Color: Rebar
Material Type: Rebar
Material Notes: Modify/Show Notes...

Weight and Mass
Weight per Unit Volume: 76.9729
Mass per Unit Volume: 7.849
Units: KN, m, C

Uniaxial Property Data
Modulus of Elasticity, E: 1.999E+08
Poisson: 0.3
Coefficient of Thermal Expansion, A: 1.170E-05
Shear Modulus, G: 76903069

Other Properties for Rebar Materials
Minimum Yield Stress, Fy: 517106.8
sile Stress, Fu: 620000.
eld Stress, Fye: 455000.
nsile Stress, Fue: 680000.

dvanced Property Display

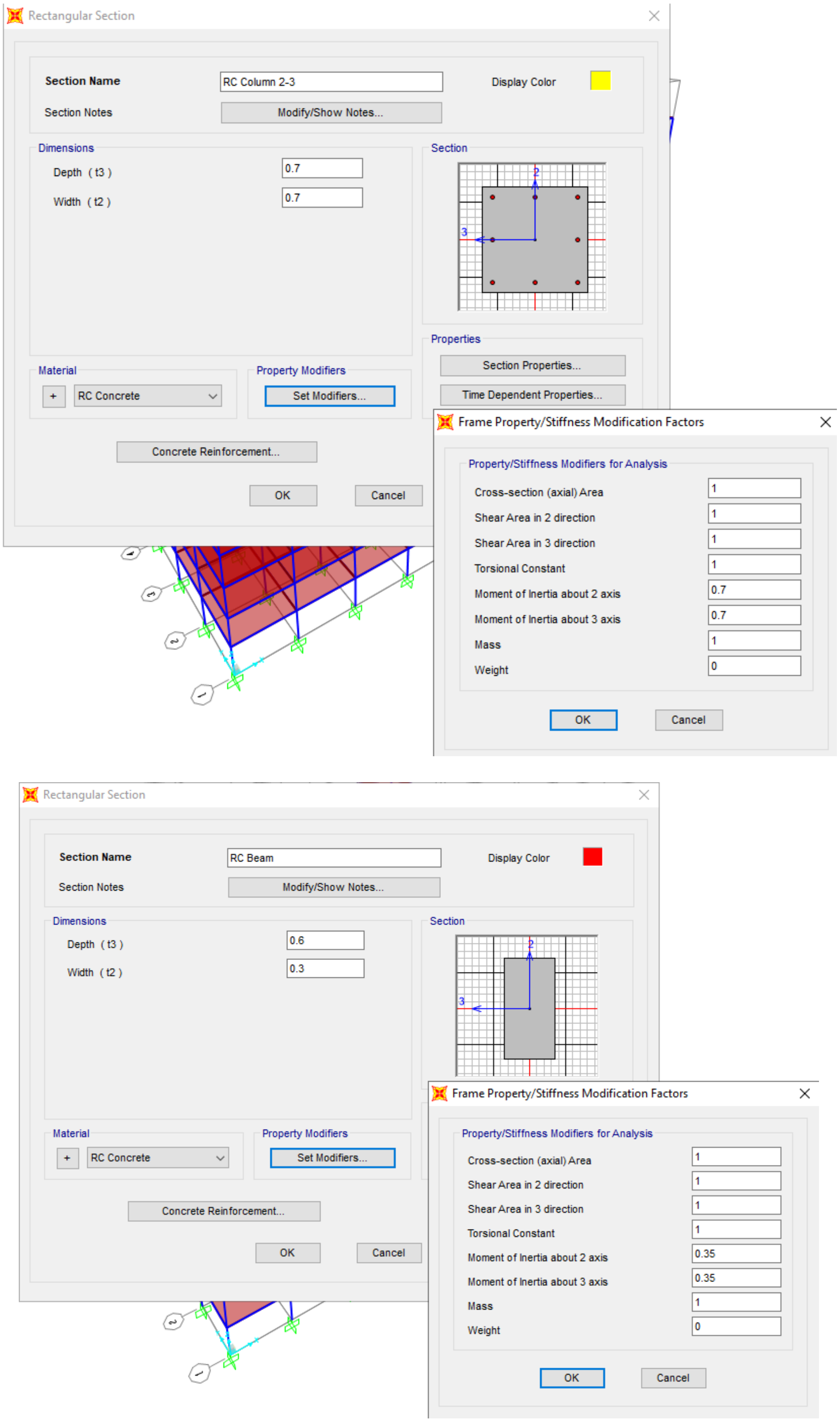
Joint Restraints

Restraints in Joint Local Directions
☑ Translation 1 ☑ Rotation about 1
☑ Translation 2 ☑ Rotation about 2
☑ Translation 3 ☑ Rotation about 3

Fast Restraints

Figure 3.5. Material Property Data and Joint Restraints

Following the material property data input, the section properties for both reinforced major beam and columns were entered into the software. As can be seen from Figure 3.6 below, the dimensions in SAP 2000 are exactly the same presented earlier in this section.



Rectangular Section

Section Name: RC Column 2-3

Depth (t3): 0.7

Width (t2): 0.7

Material: RC Concrete

Frame Property/Stiffness Modification Factors

Cross-section (axial) Area	1
Shear Area in 2 direction	1
Shear Area in 3 direction	1
Torsional Constant	1
Moment of Inertia about 2 axis	0.7
Moment of Inertia about 3 axis	0.7
Mass	1
Weight	0

Rectangular Section

Section Name: RC Beam

Depth (t3): 0.6

Width (t2): 0.3

Material: RC Concrete

Frame Property/Stiffness Modification Factors

Cross-section (axial) Area	1
Shear Area in 2 direction	1
Shear Area in 3 direction	1
Torsional Constant	1
Moment of Inertia about 2 axis	0.35
Moment of Inertia about 3 axis	0.35
Mass	1
Weight	0

Figure 3.6. Structural Element Section Properties

Now that all the necessary parameters were inputed into SAP 2000, the load assignment for dead, live, roof live loads in both 2D and 3D are presented below. As you could also observe, for 3D view two configurations of load distribution were chosen, those being Uniform Distribution to Area and Uniform Distribution to Frame.

To calculate the applied distributed load the computed values from Seciton 3.3 were used in order to consider all the possible loads in the building. Below you can find the three calculations for each load type.

$Distributed\ Dead\ Load = (5.294 + 5.874 + 5.13) * 6 = 97.20\ kN/m$

$Distributed\ Live\ Load = 4.79 * 6 = 28.74\ kN/m$

$Distributed\ Roof\ Live\ Load = 0.96 * 6 = 5.76\ kN/m$

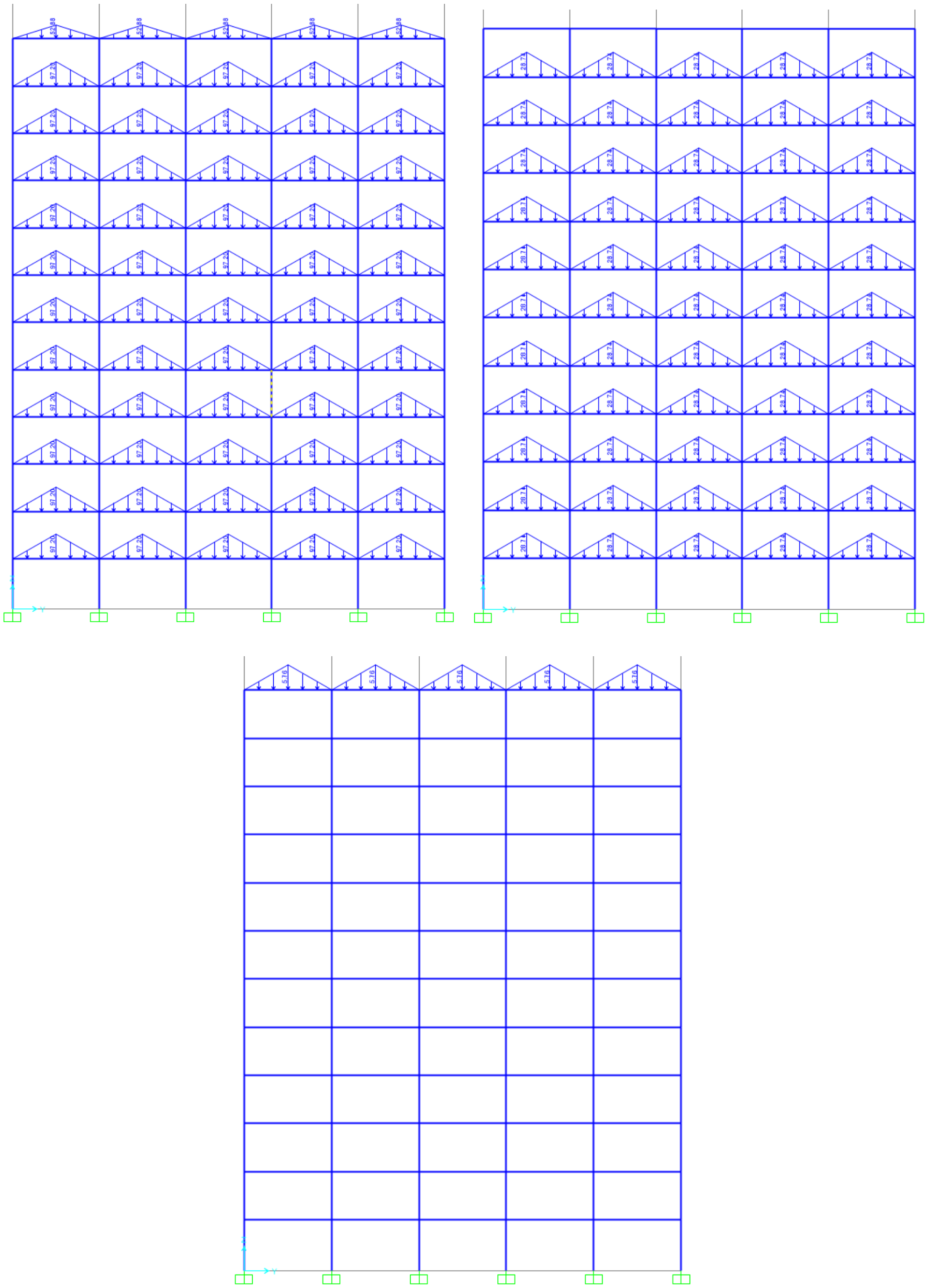


Figure 3.7 2D Load Assignment for Dead, Live and Roof Live Loads respectively

Analysis of 2D Loads Assignment is available in Appendix.

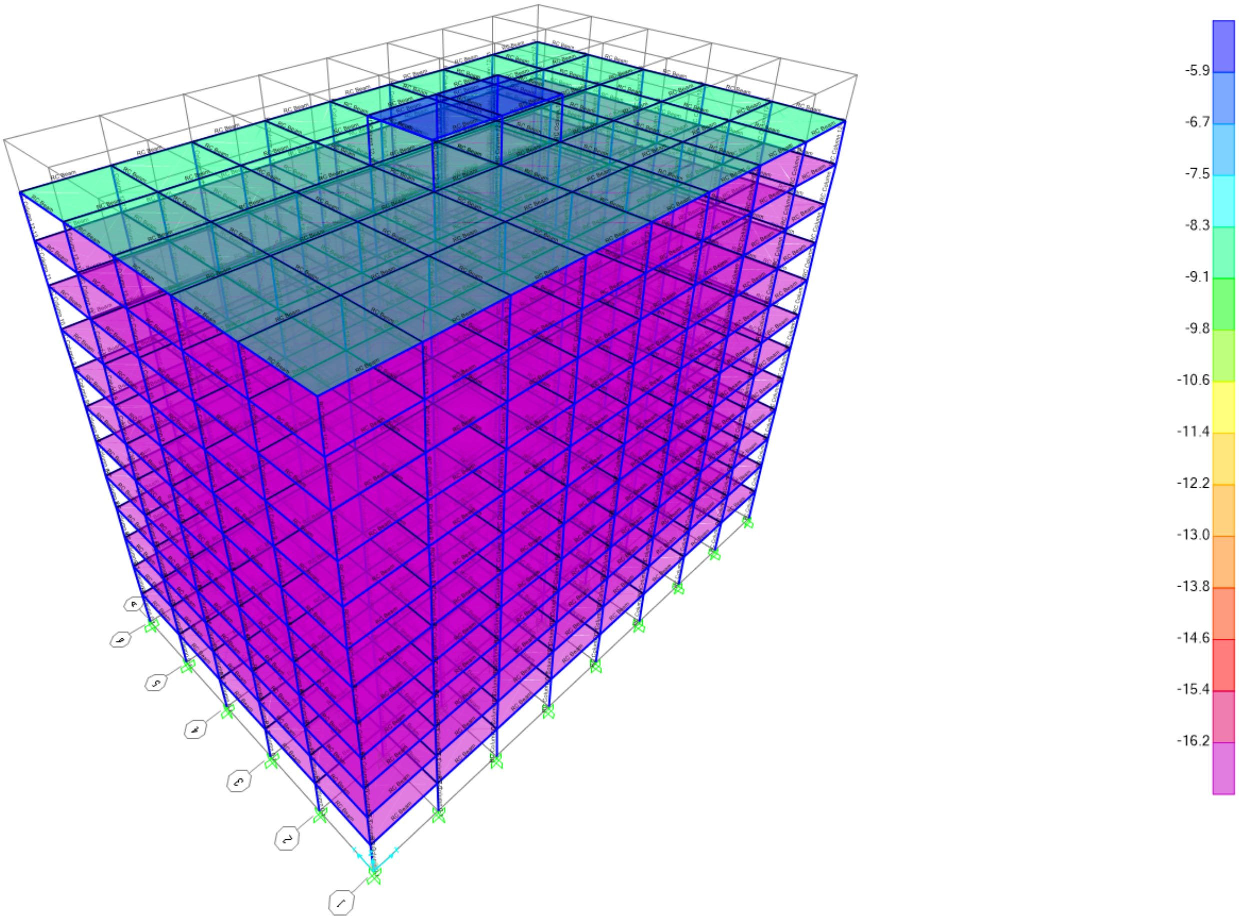


Figure 3.8 3D Uniform Dead load distributed to Area (Slab)

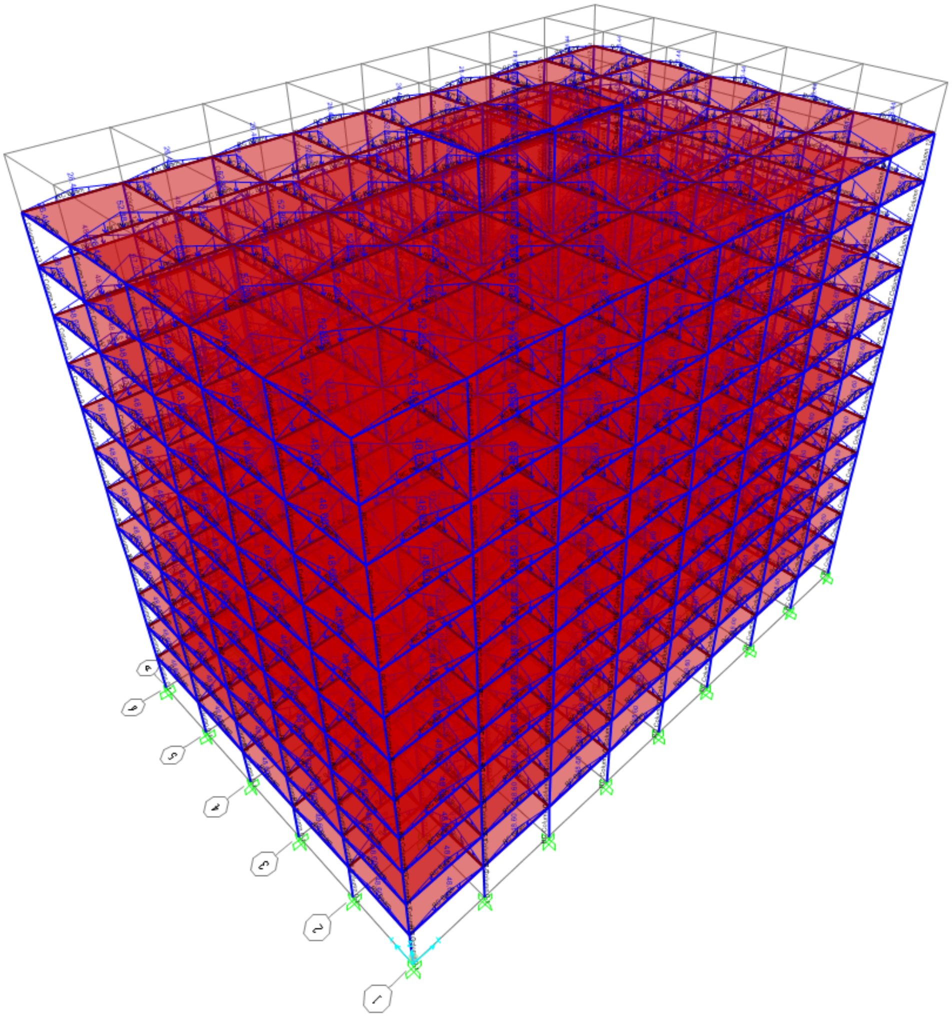


Figure 3.9. 3D Uniform Dead load distributed to Frames

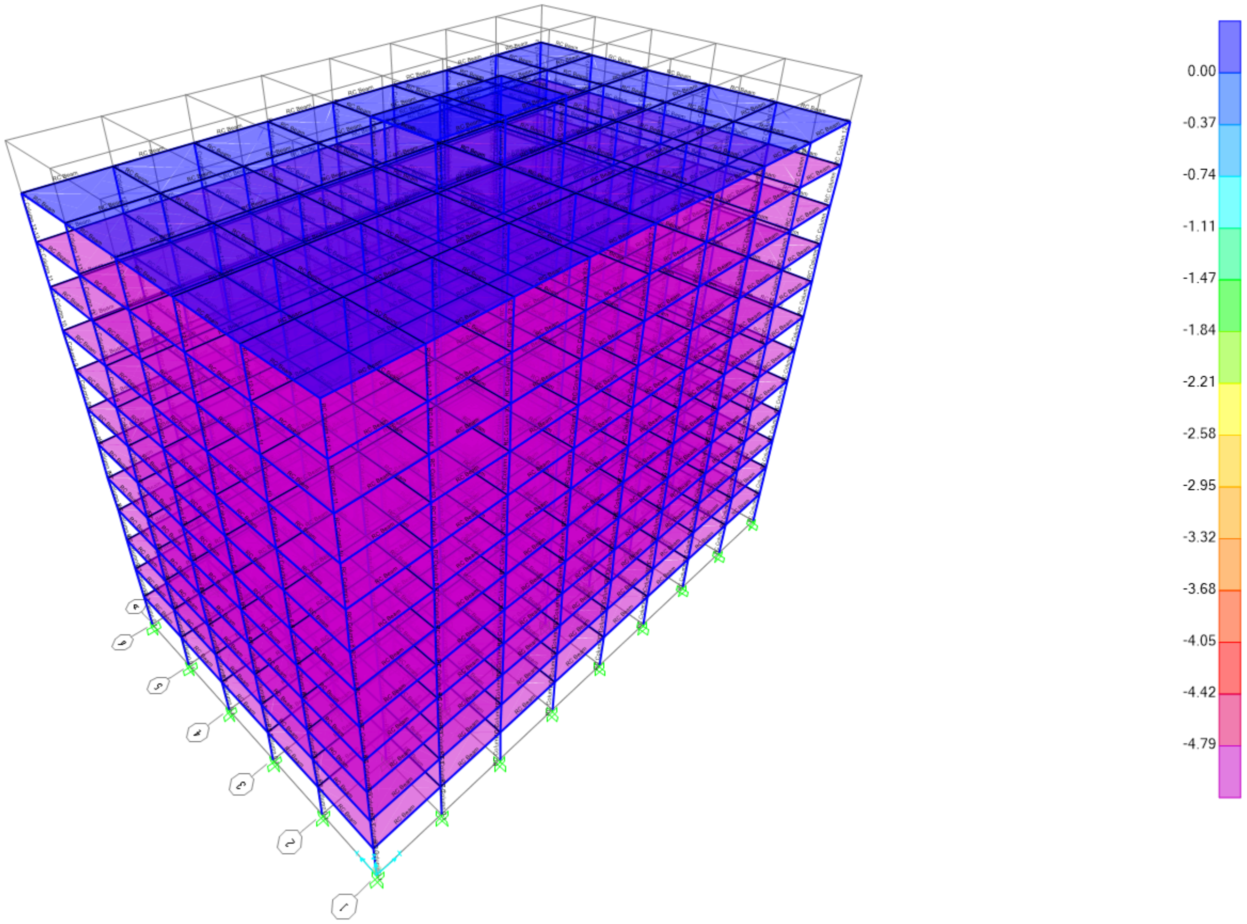


Figure 3.10. 3D Uniform Live load distributed to Area (Slab)

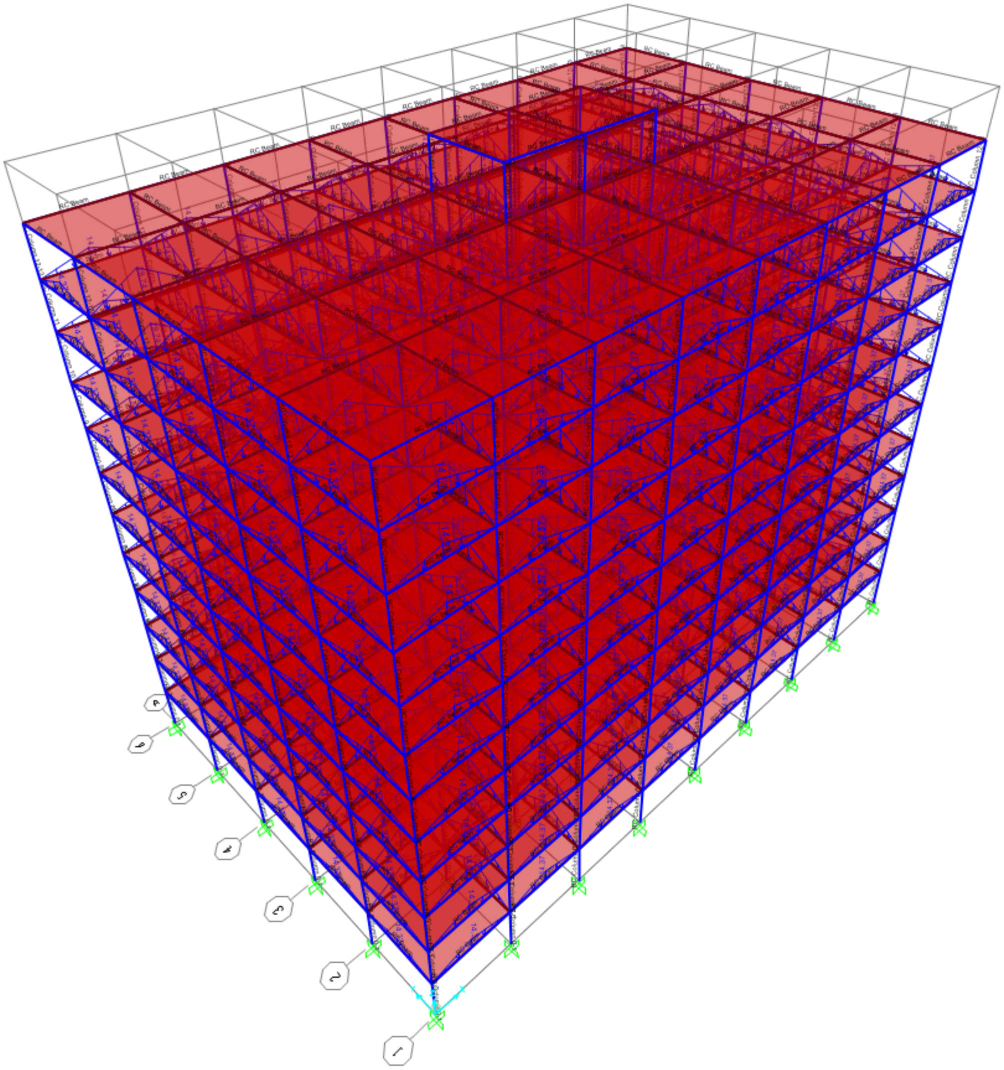


Figure 3.11. 3D Uniform Live load distributed to Frames

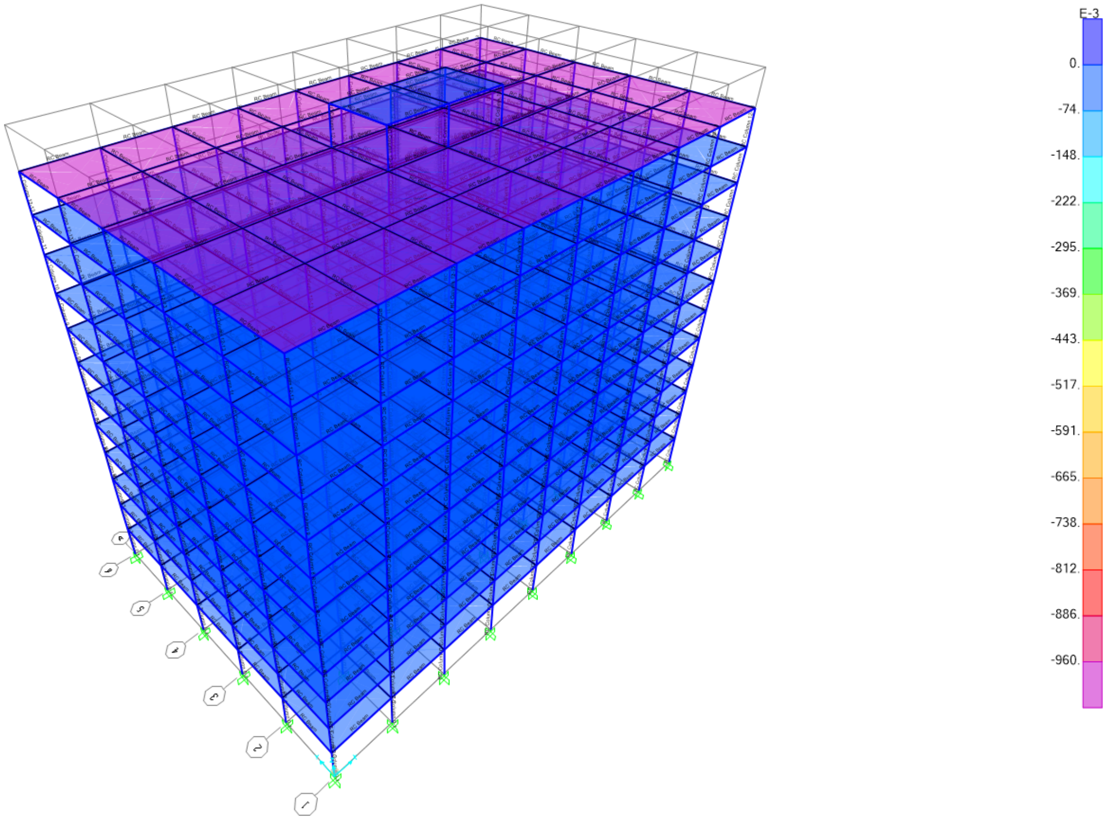


Figure 3.12. 3D Uniform Roof Live load distributed to Area (Slab)

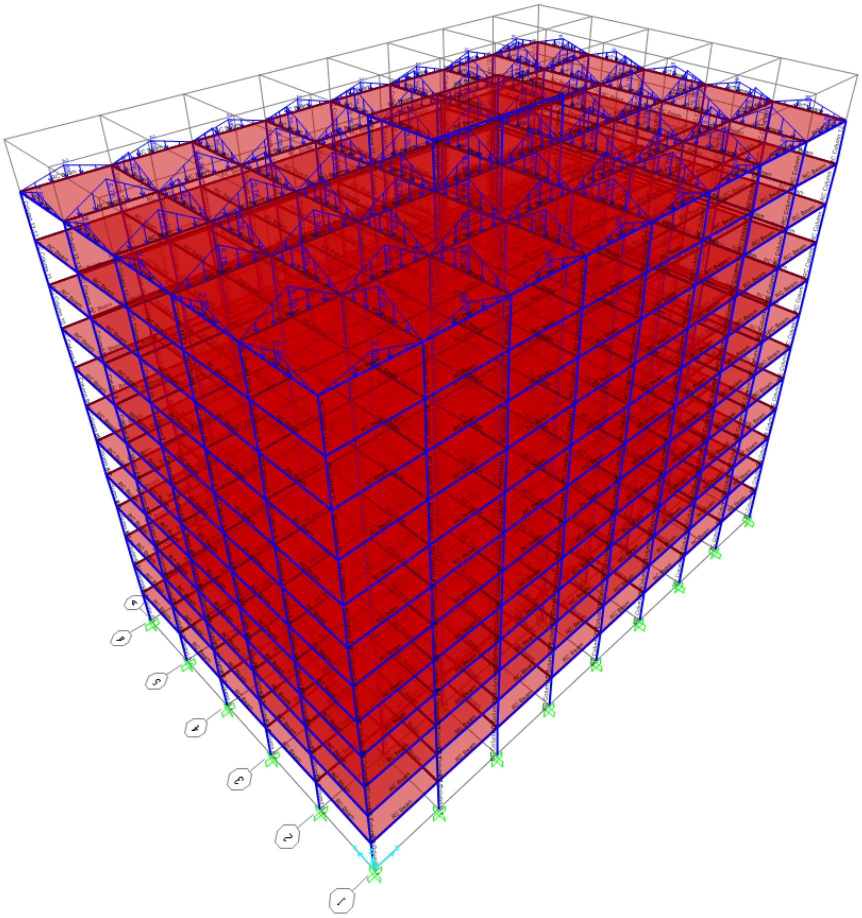


Figure 3.13. 3D Uniform Roof Live load distributed to Frames

3.4 Analysis and Design of Lateral Force Resisting System (LFRS)

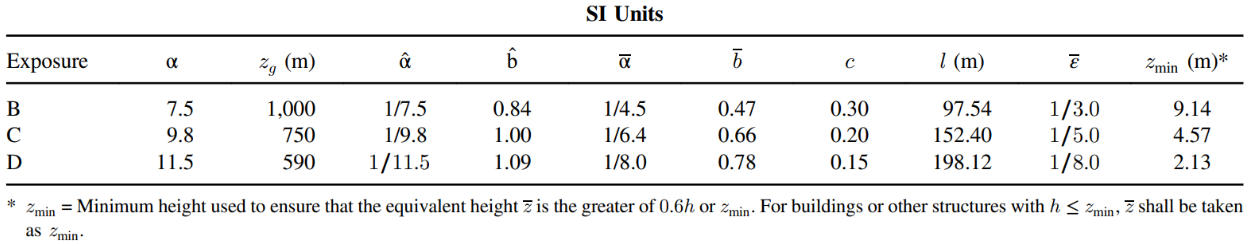
3.4.1. Wind load with torsional effect

As with GLRS calculations, wind load computations were based on the guidelines provided in ASCE 7-22 (American Society of Civil Engineers, 2022). To determine the basic wind speed in the Mission Bay area, San Francsico, California, ASCE Hazard Tool map was implemented (ASCE Hazard Tool, 2024). According to the data given in the tool, the basic wind speed in the chosen site is 41 m/s. The following data necessary for wind load calculation was taken from ASCE 7-22:

- From Table 26.6-1 Wind Directionality Factor, K_d, for Buildings with Main Wind Force Resisting System equal to 0.85.
- From Section 26.7.3, the Exposure Category for the project is B.
- Topographical factor, K_{zt} is 1 as the construction site is flat.

To define the Exposure Category B constants, data from Table X.X must be applied.

Table 3.23. Exposure Category Constants



SI Units

Exposure	α	z_g (m)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (m)	$\bar{\varepsilon}$	z_{min} (m)*
B	7.5	1,000	1/7.5	0.84	1/4.5	0.47	0.30	97.54	1/3.0	9.14
C	9.8	750	1/9.8	1.00	1/6.4	0.66	0.20	152.40	1/5.0	4.57
D	11.5	590	1/11.5	1.09	1/8.0	0.78	0.15	198.12	1/8.0	2.13

* z_{min} = Minimum height used to ensure that the equivalent height $\bar{z}$ is the greater of $0.6h$ or z_{min}. For buildings or other structures with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min}.

For Height Effect, the velocity pressure exposure coefficient, K_s, should be determined:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}, \text{ for } z < 15 \text{ ft} \tag{3.6}$$

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}, \text{ for } 15 \text{ ft} \leq z \leq z_g \tag{3.7}$$

As can be seen from the equations above, the velocity pressure value depends on the height of the level. Therefore, Table X.X below was created applying the formulas:

Table 3.24. Height Effect Coefficient Calculations

Story	h (m)	z (m)	Kz
13	3.3	41.6	1.080
12	3.3	39.8	1.067
11	3.3	36.5	1.040
10	3.3	33.2	1.013
9	3.3	29.9	0.983
8	3.3	26.6	0.951
7	3.3	23.3	0.915
6	3.3	20	0.876
5	3.3	16.7	0.832
4	3.3	13.4	0.781
3	3.3	10.1	0.721
2	3.3	6.8	0.644
1	3.5	3.5	0.575

Now, the Gust Effect for both longitudinal and transverse directions must be considered to calculate the wind loads.

Longitudinal Direction: $B = 48\, m,\ L = 30\, m,\ \#\, of\ frames = 9$

Transverse Direction: $B = 30\, m,\ L = 48\, m,\ \#\, of\ frames = 6$

Longitudinal Direction, Case 1 (Y-Axis):

Firstly, the type of the building must be determined to define the calculation procedure. As the total height of the building is 43.1, it is more than 18 m, indicating that the building is "high-rise." Thus, the Gust Effect procedure is as follows:

1) $n_a = 43.5/h^{0.9} = 43.5/136.483^{0.9} = 0.521$ < 1 Hz → Flexible Building

To clearly define the building type, equation X.X was used and as a result, it was identified that the building is flexible.

2) $\bar{z} = 0.6h = 0.6 \cdot 41.6 = 24.96 > z_{min} = 9.14\, m$

As can be seen from equation X.X, $\bar{z}$ was computed and it does satisfy the minimum height criteria as 24.96 m is more than 9.14 m.

Now, the stps below closely follow the Gust Effect procedure:

3) $I_{\bar{z}} = c(\frac{10}{\bar{z}})^{\frac{1}{6}} = 0.3(\frac{10}{24.96})^{\frac{1}{6}} = 0.258$

4) $\bar{V}_{\bar{z}} = \bar{b}(\frac{\bar{z}}{10})^{\bar{\alpha}} V = 0.45(\frac{24.96}{10})^{0.25} \cdot 41 = 23.19\, m/s$

5) $n_1 = n_a = 0.373, L_{\bar{z}} = l(\frac{\bar{z}}{10})^{\bar{\epsilon}} = 97.54(\frac{24.96}{10})^{0.33} = 132.31$ m

6) $N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.521 \cdot 132.31}{23.19} = 2.973$

7) $R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = \frac{7.47 \cdot 2.973}{(1+10.3 \cdot 2.973)^{5/3}} = 0.0702$

8) $\eta_h = \frac{4.6 n_1 H}{\bar{V}_{\bar{z}}} = \frac{4.6 \cdot 0.521 \cdot 41.6}{23.19} = 4.300$

9) $R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}) = \frac{1}{4.3} - \frac{1}{2*4.3^2}(1 - e^{-2*4.3}) = 0.206$

10) $\eta_b = \frac{4.6 n_1 B}{\overline{V}_{\bar{z}}} = \frac{4.6 \cdot 0.521 \cdot 48}{23.19} = 3.905$

$R_b = 0.181$

11) $\eta_L = \frac{15.4 n_1 L}{\overline{V}_{\bar{z}}} = \frac{15.4 \cdot 0.521 \cdot 30}{23.19} = 4.14$

12) $R_L = 0.0917$

13) $R = \sqrt{\frac{R_n R_h R_b (0.53 + 0.47 R_L)}{\beta}} = \sqrt{\frac{0.0702 \cdot 0.206 \cdot 0.181 \cdot (0.53 + 0.47 \cdot 0.0917)}{0.02}} = 0.274$

14) $g_R = \sqrt{2ln(3600 n_1)} + \frac{0.577}{\sqrt{2ln(3600 n_1)}} = \sqrt{2ln(3600 \cdot 0.521)} + \frac{0.577}{\sqrt{2ln(3600 \cdot 0.521)}} = 4.031$

$g_v = g_Q = 3.4$

15) $Q = \sqrt{\frac{1}{1 + 0.63 (\frac{B+h}{L_{\bar{z}}})^{0.63}}} = \sqrt{\frac{1}{1 + 0.63 (\frac{48+41.6}{132.31})^{0.63}}} = 0.818$

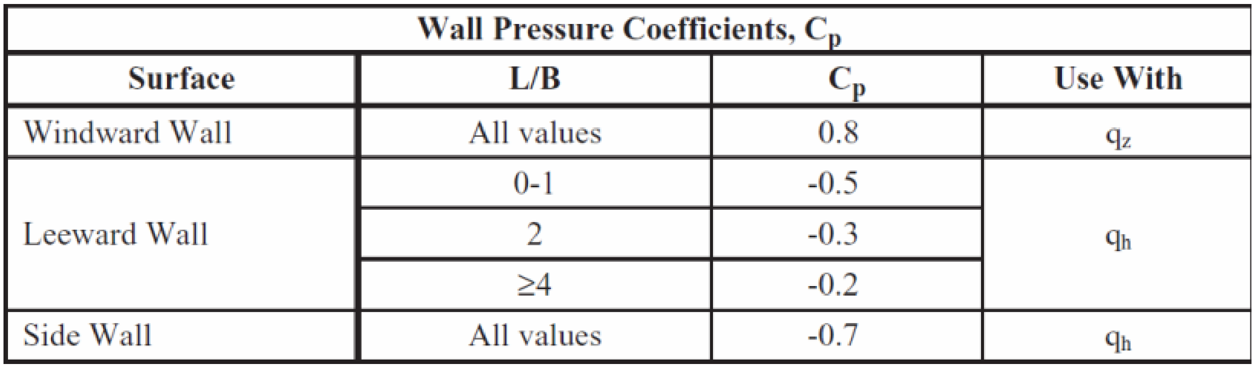
16) $G_f = 0.925 (\frac{1 + 1.7 I_z \sqrt{g_Q{}^2 Q^2 + g_R{}^2 R^2}}{1 + 1.7 g_v I_z}) = 0.925 (\frac{1 + 1.7 \cdot 0.258 \sqrt{3.4^2 \cdot 0.818^2 + 4.031^2 \cdot 0.274^2}}{1 + 1.7 \cdot 3.4 \cdot 0.258}) = 0.859$

For the external pressure coefficient, the relation between length and width of the building needs to be determined:

$$\frac{L}{B} = \frac{30}{48} = 0.625 < 1$$

Now, based on this value, external pressure coefficients, C_p can be obtained from Table 3.25 below:

Table 3.25. External Pressure Coefficients:



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

Thus, from Table 3.25: C_p for Windward Wall is 0.8 regardless of the relation value, while for the Leeward Wall C_p is -0.5 as the relation value of 0.625 falls into the range of 0-1.

Transverse Direction, Case 1 (X-Axis)

The entire calculation procedure is the same for the transverse direction with the exception of the L/B relation. Therefore:

$$G_f = 0.925(\frac{1+1.7I_z\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_z}) = 0.883$$

As for the L/B relation:

$$\frac{L}{B} = \frac{48}{30} = 1.6 > 1$$

Thus, by applying the values from Table 3.25 and linear interpolation, the external pressure coefficients are as follows:

- $C_{p, windward} = 0.8$
- $C_{p, leeward} =- 0.38$

Now, moving on to the wind load calculations, according to ASCE 7-22, there are four different cases for Wind Load shown in Figure 3.14 below.

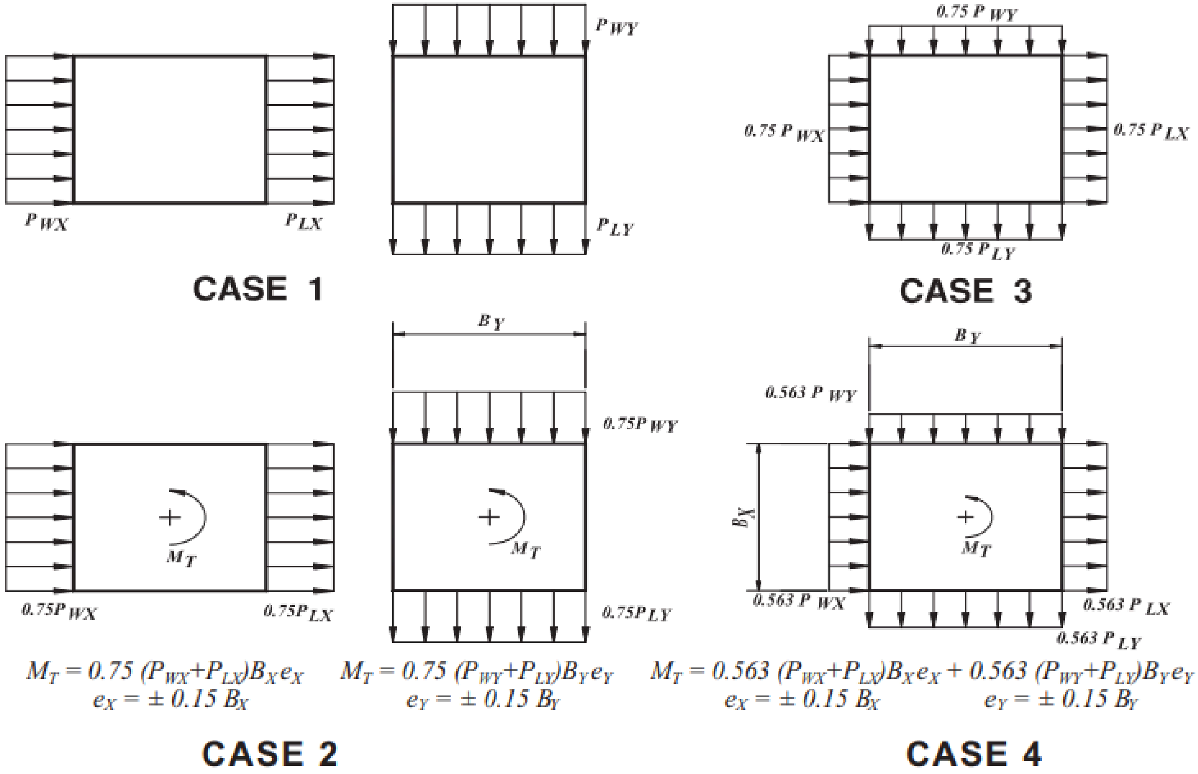


Figure 3.14. Wind Load Cases (American Society of Civil Engineers, 2022)

To determine the wind forces acting on each floor, velocity pressure and wind pressure expressions are necessary:

$$q_z = 0.613 K_z K_{zt} K_d V^2 \tag{3.8}$$

$$P = q_z G C_{p,windward} - q_h G C_{p,leeward} \tag{3.9}$$

For the calculation of the force at each level, except for top floor, the following equation is to be used:

$$F_i = \frac{B}{\#\,of\,frames} * \left(P_{i+1} * \frac{h_{i+1}}{2} + P_i * \frac{h_i}{2}\right) \tag{3.10}$$

For calculation of wind load on top floor, the equation below must be implemented:

$$F_{13} = \frac{B}{\#\,of\,frames} * \frac{h_{13}}{2} * P_{13} \tag{3.11}$$

As can be seen from Figure 3.4.1.1, the main difference between Cases 1, 3 and Cases 2, 4 is the presence of the torsional effect. For torsional effect calculation, the following expressions are necessary:

$$e_x = 0.15 \cdot B \tag{3.12}$$

$$e_x = 0.15 \cdot 48 = 7.2\, m$$

$$e_y = 0.15 \cdot L \tag{3.13}$$

$$e_y = 0.15 \cdot 30 = 4.5\, m$$

Now that all the necessary formulas are identified and all the features are taken into account, the following tables on each case of wind load can be made:

Table 3.26. Wind Forces calculated for Longitudinal Direction (Y-Axis), Case 1

Story	h (m)	Kz	q (Pa)	Gf	Pw, Pl (Pa)	P (Pa)	Fi (kN)	F per frame (kN)
13	3.3	1.080	946.02	0.859	650.00	1051.15	83.25	41.63
12	3.3	1.067	934.14	0.859	641.84	1042.99	165.86	18.43
11	3.3	1.040	911.32	0.859	626.16	1027.31	163.97	18.22
10	3.3	1.013	886.98	0.859	609.44	1010.58	161.40	17.93
9	3.3	0.983	860.84	0.859	591.48	992.63	158.65	17.63
8	3.3	0.951	832.55	0.859	572.04	973.19	155.69	17.30
7	3.3	0.915	801.63	0.859	550.79	951.94	152.47	16.94
6	3.3	0.876	767.40	0.859	527.28	928.43	148.93	16.55
5	3.3	0.832	728.87	0.859	500.80	901.95	144.97	16.11
4	3.3	0.781	684.43	0.859	470.27	871.42	140.45	15.61
3	3.3	0.721	631.32	0.859	433.78	834.93	135.14	15.02

2	3.3	0.644	563.85	0.859	387.41	788.56	128.58	14.29
1	3.5	0.807	706.85	0.859	485.67	886.82	136.95	15.22

Table 3.27. Wind Forces calculated for Transverse Direction (X-Axis), Case 1

Story	h (m)	Kz	q (Pa)	Gf	Pw, Pl (Pa)	P (Pa)	Fi (kN)	F per frame (kN)
13	3.3	1.080	946.02	0.883	668.27	981.71	48.59	24.30
12	3.3	1.067	934.14	0.883	659.87	973.32	96.77	16.13
11	3.3	1.040	911.32	0.883	643.76	957.20	95.56	15.93
10	3.3	1.013	886.98	0.883	626.56	940.00	93.91	15.65
9	3.3	0.983	860.84	0.883	608.10	921.54	92.15	15.36
8	3.3	0.951	832.55	0.883	588.11	901.55	90.24	15.04
7	3.3	0.915	801.63	0.883	566.27	879.71	88.17	14.70
6	3.3	0.876	767.40	0.883	542.09	855.53	85.89	14.32
5	3.3	0.832	728.87	0.883	514.87	828.31	83.35	13.89
4	3.3	0.781	684.43	0.883	483.48	796.92	80.45	13.41
3	3.3	0.721	631.32	0.883	445.97	770.00	77.56	12.93
2	3.3	0.644	563.85	0.883	398.30	770.00	76.23	12.71
1	3.5	0.807	706.85	0.883	499.32	812.76	80.78	13.46

Table 3.28. Direct Wind Forces Component calculated for Case 2 (Y-axis).

Story	Pw, Pl (Pa)	M_T (kN-m)	0.75Fi (kN)	F direct (kN)
13	650.00	272.46	62.44	31.22

12	641.84	270.34	124.39	13.82
11	626.16	266.28	122.98	13.66
10	609.44	261.94	121.05	13.45
9	591.48	257.29	118.99	13.22
8	572.04	252.25	116.77	12.97
7	550.79	246.74	114.35	12.71
6	527.28	240.65	111.69	12.41
5	500.80	233.79	108.72	12.08
4	470.27	225.87	105.34	11.70
3	433.78	216.41	101.36	11.26
2	387.41	204.40	96.44	10.72
1	485.67	229.86	102.71	11.41

Table 3.29. Direct Wind Forces Component calculated for Case 2 (X-axis).

Story	Pw, Pl (Pa)	MT (kN-m)	0.75Fi (kN)	Fdirect (kN)
13	668.27	99.40	36.45	18.22
12	659.87	98.55	72.58	12.10
11	643.76	96.92	71.67	11.95
10	626.56	95.18	70.43	11.74
9	608.10	93.31	69.11	11.52
8	588.11	91.28	67.68	11.28
7	566.27	89.07	66.13	11.02
6	542.09	86.62	64.42	10.74
5	514.87	83.87	62.51	10.42
4	483.48	80.69	60.34	10.06
3	445.97	76.89	58.17	9.70
2	398.30	72.06	57.17	9.53
1	499.32	82.29	60.59	10.10

To clearly demonstrate the torsional effect of wind load, separate Table 3.30 was created for Case 2 in both longitudinal and transverse directions.

Table 3.30. Wind Force Calculations for third floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	0.5825	9.70	10.28	1	-1.6395	11.26	9.62
B	0.3495	9.70	10.04	2	-1.2296	11.26	10.03
C	0.1165	9.70	9.81	3	-0.8197	11.26	10.44
D	-0.1165	9.70	9.58	4	-0.4099	11.26	10.85
E	-0.3495	9.70	9.35	5	0.0000	11.26	11.26
F	-0.5825	9.70	9.11	6	0.4099	11.26	11.67
				7	0.8197	11.26	12.08
				8	1.2296	11.26	12.49
				9	1.6395	11.26	12.90

All other Tables for the rest floors are included in Appendix. The same calculation procedure was performed for Case 3 and Case 4 in the following Tables 3.31 below:

Table 3.31. Wind Force Calculations for Case 3

Story	**0.75F1 (kN)**	**F1 per frame (kN)**	**0.75F2 (kN)**	**F2 per frame (kN)**
13	62.44	31.22	36.45	18.22
12	124.39	13.82	72.58	12.10
11	122.98	13.66	71.67	11.95
10	121.05	13.45	70.43	11.74
9	118.99	13.22	69.11	11.52
8	116.77	12.97	67.68	11.28
7	114.35	12.71	66.13	11.02
6	111.69	12.41	64.42	10.74
5	108.72	12.08	62.51	10.42
4	105.34	11.70	60.34	10.06

3	101.36	11.26	58.17	9.70
2	96.44	10.72	57.17	9.53
1	102.71	11.41	60.59	10.10

Table 3.32. Wind Force Calculations for Case 4

Story	MT (kN-m)	0.563F1 (kN)	0.563F2 (kN)	F1 direct (kN)	F2 direct (kN)
13	279.14	46.87	27.36	23.44	13.68
12	276.91	93.38	54.48	10.38	9.08
11	272.64	92.31	53.80	10.26	8.97
10	268.08	90.87	52.87	10.10	8.81
9	263.18	89.32	51.88	9.92	8.65
8	257.88	87.65	50.81	9.74	8.47
7	252.09	85.84	49.64	9.54	8.27
6	245.67	83.85	48.36	9.32	8.06
5	238.45	81.62	46.93	9.07	7.82
4	230.12	79.07	45.29	8.79	7.55
3	220.17	76.09	43.67	8.45	7.28
2	207.53	72.39	42.92	8.04	7.15
1	234.33	77.10	45.48	8.57	7.58

As was mentioned above, all other Tables are included in Appendix. Below, Table X was created to demonstrate total wind forces including both direct and torsional effect for Case in both directions.

Table 3.33. Wind Force Calculations for third floor in both directions including the torsional effect (Case 4)

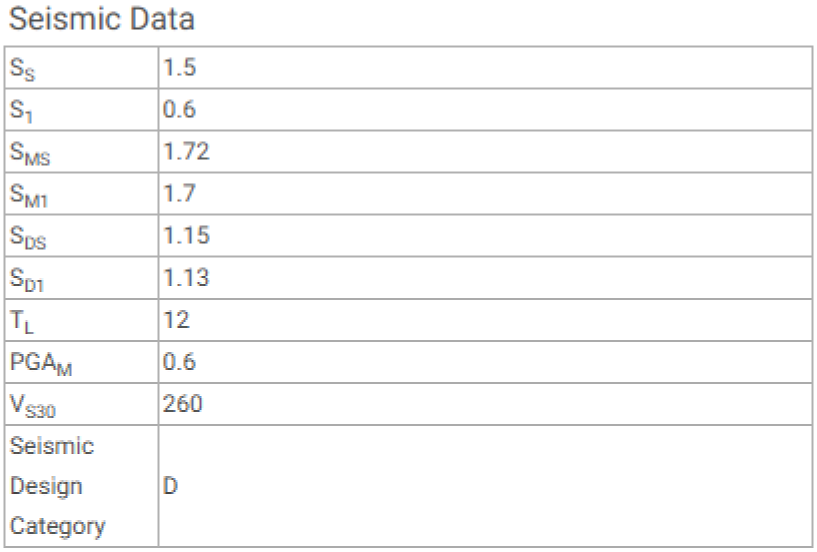
Transverse				Longitudinal			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	1.7452	7.28	9.02	1	-1.7452	8.45	6.71
B	1.0471	7.28	8.33	2	-1.3089	8.45	7.15

C	0.3490	7.28	7.63	3	-0.8726	8.45	7.58
D	-0.3490	7.28	6.93	4	-0.4363	8.45	8.02
E	-1.0471	7.28	6.23	5	0.0000	8.45	8.45
F	-1.7452	7.28	5.53	6	0.4363	8.45	8.89
				7	0.8726	8.45	9.33
				8	1.3089	8.45	9.76
				9	1.7452	8.45	10.20

3.4.2 Calculation of Seismic Loads

Equivalent Lateral Force method or ELF is the main calculating procedure for determining the seismic loads (American Society of Civil Engineers, 2022).

The site soil class must be identified first to start the ELF method. From the Geotechnical Investigation Report, it was determined that the soil class is D or stiff soil [REFERENCE]. Now, by using the ASCE Hazard tool the mapped acceleration parameters S_s and S_1, and period T_L can be found (ASCE Hazard Tool, 2024). Figure 26 below represents the findings:



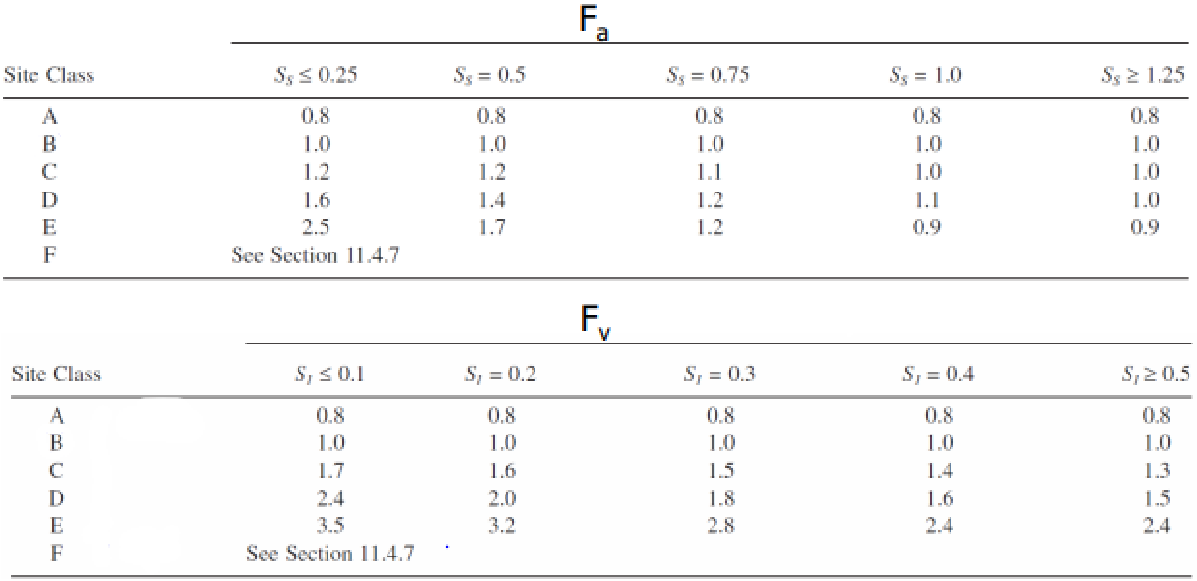
Seismic Data

S_S	1.5
S_1	0.6
S_{MS}	1.72
S_{M1}	1.7
S_{DS}	1.15
S_{D1}	1.13
T_L	12
PGA_M	0.6
V_{S30}	260
Seismic Design Category	D

Figure 3.15. ASCE Hazard Tool Seismic Design Data

From the tool: $S_s = 1.5\ g, S_1 = 0.6\ g, T_L = 12\ s$

Having identified mapped acceleration data, the next step is to determine soil condition correction coefficients from Figure 27 below:



F_a

Site Class	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
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				

F_v

Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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 3.16. F_a, F_v - Site Class Table

Thus, from Figure X: $F_a = 1.0, F_v = 1.5$

Now to compute the adjusted parameters S_{MS} and S_{M1} taking into account soil condition correction coefficients, the following formulas are used:

$$S_{MS} = F_a S_s \tag{3.14}$$

$$S_{M1} = F_v S_1 \tag{3.15}$$

$$S_{MS} = 1.0 \cdot 1.5 = 1.5\, g$$

$$S_{M1} = 1.5 \cdot 0.6 = 0.90\, g$$

Afterwards, adjustments based on design acceleration parameters are necessary, and thus:

$$S_{DS} = \frac{2}{3} S_{MS} \tag{3.16}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{3.17}$$

$$S_{DS} = \frac{2}{3} \cdot 1.5 = 1.0\, g$$

$$S_{D1} = \frac{2}{3} \cdot 0.90 = 0.60\, g$$

Having calculated all the necessary seismic coefficients, the Seismic Design Category for the construction project can be identified. Figure X below clearly depicts how exactly Seismic Design Category can be determined.

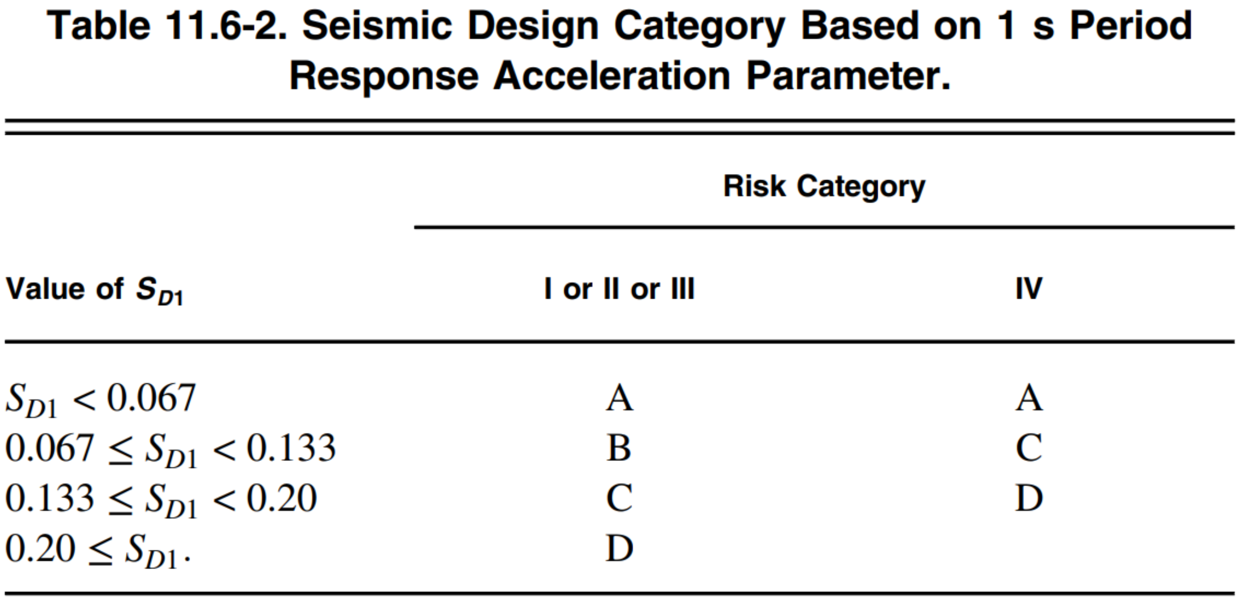


Table 11.6-2. Seismic Design Category Based on 1 s Period Response Acceleration Parameter.

	Risk Category	
Value of S_{D1}	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	

Figure 3.17. Seismic Design Category Table (American Society of Civil Engineers, 2017)

From Figure 3.17, Seismic Design Category for the construction project is D as one-second Period Response Acceleration Parameter is equal to 0.6 and the Risk Category for the building is II.

The next step is to determine the Design Coefficients and Factors for Seismic Force-Resisting System. Figure 28 below shows these factors for Moment-Resisting Frame Systems that are to be used for the construction of this building.

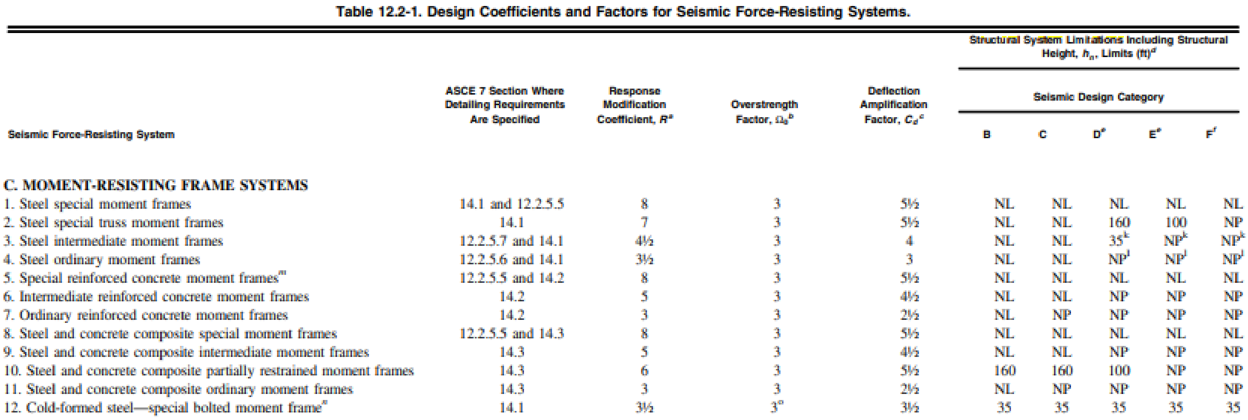


Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems.

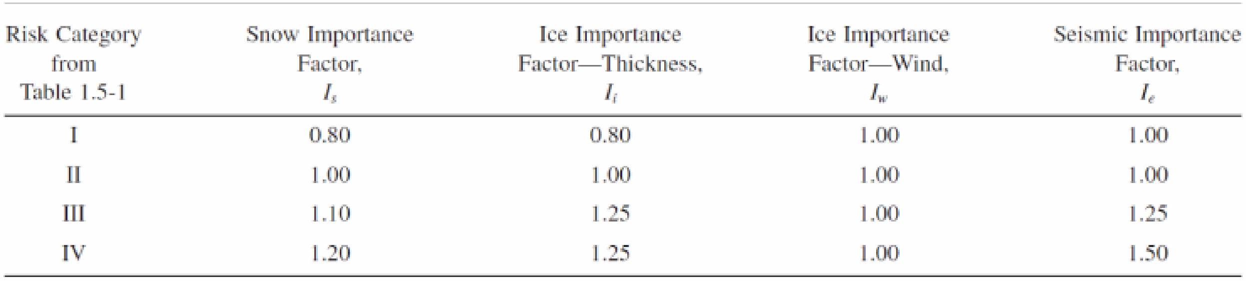
					Structural System Limitations Including Structural Height, h_n, Limits (ft)^{d}				
					Seismic Design Category				
Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^{a}	Overstrength Factor, $\Omega_0{}^{b}$	Deflection Amplification Factor, $C_d{}^{c}$	B	C	D^{e}	E^{e}	F^{f}
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^{k}	NP^{k}	NP^{k}
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP^{l}	NP^{l}	NP^{l}
5. Special reinforced concrete moment frames^{m}	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame^{n}	14.1	3½	3^{o}	3½	35	35	35	35	35

Figure 3.18. Design Coefficients and Factors for Seismic Force-Resisting System (American Society of Civil Engineers, 2017)

Therefore, from Figure 3.18 above, as the building is going to be constructed implementing reinforced concrete moment frames with no steel frame elements, Special Reinforced Concrete Moment Frame was chosen, and so the factors are:

- Response modification coefficient, R is equal to 8
- Overstrength factor, Ω is equal to 3
- Design amplification factor, C_d is equal to 5.5

For the Importance Factor, I_e, the following Figure 29 must be used:



Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

Figure 3.19. Importance Factor Table (American Society of Civil Engineers, 2022)

Based on the data from Figure 3.19 Importance Factor, I_e is equal to 1.0.

Now for the design spectrum periods:

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} \tag{3.18}$$

$$T_0 = 0.2\frac{0.60}{1.00} = 0.12\ s$$

$$T_s = \frac{S_{D1}}{S_{DS}} \tag{3.19}$$

$$T_s = \frac{0.60}{1.00} = 0.60\ s$$

Approximate fundamental period must be computed next:

$$T_a = C^t h_n^{\ x} \tag{3.20}$$

Where:

T_a - approximate fundamental period

C^t and x - approximate period parameters defined in Table 12.8-2 (ASCE 7-22, 2022)

h_n - building height above the ground to its highest point ($h_n = 43.1\ m$)

From Table 12.8-2 in ASCE 7-22, for concrete moment-resisting frames, $C^t = 0.0466,\ x = 0.9$ (American Society of Civil Engineers, 2022). As a result approximate fundamental period can be calculated:

$$T_a = 0.0466 \cdot 43.1^{0.9} = 1.379\,s$$

Now, fundamental period can also be calculated applying analytic method, and so it can be found using the expression below:

$$T = C_u T_a \tag{3.21}$$

Where,

C_u - coefficient for upper limit on calculated period demonstrated in Table 12.8-1 (American Society of Civil Engineers, 2022). C_u is 1.4 for this construction project.

$$T = 1.4 \cdot 1.379 = 1.930\,s$$

To obtain T *software modal analysis like SAP 2000 is necessary. However, as it was not performed, T * will be taken as equal to $T_a = 1.379\,s$.

Next step is to obtain base shear implementing this equation:

$$V = C_s W \tag{3.22}$$

Where,

C_s - seismic response coefficient

W - effective seismic weight

Calculation of seismic response coefficient requires multiple steps. Firstly, the minimum value for C_s needs to be determined, and for that one of the following equations can be used:

$$C_{s,min} = \frac{0.5S_1}{R/I_e} \text{ for } S_1 \geq 0.6\text{ g}$$

$$C_{s,min} = MAX[0.044S_{DS} * I_e, 0.01] \text{ for } S_1 < 0.6\text{ g}$$

Since S_1 is exactly 0.6 equation XX must be implemented:

$$C_{s,min} = \frac{0.5S_1}{R/I_e} = \frac{0.5*0.6}{8/1} = 0.0375$$

Now for the seismic response coefficient the following expression are utilized

$$C_s = \frac{S_{DS}}{(\frac{R}{I_e})} \tag{3.23}$$

$$C_s = \frac{1.0}{(\frac{8}{1})} = 0.125$$

The maximum limits for C_s are as follows:

$$C_s = \frac{S_{D1}}{T(\frac{R}{I_e})} \quad \text{for } T \leq T_L \tag{3.24}$$

$$C_s = \frac{0.6}{1.930(\frac{8}{1})} = 0.039$$

And now, the minimum value resulting from the equations (3.18) must be taken, which is 0.039. As we calculated the minimum seismic response value, 0.039 should not be less than 0.0375. Since this requirement is satisfied, C_s is equal to 0.039.

Lastly for the base shear, effective seismic weight must be found. For that the following Table 3.34 was created, where the weight of floor slab, staircase, columns, beams, and walls was taken into account.

Table 3.34. Effective Seismic Weight.

Floors	Slab and floor finishing, kN/m²	Columns, kN	Beams, kN	Exterior/ partition walls, kN/m²	Stairs, kN	Floor weight, kN
1	5.874	2657.81	2511.00	3.331	244.952	18503.0
2	5.874	2182.95	2511.00	3.331	244.952	18028.1
3	5.874	2182.95	2511.00	3.331	244.952	18028.1
4	5.874	1882.24	2511.00	3.331	244.952	17727.4
5	5.874	1882.24	2511.00	3.331	244.952	17727.4
6	5.874	1603.80	2511.00	3.331	244.952	17449.0
7	5.874	1603.80	2511.00	3.331	244.952	17449.0
8	5.874	1347.64	2511.00	3.331	244.952	17192.8
9	5.874	1113.75	2511.00	3.331	244.952	16958.9
10	5.874	902.14	2511.00	3.331	244.952	16747.3
11	5.874	712.80	2511.00	3.331	244.952	16558.0
12	5.874	545.74	2511.00	3.331	244.952	16390.9

13	0	545.74	2511.00	3.331	122.476	7915.8
Roof	5.13			0.281		7694.3
					Effective seismic weight	**224370**

Finally base shear can be computed:

$$V = C_s W = 0.039 \cdot 224370 = 8719\, kN$$

Distribution in the vertical direction must be determined applying the formula below:

$$F_x = C_{vx} V \tag{3.25}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{3.26}$$

Where,

C_{vx} - vertical distribution factor

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

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

k - an exponent related to the structure period which differs as follows:

- k = 1, if T ≤ 0.5 s
- k = 2, if T ≥ 2.5 s
- T between 0.5 s and 2.5 s can be linearly interpolated as it is true for this cas

$$k = \frac{2-1}{2.5-0.5} \cdot 1.379 + 1 = 1.439$$

Now that everything necessary to calculate the lateral seismic force was identified, Table 3.35 is created to demonstrate the seismic forces for each floor.

Table 3.35. Seismic force for each floor.

Floors	h_i, m	W_x, kN	$W_x h_x^k$	C_{vx}	F_x, kN
13	43.1	15610.1	3514384	0.159011	1386.46
12	39.8	16390.9	3290466	0.148879	1298.13
11	36.5	16558.0	2934670	0.132781	1157.76
10	33.2	16747.3	2589790	0.117177	1021.70

9	29.9	16958.9	2255688	0.102060	889.89
8	26.6	17192.8	1932538	0.087439	762.41
7	23.3	17449.0	1620900	0.073339	639.46
6	20	17449.0	1301055	0.058867	513.28
5	16.7	17727.4	1019665	0.046135	402.27
4	13.4	17727.4	742757	0.033607	293.03
3	10.1	18028.1	502845	0.022752	198.38
2	6.8	18028.1	284545	0.012874	112.26
1	3.5	18503.0	112279	0.005080	44.30
Total		205867.3	22101583		

3.4.2.1 Seismic Loads Including Torsional Effect

Torsional effect must also be considered for seismic loads as the structure of the building allows for moment and torsion actions.

Mass center must be determined first to start further calculations. The construction project is 48 meters in length and 30 meters in width, therefore the following is true

$x = 24\ m$ (as the building is symmetric in length)

$$y = \frac{A_{floor}y_{floor} - A_{elevator\ shaft}y_{elevator\ shaft} - A_{stairs\ shaft,\,1}y_{stairs\ shaft,\,1} - A_{stairs\ shaft,\,2}y_{stairs\ shaft,\,2}}{A_{floor} - A_{shafts}} \tag{3.27}$$

$$y = 14.97\ m$$

So, the mass center coordinates are x = 24 m, y = 14.97 m.

Next, the stiffness center must be calculated and for that critical moments of inertia should be identified first:

$$I_{b,cr} = \frac{1}{12}bh^3 \cdot 0.35 \tag{3.28}$$

$$I_{c,cr} = \frac{1}{12}bh^3 \cdot 0.7 \tag{3.29}$$

Afterwards, stiffness of the entire frame can be calculated:

$$D = \frac{12E}{h^2}\left(\frac{I_{c,cr} * I_{b,cr}}{L * I_{c,cr} + h * I_{b,cr}}\right) \tag{3.30}$$

And so as a result the following expression gives us required force for the inter-story rotation:

$$C_F = h \cdot D \cdot \#\ of\ columns\ per\ frame \cdot \#\ of\ frames \tag{3.31}$$

After that the stiffness center must be determined. The in-depth computations are provided in Appendix. After the calculations, stiffness center coordinates are x' = 24 m and y' = 15.96 m

For the eccentricity in X and Y axes:

$$e_x = 24 - 24 = 0\, m$$

$$e_a = 5\% \cdot L_x = 0.05 \cdot 48 = 2.4\, m$$

$$e_{xt} = e_x + e_a = 2.4\, m$$

$$e_y = 14.97 - 15.96 =- 0.99\, m$$

$$e_a = 5\% \cdot L = 0.05 \cdot 30 = 1.5\, m$$

$$e_{yt} = e_y + e_a = 0.51\, m$$

Thus, torsion, direct force, torsional force and total force can be found implementing the following list of equations:

$$T = eF \tag{3.32}$$

$$F_{direct} = \frac{F_x}{\#\, of\, frames} \tag{3.33}$$

$$F_{torsion} = \frac{T{*}C_f{*}(x_i\, or\, y_i)}{\sum C_f{*}(x_i^2\, or\, y_i^2)} \tag{3.34}$$

$$F_{total} = F_{direct} \pm F_{torsion} \tag{3.35}$$

Table 3.36 below represents horizontal and direct forces, and torsion in both transverse and longitudinal directions.

Table 3.36. Direct forces on each frame including torsion

		Longitudinal		**Transverse**	
Floors	**Fx, kN**	**F_{direct}, kN**	**T, kN-m**	**F_{direct}, kN**	**T, kN-m**
13	1386.46	231.08	710.36	154.05	3327.52
12	1298.13	216.35	665.10	144.24	3115.50
11	1157.76	192.96	593.19	128.64	2778.63
10	1021.70	170.28	523.48	113.52	2452.09

9	889.89	148.32	455.94	98.88	2135.75
8	762.41	127.07	390.62	84.71	1829.78
7	639.46	106.58	327.63	71.05	1534.71
6	513.28	85.55	262.98	57.03	1231.88
5	402.27	67.04	206.11	44.70	965.45
4	293.03	48.84	150.13	32.56	703.26
3	198.38	33.06	101.64	22.04	476.11
2	112.26	18.71	57.52	12.47	269.42
1	44.30	7.38	22.70	4.92	106.31

Now, Table 3.37 also includes the torsional effect and takes the third floor as an example as usual.

Table 3.37. Seismic load calculations for third floor including torsional effect

Transverse				**Longitudinal**			
Frame	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**	**Frame**	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**
A	0.7700	33.06	33.83	1	-3.6069	22.04	18.44
B	0.4620	33.06	33.53	2	-2.7052	22.04	19.34
C	0.1540	33.06	33.22	3	-1.8034	22.04	20.24
D	-0.1540	33.06	32.91	4	-0.9017	22.04	21.14
E	-0.4620	33.06	32.60	5	0.0000	22.04	22.04
F	-0.7700	33.06	32.29	6	0.9017	22.04	22.94
				7	1.8034	22.04	23.85
				8	2.7052	22.04	24.75
				9	3.6069	22.04	25.65

3.4.3 Assigning Lateral Forces to SAP 2000

The overall background properties and conditions stayed the same for this section as they were for Section 3.3. Therefore, no reiteration of properties for LFRS Assignment is necessary.

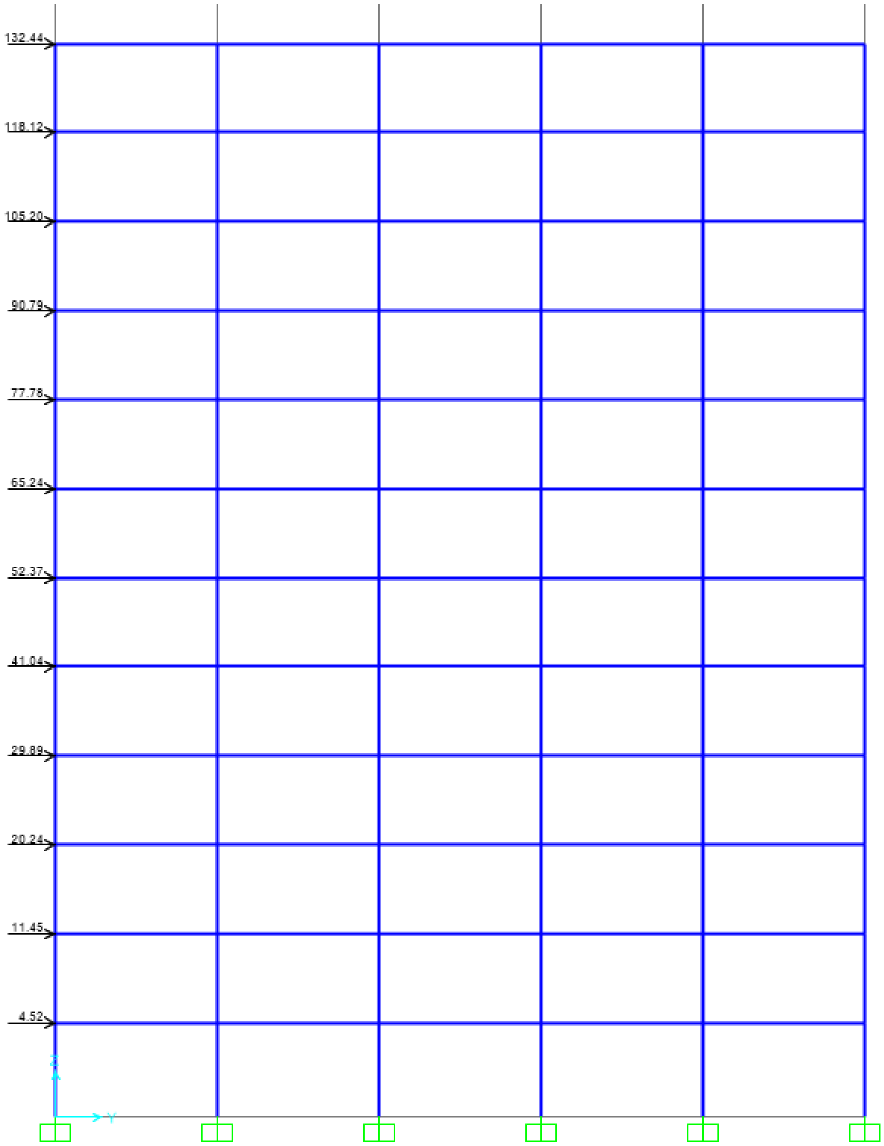


Figure 3.20. Seismic Load Assignment for Frame C

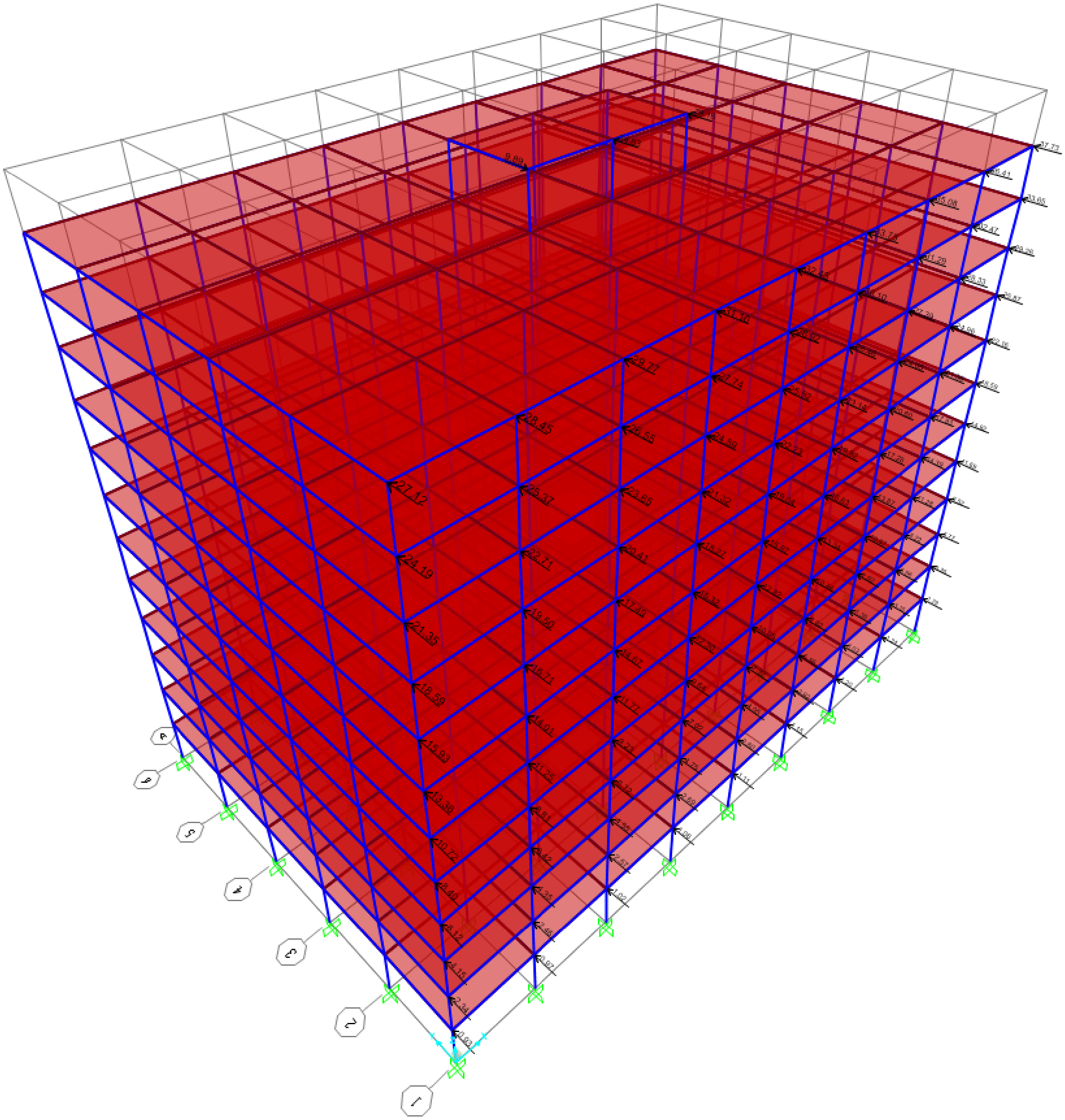


Figure 3.21. 3D Seismic Load Assignment

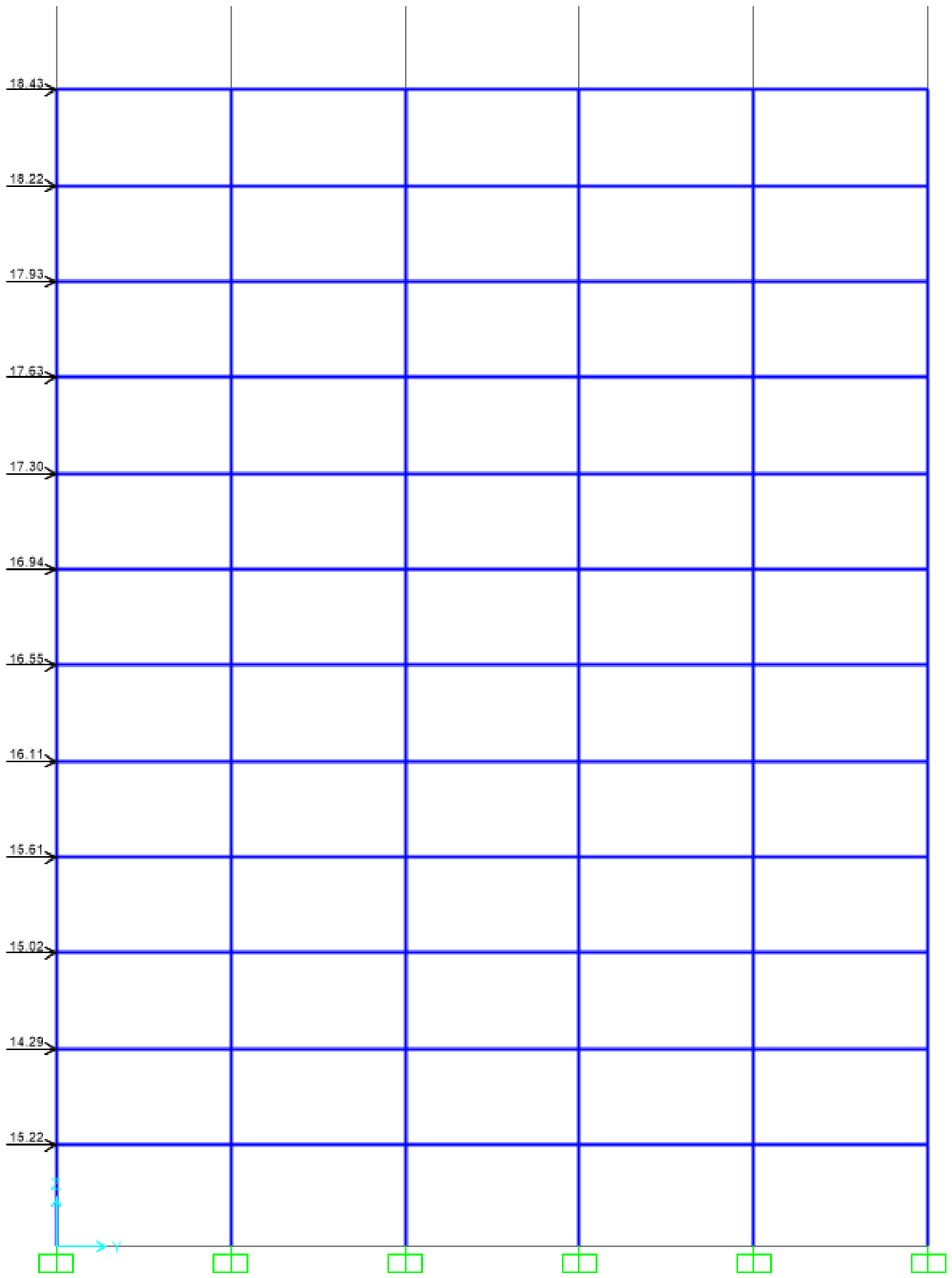


Figure 3.22. Case 1 Wind Load Assignment for Frame C

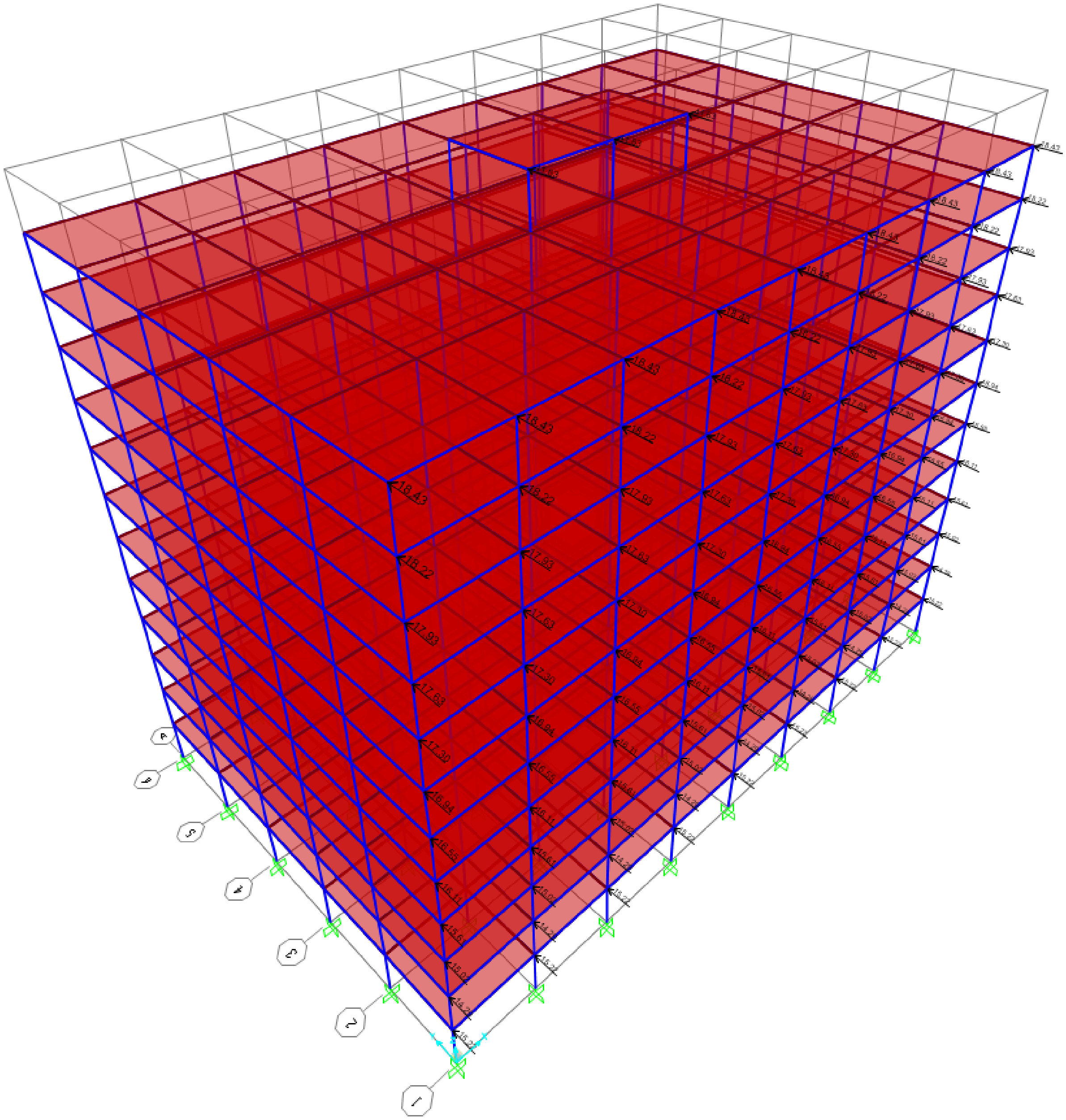


Figure 3.23. 3D Case 1 Wind Load Assignment

The rest of the Cases for Wind Forces are included in Appendix.

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

3.5.1. Wind and Seismic Drifts Calculations

As a part of the analysis of the Lateral Force Resisting System, wind forces and seismic loads' effect on the drift of the structure needs to be examined. To provide more accurate and relevant results, lateral shift data was calculated for both X and Y directions, or Frames A and Frame 3 respectively, given that the residential building has a rectangular shape. The structural layout of the residential building with the chosen frames is depicted in Figure X. Both shear and flexural drifts were computed for wind and seismic loads.

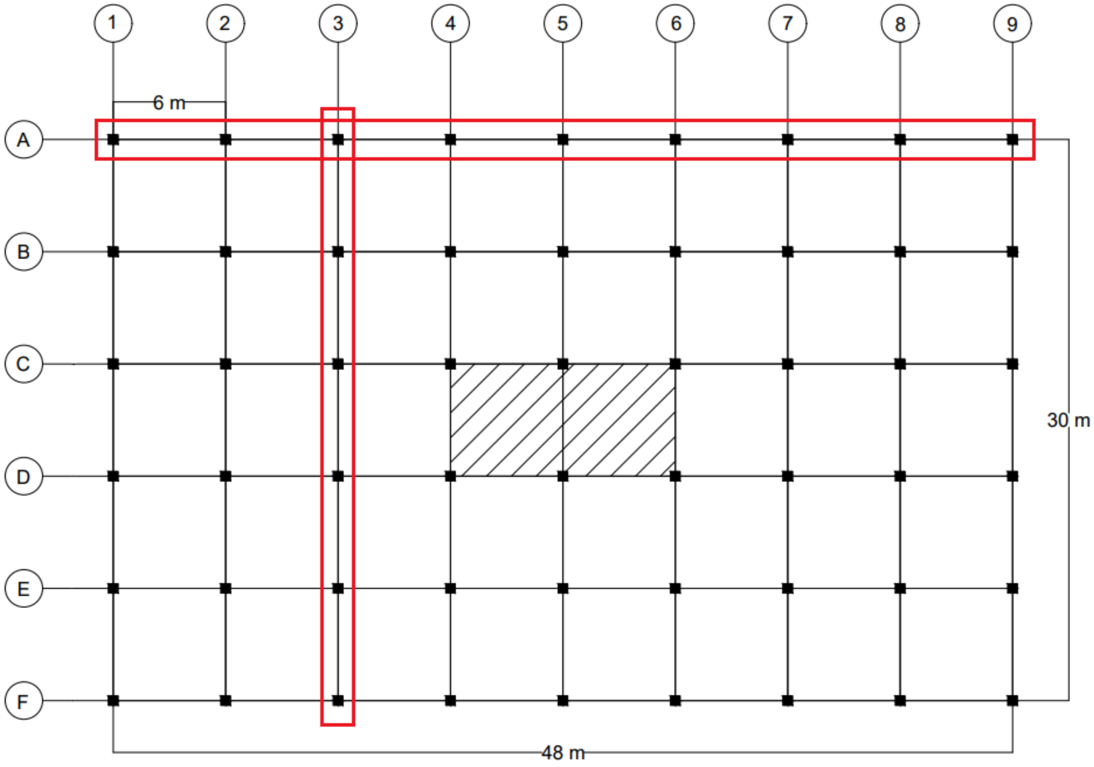


Figure 3.24. Structural layout with the chosen Frame A and Frame 3

In the following tables, we calculated the wind drifts for long and short sides of the building by using the previously found Forces per frame on each level. Shear force was determined by the formula:

$$V_i = F_i + V_{i+1} \tag{3.36}$$

And base shear for each column on the levels was determined by dividing shear force of the whole level by the number of frames (n=8).

$$V_{i-col} = \frac{V_i}{n} \tag{3.37}$$

For calculations we also used critical moments of inertia, Ic,cr and Ib,cr. Formulas that we used to determine deformations are the following:

$$\delta_b^{\,i} = \frac{V_{i-avg} h_i^{2} * L}{12 * E * I_{b,cr}} \text{ for floors 1-11} \tag{3.38}$$

$$\delta_b^{\,i} = \frac{V_{i-avg} * h_i^{2} * L}{48 * E * I_{b,cr}} \text{ for floor 12} \tag{3.39}$$

$$\delta_c^{\,i} = \frac{V_{i-avg} * h_i^{3}}{12 * E * I_c} \text{ for floors 1-11} \tag{3.40}$$

$$\delta_c^{\,i} = \frac{V_{i-avg} * h_i^{3}}{24 * E * I_c} \text{ for floor 12} \tag{3.41}$$

$$\delta_c^{\,i} = \frac{V_{i-avg} * h_i^{3}}{3 * E * I_c} \text{ for floor 1} \tag{3.42}$$

$$\delta_t^{\,i} = \delta_b + \delta_c \tag{3.43}$$

$$\delta_i = \delta_t^{\,i} + (\delta_t^{\,i-1}/2) \text{ for floor} \tag{3.44}$$

$$\delta_i = \delta_t^{\,i}/2 + (\delta_t^{\,i-1}/2) \text{ for floors} \tag{3.45}$$

$$\delta_i = (\delta_t^{\,i}/2) + \delta_t^{\,i-1} \text{ for floor 1} \tag{3.46}$$

Where: $\delta_b^{\,i}$ - inter-story drift calculated for major beams; $\delta_c^{\,i}$ - inter-story drift calculated for columns; $\delta_t^{\,i}$ - total inter-story drift.

Tables 3.38 below represent all the calculated data for the lateral drifts.

Table 3.38. Lateral Shear Drift due to Wind Load in X-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)
12	3300	3300	16.13	16.13	2.02	2.02	8.75E+08	1.89E+09	8.75E+08
11	3300	3300	15.93	32.06	4.01	3.01	1.49E+09	1.89E+09	1.18E+09
10	3300	3300	15.65	47.71	5.96	4.99	2.39E+09	1.89E+09	1.94E+09

9	3300	3300	15.36	63.07	7.88	6.92	3.65E+09	1.89E+09	3.02E+09
8	3300	3300	15.04	78.11	9.76	8.82	5.34E+09	1.89E+09	4.49E+09
7	3300	3300	14.70	92.80	11.60	10.68	7.56E+09	1.89E+09	6.45E+09
6	3300	3300	14.32	107.12	13.39	12.49	7.56E+09	1.89E+09	7.56E+09
5	3300	3300	13.89	121.01	15.13	14.26	1.04E+10	1.89E+09	8.99E+09
4	3300	3300	13.41	134.42	16.80	15.96	1.04E+10	1.89E+09	1.04E+10
3	3300	3300	12.93	147.34	18.42	17.61	1.40E+10	1.89E+09	1.22E+10
2	3300	3300	12.71	160.05	20.01	19.21	1.40E+10	1.89E+09	1.40E+10
1	3500	3400	13.46	173.51	21.69	20.85	1.85E+10	1.89E+09	1.62E+10
0		1750				10.84			

Table 3.39.. Lateral Shear Drift due to Wind Load in X-direction (Continuation)

Story	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	abs (mm)
12	0.05	0.12	0.16	0.44	15.51
11	0.29	0.26	0.55	0.64	15.07
10	0.48	0.26	0.74	0.82	14.43
9	0.67	0.23	0.90	0.98	13.61
8	0.86	0.20	1.05	1.13	12.63
7	1.04	0.17	1.20	1.29	11.50
6	1.21	0.17	1.38	1.46	10.21
5	1.38	0.16	1.54	1.62	8.75
4	1.55	0.15	1.70	1.78	7.13
3	1.71	0.15	1.85	1.93	5.35
2	1.86	0.14	2.00	2.21	3.43
1	2.27	0.15	2.43	1.21	1.21
0		0.00	0.00		0.00

To determine the lateral flexural drift of the short side of the building, firstly we determined the moment of each level by multiplying the height of the floor and shear base for the highest floor, and for the rest of floors we had to add the next floor to the one we are calculating.

After that we determined ϕ_i by using moment and cross sectional area of columns, and some constants as well. For the rest of the units that we needed, the formulas are below:

$$M_i = h_i * V_i,\ \text{(i=12)} \tag{3.47}$$

$$M_i = M_{i+1} + V_i h_i,\ \text{(i=1-11)} \tag{3.48}$$

$$\phi_i = \frac{2M_i}{E(mL)^2 * Ac} \tag{3.49}$$

$$\Delta\theta = \phi_i * h_i \tag{3.50}$$

$$\theta_i = \sum_i^n \Delta\theta_i \rightarrow \Delta\theta_i + \theta_{i-1} \tag{3.51}$$

$$\delta_i = \theta_i h_i \tag{3.52}$$

$$\Delta_i = \sum_i^n \delta_i \tag{3.53}$$

Table 3.40. Lateral Flexural Drift due to Wind Load in X-direction

Story	**M (N-mm)**	**a=b (m)**	**A (mm^2)**	ϕ_i	**Δθ (rad)**	**θ (rad)**	δ_i **(mm)**	Δ_i**(mm)**
12	5.32E+07	0.35	122500	1.27E-11	4.19E-08	4.26E-06	0.01	0.12
11	1.59E+08	0.4	160000	2.90E-11	9.58E-08	4.22E-06	0.01	0.11
10	3.16E+08	0.45	202500	4.56E-11	1.51E-07	4.12E-06	0.01	0.09
9	5.25E+08	0.5	250000	6.13E-11	2.02E-07	3.97E-06	0.01	0.08
8	7.82E+08	0.55	302500	7.55E-11	2.49E-07	3.77E-06	0.01	0.07
7	1.09E+09	0.6	360000	8.83E-11	2.91E-07	3.52E-06	0.01	0.05
6	1.44E+09	0.6	360000	1.17E-10	3.86E-07	3.23E-06	0.01	0.04
5	1.84E+09	0.65	422500	1.27E-10	4.20E-07	2.84E-06	0.01	0.03

4	2.28E+09	0.65	422500	1.58E-10	5.21E-07	2.42E-06	0.01	0.02
3	2.77E+09	0.7	490000	1.65E-10	5.45E-07	1.90E-06	0.01	0.01
2	3.30E+09	0.7	490000	1.97E-10	6.49E-07	1.36E-06	0.00	0.01
1	3.91E+09	0.75	562500	2.03E-10	7.10E-07	7.10E-07	0.00	0.00

For determining lateral shear drift on the long side of the building, we basically used the same formulas as for the short side. Additionally, we used forces per frame found beforehand per each level of the building for the long side. Number of frames on the long side is 5 (n=5), so the formula for base shear for each column is different (Vi-col).

$$V_i = F_i + V_{i+1} \tag{3.54}$$

$$V_{i-col} = \frac{V_i}{n} \tag{3.55}$$

Table 3.41. Lateral Shear Drift due to Wind Load in Y-direction

Story	**hi (mm)**	**hi-avg (mm)**	**Fi (kN)**	**Vi (kN)**	**Vi-col (kN)**	**Vavg (kN)**	**Ic,cr (mm^4)**	**Ib,cr (mm^4)**	**Ic-avg (mm^4)**
12	3300	3300	18.43	18.43	3.69	3.69	8.75E+08	1.89E+09	8.75E+08
11	3300	3300	18.22	36.65	7.33	5.51	1.49E+09	1.89E+09	1.18E+09
10	3300	3300	17.93	54.58	10.92	9.12	2.39E+09	1.89E+09	1.94E+09
9	3300	3300	17.63	72.21	14.44	12.68	3.65E+09	1.89E+09	3.02E+09
8	3300	3300	17.30	89.51	17.90	16.17	5.34E+09	1.89E+09	4.49E+09
7	3300	3300	16.94	106.45	21.29	19.60	7.56E+09	1.89E+09	6.45E+09
6	3300	3300	16.55	123.00	24.60	22.94	7.56E+09	1.89E+09	7.56E+09
5	3300	3300	16.11	139.10	27.82	26.21	1.04E+10	1.89E+09	8.99E+09
4	3300	3300	15.61	154.71	30.94	29.38	1.04E+10	1.89E+09	1.04E+10

3	3300	3300	15.02	169.73	33.95	32.44	1.40E+10	1.89E+09	1.22E+10
2	3300	3300	14.29	184.01	36.80	35.37	1.40E+10	1.89E+09	1.40E+10
1	3500	3400	15.22	199.23	39.85	38.32	1.85E+10	1.89E+09	1.62E+10
0		1750			0.00	19.92			

Table 3.42. Lateral Shear Drift due to Wind Load in Y-direction (Continuation)

Story	db (mm)	dc (mm)	dt (mm)	Interstory	abs
12	0.09	0.21	0.30	0.80	28.50
11	0.53	0.47	1.00	1.18	27.70
10	0.88	0.47	1.36	1.50	26.52
9	1.23	0.42	1.65	1.79	25.01
8	1.57	0.36	1.93	2.07	23.22
7	1.90	0.31	2.21	2.37	21.16
6	2.22	0.31	2.53	2.68	18.79
5	2.54	0.29	2.83	2.98	16.11
4	2.85	0.28	3.13	3.27	13.12
3	3.14	0.27	3.41	3.55	9.85
2	3.43	0.25	3.68	4.07	6.30
1	4.18	0.28	4.46	2.23	2.23
0		0.00	0.00		0.00

Formulas for lateral flexural drift on the long side of the building are the same as for the short side.

$$M_i = h_i * V_i, \text{ (i=13)} \quad (3.56)$$

$$M_i = M_{i+1} + V_i h_i, \text{ (i=1-12)} \quad (3.57)$$

$$\phi_i = \frac{2M_i}{E(mL)^2 * Ac} \quad (3.58)$$

$$\Delta\theta = \phi_i * h_i \quad (3.59)$$

$$\theta_i = \sum_i^n \Delta\theta_i \rightarrow \Delta\theta_i + \theta_{i-1} \quad (3.60)$$

$$\delta_i = \theta_i h_i \quad (3.61)$$

$$\Delta_i = \sum_{i}^{n} \delta_i \tag{3.62}$$

Table 3.43. Lateral Flexural Drift due to Wind Load in Y-direction

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
12	6.08E+07	0.35	122500	3.71E-11	1.22E-07	1.25E-05	0.04	0.35
11	1.82E+08	0.4	160000	8.49E-11	2.80E-07	1.24E-05	0.04	0.31
10	3.62E+08	0.45	202500	1.34E-10	4.41E-07	1.21E-05	0.04	0.27
9	6.00E+08	0.5	250000	1.79E-10	5.92E-07	1.17E-05	0.04	0.23
8	8.96E+08	0.55	302500	2.21E-10	7.30E-07	1.11E-05	0.04	0.19
7	1.25E+09	0.6	360000	2.59E-10	8.54E-07	1.03E-05	0.03	0.16
6	1.65E+09	0.6	360000	3.43E-10	1.13E-06	9.49E-06	0.03	0.12
5	2.11E+09	0.65	422500	3.74E-10	1.23E-06	8.36E-06	0.03	0.09
4	2.62E+09	0.65	422500	4.64E-10	1.53E-06	7.13E-06	0.02	0.06
3	3.18E+09	0.7	490000	4.86E-10	1.60E-06	5.60E-06	0.02	0.04
2	3.79E+09	0.7	490000	5.78E-10	1.91E-06	4.00E-06	0.01	0.02
1	4.49E+09	0.75	562500	5.96E-10	2.09E-06	2.09E-06	0.01	0.01

For calculating lateral shear drift regarding seismic loads, we use the same formulas and same principles as for calculation of the wind loads shear drift. The difference is only in the forces that we found on seismic loads. Everything else is the same, including the constants.

Table 3.44. Lateral Shear Drift due to Seismic Load in X-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)
12	3300	3300	144.24	144.24	18.03	18.03	8.75E+08	1.89E+09	8.75E+08
11	3300	3300	128.64	272.88	34.11	26.07	1.49E+09	1.89E+09	1.18E+09
10	3300	3300	113.52	386.40	48.30	41.20	2.39E+09	1.89E+09	1.94E+09
9	3300	3300	98.88	485.28	60.66	54.48	3.65E+09	1.89E+09	3.02E+09
8	3300	3300	84.71	569.99	71.25	65.95	5.34E+09	1.89E+09	4.49E+09
7	3300	3300	71.05	641.04	80.13	75.69	7.56E+09	1.89E+09	6.45E+09
6	3300	3300	57.03	698.07	87.26	83.69	7.56E+09	1.89E+09	7.56E+09
5	3300	3300	44.70	742.77	92.85	90.05	1.04E+10	1.89E+09	8.99E+09
4	3300	3300	32.56	775.33	96.92	94.88	1.04E+10	1.89E+09	1.04E+10
3	3300	3300	22.04	797.37	99.67	98.29	1.40E+10	1.89E+09	1.22E+10
2	3300	3300	12.47	809.84	101.23	100.45	1.40E+10	1.89E+09	1.40E+10
1	3500	3400	4.92	814.76	101.85	101.54	1.85E+10	1.89E+09	1.62E+10
0		1750			0.00	50.92			

Table 3.45. Lateral Shear Drift due to Seismic Load in X-direction (Continuation)

Story	db (mm)	dc (mm)	dt (mm)	Interstory (mm)	Absolute (mm)
12	0.44	1.04	1.47	3.85	97.53
11	2.53	2.22	4.74	5.44	93.69
10	3.99	2.14	6.13	6.61	88.25
9	5.28	1.82	7.10	7.48	81.63
8	6.39	1.48	7.87	8.19	74.15
7	7.34	1.18	8.52	8.87	65.95

6	8.11	1.12	9.23	9.48	57.08
5	8.73	1.01	9.74	9.93	47.60
4	9.20	0.92	10.11	10.23	37.67
3	9.53	0.81	10.34	10.40	27.45
2	9.74	0.72	10.46	11.14	17.05
1	11.07	0.75	11.82	5.91	5.91
0		0.00	0.00		0.00

Table 3.46. Lateral Flexural Drift due to Seismic Load in X-direction

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
12	4.76E+08	0.35	122500	2.90E-10	9.59E-07	7.58E-05	0.25	2.06
11	1.38E+09	0.4	160000	6.43E-10	2.12E-06	7.48E-05	0.25	1.81
10	2.65E+09	0.45	202500	9.79E-10	3.23E-06	7.27E-05	0.24	1.56
9	4.25E+09	0.5	250000	1.27E-09	4.20E-06	6.95E-05	0.23	1.32
8	6.13E+09	0.55	302500	1.52E-09	5.00E-06	6.53E-05	0.22	1.09
7	8.25E+09	0.6	360000	1.71E-09	5.65E-06	6.03E-05	0.20	0.88
6	1.06E+10	0.6	360000	2.19E-09	7.23E-06	5.46E-05	0.18	0.68
5	1.30E+10	0.65	422500	2.30E-09	7.59E-06	4.74E-05	0.16	0.50
4	1.56E+10	0.65	422500	2.75E-09	9.09E-06	3.98E-05	0.13	0.34
3	1.82E+10	0.7	490000	2.78E-09	9.16E-06	3.07E-05	0.10	0.21
2	2.09E+10	0.7	490000	3.18E-09	1.05E-05	2.15E-05	0.07	0.11
1	2.37E+10	0.75	562500	3.15E-09	1.10E-05	1.10E-05	0.04	0.04

Table 3.47. Lateral Shear Drift due to Seismic Load in Y-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm^4)	Ib,cr (mm^4)	Ic-avg (mm^4)
12	3300	3300	216.35	216.35	43.27	43.27	8.75E+08	1.89E+09	8.75E+08
11	3300	3300	192.96	409.31	81.86	62.57	1.49E+09	1.89E+09	1.18E+09
10	3300	3300	170.28	579.60	115.92	98.89	2.39E+09	1.89E+09	1.94E+09
9	3300	3300	148.32	727.91	145.58	130.75	3.65E+09	1.89E+09	3.02E+09
8	3300	3300	127.07	854.98	171.00	158.29	5.34E+09	1.89E+09	4.49E+09
7	3300	3300	106.58	961.56	192.31	181.65	7.56E+09	1.89E+09	6.45E+09
6	3300	3300	85.55	1047.11	209.42	200.87	7.56E+09	1.89E+09	7.56E+09
5	3300	3300	67.04	1114.15	222.83	216.13	1.04E+10	1.89E+09	8.99E+09
4	3300	3300	48.84	1162.99	232.60	227.71	1.04E+10	1.89E+09	1.04E+10
3	3300	3300	33.06	1196.05	239.21	235.90	1.40E+10	1.89E+09	1.22E+10
2	3300	3300	18.71	1214.76	242.95	241.08	1.40E+10	1.89E+09	1.40E+10
1	3500	3400	7.38	1222.14	244.43	243.69	1.85E+10	1.89E+09	1.62E+10
0		1750			0.00	122.21			

Table 3.48. Lateral Shear Drift due to Seismic Load in Y-direction (Continuation)

Story	db (mm)	dc (mm)	dt (mm)	Interstory	abs
12	1.05	2.49	3.54	9.23	234.08
11	6.06	5.32	11.39	13.05	224.85
10	9.58	5.13	14.71	15.87	211.80
9	12.67	4.36	17.04	17.96	195.92
8	15.34	3.55	18.89	19.67	177.96

7	17.61	2.84	20.44	21.29	158.29
6	19.47	2.68	22.14	22.76	137.00
5	20.95	2.42	23.37	23.82	114.24
4	22.07	2.20	24.27	24.54	90.42
3	22.86	1.95	24.81	24.95	65.88
2	23.37	1.73	25.10	26.74	40.92
1	26.57	1.80	28.37	14.19	14.19
0		0.00	0.00		0.00

Table 3.49. Lateral Flexural Drift due to Seismic Load in Y-direction

Story	M (N-mm)	a=b (m)	A (mm2)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute
12	7.14E+08	0.35	122500	1.70E-10	5.62E-07	4.44E-05	0.15	1.21
11	2.06E+09	0.4	160000	3.77E-10	1.24E-06	4.38E-05	0.14	1.06
10	3.98E+09	0.45	202500	5.74E-10	1.89E-06	4.26E-05	0.14	0.92
9	6.38E+09	0.5	250000	7.45E-10	2.46E-06	4.07E-05	0.13	0.77
8	9.20E+09	0.55	302500	8.88E-10	2.93E-06	3.82E-05	0.13	0.64
7	1.24E+10	0.6	360000	1.00E-09	3.31E-06	3.53E-05	0.12	0.51
6	1.58E+10	0.6	360000	1.28E-09	4.24E-06	3.20E-05	0.11	0.40
5	1.95E+10	0.65	422500	1.35E-09	4.45E-06	2.78E-05	0.09	0.29
4	2.33E+10	0.65	422500	1.61E-09	5.32E-06	2.33E-05	0.08	0.20
3	2.73E+10	0.7	490000	1.63E-09	5.37E-06	1.80E-05	0.06	0.12
2	3.13E+10	0.7	490000	1.87E-09	6.16E-06	1.26E-05	0.04	0.06
1	3.56E+10	0.75	562500	1.85E-09	6.46E-06	6.46E-06	0.02	0.02

3.5.2. Comparison of Inter-Story Drifts

Now that all of the Inter-Story Drifts were calculated the hand-calculated drifts should be compared with the 2D Frame and 3D Frame SAP 2000 simulations. Below you can find Tables XXX, where the interstory displacement for Frame A and Frame 3, transverse and longitudinal directions, are demonstrated.

Table 3.50. Comparison of interstory displacements on LFRS

Comparison of interstory displacements, Frame 3						
Levels	Interstory displacement from wind load			Interstory displacement from seismic load		
	2D	3D	Hand	2D	3D	Hand
12	0.81	1.219	0.84	9.3	8.467	9.38
11	1.24	1.504	1.22	13.52	12.078	13.19
10	1.56	1.709	1.54	16.33	14.31	16.01
9	1.86	1.91	1.83	18.45	15.94	18.10
8	2.15	2.113	2.10	20.12	17.217	19.79
7	2.41	2.311	2.40	21.4	18.206	21.41
6	2.74	2.592	2.71	22.97	19.574	22.86
5	2.94	2.751	3.01	23.28	19.828	23.91
4	3.09	2.901	3.30	23.22	19.934	24.62
3	2.99	2.814	3.57	21.3	18.454	25.01
2	2.51	2.395	4.09	17.02	15.046	26.78
1	1.23	1.202	2.24	7.97	7.207	14.21

Table 3.51. Comparison of interstory displacements, Frame A

Comparison of interstory displacements, Frame A						
Levels	Interstory displacement from wind load			Interstory displacement from seismic load		
	2D	3D	Hand	2D	3D	Hand
12	0.4	0.408	0.45	3.38	3.592	4.10
11	0.54	0.609	0.66	4.68	5.122	5.68
10	0.68	0.756	0.84	5.44	6.053	6.85
9	0.78	0.885	0.99	5.93	6.73	7.71
8	0.87	1.009	1.14	6.37	7.261	8.41
7	0.98	1.126	1.30	6.63	7.676	9.07
6	1.1	1.283	1.47	7.17	8.265	9.66
5	1.19	1.378	1.63	7.3	8.38	10.08

4	1.36	1.469	1.78	7.3	8.447	10.36
3	1.2	1.439	1.93	6.9	7.849	10.50
2	1.1	1.244	2.22	5.8	6.443	11.21
1	0.6	0.64	1.22	2.8	3.111	5.95

After calculating wind and seismic drift for both Frame A and Frame 3 and creating the comparison tables, the following figures were created to better represent the difference between hand calculations and SAP 2000 values. Figures XX below represent the inter-story wind and seismic drifts when force was applied in X and Y directions. It is important to mention that the following graphs represent the drifts under the assumption of the elastic zone. Since the residential building is to be constructed in the seismically active area, further measures must be taken in order to consider the real impact of potential earthquakes.

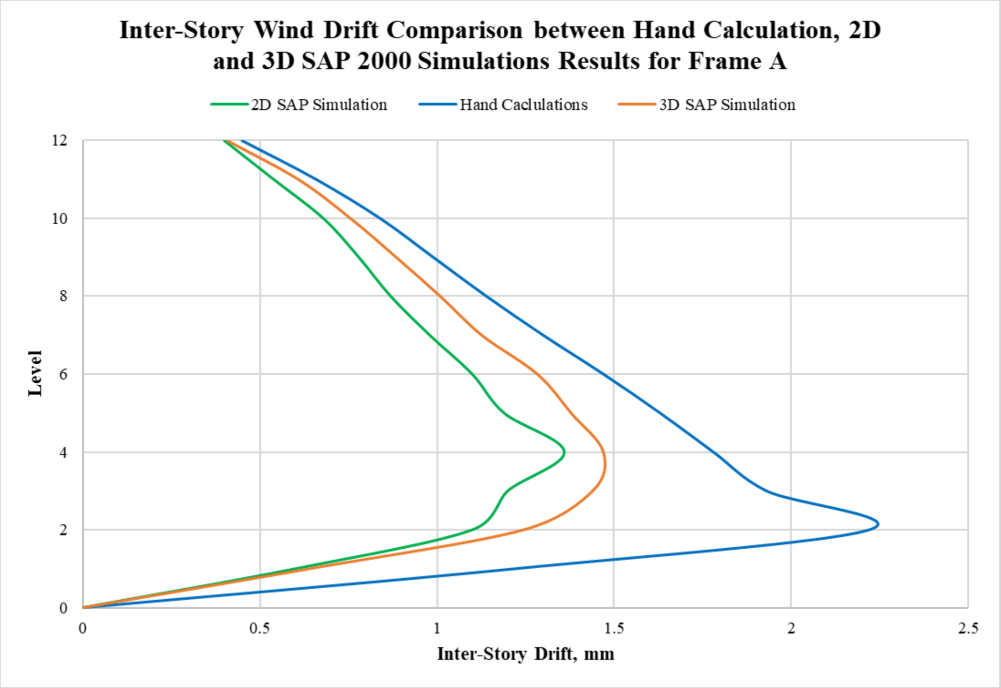


Figure 3.25. Inter-Story Wind Drift Comparison between Hand Calculations and SAP 2000 for Frame A, X - Direction

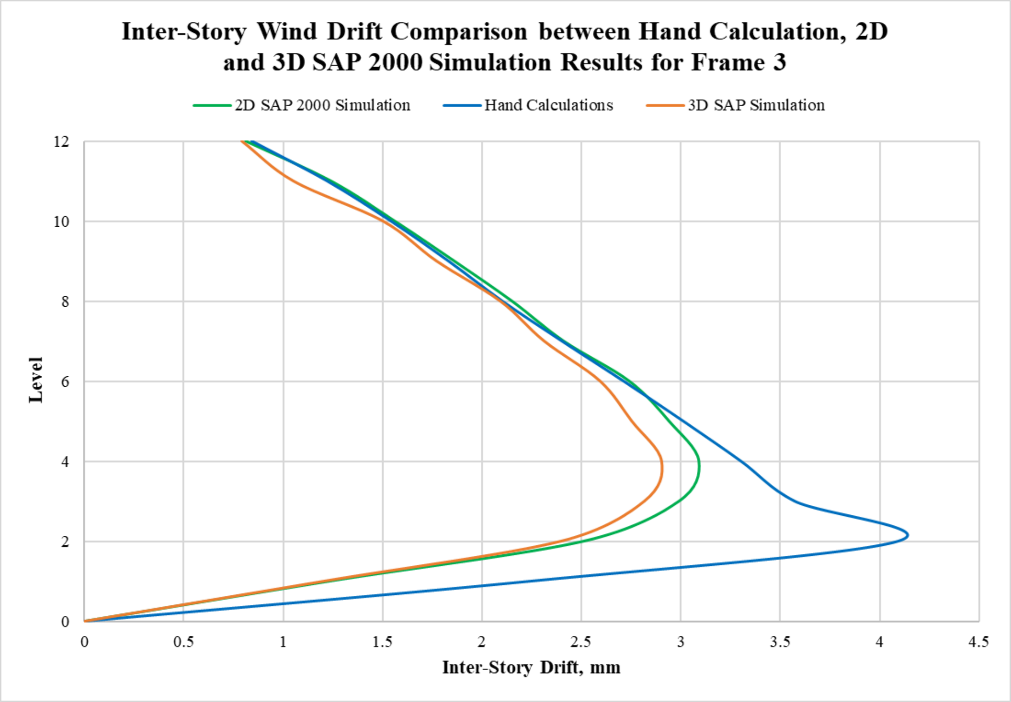


Figure 3.26. Inter-Story Wind Drift Comparison between Hand Calculations and SAP 2000 for Frame 3, Y - Direction

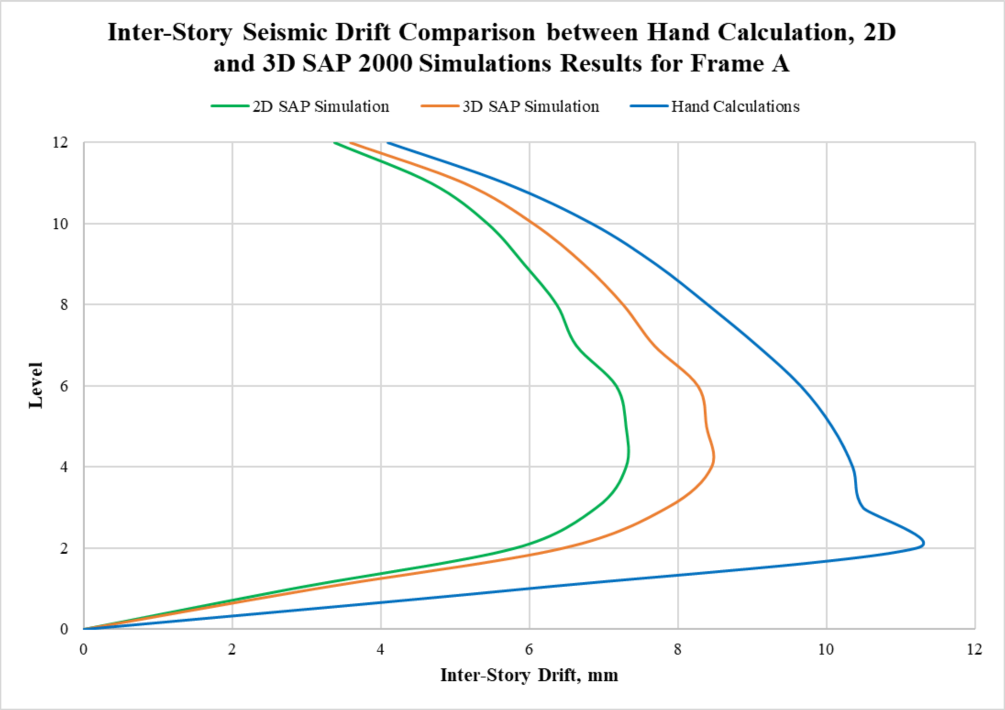


Figure 3.27. Inter-Story Seismic Drift Comparison between Hand Calculations and SAP 2000 for Frame A, X - Direction

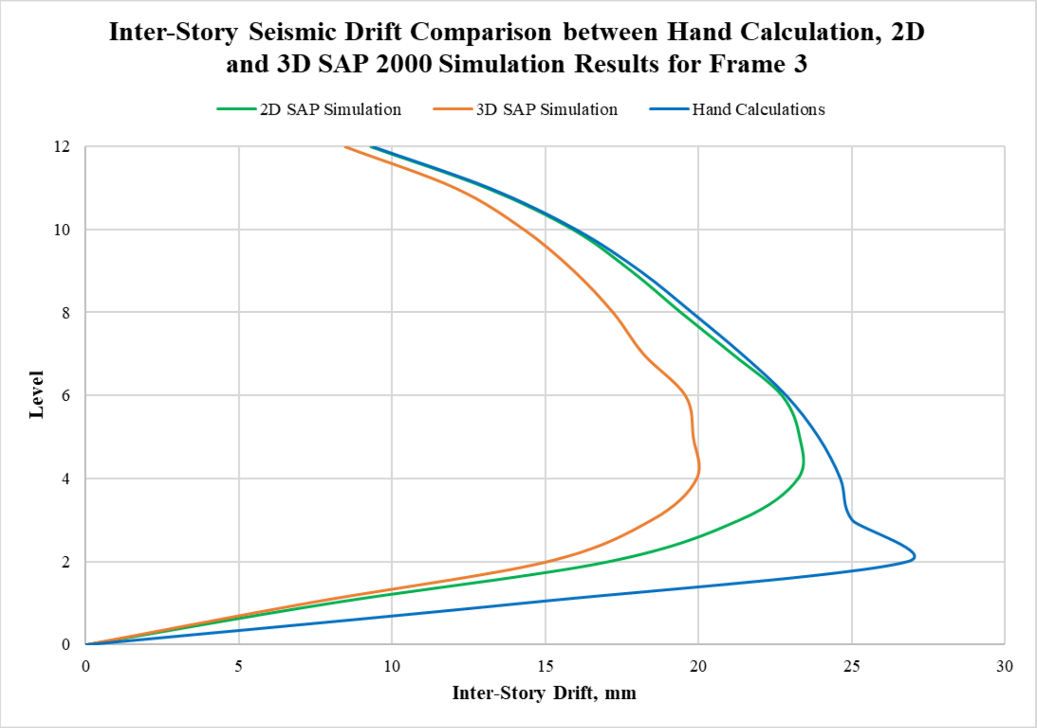


Figure 3.28. Inter-Story Seismic Drift Comparison between Hand Calculations and SAP 2000 for Frame 3, Y - Direction

As was mentioned before, seismic loads must be accounted for by implementing the amplification factor, C_d, in this case $C_d = 5.5$. This ensures that the interstory displacements are approximated to the most realistic scenario that is possible under the real world circumstances. The obtained values must be compared with the allowable limit for seismic drift. The following equations demonstrates the calculation of the allowable limit:

$$Allowable\ Limit\ = 2\%\ of\ the\ story\ height$$
$$First\ Floor\ Allowable\ Limit = 0.02 * 3500 = 70\ mm$$
$$2 - 12\ Floor\ Allowable\ Limit = 0.02 * 3300 = 66\ mm$$

As a result, below Figures XXX depict how the interstory drift displacements compare with the allowable limit for seismic drift. Clearly, all of the values are below the limit proving the design accuracy of the lateral force resisting system.

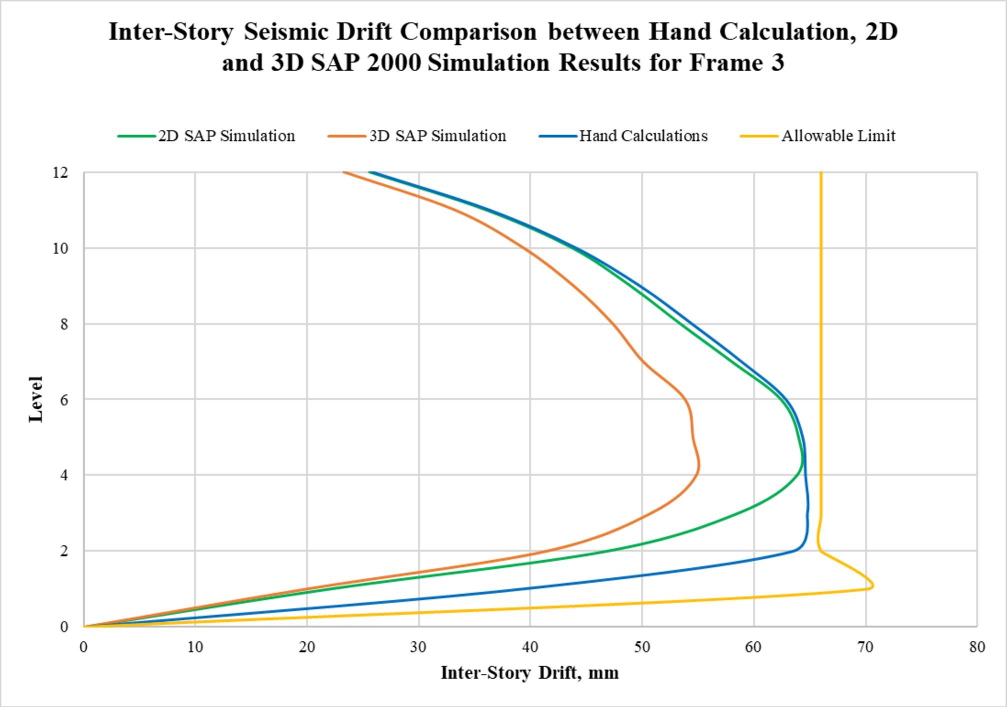


Figure 3.29. Amplified Inter-Story Seismic Drift Comparison between Hand and SAP 2000 Calculations with the Allowable Limit for Frame 3, Y - Direction

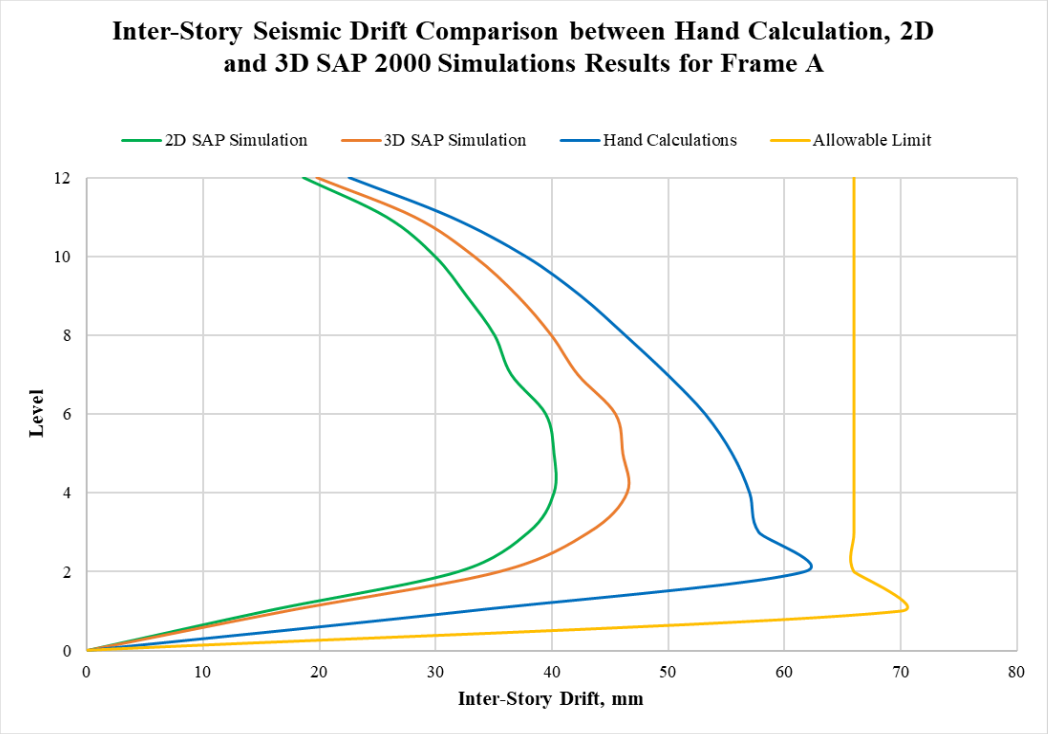


Figure 3.30. Amplified Inter-Story Seismic Drift Comparison between Hand and SAP 2000 Calculations with the Allowable Limit for Frame A, X - Direction

3.5.3. Stability Analysis

Another important point is to consider the gravity effect on the lateral load resisting system, for this case, it is P-delta effect. To evaluate whether this effect should be ignored, the stability coefficient θ must be found. The following formula below shows the calculation method:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \qquad (3.63)$$

P-delta effect can be ignored only in case the stability coefficient is lower than 0.1. By utilizing the previously found values for the axial load and seismic forces, those being Pu and Vu respectively, the stability coefficient can be determined. Tables XXX below demonstrate the utilized values along with the determined stability coefficient for both Frame 3 and Frame A to consider the long and short sides of the residential building.

Table 3.52. Stability Coefficient for Frame 3

Stability Coefficient Calculations for Frame 3 (Transverse)					
Level	**Pu (kN)**	**Vu (kN)**	**Inter-Story Drift, mm**	**Story height, mm**	**Theta**
12	2022.278	1298.13	9.38	3300	0.0008
11	4692.757	2455.89	13.19	3300	0.0014
10	8014.452	3477.59	16.01	3300	0.0020
9	11991.354	4367.49	18.10	3300	0.0027
8	16627.92	5129.89	19.79	3300	0.0035
7	21929.073	5769.36	21.41	3300	0.0045
6	27894.517	6282.64	22.86	3300	0.0056
5	34529.937	6684.91	23.91	3300	0.0068
4	41841.189	6977.93	24.62	3300	0.0081
3	49827.919	7176.31	25.01	3300	0.0096
2	58496.802	7288.57	26.78	3300	0.0118
1	67847.457	7332.86	14.21	3500	0.0068

Table 3.53. Stability Coefficient for Frame A

Stability Coefficient Calculations for Frame A (Longitudinal)					
Level	**Pu (kN)**	**Vu (kN)**	**Inter-Story Drift, mm**	**Story height, mm**	**Theta**
12	2022.278	1298.13	4.10	3300	0.0004
11	4692.757	2455.89	5.68	3300	0.0006

10	8014.452	3477.59	6.85	3300	0.0009
9	11991.354	4367.49	7.71	3300	0.0012
8	16627.92	5129.89	8.41	3300	0.0015
7	21929.073	5769.36	9.07	3300	0.0019
6	27894.517	6282.64	9.66	3300	0.0024
5	34529.937	6684.91	10.08	3300	0.0029
4	41841.189	6977.93	10.36	3300	0.0034
3	49827.919	7176.31	10.50	3300	0.0040
2	58496.802	7288.57	11.21	3300	0.0050
1	67847.457	7332.86	5.95	3500	0.0029

Therefore, as can be observed from Tables 3.53, the stability coefficient is less than 0.1 for each level of the residential building. This means that the gravity loads due to the P-delta can be ignored in the design of the lateral load resisting system.

3.5.4. Torsional Irregularity Check

As the seismic force is the most prominent among the others, it may cause torsional irregularity which leads to potential hazards. Thus, Torsional Irregularity Check must be conducted. For that the maximum inter-story displacement to average inter-stroy drift ratio must be defined and compared to the maximum available value. According to ACI 318-19 the ratio of the interstory displacements shall not exceed the value of 1.2 (ACI, 2019). Table X below represents the torsional irregularity check conducted for each floor of the residential building. As can be seen none of the floors exceed this limit. Thus, the building is not susceptible to torsional irregularity.

Table 3.54. Torsional Irregularity Check

Torsional Irregularity Check		
Level	**Ratio for Transverse**	**Ratio for Longitudinal**
12	0.478	0.494
11	0.673	0.685
10	0.817	0.826
9	0.923	0.929
8	1.010	1.013
7	1.092	1.093
6	1.166	1.164

5	1.170	1.175
4	1.178	1.183
3	1.185	1.189
2	1.199	1.196
1	0.725	0.717

3.6. Internal Force Analysis

3.6.1. Dead Load Internal Force Calculations

For the internal force analysis dead load and wind load were chosen to demonstrate the internal forces of the building frames. Below you can find all the Dead Load internal force results provided by the SAP 2000 software for both 3D and 2D frames.

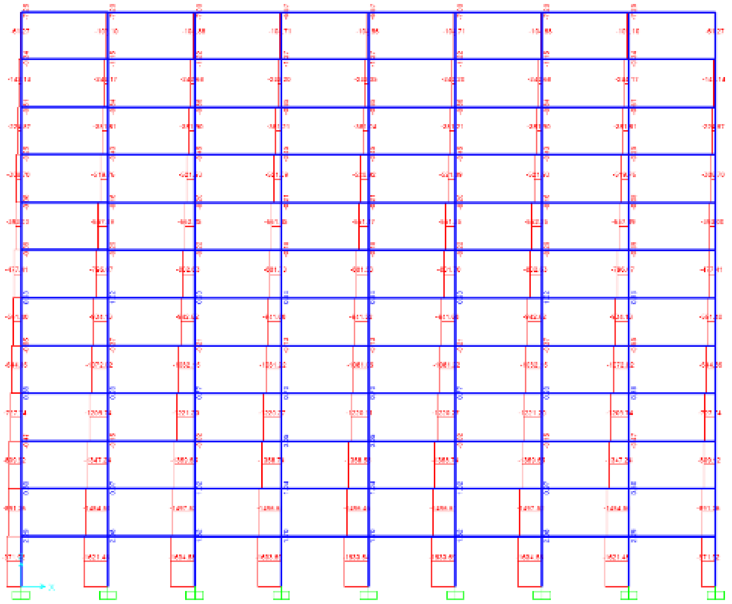


Figure 3.31. Axial Force from Dead Load in SAP2000 3D frame model

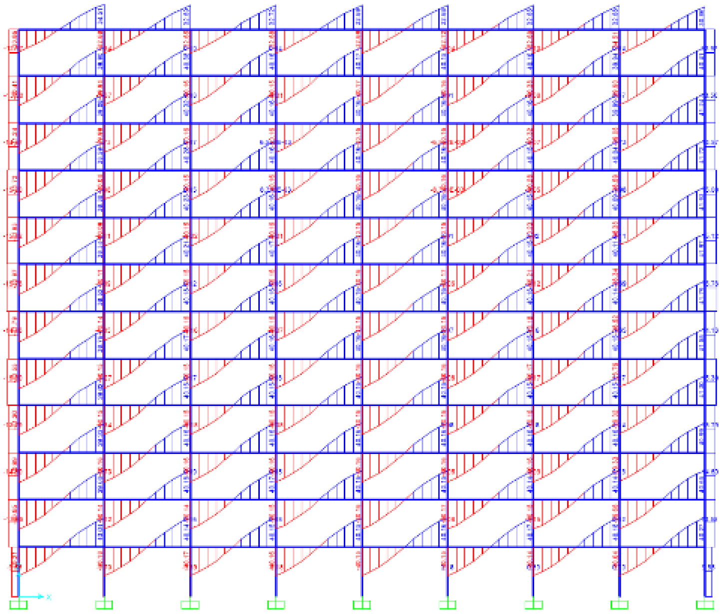


Figure 3.32. Shear Force from Dead Load in SAP2000 3D frame model

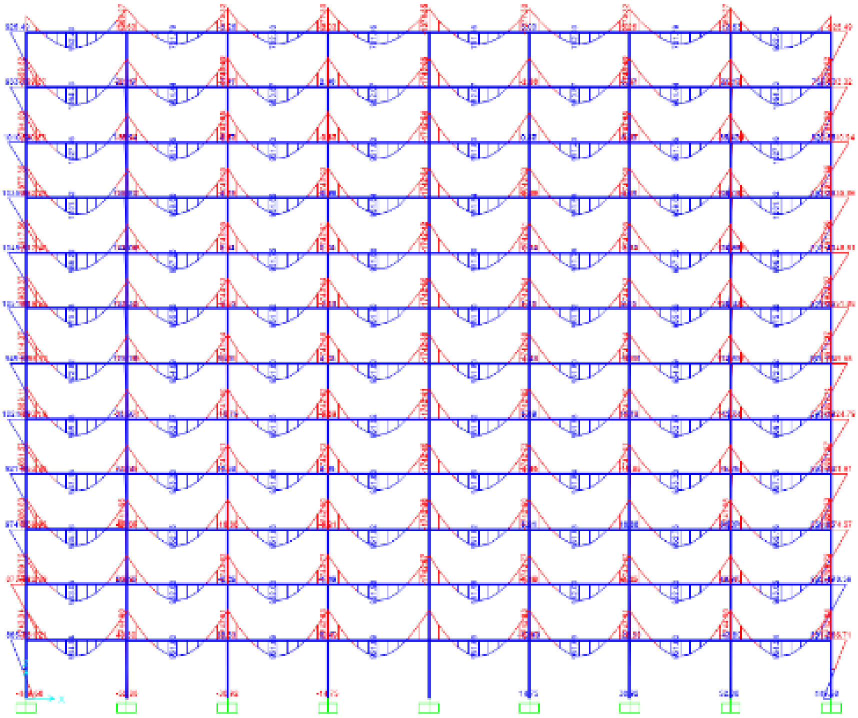


Figure 3.33. Moment Diagram of Dead Load from SAP2000 3D frame model

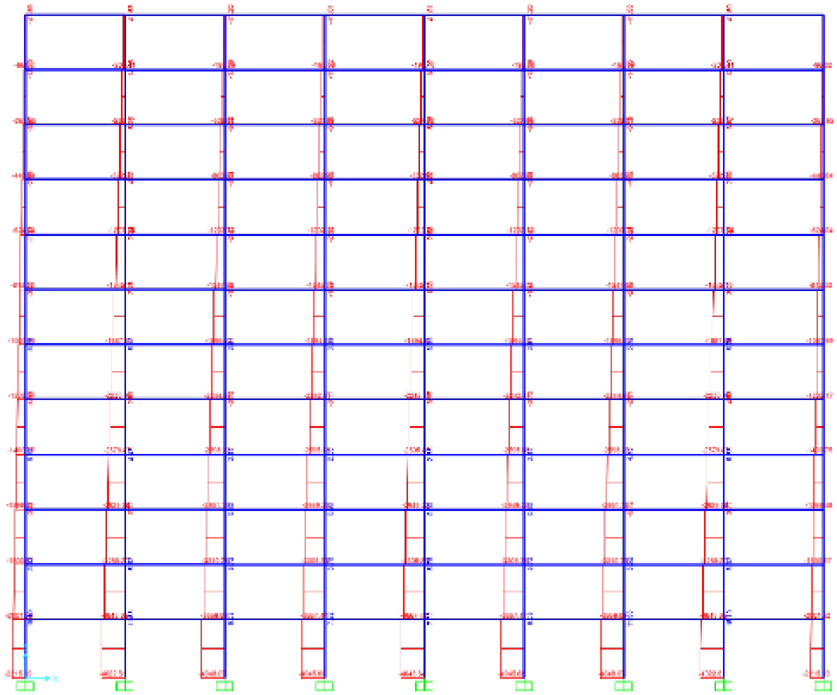


Figure 3.34. Axial Force from Dead Load in SAP2000 2D frame model

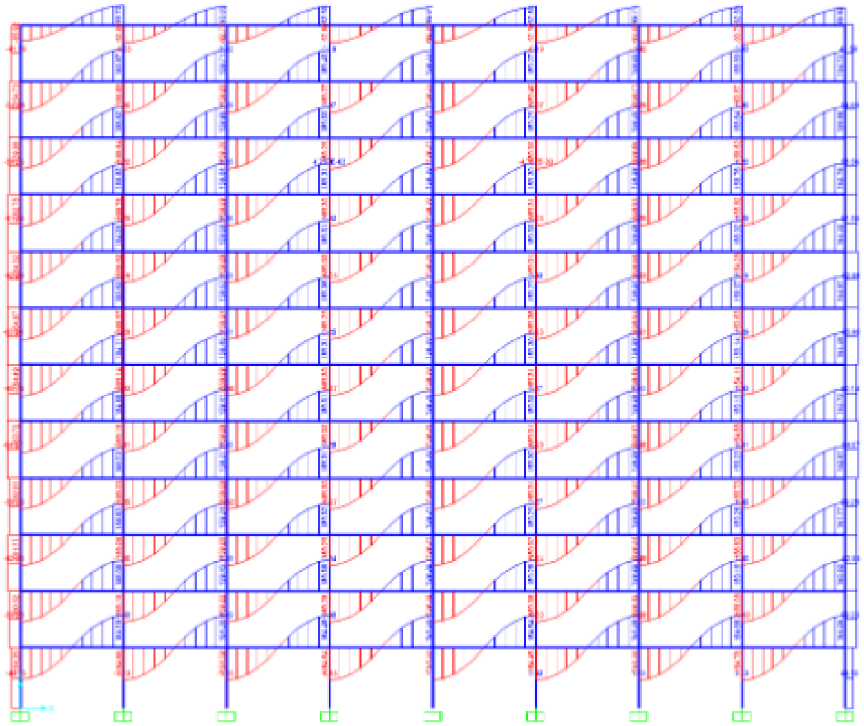


Figure 3.35. Shear Force from Dead Load in SAP2000 2D frame model

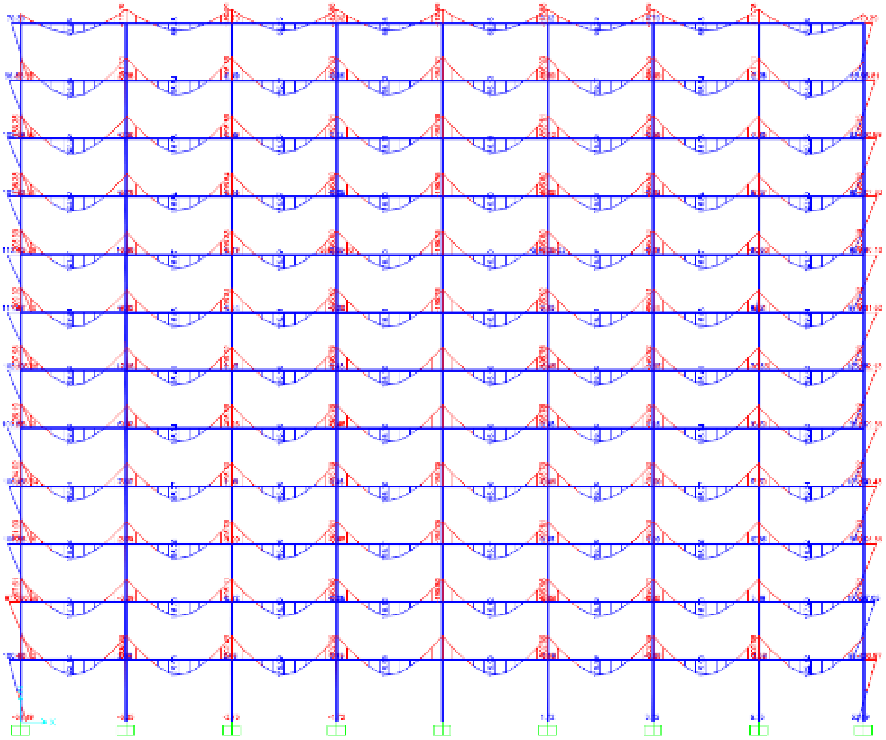


Figure 3.36. Moment Diagram from SAP2000 2D frame model

Now to verify the SAP results hand calculations must be made. Besides, they must be made to conduct comparative analysis and create internal force comparison graphs.

Below you can find the schematic diagram that represents the method used for calculating the internal forces under the Gravity Loads.

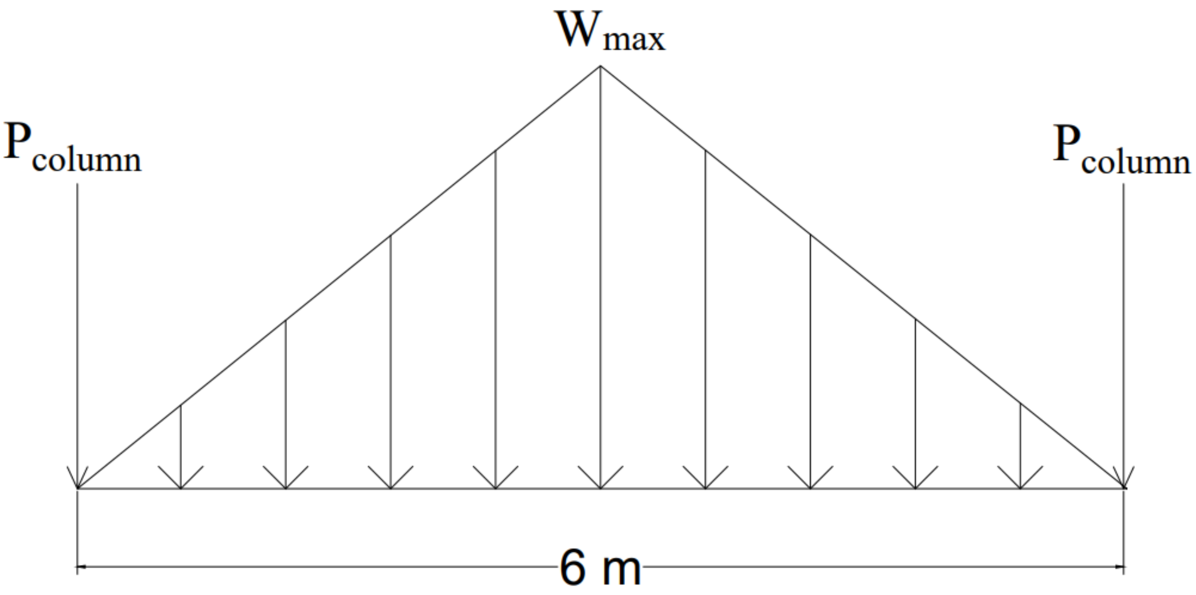


Figure 3.37. Applied Dead Load Illustration

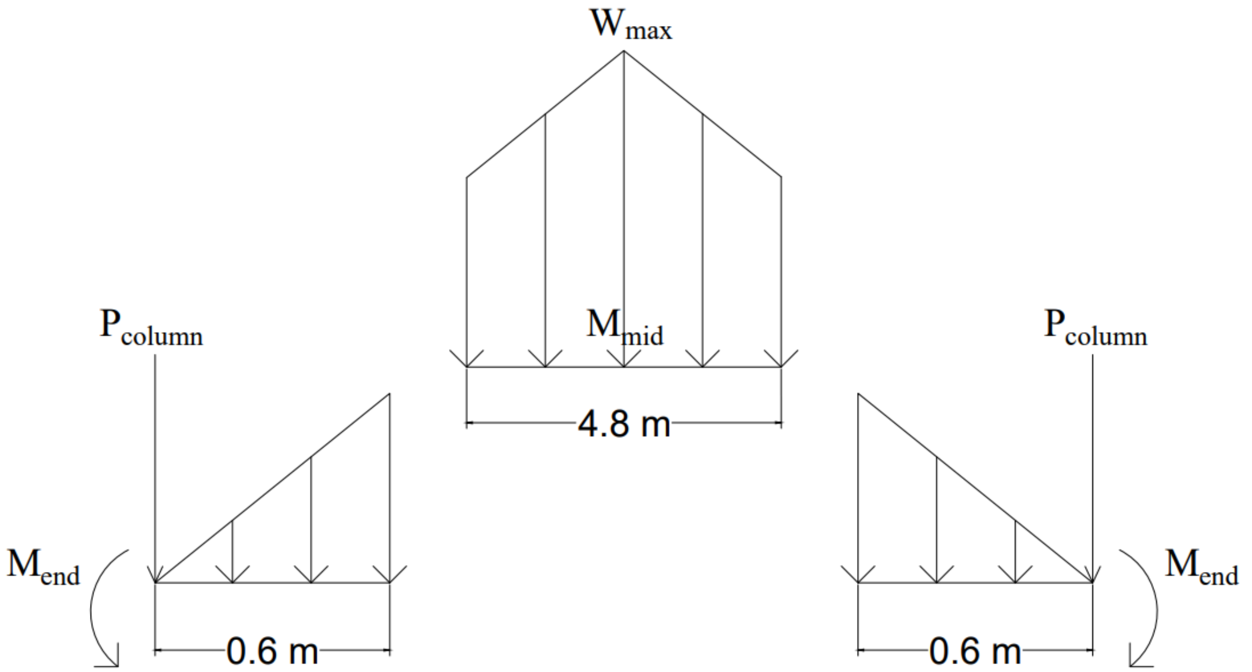


Figure 3.38. Analyzed Vertical Load Frame

As a result of hand calculations and SAP 2000 simulations the comparative tables were made to demonstrate the difference between these two methods. Below you can find Table 3.55 and Figure 3.39 for axial load, shear force and moment created under the dead load.

Table 3.55. Comparison of Axial Forces under Dead Load

Levels	Axial force in 3D, kN	Axial force in 2D, kN	Axial force in Hand, kN
12	272.55	151.5	60
11	632.29	515	400
10	1000.29	878.5	740
9	1373.18	1242	1080
8	1748.19	1605.5	1420
7	2123.66	1969	1760
6	2497.59	2332.5	2100
5	2868.51	2696	2440
4	3237.19	3059.5	2780
3	3602.77	3423	3120

2	3965.07	3786.5	3460
1	4328.57	4150	3800

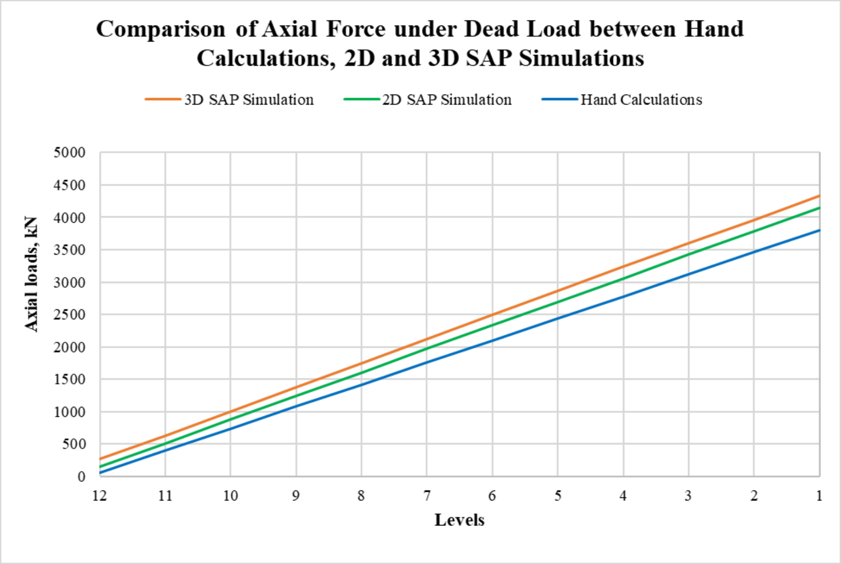


Figure 3.39. Dead Load Axial Force Comparison between Hand Calculations and SAP 2000

Table 3.56. Comparison of Shear Forces under Dead Load

Levels	Shear force in 3D, kN	Shear force in 2D, kN	Shear force in hand, kN
12	57.71	41.69	64
11	60.06	54.69	64
10	64.82	58.56	64
9	66.71	61.55	64
8	67.28	62.88	64
7	70.21	65.9	64
6	63.1	60.14	64
5	68.07	64.67	64
4	61.35	59.29	64
3	64.91	62.55	64
2	60.72	59.23	64
1	43.23	44.13	64

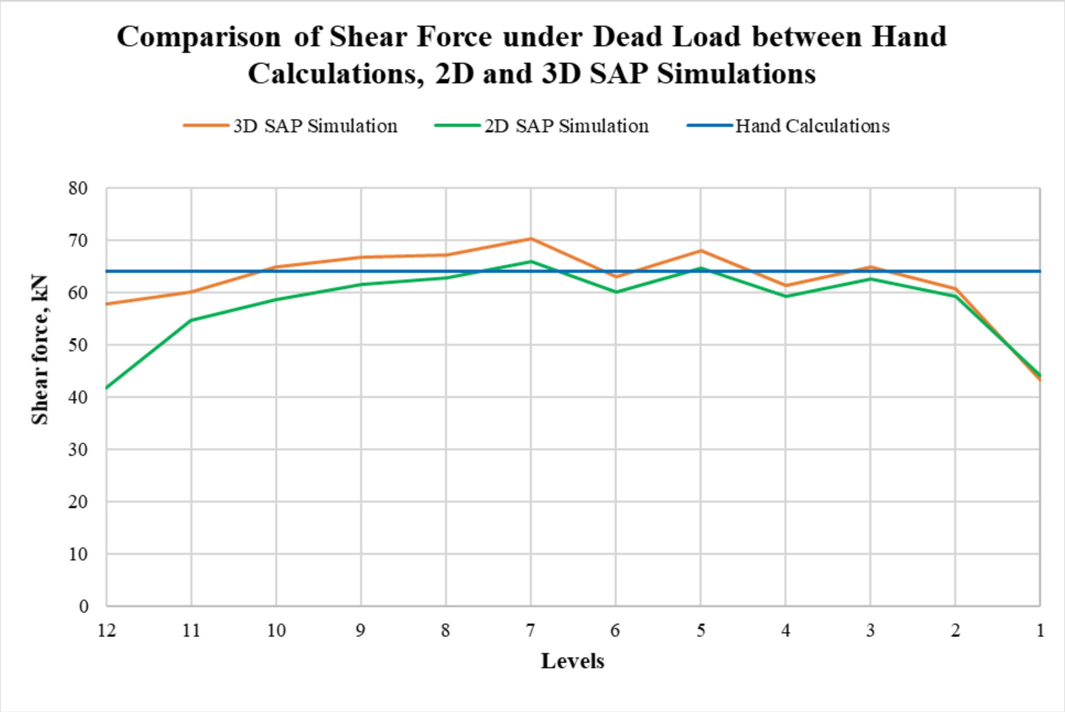


Figure 3.40. Dead Load Shear Force Comparison between Hand Calculations and SAP 2000

Table 3.57. Comparison of Moments under Dead Load

Levels	Moment in 3D, kN-m	Moment in 2D, kN-m	Moment in hand, kN-m
12	107.61	87.8	115
11	120.25	134.46	115
10	116.1	127.55	115
9	113.16	123.9	115
8	111.43	121.51	115
7	110.19	119.84	115
6	109.91	119.43	115
5	109.55	118.93	115
4	109.27	118.54	115
3	109.06	118.22	115
2	108.8	117.9	115
1	110.97	122.12	115

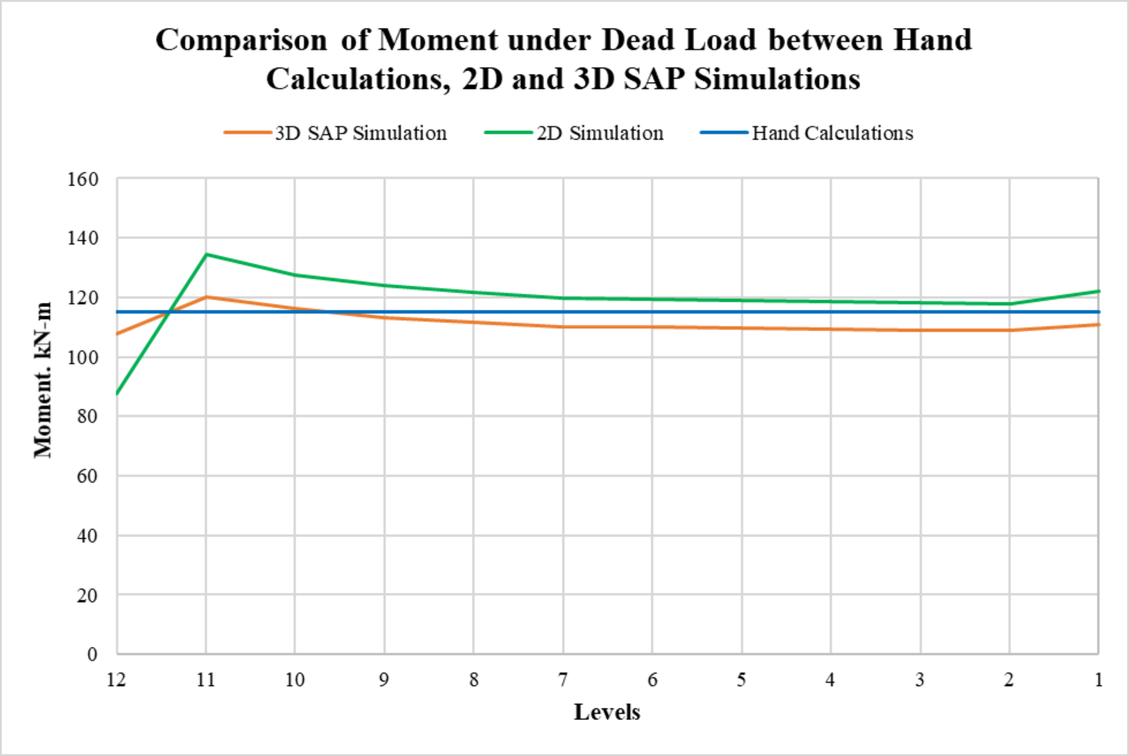


Figure 3.41. Dead Load Moment Comparison between Hand Calculations and SAP 2000

3.6.2. Wind Load Internal Force Calculations

Now moving on to the calculation of internal forces under the wind load, the hand calculations were made using the Portal Frame method. All of the calculations regarding the Portal Frame are attached in Appendix. The hand calculations clearly demonstrate the procedure of determining the internal forces (including axial load, shear force and moment) for floors 11 and 12. The same algorithm is applied to the rest of the floors.

As always the hand calculations must be compared to SAP 2000 simulation results. Therefore, as in the previous section, the following Tables 3.58 and Figures 3.42 - 3.44 were created to demonstrate the difference between the hand calculations and SAP. It is worth mentioning that SAP 2000 simulations are always more accurate as they use more precise methods of determining the internal forces, while the hand calculations use approximate methods to find out these variables.

Portal frame method itself assumes the following: 1) the internal frames have twice as much shear as the external frames as the “portals” are adding to one another; 2) the moment at the center of a column and a beam are taken as zero.

Table 3.58. Comparison of internal forces under wind load (2D, 3D, and hand calculations)

Floor	Frame 3 under the wind load (External Column)								
	Hand			2D			3D		
	Shear force, kN	Axial force, kN	Momen t, kN-m	Shear force, kN	Axial force, kN	Momen t, kN-m	Shear force, kN	Axial force, kN	Momen t, kN-m
12	1.84	1.01	3.04	1.94	1.21	2.29	1.91	1.17	2.36
11	3.66	4.04	6.05	3.93	4.43	4.96	3.8	4.41	5
10	5.46	9.06	9.01	5.65	9.71	7.09	5.41	9.75	7.09
9	7.22	16.03	11.91	7.33	16.99	8.83	6.94	17.11	8.8
8	8.95	24.93	14.77	9.04	26.2	10.51	8.52	26.43	10.46
7	10.64	35.71	17.56	10.34	37.23	11.1	9.63	37.64	10.83
6	12.30	48.33	20.29	12.98	49.91	15.42	12.2	50.58	15
5	13.91	62.74	22.95	13.8	64.29	17.37	12.75	65.32	16.1
4	15.47	78.90	25.53	16.88	79.82	26.08	15.78	81.36	23.71
3	16.97	96.74	28.00	18.47	95.8	38.72	17.13	98.11	33.95
2	18.40	116.20	30.36	22.76	110.71	68.81	21.59	113.99	61.35
1	19.92	138.56	34.86	30.79	121.17	144.96	29.53	125.43	133.12

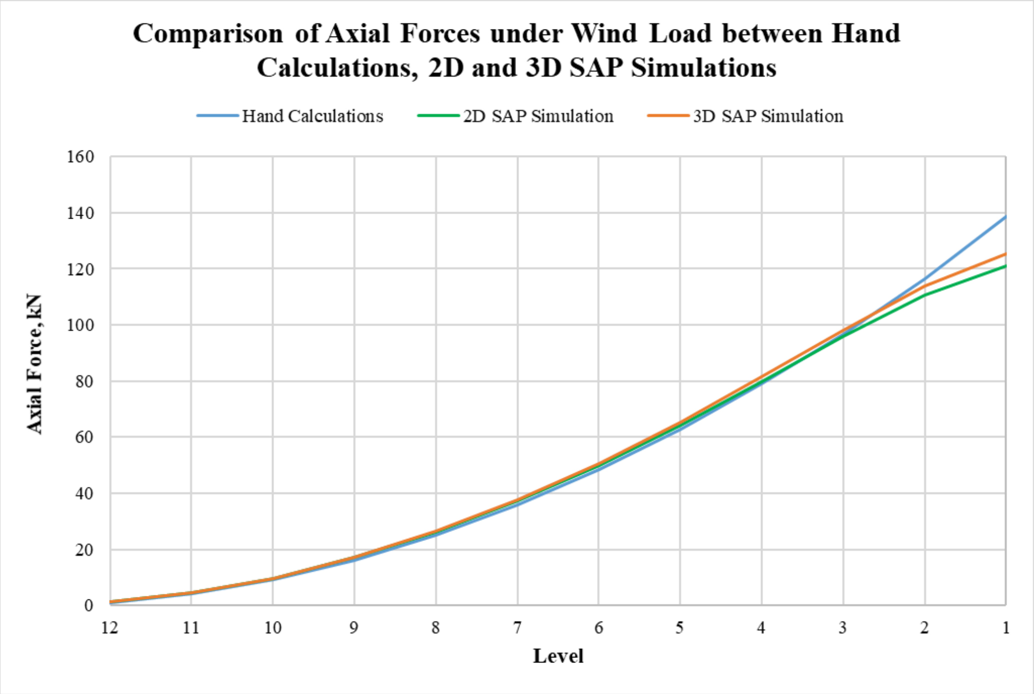


Figure 3.42. Wind Load Axial Force Comparison between Hand Calculations and SAP 2000

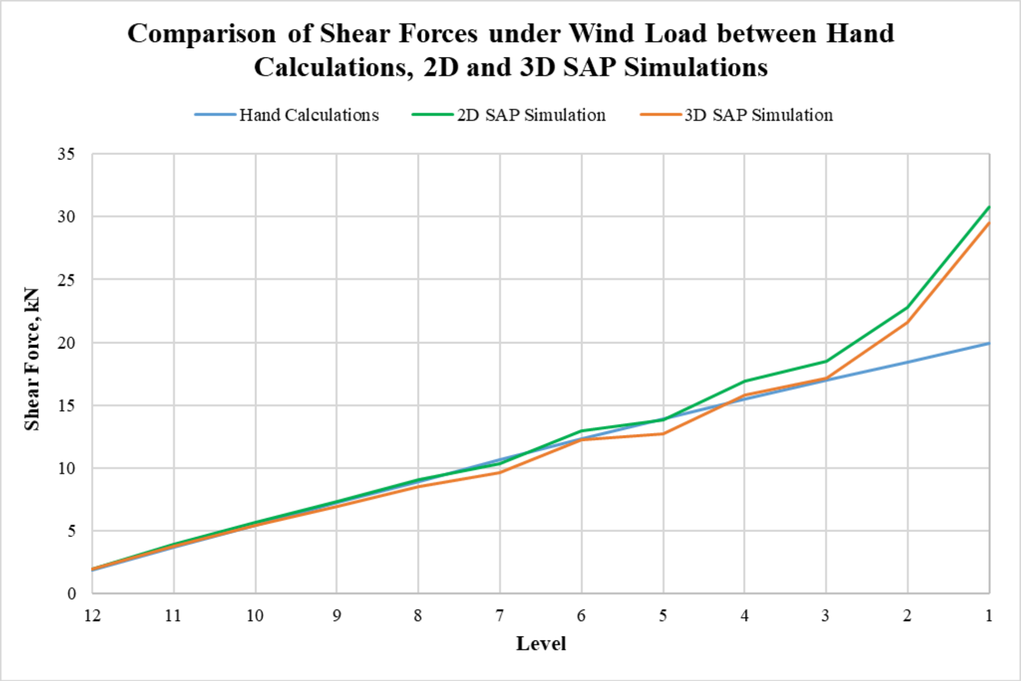


Figure 3.43. Wind Load Shear Force Comparison between Hand Calculations and SAP 2000

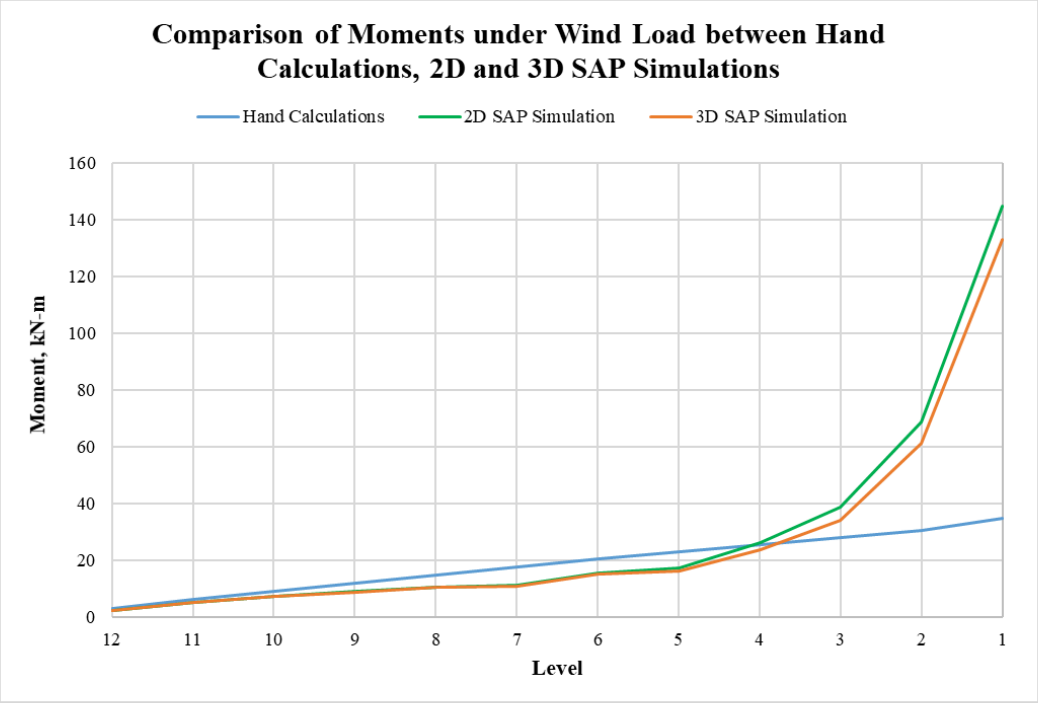


Figure 3.44. Wind Load Moment Comparison between Hand Calculations and SAP 2000

3.6.3. Load Combinations

In the following section of Structural Design, the nex load combinations depicted in Figure X will be used to account for any possible case or scenario that may happen to the building. As can be seen from the figure, all of the load combinations are in compliance with the ASCE-7 code

ASCE-7

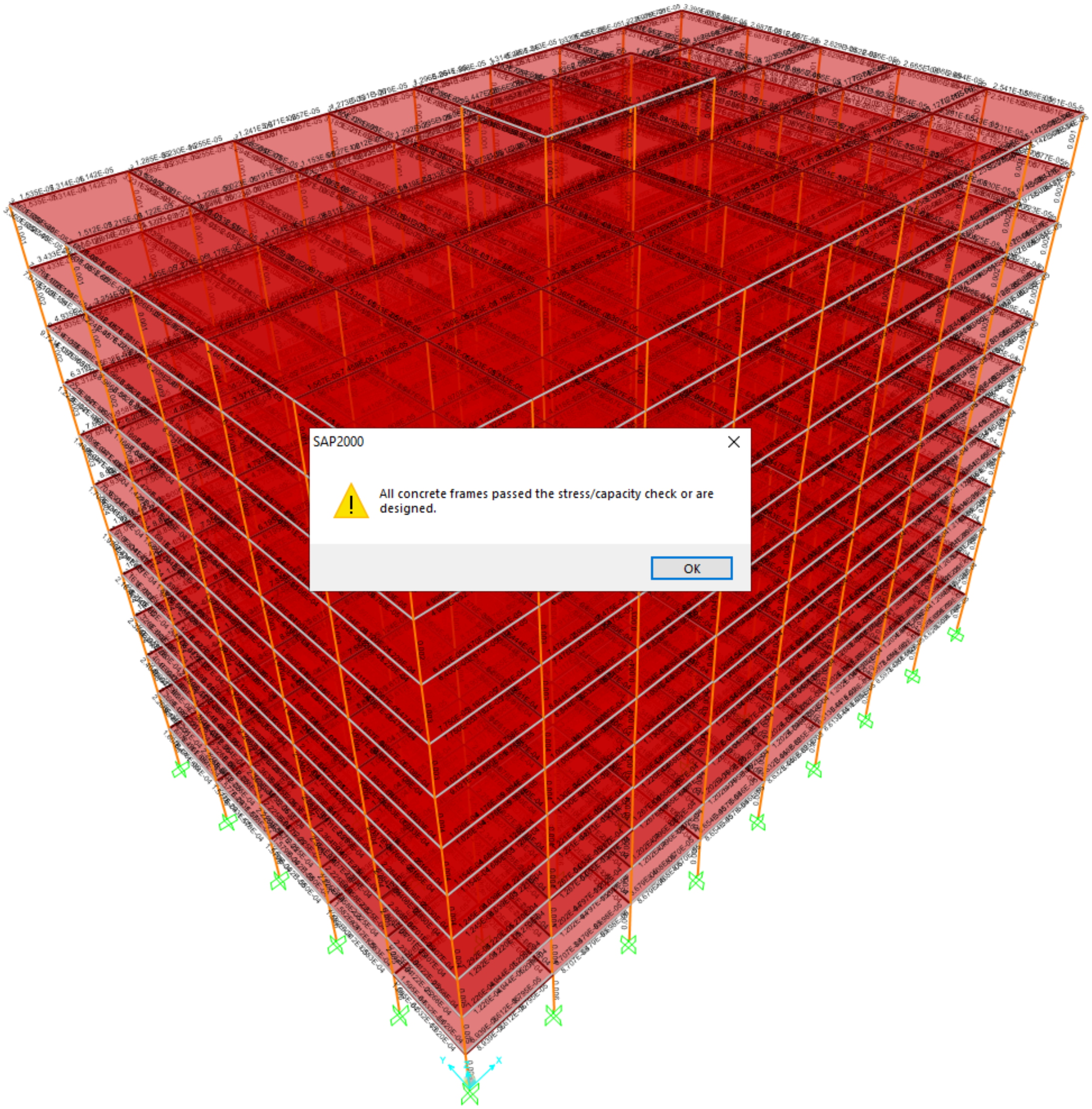
1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Figure 3.45. Load Combinations

3.7. Structural Members Design

SAP 2000 was extensively used in the process of designing the structural members. The software displays steel rebar percentages for specific loading cases. All of the values taken from SAP are verified by hand calculations you can find below. SAP 2000 conforms with the requirements established by ACI 318-19.

Below Figure X depicts the Structural Design Verification procedure to obtain the information whether the structural members pass the applied loading combinations. As can be seen all of the frames successfully pass all the structural load combinations, and therefore, the model can be used in designing the structural members.



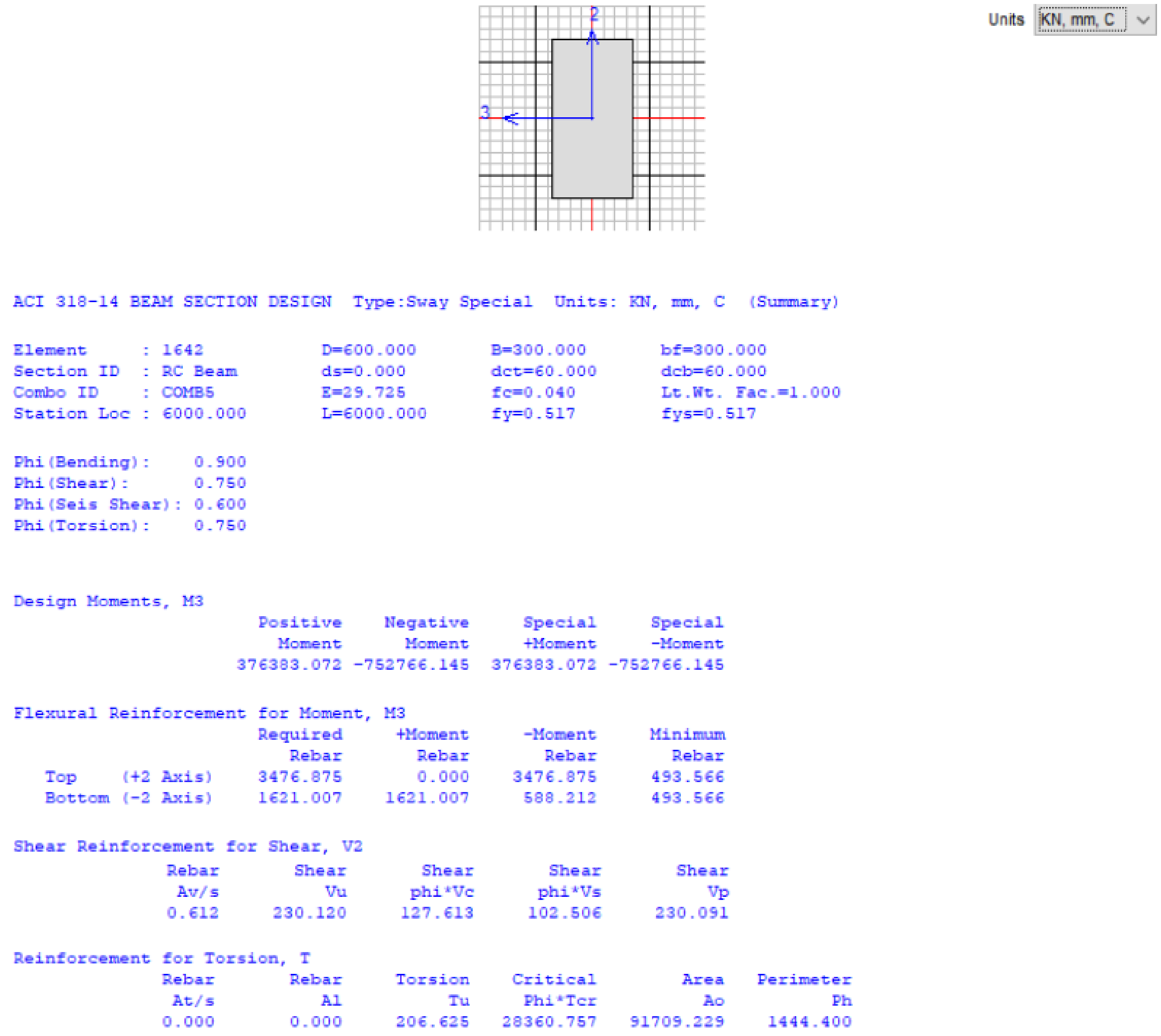
All concrete frames passed the stress/capacity check or are designed.

Figure 3.46. Capacity Check Verification

3.7.1. Major Beam Design

3.7.1.1. Longitudinal Reinforcement Design

Below you will find Figure X taken from SAP 2000 to verify the value of the required rebar. As an example, we will show the entire design procedure for the case when the negative moment governs the major beam. In this specific situation the major beam's negative moment is equal to -752.766 kN/m. Negative moment will be used to design the top reinforcement, while positive moment in the next figure will be applied to create a bottom reinforcement design, as both of these SAP 2000 summaries are for the END section of the major beam, where the compressive forces are higher than the tensile ones.



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

Element      : 1642          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000      L=6000.000      fy=0.517        fys=0.517

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


Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     376383.072 -752766.145  376383.072 -752766.145

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    3476.875       0.000    3476.875     493.566
   Bottom (-2 Axis)    1621.007    1621.007     588.212     493.566

Shear Reinforcement for Shear, V2
                  Rebar       Shear       Shear       Shear       Shear
                   Av/s          Vu     phi*Vc      phi*Vs          Vp
                  0.612     230.120     127.613     102.506     230.091

Reinforcement for Torsion, T
                  Rebar       Rebar     Torsion    Critical        Area   Perimeter
                   At/s          Al          Tu     Phi*Tcr          Ao          Ph
                  0.000       0.000     206.625   28360.757   91709.229    1444.400
```

Figure 47. Major Beam Reinforcement Design for Negative Moment

$$M_u = 752.77\, kN - m$$

$$b = 300\, mm,\ h = 600\, mm,\ d = 700 - 2.5\, in \cdot 25.4\, mm/in = 536.5\, mm \sim 530\, mm$$

$$f'_c = 40\, MPa,\ f_y = 517.1\, MPa$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{752.77}{0.9 \cdot 0.3 \cdot 0.53^2} = 8717.65\, kN/m^2$$

$$\rho = \frac{0.85 f'_c}{f_y}[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}] = 0.01986$$

$$A_s = \rho b d = 0.01886 \cdot 300 \cdot 530 = 3196.02\, mm^2$$

$$A_s f_y = 0.85 f'_c a b \rightarrow a = \frac{3196{,}02 \cdot 517.1}{0.85 \cdot 40 \cdot 300} = 162.06\, mm$$

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

$$c = \frac{a}{\beta_1} = \frac{162.06}{0.76} = 213.2\,mm$$

$$\varepsilon_t = 0.003(\frac{d-c}{c}) = 0.003(\frac{530-202.5}{202.5}) = 0.0055 > 0.005 \rightarrow \text{tension-controlled and ductile}$$

A_s calculated by SAP2000 is 3477 mm^2, which is greater than the value calculated above. It would be reasonable to find the error first:

$$Error = \frac{3477-3196.02}{3477} \times 100\% = 8.08\%$$

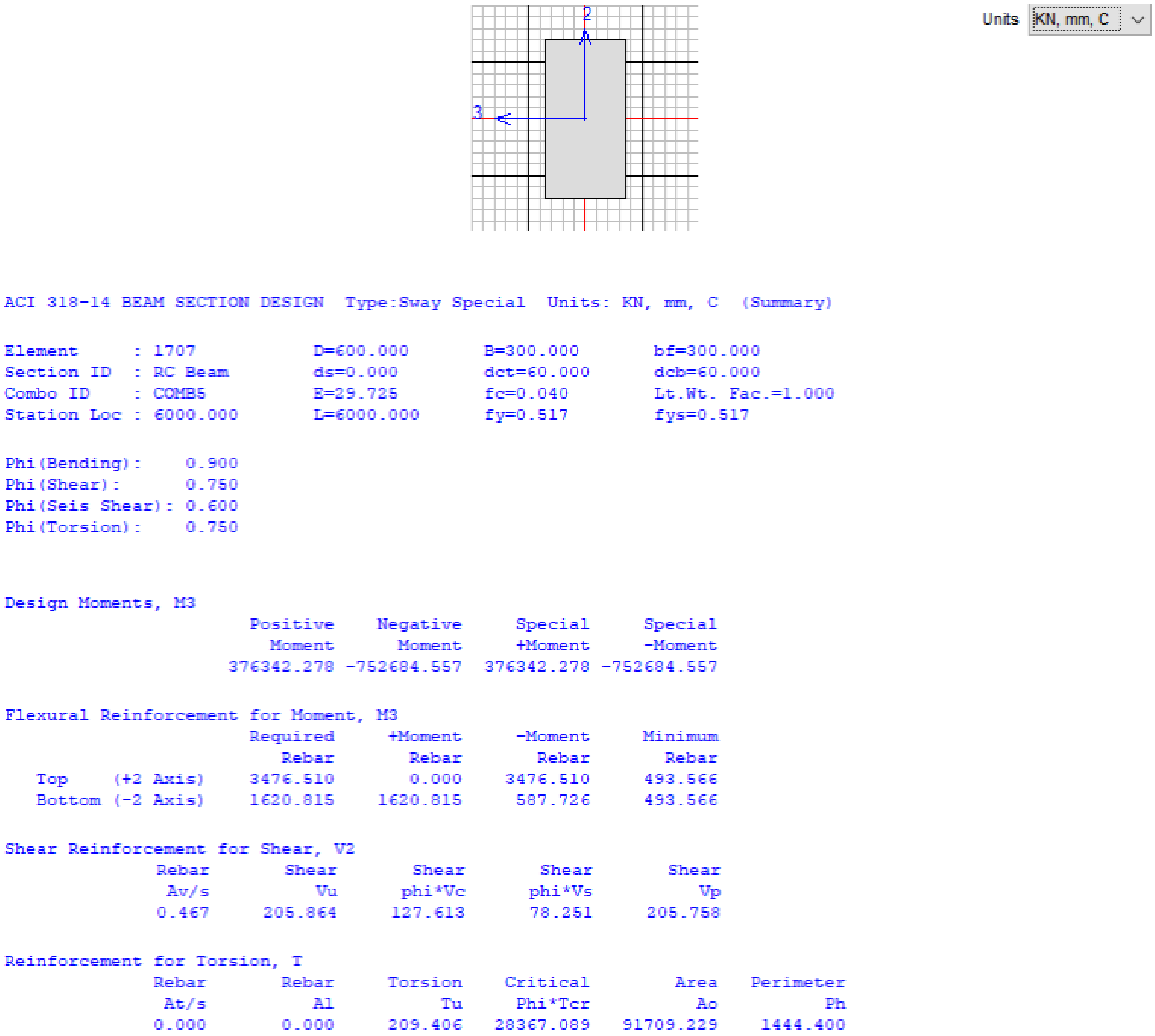
The error is not too significant and can be explained by a variety of different factors. The minimum steel area can also be checked, and as can be seen from lower equations, this criterion is satisfied:

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 613\,mm^2 > 537\,mm^2$$

$$A_s = 3477\,mm^2 > 613\,mm^2 \rightarrow \text{sufficient reinforcement}$$

Therefore, the A_s = 3477 mm^2 (5.389 in^2) can be used to select the bar size. Thus, at top reinforcements 6#9 bars will be implemented (A_s = 5.39 in^2). The stirrup chosen is #3 at 130 mm center-to-center. All of the detailed characteristics of the major beam reinforcements, dimension, spacing, clear cover, edge distance can be found in **Technical Drawings**.

Now for the positive moment used to design the bottom reinforcement at the END section of the major beam. It is important to mention that the reinforcement design was made for each individual section of the major beam on each of the floors. Here we only provide the calculation procedure necessary to prove the SAP 2000 steel percentage. All of the detailed design decisions are attached to **Apendix**.



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

Element      : 1707          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000      L=6000.000      fy=0.517        fys=0.517

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


Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                     376342.278  -752684.557   376342.278  -752684.557

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)   3476.510        0.000     3476.510      493.566
   Bottom  (-2 Axis)   1620.815     1620.815      587.726      493.566

Shear Reinforcement for Shear, V2
                  Rebar        Shear        Shear        Shear        Shear
                   Av/s           Vu      phi*Vc       phi*Vs           Vp
                  0.467      205.864      127.613       78.251      205.758

Reinforcement for Torsion, T
                  Rebar        Rebar      Torsion     Critical         Area    Perimeter
                   At/s           Al           Tu      Phi*Tcr           Ao           Ph
                  0.000        0.000      209.406    28367.089    91709.229     1444.400
```

Figure 48. Major Beam Reinforcement Design for Positive Moment

$$M_u = 376.34\ kN - m$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{376.34}{0.9 \cdot 300 \cdot 536.5} = 4358.35\ kN/m^2$$

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

$$A_s = \rho b d = 0.009 \cdot 300 \cdot 536.5 = 1456.8\ mm^2$$

$$A_s f_y = 0.85 f'_c ab \rightarrow a = \frac{1456.8 \cdot 517.1}{0.85 \cdot 40 \cdot 300} = 73.86\ mm$$

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

$$c = \frac{a}{\beta_1} = \frac{73.86}{0.76} = 97.19\ mm$$

$$\varepsilon_t = 0.003(\frac{d-c}{c}) = 0.003(\frac{536.5-97.19}{97.19}) = 0.01356 > 0.005 \rightarrow \text{tension-controlled and ductile}$$

A_s calculated by SAP2000 is 1621 mm^2, which is greater than the value calculated above. It would be reasonable to find the error first:

$$Error = \frac{1621-1456.8}{1621} \times 100\% = 10.1\%$$

The error is not too significant and can be explained by a variety of different factors. The minimum steel area can also be checked, and as can be seen from lower equations, this criterion is satisfied:

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 612.9\ mm^2 > 537\ mm^2$$

$$A_s = 1621\ mm^2 > 1199\ mm^2 \rightarrow \text{sufficient reinforcement}$$

Therefore, A_s = 1621 mm^2 (2.512 in^2) can be used to select the bar size. Thus, at bottom reinforcements 3#9 bars will be implemented (A_s = 3 in^2). The stirrup chosen is #3 at 130 mm center-to-center. All of the detailed characteristics of the major beam reinforcements, dimension, spacing, clear cover, edge distance can be found in **Technical Drawings**.

As was mentioned before, the major beam reinforcement design was made for each individual section of the beam, being END-MID-END. As you can see SAP 2000 values were verified and now Table X like the one below can be made for each beam on every floor of the building. For other tables, refer to the **Appendix** section

Table 3.59. Detailed Major Beam Reinforcement Design for Floor 4

Major Beam for Floor 4			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	123	945	1621
Area, in2	0.19	1.46	2.51
SAP 2000 Compression Area, mm2	247	777	3477
Area, in2	0.38	1.20	5.39
Tension Bars	**2#3**	**2#8**	**3#9**
Compression Bars	**2#4**	**4#5**	**6#9**

Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

3.7.1.2. Shear Reinforcement Analysis

From Figure X the maximum $V_u = 230.12$ kN. $\lambda = 1$ for normal-weight concrete.

$$\phi V_c = \phi \cdot 2\lambda\sqrt{f'_c} b_w d = 0.75 \cdot 2 \cdot \sqrt{40000} \cdot 300 \cdot 530 = 48.29\, kN$$

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

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

$$V_s = \frac{V_u - \phi V_c}{\phi} = 242.45\, kN < V_{c2}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{242.45}{0.517 \cdot 530} = 0.874\, mm^2/mm$$

The value of A_v/s calculated by SAP2000 is 0.612 mm^2/mm.

$$Error = \frac{0.874 - 0.612}{0.874} \times 100\% = 8\%$$

The error is not significant. The shear reinforcement to be used is #3 at spacing 130 mm center-to-center.

3.7.1.2. Torsional Reinforcement Analysis

From Figure X, in SAP 2000 Tu is equal to 0.21 kN-m. This is significantly smaller than the design Tn value of 2.7 kN-m. The torsional reinforcement design value was calculated using this formula:

$$T_n = \phi\lambda\sqrt{f'_c}(\frac{A_{cp}^{2}}{P_{cp}}) = 2.7\, kN - m > T_u = 0.21\, kN - m$$

Thus, torsional reinforcement is ***NOT*** required.

3.7.2. Column Design

3.7.2.1. Slenderness Ratio

Slenderness Ratio Calculations

From SAP 2000, $P_u = 10297\, kN$

$I_{column} = 0.018457\, m^4$, $I_{beam} = 0.00189\, m^4$,

$L_{column} = 3.5\, m$, $L_{beam} = 6\, m$

1. Interior column ($750\, mm \times 750\, mm$):

$$\Psi = \frac{\Sigma EI/l_c \, of \, columns}{\Sigma EI/l_c \, of \, beams} = 8.37 \rightarrow K = 2.7$$

$$\frac{Kl_u}{r} = 34.8 > 28 \rightarrow slender \ column$$

2. Corner column ($750\,mm \times 750\,mm$):

$$\Psi = \frac{\Sigma EI/l_c \, of \, columns}{\Sigma EI/l_c \, of \, beams} = 16.74 \rightarrow K = 3.6$$

$$\frac{Kl_u}{r} = 46.4 > 28 \rightarrow slender \ column$$

3. Exterior column ($750\,mm \times 750\,mm$):

$$\Psi = \frac{\Sigma EI/l_c \, of \, columns}{\Sigma EI/l_c \, of \, beams} = 11.16 \rightarrow K = 3.2$$

$$\frac{Kl_u}{r} = 32.7 > 28 \rightarrow slender \ column$$

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

1) Firstly, the column buckling is calculated: $P_c = \frac{\pi^2 EI}{(kl_u)^2} = 89137\,kN$
2) After that the sway magnification coefficient is determined:

$$\delta_S = \frac{1}{1 - \frac{\Sigma P_U}{0.75 \Sigma P_C}} = 1.182 < 2.5$$

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

$$\frac{l_u}{r} < \frac{35}{\sqrt{P_u/(f_c{}'A_g)}} = 12.89 < 51.74 \rightarrow No \ further \ magnification \ is \ required$$

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

$$M_1 = \delta_S M_1 = 253.03\,kN - m$$

$$M_2 = \delta_S M_2 = 459.33\,kN - m$$

3.7.2.2. Longitudinal Rebars

Interior (General) Column Design

1) $P_n = \frac{P_u}{\Phi} = 15838\,kN$
2) $M_n = \frac{M_{crit}}{\Phi} = 706.7\,kN - m$
3) $e = \frac{M_n}{P_u} = 0.044\,m$
4) $K_n = \frac{P_n}{f_c{}'A_g} = 0.704$
5) $R_n = \frac{K_n e}{h} = 0.0419$

6) $\rho = 0.01 \rightarrow A_S = 0.005625\, m^2 \rightarrow 8\ \#9\ bars$

3.7.2.3. Shear Reinforcement

Shear Strength Check

1) $V_{u1} = 91.461\, kN;\ V_{u2} = 1.189\, kN;\ V_c = 23755\, kN$
2) $\Phi V_C = 17816\, kN$
3) $\frac{\Phi V_c}{2} = 8908\, kN$
4) $V_u < 0.5\Phi V_c \rightarrow Only\ minimum\ reinforcement\ is\ required \rightarrow \#3\ ties\ @\ 200\ mm$

3.7.2.4. Biaxial Bending

Biaxial Bending Check:

1) $M_{ux} = 253.03\, kN - m \rightarrow M_{nx} = 389.28\, kN - m$
2) $M_{uy} = 459.33\, kN - m \rightarrow M_{yx} = 706.67\, kN - m$
3) $\gamma_x = \gamma_y = 0.82;\ e_x = 0.025m;\ e_y = 0.045\, m$
4) $P_{nx} = 45000\, kN;\ P_{ny} = 24231\, kN;\ P_o = 19128\, kN;\ P_n = 89189\, kN$
5) $P_n > 0.1P_{n0} \rightarrow The\ method\ is\ feasible$
6) $\varphi P_n > P_u \rightarrow 57972\, kN > 10297\, kN \rightarrow Safe\ against\ biaxial\ bending$

3.7.3. Two-Way Slab

For the two-way slab design, the procedure similar to that of the major beam was implemented, where the two-way slab was divided into separate strips that were further divided into separate sections. As the two-way slab contains no minor beams, its dimensions are 6000 mm by 6000 mm with slab thickness of 150 mm.

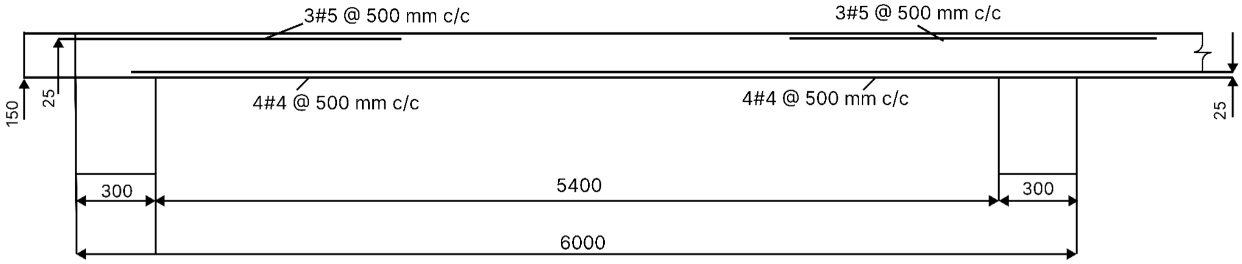


Figure 49. Two-Way Slab Design

3.7.3.1. Flexural analysis

$$\overline{y} = \frac{13500*75+150*375}{13500+150} = 170\,mm$$

$$I_b = \frac{1050*150^3}{12} + 13500(170 - 75)^2 + (150 * 450^3)/12 + 150 * (375 - 75)^2 = 5.9 * 10^9 mm^4$$

$$I_s = 6000/2 * 150^3/12 = 8.9 * 10^8\,mm^4$$

$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = 4.52$$

Minimum height can be determined through the following

$$h_{min} = \frac{l_n(0.8+\frac{f_y}{1400})}{36+9\beta} = 2625 * (0.8 + 420/1400)/(36 + 9 * 1) = 64.49\,mm \qquad (3.64)$$

Factored load is calculated here:

$$q_u = 1.2q_d + 1.6q_l = 1.2 * 5.6568 + 1.6 * 4.79 = 14.88\,kN/m^2$$

Total static moment:

$$M_0 = \frac{q_u l_2 l_0^2}{8} = \frac{14.45*6.75*6^2}{8} = 448.45\,kN - m \qquad (3.65)$$

$$M(-) = 0.65 * M_0 = 0.65 * 438.98 = 275.32\,kN - m$$

$$M(+) = 438.98 - 285.34 = 150.14\,kN - m$$

$$\alpha_{f1} = \alpha_s = 4.52 \geq 1.0$$

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

$$\overline{y} = \frac{67500*75+0*375}{67500+0} = 80\,mm$$

$$I_b = 1.27 * 10^8\,mm^4$$

$$I_s = 7.38 * 10^8\,mm^4$$

$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = 4.48$$

As α_{fm} is greater than 2.0, minimum height can be calculated as follows:

$$h_{min} = \frac{l_n(0.8+\frac{f_y}{1400})}{36+9\beta} = 2625 * (0.8 + 420/1400)/(36 + 9 * 1) = 64.19\,mm$$

As the thickness is satisfied, factored loads can be calculated:

$$q_u = 1.2q_d + 1.6q_l = 1.2 * 5.6568 + 1.6 * 4.79 = 14.54\ kN/m^2$$

Table 3.60. Two-Way Slab Design Procedure for exterior panel

Exterior	**Column Strip**			**Middle Strip**		
	Negative	**Positive**	**Negative**	**Negative**	**Positive**	**Negative**
Moment %	97.29	75.00	75.00	2.71	25.00	25.00
M_u, kN-m	192.34	79.84	148.27	5.35	26.61	49.42
$M_{u,slab}$, kN-m - 15% to slab	28.85	11.98	22.24	0.80	3.99	7.41
b, m	1.65	1.65	1.65	1.65	1.65	1.65
R_n, kPa	1491.38	619.06	1149.68	41.52	206.35	383.23
ρ	0.0030	0.0012	0.0023	0.0001	0.0004	0.0007
A_s, m^2	0.0006	0.0002	0.0004	0.00002	0.00008	0.00014
A_s, in^2	0.861	0.352	0.660	0.023	0.117	0.218
$A_{s,min}, in^2$	0.691	0.691	0.691	0.691	0.691	0.691
$A_{s,final}, in^2$	0.861	0.691	0.691	0.691	0.691	0.691
Bar Selected	3#5	4#4	4#4	4#4	4#4	4#4
Spacing, m	0.5	0.5	0.5	0.5	0.5	0.5
Minimum Spacing, mm	25.4	25.4	25.4	25.4	25.4	25.4

Table 3.61. Two-Way Slab Design Procedure for interior panel

Interior	**Column Strip**			**Middle Strip**		
	Negative	**Positive**	**Negative**	**Negative**	**Positive**	**Negative**
Moment %	75.00	75.00	75.00	25.00	25.00	25.00
new Mu (kN-m)	148.27	79.84	148.27	49.42	26.61	49.42
Mu slab (kN-m)	22.24	11.98	22.24	7.41	3.99	7.41
b, m	2.06	2.06	2.06	2.06	2.06	2.06
Rn (kPa)	919.74	495.24	919.74	306.58	165.08	306.58
ρ	0.0018	0.0010	0.0018	0.0006	0.00032	0.00060
As (m^2)	0.00042	0.00023	0.00042	0.00014	0.00008	0.00014
As (in^2)	0.65801	0.35204	0.65801	0.21731	0.11677	0.21731
Asmin (in^2)	0.691	0.691	0.691	0.691	0.691	0.691
As final (in^2)	0.691	0.691	0.691	0.691	0.691	0.691

Bar Selected	4#4	4#4	4#4	4#4	4#4	4#4
Spacing (m)	478	500	500	500	500	500
Minimum Spacing (mm)	25.4	25.4	25.4	25.4	25.4	0.4
Minimum Spacing (in)	1	1	1	1	1	1

3.7.3.2. Shear analysis

$$d_{average} = 4.92 - 3/4 - 5/8 = 3.755\, in$$

$$d_{average} = 3.545 * 25.4 = 92.95\, mm$$

$$V_u = [l_1 l_2 - (b + d_{avg})(h + d_{avg})] * q_u \qquad (3.66)$$

$$V_u = (36 * (0.75 + 90.043/1000)) * 14.45 = 510.08\, kN$$

$$\lambda = 1.0$$

$$\varphi V_{c1} = \varphi(0.17\lambda\sqrt{f'_c} + \frac{N_u}{6A_g})b_w d = 358.47\, kN \qquad (3.67)$$

$$\varphi V_{c2} = \varphi(0.66\lambda\rho^{(1/3)}\sqrt{f'_c} + \frac{N_u}{6A_g})b_w d = 31.36\, kN$$

3.7.4. Joint design

The following procedure below is in accordance with ACI 318-19. Joint design can be presented as follows:

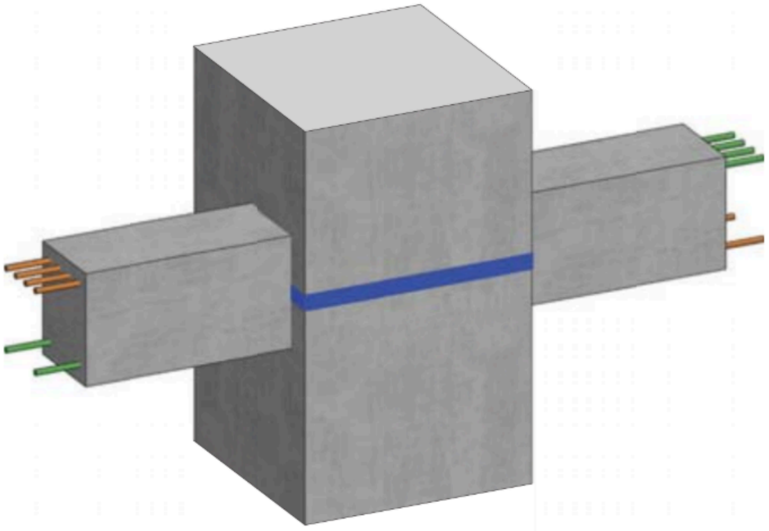


Figure 3.50. Joint design between the column and beam (ACI, 2019)

Interior beam and column were chosen as an example to show the following procedure. The shear between these two structural elements was computed by applying the following equation.

$$V_{u,joint} = T_{pr1} + C_{pr2} - V_{col} \tag{3.68}$$

To determine the joint shear, the reinforcement details were found by utilizing SAP2000 for the interior beam-column joint (see Figure 3.26).

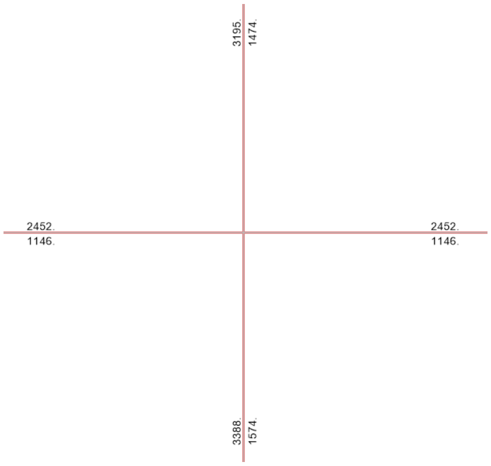


Figure 3.51. Reinforcement details at the beam-column joint.

The parameters presented in the previous equation have been computed as follows:

$$T_{pr1} = \alpha A_s f_y = 1.25 \cdot 2452 \cdot 0.517 = 1584.61\ kN$$

$$C_{pr1} = \alpha A_s f_y = 1.25 \cdot 1146 \cdot 0.517 = 740.60\ kN$$

SAP 2000's results are the following:

$$V_{col} = 424\ kN$$

Respectively, the shear demand on joint:

$$V_{u,joint} = 1584.61 + 740.60 - 424 = 1901.21\ kN$$

To determine the shear strength of the joint, the effective width, b_j, was calculated first:

$$b_j = \frac{b_b + b_{col}}{2} = \frac{300+750}{2} = 525\ mm < b_b + h_{col} = 300 + 750 = 1050\ mm$$

$$b_j = 525\ mm < b_b + 2x = 300 + 750 = 1050\ mm$$

To calculate shear capacity, we use the following formula:

$$V_n = \gamma \sqrt{f'_c} A_j \tag{3.69}$$

From that formula we have γ = 1.7 for confined interior joint and A_j is the product of b_j and h_{col}. Therefore:

$$V_n = \gamma\sqrt{f'_c}A_j = 1.7\sqrt{40} \cdot 525 \cdot 750 = 4233.5\,kN$$

The nominal shear strength must satisfy $\phi V_n \geq V_u$. To account for the strain hardening of the reinforcement, the strength reduction factor φ is taken as 0.85.

$$0.85 \cdot 4233.5 = 3598.48\,kN \geq V_u = 1901.21\,kN$$

As a result, it was confirmed that the joint is safe under shear forces. The same method was used to check the joint's shear capacity in other directions, and it was also found to be safe. In addition, the spacing requirements were calculated based on ACI 15.4.2, using the following formula:

$$A_v = max(\frac{0.062\sqrt{f'_c}b_c s}{f_{yt}}, \frac{0.35 b_c s}{f_{yt}}) \tag{3.70}$$

$$s = min(\frac{A_v f_{yt}}{0.062\sqrt{f'_c}b_c}, \frac{A_v f_{yt}}{0.35 b_c}) \tag{3.71}$$

The minimum spacing with 2#3 ties:

$$s = min(\frac{2\cdot71\cdot517.1}{0.062\sqrt{40}\cdot750}, \frac{2\cdot71\cdot517.1}{0.35\cdot750}) = min(249.68, 279.73) = 249.68\,mm$$

Therefore, two #3 ties spaced at 240 mm will be used to satisfy the code requirements. Additionally, the following check should be carried out to make sure that plastic hinges form in the beams rather than in the columns.

$$\Sigma M_{nc} \geq \frac{6}{5}\Sigma M_{nb} \tag{3.72}$$

$$537.5 + 478.3 = 1015.8\,kN - m \geq \frac{6}{5}(153.6 + 114.7) = 268.3\,kN - m$$

3.7.5 Bar Selection

Table 3.62 below summarizes the selected rebars. As can be seen there is no major beam in the table as each individual beam on each floor has its own unique reinforcement design. Thus, to explore each beam individually, refer to **Appendix**.

Table 3.62. Final selected bars.

Component	Top reinforcement	Bottom reinforcement	Stirrups
Two-way slab	3#5	4#4	
Column types		**Reinforcement**	**Ties**
Column 0-1 floors (750 x 750 mm)		8#9	#3 at 200 mm
Column 2-3 floors (700 x 700 mm)		8#9	#3 at 200 mm
Column 4-5 floors (650 x 650 mm)		8#9	#3 at 200 mm
Column 6-7 floors (600 x 600 mm)		8#8	#3 at 200 mm
Column 8 (550 x 550 mm)		8#7	#3 at 200 mm
Column 9 (500 x 500 mm)		8#6	#3 at 200 mm
Column 10 (450 x 450 mm)		8#6	#3 at 200 mm
Column 11 (400 x 400 mm)		8#5	#3 at 200 mm
Column 12-13 (350 x 350 mm)		8#5	#3 at 200 mm

3.7.6. Development length

ACI 318-19 was used in calculation of development length. The chosen bars are #8 and #9 which have 1 in and 1.128 in diameters in turn. The equation below was used to compute the tension reinforcement:

$$\frac{l_d}{d_b} = \frac{f_y \Psi_t \Psi_e \Psi_g}{20\lambda\sqrt{f_c'}} \quad (for\ bars > \#7) \rightarrow l_d = 1410\ mm > 750\ mm\ (column\ width)$$

As can be seen from the previous equation the hooked bar is needed to accommodate for the entire length of the longitudinal reinforcement as it does not fit into the largest dimension-size column. The equation below is to be used to find the hooked rebars:

$$l_{dh} = \left(\frac{f_y \Psi_e \Psi_r \Psi_o \Psi_c}{55\lambda\sqrt{f_c'}}\right) d_b^{1.5}\ (modification\ factor) \rightarrow l_{dh} = 381.4\ mm > 8d_b = 229\ mm$$

Thus, l_{dh} = 381.4 mm for the bars in tension. Subsequently, the 90° hook's parameters were found:

$$D = 6d_b = 9.024\ in,\ r = D/2 = 4.512\ in\ or\ 114.6\ mm$$

$$\text{Hook distance: } 12d_b = 13.56\ in\ or\ 343.8\ mm$$

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

For the bars in compression, the development length was calculated using the formula below:

$$l_{dc} = (\frac{f_y \Psi_r}{50\lambda\sqrt{f_c'}})d_b \geq 0.0003f_y\Psi_r d_b \rightarrow l_{dc} = 250\ mm < 564\ mm$$

Therefore, as can be seen all of the ACI 318-19 requirements on the development length were satisfied

3.7.7. Lap splices

The lap splices were calculated for each individual member. Tension controlled beams have the following lap splice value:

$$l_{st} = l_{dh} = 380\ mm$$

While beam in compression require the following

$$l_{sc} = 0.0005f_y d_b = 38\ in\ or\ 965mm$$

To calculate the lap splice value for columns, the following Table 3.63 is created to demonstrate how the lap splices differ on each floor.

Table 3.63. Column Lap Splices

Component	Lap splice, mm
Column 1-2 floors (700 x 700 mm)	1074
Column 3-4 floors (650 x 650 mm)	1074
Column 5-6 floors (600 x 600 mm)	953
Column 7-8 floors (550 x 550 mm)	833
Column 9-10 (500 x 500 mm)	714
Column 11-12 (450 x 450 mm)	714

3.8. Special seismic provisions for reinforcement detailing

3.8.1. Beams

Due to the building's location in a high seismicity zone, special seismic detailing was applied. Both the clear span and the width-to-depth ratio comply with the requirements of ACI 318-19 Section 18.6. In addition, at least two continuous longitudinal bars were placed at both the top and bottom of the beams, as specified in Section 18.6.3.1. The positive moment capacity at the face of the joint was designed to exceed half of the negative moment capacity. To ensure confinement, hoops were installed along a length equal to twice the member depth from the face of the support, with spacing limited to the smallest of d/4, 6db, or 150 mm. The first hoop was positioned 50 mm away from the support face.

3.8.2. Columns

Seismic detailing for the columns was carried out in accordance with Section 18.7 of ACI 318-19. The reinforcement area in the columns was designed to range between 1% and 6% of the cross-sectional area. Transverse reinforcement was provided on both sides of the joint over a length l_0, where l_0 was taken as one-sixth of the clear span of the column, resulting in l_0 = 400 mm.

3.8.3. Joints

The longitudinal reinforcement of the beams was extended to the far face of the columns at the joints. In cases where the longitudinal bars continued through the columns, the column depth h was designed to be greater than 20 times the bar diameter (20db).

The development lengths of the reinforcement bars were designed to meet the required criterion:

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

3.9. Structural serviceability design

3.9.1. Minimum Deflection

Minimum deflection requirements for structural components like beams and slabs are established by ACI 318-14. When these elements do not match the required minimum thickness requirements, their deflections must be calculated. In this case, the fact that the minimum thickness requirements were satisfied suggests that the deflections are within acceptable bounds.

$$h_b = \frac{L}{21} \tag{3.73}$$

Where L is the length of the major beam. In this case, the following is true:

$$h_b = \frac{6000}{21} = 286\, mm \,<\, 300\, mm \text{ (depth of minor beam)}$$

The minimum dimension thickness of the major beam is greater than the minimum thickness, and so the requirements are satisfied.

3.9.2. Crack width

Crack control in reinforced concrete design plays a critical role in ensuring the structural safety under applied loads. For this reason, it is essential to verify that the crack widths remain within the permissible limits defined by building codes. To evaluate the maximum flexural crack width, the following equation from McCormac and Brown (2016) will be used:

$$w = 0.076\beta_h f_s \sqrt[3]{d_c A} \tag{3.74}$$

Where, w - estimated crack width (in), β_h - the ratio between the distance from the extreme tension fiber to the neutral axis and the distance from the centroid of the reinforcement, which can be assumed as 1.2, f_s is the steel stress, d_c is the cover and A is the effective tension area.

$$d_c = 2.5\, in$$

$$A = \frac{(5\, in)(19.7\, in)}{6} = 16.4\, in^2$$

$$w = 0.076\beta_h f_s \sqrt[3]{d_c A} = 0.012\, in \,<\, 0.016\, in$$

The allowable crack width for this structure is 0.016 inches, and the calculated crack width remains within this limit. As a result, the crack width is considered acceptable. Additionally, the maximum permissible spacing for the longitudinal reinforcement can be determined using the following formula:

$$s(in) = [15(\frac{40}{f_s}) - 2.5C_c] \tag{3.75}$$

$$s = 15(\frac{40}{(2/3)60} - 2.5 \cdot 2.5) = 9.83\ in\ or\ 250mm$$

Therefore, the shear reinforcement spacing should not exceed 250 mm to satisfy the serviceability crtireia.

4. Geotechnical Design

4.1 Site Description

The site is located in San Francisco, California within a parking lot bordered by Mission Bay Boulevard South to the north, Nelson Rising Ln to the south, and 5th and 6th streets to the east and west. The surrounding area consists of buildings ranging from five to twelve stories. The project involves clearing the existing pavement and utilizing a 1,500 m² area for the construction of a new 12-story building with a single basement level.

4.2 Field investigation

The geotechnical assessment, conducted by Langan Treadwell Rollo, involved soil testing through four boreholes (B-1 to B-4). The depths of B-1, B-2, and B-4 ranged from 21.34 m to 27.8 m, while B-3 was shallower at 15.24 m. The soil data is extracted from the boring with greatest depth B-4. The Unified Soil Classification System (USCS) is applied for the soil classification.

The position of the groundwater table is another key factor in geotechnical design, as it influences both the foundation's bearing capacity and the soil's potential for liquefaction. Groundwater was detected at depths of approximately 6.8 m and 7.5 m in boreholes B-2 and B-3, respectively, but was not recorded in B-1 and B-4. Groundwater levels can fluctuate due to seasonal changes, underground drainage, and regional hydrological conditions.

The geotechnical report determined a shear wave velocity of $\nu_{s(30)} = 340m/s$ (approximately 1120 ft/sec), classifying the site as a Site Class D soil type.

4.2.1 Soil Profile

To ensure an accurate geotechnical design, the soil profile of the selected site must be thoroughly investigated and described. The soil investigation report provides data limited to SPT blow counts. Further analysis was conducted using standard SPT correlations, with the correlation methodologies detailly described in the Capstone Project I Report. SPT N value was utilized to calculate the standard penetration number N_{60}. Elastic modulus values for different soil layers were derived using the methods of Bowles (1996) and Kulhawy & Mayne (1990). Poisson's ratio was obtained through the equation provided by Das & Sivakugan (2019). The soil's friction angle was determined using three different formulas, with the average taken for accuracy. Cohesion values for various soil types were estimated using empirical equations selected based on soil characteristics. The resulting analyzed soil profile is presented in Table 4.2.1.1.

Table 4.2.1.1. Soil Profile.

	Soil type	z, m	$\gamma,\ kN/m^3$	μ_s	ϕ', °	δ', °	E, MPa	c_u, kN/m^2
1	Clayey sand with gravel	2.134	16.7	0.143	27.89	22.31	8470	24.5
2	Silty sand with gravel	4.268	16.8	0.163	20.19	16.15	8982	0.7272
3	Gravel with clay and sand	5.792	18.2	0.398	44.86	35.89	20255	2
4	Gravel with silt and clay	7.316	17.5	0.308	38.84	31.07	14056	1.36
5	Sandy clay	10.059	18.9	0.400	44.98	35.98	35000	58
6	Silty sand	11.888	18.8	0.438	47.55	38.04	23334	2.55
7	Clayey sand	13.412	17.1	0.242	34.44	27.55	10414	1
8	Sand	23.772	21.8	0.343	41.17	32.94	15835	5.27
9	Clayey peat	25.296	17.4	0.292	37.82	30.26	10000	1.27
10	Clay	27.430	19.3	0.461	49.05	39.24	35000	115.5

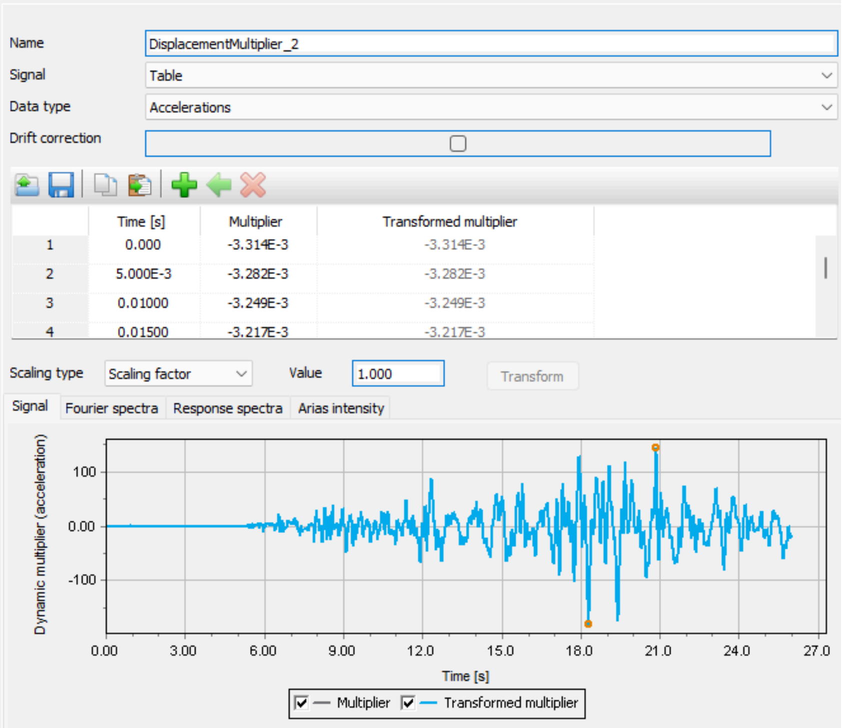
4.2.2 Liquefaction

Liquefaction is a concern in seismically active regions with loose, saturated soils, particularly where groundwater is close to the surface. During an earthquake, increased pore-water pressure can cause granular soil to lose strength and behave like a liquid (Dixit, Dewaikar, & Jangid, 2012).

Idriss and Boulanger (1971) and Luna and Frost (1998) equations were used to estimate LPI. It was equal to zero, except for the first layer with an LPI value of 3.164. Since it is lower than 5, liquefaction risk is considered to be low.

4.3 Site Response analysis

For the site response analysis of the project location, an earthquake with a magnitude of 6.0 that occurred in Napa, California was selected, based on data from the USGS government website (Geological Survey, n.d.). The analysis was conducted using PLAXIS 2D software, which involves modeling the soil layers, setting appropriate boundary conditions, and assigning relevant material properties. After specifying the type of dynamic analysis, the software simulates the soil's response to seismic loading. A dynamic multiplier was incorporated into the PLAXIS 2D model, and ground motion data from the Chicago event was inputted as a "table" format. The data type was selected as "accelerations," as illustrated.



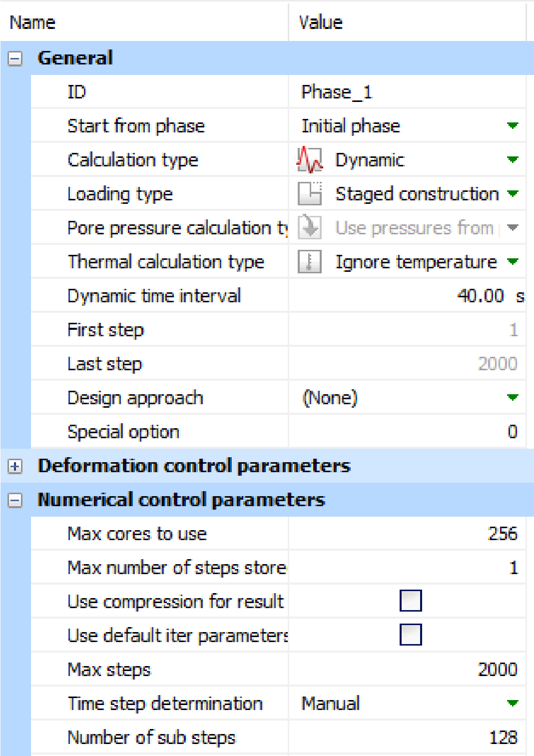
Name: DisplacementMultiplier_2
Signal: Table
Data type: Accelerations
Drift correction: ☐

	Time [s]	Multiplier	Transformed multiplier
1	0.000	-3.314E-3	-3.314E-3
2	5.000E-3	-3.282E-3	-3.282E-3
3	0.01000	-3.249E-3	-3.249E-3
4	0.01500	-3.217E-3	-3.217E-3

Scaling type: Scaling factor
Value: 1.000

Figure 4.3.1. Site Response analysis.

The model consisted of two stages: the initial phase and “Phase_1,” during which the seismic signal was applied and the dynamic analysis was carried out. A time interval of 40 seconds was selected, corresponding to the duration of the earthquake signal. Details of the phase setup and the required configurations are illustrated in Figure below.



Name	Value
General	
ID	Phase_1
Start from phase	Initial phase
Calculation type	Dynamic
Loading type	Staged construction
Pore pressure calculation ty	Use pressures from
Thermal calculation type	Ignore temperature
Dynamic time interval	40.00 s
First step	1
Last step	2000
Design approach	(None)
Special option	0
Deformation control parameters	
Numerical control parameters	
Max cores to use	256
Max number of steps store	1
Use compression for result	☐
Use default iter parameters	☐
Max steps	2000
Time step determination	Manual
Number of sub steps	128

Figure 4.3.2. Site Response analysis setup details.

Figure 4.3.3. illustrates the soil profile both before and after the application of dynamic loading conditions. It is evident that the soil underwent deformations in both the x and y directions as a result of the applied ground motion. The maximum value of the soil deformation is presented below.

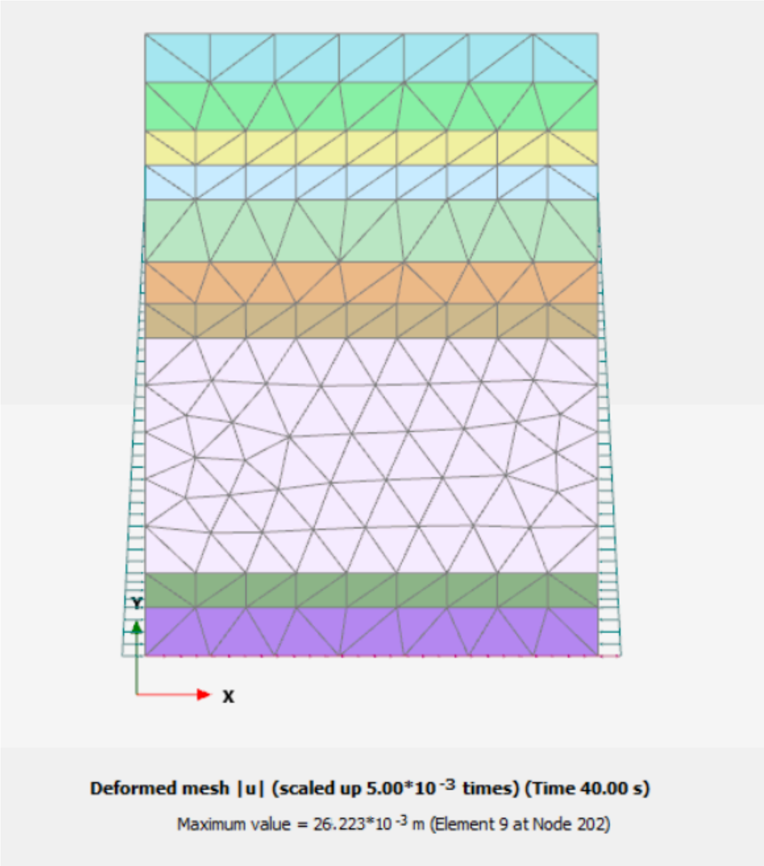


Figure 4.3.3. Soil deformation.

Following the completion of calculations in PLAXIS 2D, acceleration versus time and peak ground acceleration versus period graphs were generated and presented in Figure. The ground acceleration data was recorded over a duration of 40 seconds, with the peak acceleration observed at X occurring at X seconds. The amplitude of the ground motion was obtained by applying a Fast Fourier Transformation (FFT) to the accelerator plot, as illustrated in Figure 4.3.4.

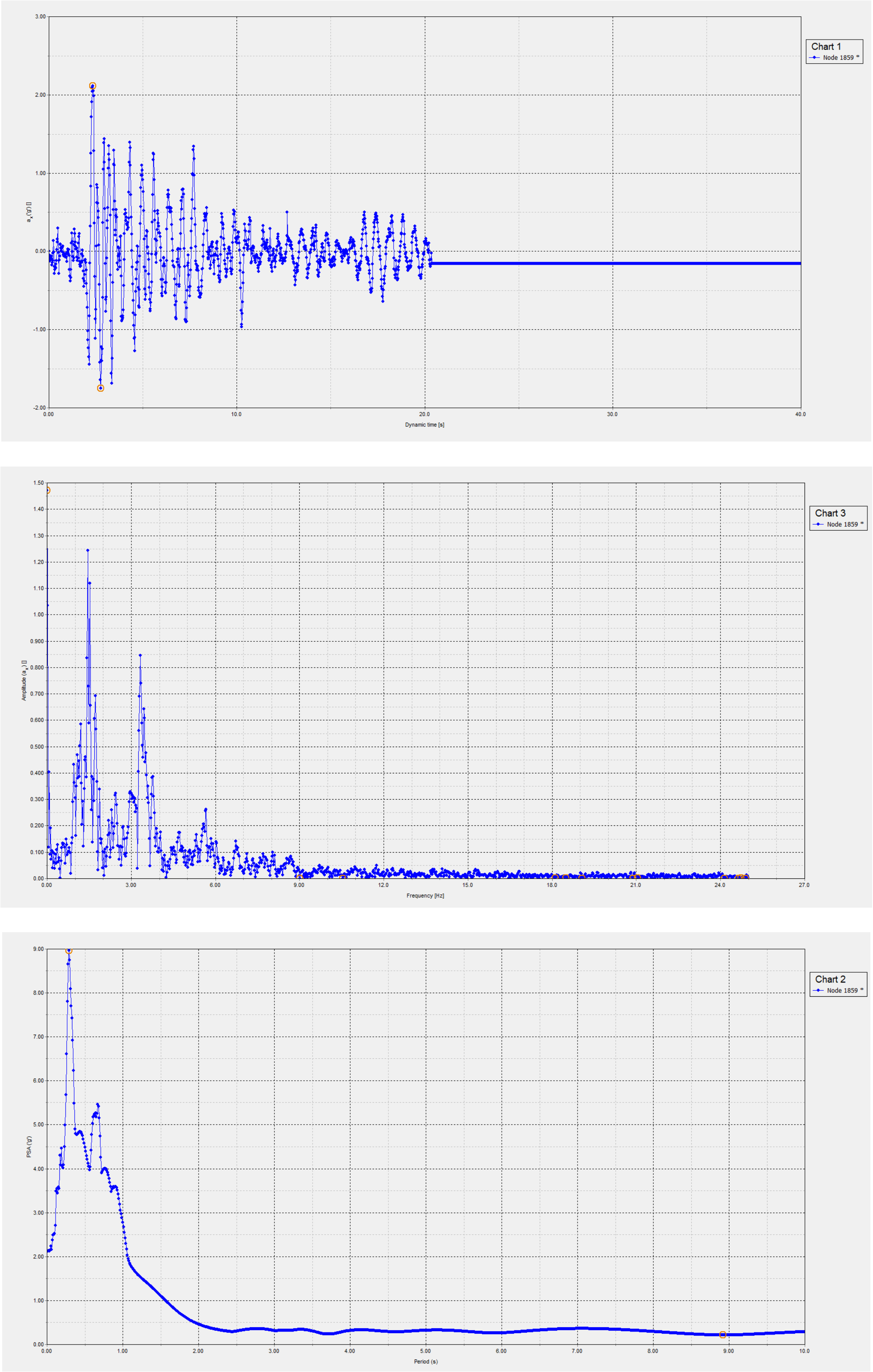


Figure 4.3.4. PLAXIS 3D output.

The output of a site response analysis was used further as input for software analysis of the foundation. It accurately reflects local soil conditions and translates bedrock-level seismic motions into surface-level ground motions that account for local soil amplification or damping effects.

4.4 Foundation Design

4.4.1 Foundation Selection

The assessment of various foundation types based on load-bearing capacity showed that the group pile foundation is the most suitable for our structure. Shallow, mat and single-pile foundations' load-bearing capacity failed to exceed the design load.

Several types of piles are available in practice, with their selection depending on factors such as structural load demands, soil conditions, installation methods, and potential impacts from nearby excavations. Based on recent literature, steel pipe piles were chosen for the current site due to their superior strength, installation efficiency, and performance under dynamic conditions, especially in urban excavation environments.

Steel pipe piles exhibit significantly higher stiffness and bearing resistance compared to concrete piles. This makes them ideal for deep foundation systems where strength and deformation control are critical. Yao et al. (2016) report that the elastic modulus of steel (210,000 MPa) greatly exceeds that of concrete (25,000 MPa), enabling steel piles to resist deformation and horizontal displacement more effectively. In their study, concrete piles displayed approximately twice the horizontal displacement compared to steel pipe piles, especially after reaching critical inflection points. These qualities also translate to increased stability under lateral soil pressures in urban excavation conditions. From an installation perspective, steel pipe piles offer construction advantages due to their prefabricated nature. Unlike cast-in-place concrete piles, steel piles eliminate curing time and reduce on-site variability, leading to faster installation and better quality control (Yao et al., 2016).

The decision to use driven piles—rather than bored or drilled shafts—was informed by findings from Massad (2014) and Bian et al. (2021). Driven piles, including steel pipe types, compact the surrounding soil during installation, which enhances both end-bearing and shaft friction resistance. This process ensures minimal residual displacement under dynamic loads and provides excellent load transfer characteristics, critical for structures sensitive to settlement (Massad, 2014). Furthermore, driven piles are suitable for heterogeneous soil layers, such as alternating clay and sand strata. Additionally, Bian et al. (2021) discuss the beneficial pile setup effect, where dissipation of pore water pressure and soil consolidation over time increases pile resistance. This natural strengthening process enables shorter pile lengths or reduced pile numbers without compromising safety or performance, ultimately lowering construction costs while improving reliability. Therefore, for this project, steel pipe driven piles were selected based on their mechanical properties, performance under excavation-induced stresses, adaptability to varied soil conditions, and cost-effectiveness over the project lifespan.

In the current project, the design does not require consideration of negative skin friction, as none of the typical conditions that warrant such analysis are present. The expected settlements are well below the critical limits, the embankment height is moderate, the compressible soil layer is relatively thin, no significant drawdown of the water table is anticipated, and the pile lengths are within acceptable ranges. Thus, the risk of downdrag is minimal, and additional design measures to account for negative skin friction are not necessary.

4.4.2. Foundation Design under Axial Loading

4.4.2.1 Hand calculations of axial bearing capacity

To assess whether a group of piles behaves as a single block or as individual piles, the efficiency of the pile group must be determined using the following formula:

$$\eta = \frac{2(n_1+n_2-2)*d+4D}{pn_1n_2} \tag{4.1}$$

where

n_1, n_2 - number of piles in a group;

p - perimeter of the pile;

d - minimum spacing between piles $= 2.5D$.

There are two outcomes from this formula: 1) if $\eta < 1$, then their bearing capacity will be calculated as $Q_{g(u)} = \Sigma Q_u$, 2) if $\eta \geq 1$, the bearing capacity will be equal to $Q_{g(u)} = n_1 n_2 \Sigma Q_u$.

Additionally, the dimensions of pile cap L_g and B_g will be calculated using:

$$L_g = (n_1 - 1)d + 2(D/2) \tag{4.2}$$

$$B_g = (n_2 - 1)d + 2(D/2) \tag{4.3}$$

Several iterations on the number of piles and diameter values were done to find out the best suited dimensions. In California, the allowable capacities of piles are regulated by the California Building Code (CBC), which is based on the International Building Code (IBC) with state-specific amendments. Given California's high seismicity, the CBC also includes requirements to ensure piles can resist seismic loads. CBC set Q_{all} to be higher than Q_{design} for 50-60% to account for uncertainties. Using $1.5Q_{design}$ value, iteration results showed the most suitable diameter and configuration as displayed in Table 4.4.2.1.1.

Table 4.4.2.1.1. Parameters of group of piles under each column type.

	$L,\ m$	$D,\ m$	$d,\ m$	n_1	n_2	B_g	L_g	$P,\ m$	$A,\ m^2$
Exterior	8	0.45	1.13	2	3	1.58	2.7	1.41	0.16
Interior	8	0.5	1.25	2	3	1.75	3	1.57	0.20
Corner	8	0.45	1.13	2	2	1.70	1.7	1.41	0.16

The figure below illustrates the placement of each pile group according to the building's layout.

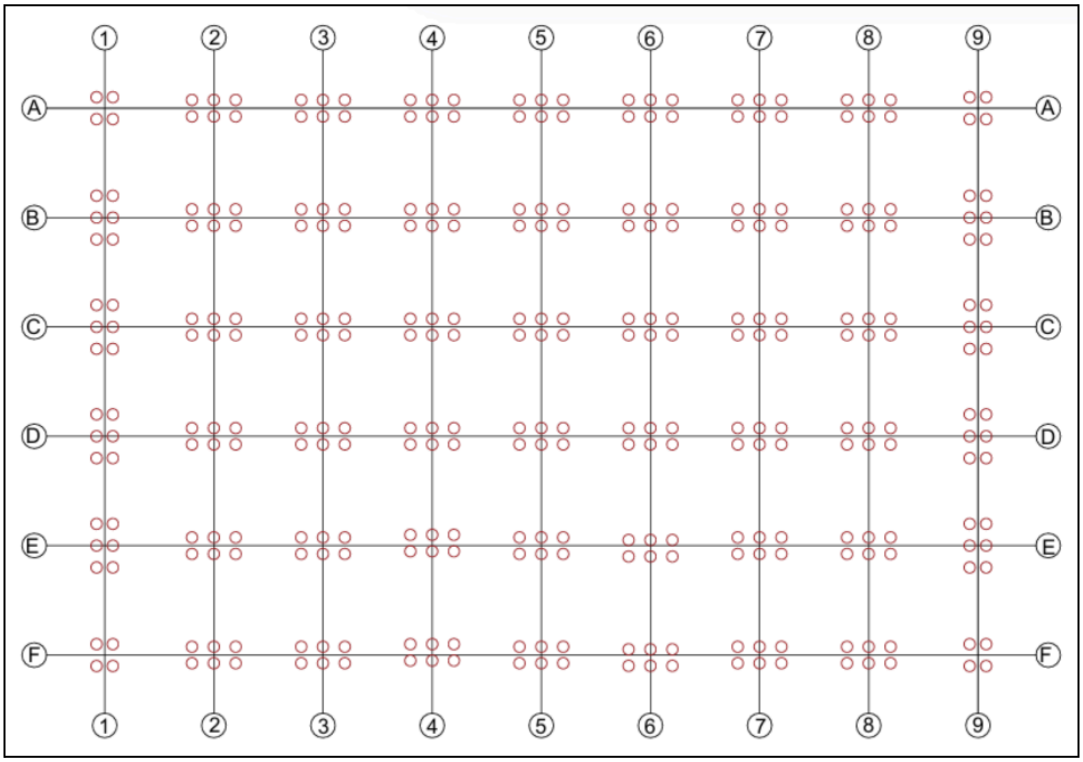


Figure 4.4.2.1.1. Group pile distribution under the building.

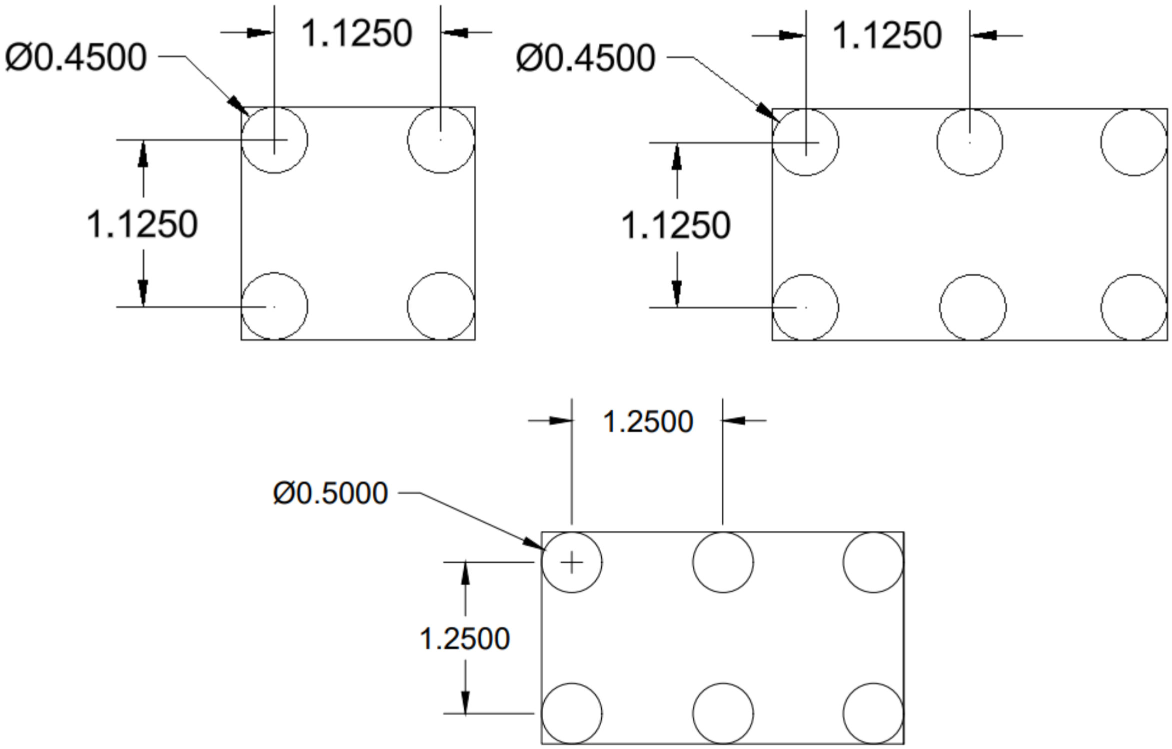


Figure 4.4.2.1.2. Dimensions of group piles under each column type.

The bearing capacity of the group of piles was calculated for each column type. The procedure was performed similar to the bearing capacity calculation for the single pile.

$$\eta_{exterior} = \frac{2(3+2-2)*1+4*0.45}{1.41*3*2} = 1.0085$$

$$\eta_{interior} = \frac{2(3+2-2)*1+4*0.5}{1.57*3*2} = 1.0085$$

$$\eta_{corner} = \frac{2(2+2-2)*1+4*0.45}{1.41*3*2} = 1.2031$$

Since η $\geq$ 1 group piles, then ultimate bearing capacity will be found as following $Q_{g(u)} = n_1 n_2 \Sigma Q_u$. Factor of safety is taken as 3.

Table 4.4.2.1.2. Bearing capacity of group piles.

	η	Q_p, kN	Q_s, kN	Q_u, kN	ΣQ_u, kN	Q_{all}, kN	Q_{design}, kN
Exterior	1.0085	5297.44	1173.13	6470.57	38823.44	10292.4	6161.26
Interior	1.0085	6722.04	1292.58	8014.62	48087.72	11669.2	7293.29
Corner	1.2031	5297.44	1173.13	6470.57	25882.29	7020.88	4483.93

4.4.2.2 Hand calculations of settlement

Vesic method

1) Elastic settlement S_{e1} for each pile is determined using the formula:

$$S_{e1} = \frac{L\times(Q_{wp}+\xi Q_{ws})}{A_p \times E_p} \tag{4.4}$$

Where,

A_p − cross sectional area, $0.16\, m^2$;

Q_{wp} − pile load at working conditions;

$Q_{wp} = \frac{6215.78}{3} = 2071.927\, kN$

Q_{ws} − frictional skin resistance load;

$Q_{ws} = \frac{843.85}{3} = 281.283\, kN$

ξ − assumed to be equal to 0.67;

L − length of the pile;

E_p − modulus of elasticity;

$E_p = 4700\sqrt{f_c'} = 29725.41\ MPa$

2) Next, pile load settlement is calculated using:

$$S_{e2} = \frac{q_{wp} \times D}{E_s}(1 - \mu_s^2) \times I_{wp} \tag{4.5}$$

Where,

q_{wp} − point load per unit area;

$q_{wp} = \frac{Q_{wp}}{A_p} = \frac{6215.78/3}{0.16} = 12949.542\ kN/m\hat{}2$

D − diameter of a single pile.

E_s − modulus of elasticity, 23334 Pa;

μ_s − Poisson ratio, 0.438;

I_{wp} − influence factor, 0.85 as stated by Das & Sivakugan (2019).

3) The settlement caused by load transmission along the pile shaft was calculated using the formula below.

$$S_{e3} = \frac{Q_{ws}}{p \times L} \times \frac{D}{E_s} \times (1 - \mu_s^2) \times I_{ws} \tag{4.6}$$

Where,

p − pile perimeter; $P = \pi D = \pi * 0.6 = 1.89\ m$,

L − embedded length; 8m

I_{ws} − influence factor;

In this formula, average values for E_s and μ_s should be used. And Influence factor is found by equation:

$$I_{ws} = 2 + 0.35 \times \sqrt{\frac{L}{D}} \tag{4.7}$$

Finally, the sum of the three settlements would give the total settlement of one pile:

$$S_e = S_{e1} + S_{e2} + S_{e3}$$

Results are further compared with software analysis, and can be seen on Table 4.4.2.3.2.

4.4.2.3 Software analysis of axial bearing capacity and settlement

The bearing capacity analysis was performed using GEO5, while the settlement of the foundation was evaluated using both PLAXIS 3D and GEO5 software.

The soil parameters and pile parameters such as diameter, length, and material properties were defined in the GEO5 software,. The groundwater table level was also specified. After that, external loads were applied based on the structural demands, and the appropriate design code (Eurocode) was selected for the analysis. These load combinations and their corresponding values are detailed in Table 4.4.2.3.1.

Table 4.4.2.3.1. Most Critical Load Combination

Column Type	Combination Type	Fx, kN	Fy, kN	Fz, kN	Mx, kNm	My, kNm	Mz, kNm
Corner Column	1.2D+1.0E+1.0L	-138.49	-239.88	7170.48	956.61	-455.50	0.23
Exterior Column		-166.94	-224.69	10176.21	939.32	-487.50	0.18
Interior Column	1.2D+1.6Lr+1.0L	-3.884E-13	1.18	17710.42	-2.16	-9.645E-13	-2.477E-14

GEO5 then carried out calculations to evaluate both the bearing capacity and the expected settlement of the pile foundation. Hand calculations and Geo5 software results comparison showed that the ultimate and design bearing capacities for all pile group configurations—interior, exterior, and corner, as well as safety factors are similar and within allowable limits, as shown in Table 4.4.2.3.2.

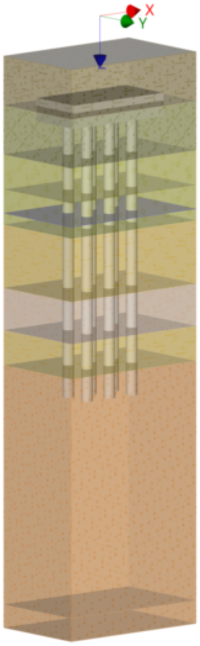


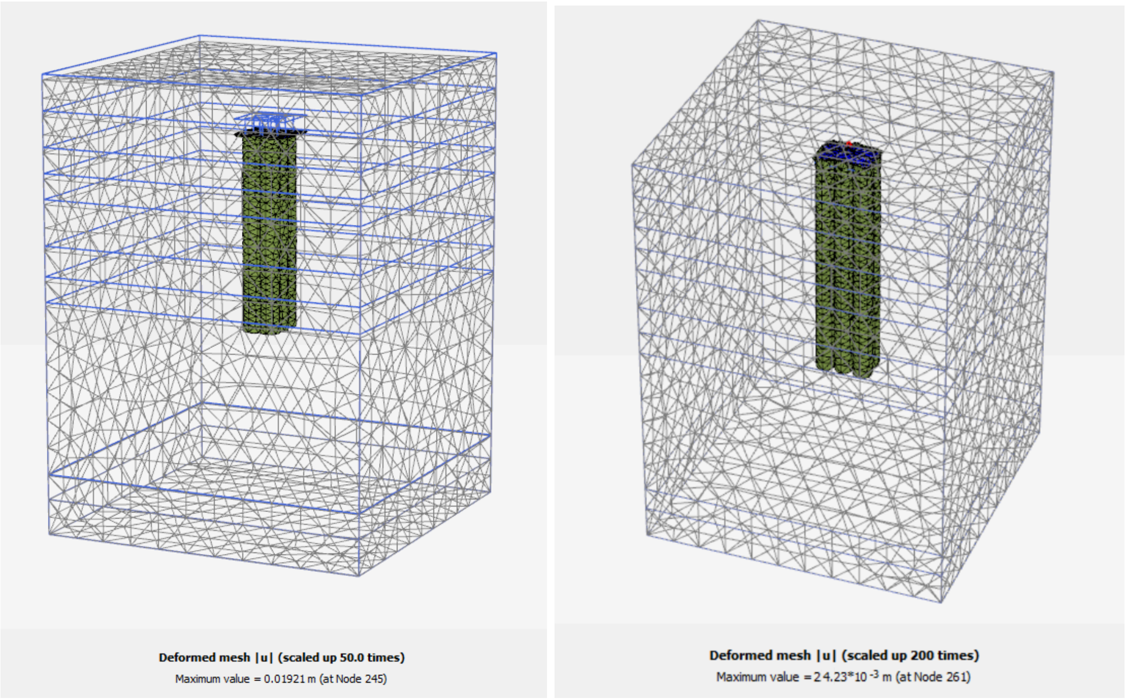
Figure 4.4.2.3.1. Geo5 Soil Profile.

Table 4.4.2.3.2. Comparison of bearing capacity values from hand calculation and Geo5.

	Interior		Exterior		Corner	
	Hand calculation	Geo5 Software	Hand calculation	Geo5 Software	Hand calculation	Geo5 Software
Q_u, kN (single)	3217.81	3100.19	3217.81	4088.21	2378,21	2910.21
Q_u, kN (pile)	48087.72	48112.24	38823.44	40123.44	25882.29	26282.29
Q_{all}, kN	11669.2	-	10292.4	-	7020.88	-
Q_{design}, kN	7293.29	-	6161.26	-	4483.93	-
FS	3	3.71	3	3.25	3	3.34

In PLAXIS, the piles were modeled as embedded beams, while the soil profile was constructed using the Mohr-Coulomb model, with all relevant soil parameters taken from the site-specific soil data. Pile caps were represented as plate elements, and the load combinations with the highest axial loads were applied to the model.

The resulting deformation of the pile groups is illustrated in Figures 4.4.2.3.2. and 4.4.2.3.3. The settlement values from hand calculation, Geo5, and Plaxis 3D are summarized in Table 4.4.2.3.3.



Deformed mesh |u| (scaled up 50.0 times)
Maximum value = 0.01921 m (at Node 245)

Deformed mesh |u| (scaled up 200 times)
Maximum value = 24.23*10 $^{-3}$ m (at Node 261)

Figure 4.4.2.3.2. Deformation of the exterior (left) and interior (right) pile groups.

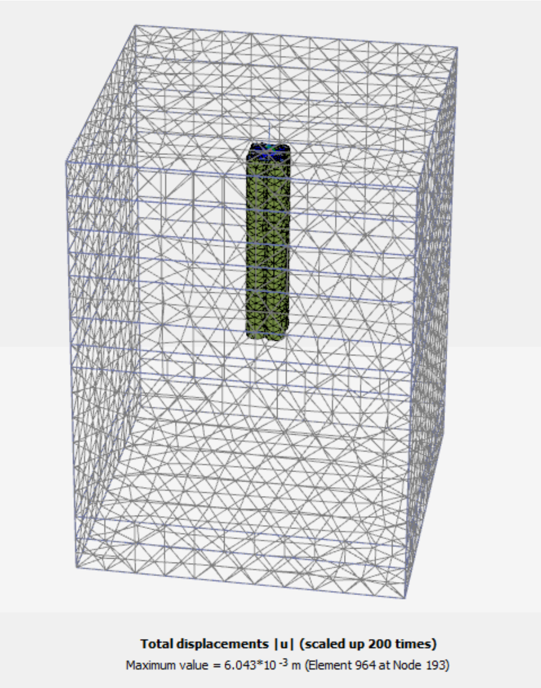


Figure 4.4.2.3.3. Deformation of corner pile group.

Table 4.4.2.3.3. Summary of the settlement values from hand calculation, Geo5, and Plaxis 3D.

	Interior			Exterior			Corner		
	Hand calculation	Geo5	Plaxis	Hand calculation	Geo5	Plaxis	Hand calculation	Geo5	Plaxis
S_e, mm	18.25	16.79	24.13	16.94	15.01	19.21	11.14	8.84	6.04

Both numerical simulations and hand calculations confirmed that the settlement of each pile group remained within the allowable limits. Hand calculations often rely on simplified assumptions and empirical formulas that may not capture the full complexity of soil-structure interaction, especially in layered soil conditions or non-linear behavior. In contrast, software tools like PLAXIS 3D and GEO5 use advanced numerical methods which provide a more detailed and realistic representation of the soil and structural behavior under loading. Overall, the findings indicate that the design is structurally sound and that the pile groups are capable of withstanding both lateral and axial loads imposed by the columns.

4.4.3 Foundation design under lateral loading

4.4.3.1. Hand calculations of lateral bearing capacity

The lateral bearing capacity of the pile group was determined using the lateral forces and moments obtained from the SAP 2000 software. For the analysis, the maximum loads were considered for three configurations: internal, external, and corner pile groups.

Table 4.4.3.1.1. Lateral loads.

Column Type	Combination Type	Fx, kN	Fy, kN	Number of piles	Lateral Load
Corner Column	1.2D+1.0E+1.0L	-138.489	273.04	4	68.26
Exterior Column		-166.939	224.691	6	37.45
Interior Column		-101.7	236.711	6	39.45

A vertical pile resists lateral loads by generating passive pressure from the surrounding soil. The distribution of this soil reaction depends on the pile's stiffness, the soil's stiffness, and how the pile is fixed at its ends (Das & Sivakugan, 2019). Laterally loaded piles are generally classified as either short or long. Broms (1965) proposed a simplified method for analyzing such piles: assuming shear failure in the soil for short piles and bending governed by the pile’s plastic yield for long piles (Das & Sivakugan, 2019). Restrained piles offer greater bearing capacity compared to free-headed piles, confirming the effectiveness of using a pile cap.

First, Rankine passive earth pressure coefficient was found. The value of constant modulus of horizontal subgrade reaction, n_h was taken from Table 12.16 from Das & Sivakugan (2019). The calculation was done for the pile length only, so the basement depth was subtracted from the initial soil layer depths. The resulting values are presented in Table 4.4.3.1.2.

$$K_p = tan^2(45 + \frac{\phi'}{2}) \qquad (4.8)$$

Table 4.4.3.1.2. Calculation of passive earth pressure coefficient and subgrade reaction.

Layer	Layer depth, m	Soil type	ϕ'	K_p	n_h
1	-	Clayey sand with gravel	27.89	2.75	2000
2	0.968	Silty sand with gravel	20.19	2.05	2000
3	1.524	Gravel with clay and sand	44.86	5.78	6250
4	1.524	Gravel with silt and clay	38.84	4.37	6250
5	2.743	Sandy clay	44.98	5.82	6250
6	1.829	Silty sand	47.55	6.63	6250
7	1.524	Clayey sand	34.44	3.60	6250
8	10.36	Sand	41.17	4.85	16500

The characteristic length of the soil–pile system, T was calculated. E_p value was assumed to be $207 * 10^6 kN/m^2$. Moment of inertia for circular pile was calculated using the diameter defined in Table X as:

$$I_p = \frac{\pi * d^4}{64} \tag{4.9}$$

$$I_{p\,(exterior)} = \frac{\pi * d^4}{64} = \frac{\pi * 0.45^4}{64} = 2.01 * 10^{-3}\, m^4$$

$$I_{p\,(interor)} = \frac{\pi * d^4}{64} = \frac{\pi * 0.5^4}{64} = 3.07 * 10^{-3}\, m^4$$

$$I_{p\,(corner)} = \frac{\pi * d^4}{64} = \frac{\pi * 0.45^4}{64} = 2.01 * 10^{-3}\, m^4$$

Further calculation will be carried out for the lateral bearing capacity of exterior column.

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \tag{4.10}$$

$$T = \sqrt[5]{\frac{207 * 10^6 * 2.01 * 10^{-3}}{6250}} = 2.32$$

$L/T = 12/2.32 = 5.18 > 5$, so the analysis was completed for the case of long pile. The bending failure analysis was performed. Yield stress of the pile material was taken as $F_y = 248MN/m^2$ (Das & Sivakugan, 2019).

$$M_y = S^* F_y = \frac{I_p}{\frac{d_1}{2}} F_y \tag{4.11}$$

$$M_y = \frac{2*2.01*10^{-3}*248000}{0.45} = 2217.53\ kNm$$

$$\frac{M_y}{D^4\gamma K_p} = \frac{2217.53}{0.45^4*16.8*2.05} = 1570.20$$

Using Figure 12.42a from Das & Sivakugan (2019), for the value of yield moment $\frac{M_y}{D^4\gamma K_p}$ the corresponding magnitude of ultimate lateral resistance $\frac{Q_{u(g)}}{K_p D^3\gamma}$ was found to be around 435.00 and $Q_{u(g)}$ was calculated. The outcomes are summarized in Table 4.4.3.1.3.

$$Q_{u(g)} = 435 * K_p D^3\gamma = 435 * 2.05 * 0.45^3 * 16.8 = 1365.18$$

Table 4.4.3.1.3. Ultimate lateral resistance values.

Layer	Soil type	M_y	$\frac{M_y}{D^4\gamma K_p}$	$\frac{Q_{u(g)}}{K_p D^3\gamma}$	$Q_{u(g)}$
1	Clayey sand with gravel	2217.53	1177.52	330.00	1381.02
2	Silty sand with gravel	2217.53	1570.20	435.00	1365.18
3	Gravel with clay and sand	2217.53	514.07	140.00	1342.04
4	Gravel with silt and clay	2217.53	707.13	190.00	1324.07
5	Sandy clay	2217.53	491.63	135.00	1353.18
6	Silty sand	2217.53	433.86	120.00	1362.98
7	Clayey sand	2217.53	878.46	240.00	1346.32

8	Sand	2217.53	511.47	140.00	1348.85

Next, an assessment of pile head deflection was conducted.

$$\eta = \sqrt[5]{\frac{n_h}{E_p I_p}} \qquad (4.12)$$

$$\eta = \sqrt[5]{\frac{2000}{207*10^6*2.01*10^{-3}}} = 0.34$$

$$\eta L = 0.34 * 0.968 = 0.33$$

The value of dimensionless length ηL was used to find the respective value of dimensionless lateral deflection $\frac{x_o(E_p I_p)^{3/5}(n_h)^{2/5}}{Q_g L} \approx 6.8$ from Figure 12.43a from Das & Sivakugan (2019). Thus, the allowable lateral load is equal to:

$$Q_g = \frac{0.02*(207*10^6*2.01*10^{-3})^{3/5}(2000)^{2/5}}{6.8*12} = 10.22\ kN$$

Table 4.4.3.1.4. The allowable lateral load values.

Layer	Soil type	η	ηL	$\frac{x_o(E_p I_p)^{3/5}(n_h)^{2/5}}{Q_g L}$	Q_g
1	Clayey sand with gravel	0.34	0.00	0.00	-
2	Silty sand with gravel	0.34	0.33	6.80	10.22
3	Gravel with clay and sand	0.43	0.66	5.50	19.93
4	Gravel with silt and clay	0.43	0.66	5.50	19.93
5	Sandy clay	0.43	1.18	4.20	26.09
6	Silty sand	0.43	0.79	5.10	21.49
7	Clayey sand	0.43	0.66	5.50	19.93
8	Sand	0.52	2.04	2.10	76.96

Hence, $Q_g = 10.22\ kN\ (< 1365.18\ kN)$. However this method was further modified for the group pile design. Abdrabbo & Gaaver (2012) suggest implementing p-multiplier factors to better estimate the distribution of lateral load. The lateral capacity of the first pile in the row is multiplied by 0.79, the second by 0.58, and the third by 0.46. The adjusted values are listed in Table 4.4.3.1.5.

Table 4.4.3.1.5. Adjusted values of lateral bearing capacity.

Layer	Soil type	$Q_{g,1}$	$Q_{g,2}$	$Q_{g,3}$
1	Clayey sand with gravel	-	-	-
2	Silty sand with gravel	8.07	5.93	4.70
3	Gravel with clay and sand	15.74	11.56	9.17
4	Gravel with silt and clay	15.74	11.56	9.17
5	Sandy clay	20.62	15.14	12.00
6	Silty sand	16.98	12.46	9.89
7	Clayey sand	15.74	11.56	9.17
8	Sand	60.80	44.64	35.40
Sum		153.69	112.84	89.49

Since the configuration for the exterior pile is 3x2, the adjustments were made for the 1 row, and then multiplied by 2. These values are then summed to obtain the total lateral capacity. Factor of safety of 3 was applied. Final values of lateral bearing capacity and comparison to lateral loads are presented in Table 4.4.3.1.6. The calculations demonstrated that the bearing capacities of the pile groups satisfied the performance criteria.

$$Q_{g,final} = \frac{(153.69+112.84+89.49)*2}{3} = \frac{356.02*2}{3} = 237.35\ kN$$

Table 4.4.3.1.6. Lateral load comparison.

Column type	Interior	Exterior	Corner
Q_g, kN	507.93	712.04	383.55
$Q_g/FS,\ kN$	169.31	237.35	127.85
Lateral Load, kN	39.45	37.45	68.26

4.4.4. Design of Group Piles

In this section, the structural design of group piles and their reinforcement requirements are shown. The design consists of both the pile cap transferring load to more than one pile and the pile reinforcement for single piles to ensure structural safety against combined axial load, shear, and bending.

4.4.4.1. Pile Cap Reinforcement

First of all, the depth is calculated based on equations provided in Reinforced Concrete Designer's Handbook by Reynolds (Reynolds et al., 2007). The required depth depends on the pile diameter:

Case 1: Pile Diameter ≤ 550 mm

$$h = (2d_p + 100) \tag{4.13}$$

Case 2: Pile Diameter > 550 mm

$$h = \frac{1}{3}(8d_p + 600) \geq 500 \tag{4.14}$$

Next step is to identify effective depth in two directions based on equation below:

$$d = h - cover - \frac{d_{rein\,bar}}{2} \tag{4.15}$$

The 75 mm cover of concrete was chosen to provide adequate protection against soil moisture and corrosion, which also meets standard codes (Standard, 2000) and regulations, and provide adequate reinforcement anchorage space as well as safe construction.

To determine the required reinforcement area, the following equation was used:

$$A_s = \frac{M}{\phi f_y z} \tag{4.16}$$

$$z = 0.9d \tag{4.17}$$

Where,

ϕ − $strength\ reduction\ factor$

f_y − $steel\ yield\ strength$

The total number of bars is determined from the ratio:

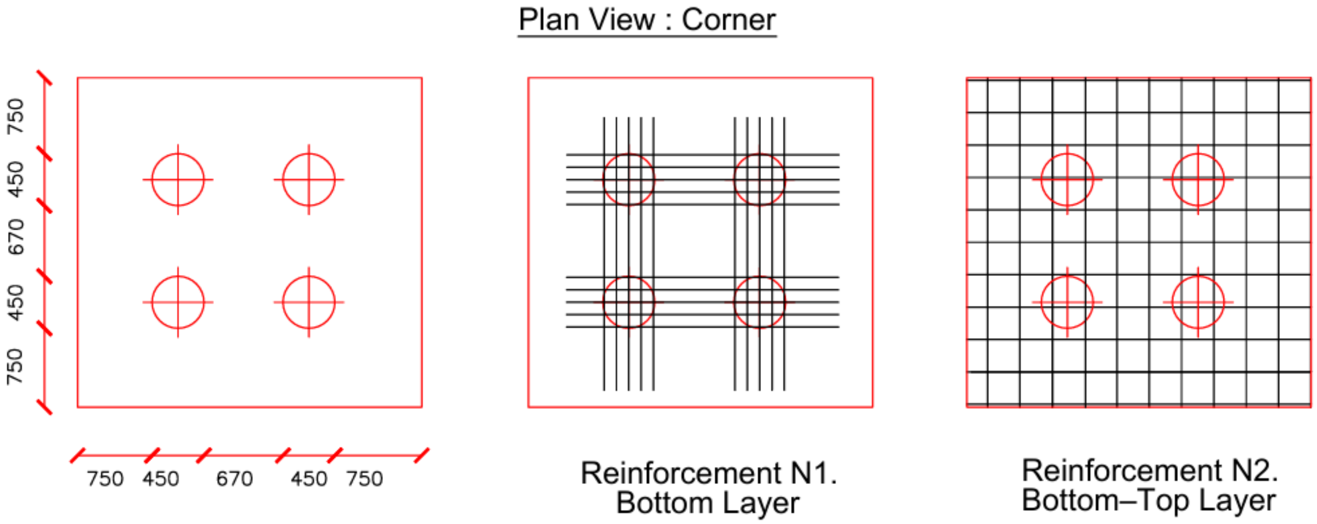
$$№\ of\ bars\ = A_s/A_{bar} \tag{4.18}$$

The reinforcement design values for each pile group type are summarized in **Table 4.4.4.1.1** below.

Table 4.4.4.1.1. Pile Cap Reinforcement Design

Group of Piles	d_p, mm	h, mm	d_x, mm	d_y, mm	Load (kN)	Reinf. in X-dir	Reinf. in Y-dir
Exterior	450	1000	915	915	1418.4	17T20 @ 150mm	17T20 @ 150mm
Interior	500	1100	1012.5	1012.5	2500	10T25 @ 100mm	18T25 @ 125mm
Corner	450	1000	917	917	3687.5	17T16 @ 200mm	17T16 @ 200mm

To visualize the pile layout and reinforcement placement, **Figure 4.4.4.1.1** provides a detailed plan view of the corner pile group, as an example, and plan views with drawings for interior and exterior walls are included in the architectural drawings file.



Plan View : Corner

Reinforcement N1.
Bottom Layer

Reinforcement N2.
Bottom–Top Layer

Figure 4.4.4.1.1. Corner Plan View.

4.4.4.2. Single Pile Reinforcement

The effective length of each pile was taken as 1.2 times the actual length, as recommended for piles with fixed heads under a pile cap and embedded in soil.

$$l_e = \beta * L \tag{4.19}$$

This gives $l_e = 9.6$ m, which is then used to determine slenderness and whether moment magnification is required in the design.

$$l_e/D > 10 \Rightarrow \text{ Slender pile } \Rightarrow \text{ Need moment magnification}$$

$$M_{add} = Q * e = \frac{N*l_e^2}{2EA} \tag{4.20}$$

Where:

N is the axial load,

E is the modulus of elasticity of concrete,

A is the cross-sectional area.

Then, the magnified moment is calculated as:

$$M_{mag} = M + M_{add} \tag{4.21}$$

$$A_s = \frac{M_{mag}}{\phi f_y z} \tag{4.16}$$

After what, steel area is used to determine the number of bars and their arrangement:

$$A_s = n * A_{bar} \tag{4.22}$$

And the reinforcement ratio is evaluated to check if it falls within the permissible range:

$$\rho = A_s/A_g * 100\% \tag{4.23}$$

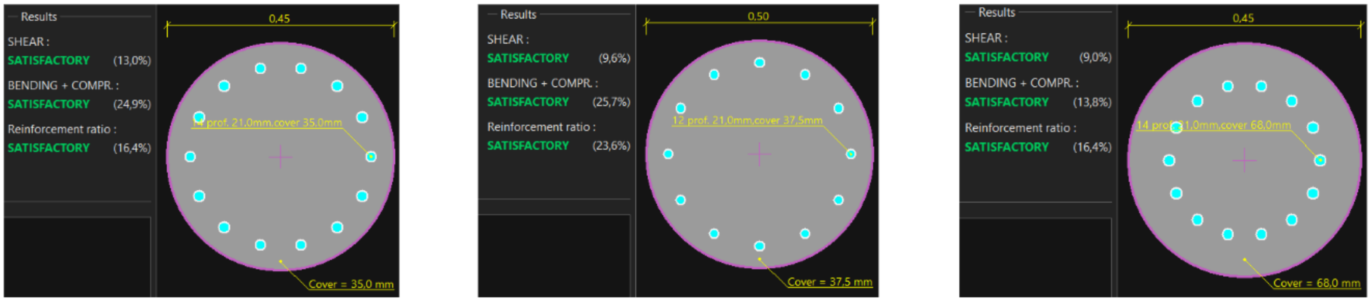
It shows the percentage of steel within the concrete section. Calculated results between 2-3% indicates that value is in acceptable range and piles are neither under- nor over-reinforced.

Table 4.4.4.2.1 also includes value for normalized moment intensity M_{mag}/d^3 to access moment demands relative to section depth. The higher values show the need for more reinforcement.

Table 4.4.4.2.1. Single Pile Reinforcement

Piles	D (m)	$\frac{l_e}{D}$	M (kNm)	M_{mag} (kNm)	$\frac{M_{mag}}{d^3}$	A_s (mm²)	ρ (%)	Design
Interior	0.50	19.2 > 10	129.64	267.8	2.14	4152	2.1	12Ø21 mm
Exterior	0.45	21.3 > 10	170.28	180.76	2.82	4152	2.6	12Ø21 mm
Corner	0.45	21.3 > 10	251.07	267.8	3.69	4844	2.8	14Ø21 mm

After hand calculations, the Geo5 was used to verify the reinforcement design. The software confirms that the pile performs satisfactorily under shear, bending & compression as well as shows that reinforcement ratio is within allowed limits. Results verifies that the reinforcement of individual piles under the pile cap the theoretical results align with software-based outcomes, thus ensuring the structure meets design safety requirements.



a. Exterior

```
Results
SHEAR :
SATISFACTORY      (13,0%)
BENDING + COMPR. :
SATISFACTORY      (24,9%)
Reinforcement ratio :
SATISFACTORY      (16,4%)
```

b. Interior

```
Results
SHEAR :
SATISFACTORY      (9,6%)
BENDING + COMPR. :
SATISFACTORY      (25,7%)
Reinforcement ratio :
SATISFACTORY      (23,6%)
```

c. Corner

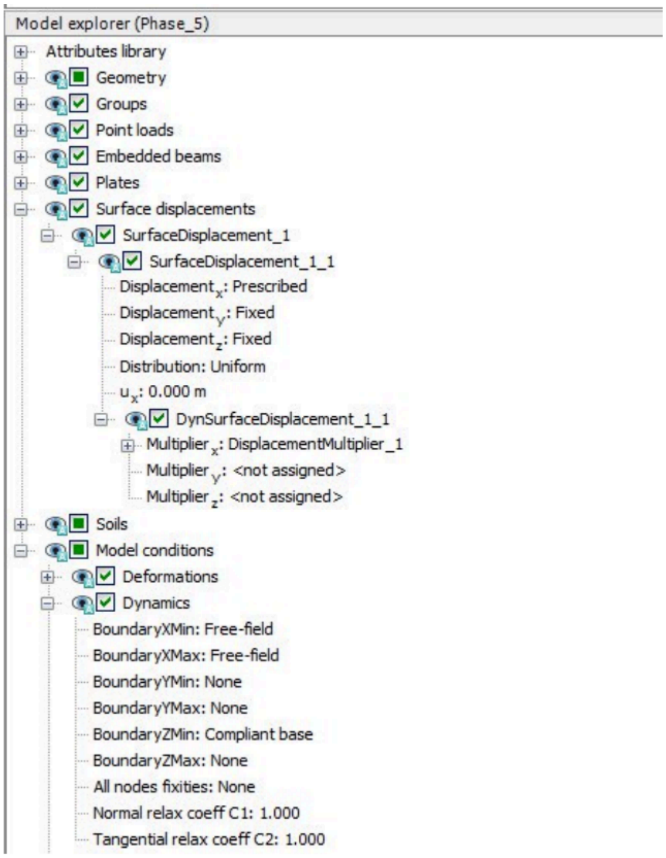
```
Results
SHEAR :
SATISFACTORY      (9,0%)
BENDING + COMPR. :
SATISFACTORY      (13,8%)
Reinforcement ratio :
SATISFACTORY      (16,4%)
```

Figure 4.4.4.2.1. Single Pile Reinforcement in Geo5.

4.4.4.3. Software analysis of all pile groups under axial and lateral loadings.

PLAXIS 3D was utilized to evaluate the deformation behavior of the pile foundation under both axial and lateral loads. The numerical modeling included the entire building layout, incorporating each column and their most critical load combinations. Three configurations of pile groups were modeled: corner, exterior, and interior piles. The design also included pile caps and sheet piles. All relevant boundary conditions were implemented in the model. Piles were represented as embedded beams, while the pile caps were modeled by assigning plate elements to the basement floor slab.

To accurately account for local soil behavior under seismic loading, a site response analysis was carried out using PLAXIS 2D. The dynamic output from PLAXIS 2D was subsequently integrated into the PLAXIS 3D model to ensure realistic seismic input for the structural system. Specifically, the displacement results obtained from the 2D site response analysis were applied in PLAXIS 3D through the Surface Displacement tab, utilizing the Dynamic Surface Multiplier function. The multiplier (denoted as "Multiplier X") was used to scale the input motion appropriately at the ground surface, thereby linking the 2D ground response with the 3D foundation behavior. This method allowed for a consistent and more representative simulation of seismic loading conditions across both models.

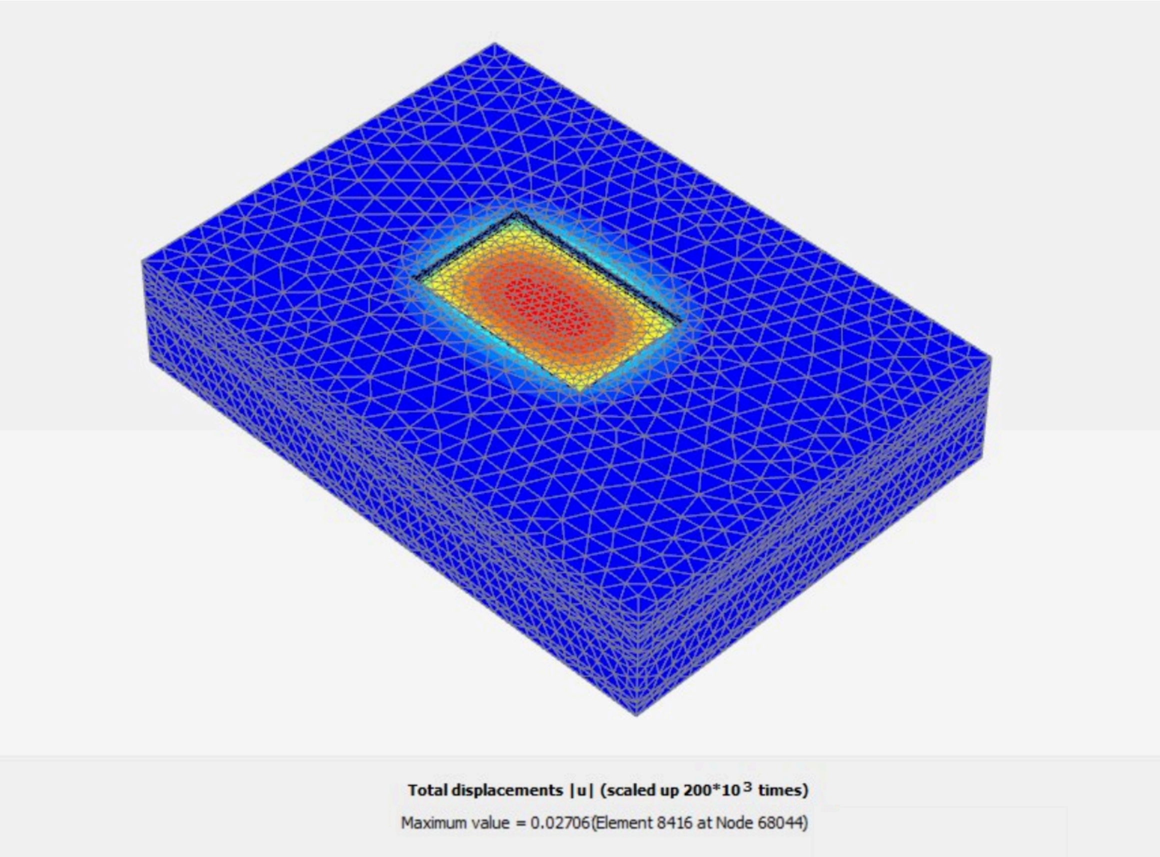


Model explorer (Phase_5)

- Attributes library
- ■ Geometry
- ☑ Groups
- ☑ Point loads
- ☑ Embedded beams
- ☑ Plates
- ☑ Surface displacements
  - ☑ SurfaceDisplacement_1
    - ☑ SurfaceDisplacement_1_1
      - Displacement$_x$: Prescribed
      - Displacement$_y$: Fixed
      - Displacement$_z$: Fixed
      - Distribution: Uniform
      - u_x: 0.000 m
      - ☑ DynSurfaceDisplacement_1_1
        - Multiplier$_x$: DisplacementMultiplier_1
        - Multiplier$_y$: <not assigned>
        - Multiplier$_z$: <not assigned>
- ■ Soils
- ■ Model conditions
  - ☑ Deformations
  - ☑ Dynamics
    - BoundaryXMin: Free-field
    - BoundaryXMax: Free-field
    - BoundaryYMin: None
    - BoundaryYMax: None
    - BoundaryZMin: Compliant base
    - BoundaryZMax: None
    - All nodes fixities: None
    - Normal relax coeff C1: 1.000
    - Tangential relax coeff C2: 1.000

Figure 4.4.4.3.1 PLAXIS Model explorer view

The numerical analysis in PLAXIS 3D has shown that the deformation of the whole building was equal to 27.06 mm.



Total displacements |u| (scaled up $200*10^3$ times)

Maximum value = 0.02706(Element 8416 at Node 68044)

Figure 4.4.4.3.2 Total displacement PLAXIS 3D

4.5. Sheet Pile Design

In regions with significant seismic activity and fluctuating groundwater levels, sheet pile walls present a suitable solution as they belong to the flexible retaining structures category. Unlike rigid retaining walls that rely on a large volume of mass for stability, sheet piles exploit the stiffness, strength, and thickness of materials, allowing them to absorb ground movements more effectively (Torrabadella, 2013). This flexibility is particularly advantageous in seismic zones, where rigid walls may experience excessive cracking or failure due to abrupt ground motion. Additionally, sheet piles are efficient in excavation projects, as they resist earth pressure, prevent water ingress, and limit ground deformations in the surrounding soil. Given that our site has a groundwater table at 6.8m, preventing water seepage is a critical consideration, further justifying the selection of sheet pile walls over conventional rigid structures (Torrabadella, 2013).

4.5.1. Hand calculations

To design the sheet piles, the Rankine earth pressure coefficients were derived. K_p is usually adjusted at the beginning of the design using a safety factor of 2 (Das & Sivakugan, 2019).

$$K_a = tan^2(45 - \frac{\varphi'}{2}) \tag{4.24.1}$$

$$K_p = tan^2(45 + \frac{\varphi'}{2}) \tag{4.24.2}$$

Table 4.5.1.1. Soil parameters for sheet pile design.

Depth, m	$\gamma,\ kN/m^3$	φ', °	K_a	K_p	$K_{p\ (design)}$
0-3.3	16.8	27.89	0.36	3.76	1.88

$$K_{p(design)} - K_a = 1.52$$

The active pressure at a depth 3.3m was calculated:

$$\sigma'_2 = \gamma L K_a \tag{4.25}$$

$$\sigma'_2 = 16.8 * 3.3 * 0.36 = 19.96\, kN/m^2$$

The procedure from Das & Sivakugan (2019) was followed, and L_3 and σ'_5 were calculated using equations:

$$L_3 = \frac{\sigma'_2}{\gamma(K_{p(design)} - K_a)} = \frac{LK_a}{(K_{p(design)} - K_a)} \tag{4.26}$$

$$\sigma'_5 = \gamma L K_{p(design)} + \gamma L_3 (K_{p(design)} - K_a) \tag{4.27}$$

$$L_3 = \frac{3.3*0.36}{1.52} = 0.78m$$

$$\sigma'_5 = 16.8 * 3.3 * 1.88 + 16.8 * 0.78 * 1.52 = 124.15\, kN/m^2$$

The area of the pressure diagram was found summing the pressure areas and the center of pressure for the area was found by taking the moment:

$$P = \frac{1}{2}\sigma'_2 L + \frac{1}{2}\sigma'_2 L_3 \tag{4.28}$$

$$P = \frac{1}{2} * 19.96 * 3.3 + \frac{1}{2} * 19.96 * 0.78 = 40.72\, kN/m$$

$$\bar{z} = \frac{32.94*(0.78+\frac{3.3}{3})+7.78*(\frac{2*0.78}{3})}{40.72} = 1.62\, m$$

Further step involved calculating A_1', A_2', A_3', A_4':

$$A_1' = \frac{\sigma_5'}{\gamma(K_{p(design)} - K_a)} \tag{4.29}$$

$$A_1' = \frac{124.15}{16.8*1.52} = 4.86\, m$$

$$A_2' = \frac{8P}{\gamma(K_{p(design)} - K_a)} \tag{4.30}$$

$$A_2' = \frac{8*40.72}{16.8*1.52} = 12.76\, m^2$$

$$A_3' = \frac{6P[2\bar{z}\gamma(K_{p(design)} - K_a) + \sigma_5']}{\gamma^2(K_{p(design)} - K_a)^2} \tag{4.31}$$

$$A_3' = \frac{6*40.72*[2*1.62*16.8*1.52+124.15]}{16.8^2 1.52^2} = 77.5\, m^3$$

$$A_4' = \frac{P(6\bar{z}\sigma'_5 + 4P)}{\gamma^2(K_{p(design)} - K_a)^2} \tag{4.32}$$

$$A_4' = \frac{40.72[6*1.62*124.15+4*40.72]}{16.8^2 1.52} = 85.53\, m^4$$

The derived values were used to construct an equation to estimate L_4:

$$L_4^4 + A_1'L_4^3 - A_2'L_4^2 - A_3'L_4 - A_4' = 0$$

$$L_4 = 4.1m$$

Next, the theoretical depth of penetration was computed:

$$D_{theory} = L_3 + L_4 \tag{4.33}$$

$$D_{theory} = 0.78 + 4.1 = 4.88\, m$$

The theoretical depth of embedment D_{theory} is increased by 20 to 30% to provide an additional safety factor for the structure (Das & Sivakugan, 2019).

$$D_{actual} = 1.3D_{theory} \tag{4.34}$$

Finally, total length was estimated:

$$L + 1.3 * D_{theory} = 3.3 + 1.3 * 4.88 = 9.64\, m$$

σ'_3, σ'_4 and L_5 were derived using following equations:

$$\sigma'_3 = L_4(K_{p(design)} - K_a)\gamma \tag{4.35}$$

$$\sigma'_3 = 4.1 * 1.52 * 16.8 = 104.7\ kN/m^2$$

$$\sigma'_4 = \sigma'_5 + \gamma L_4(K_{p(design)} - K_a) \tag{4.36}$$

$$\sigma'_4 = 124.15 + 16.8 * 4.1 * 1.52 = 228.85\ kN/m^2$$

$$L_5 = \frac{104.7*3.3-2*40.72}{104.7+228.85} = 0.79\ m$$

Point of zero shear force was found:

$$z' = \sqrt{\frac{2*40.72}{16.8*1.52}} = 1.79\ m$$

Maximum bending moment was computed:

$$M_{max} = 40.72(1.62 + 1.79) - \frac{1}{2} * 16.8 * 1.79^2 * 1.52 * \frac{1}{3} * 1.79 = 114.45$$

The allowable stress for the sheet pile was selected in accordance with ASTM A-328, which is the standard specification for steel sheet piling in the USA. Based on this specification, an allowable stress value of 34 ksi (234 MPa) was used for design purposes. Additionally, to account for seismic conditions, a seismic load reduction factor of 0.89 was applied. Thus, section modulus was equal to:

$$S = \frac{114.45}{234000*0.89} = 48.9 * 10^{-5}\ m^3/m$$

Based on the computed section modulus and maximum bending moment, the PZ-22 sheet pile section was selected (Das & Sivakugan, 2019). This section features a width of 558.8 mm and a height of 228.6 mm. It includes a flange thickness of 9.525 mm and a web thickness of 59.974 mm. The PZ-22 weighs 107.316 kN/m. A cross-sectional illustration of the PZ profile is provided below:

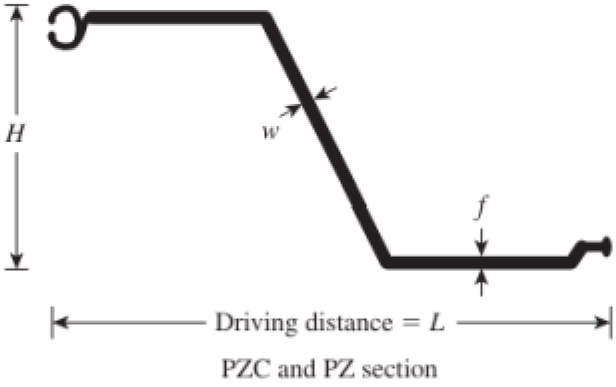


Figure 4.5.1.1 Cross-section of PZ section of steel sheet piles.

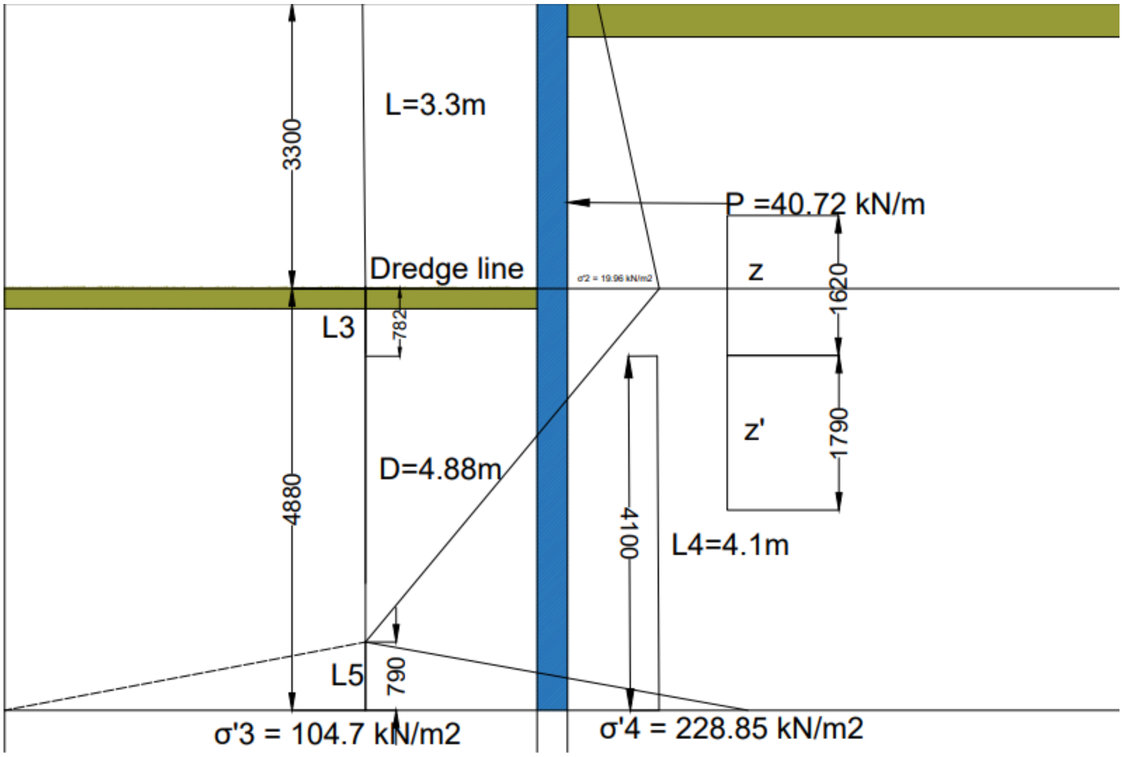


Figure 4.5.1.2. Sheet pile design.

4.5.2. Sheet pile design - Software analysis

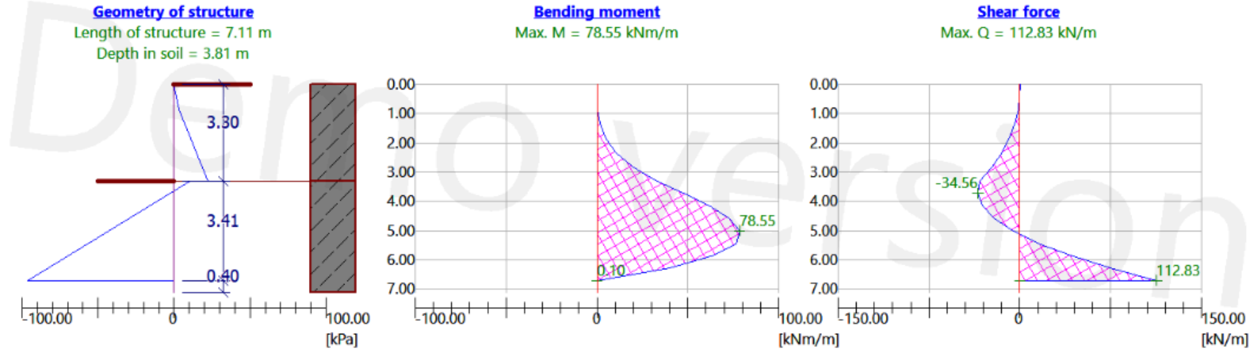


Figure 4.5.2.1. Software analysis of sheet pile design

The sheet pile wall was designed to retain soil with a structure embedded in the soil. The exposed height of the wall above the excavation level is 3.30 m, with a support level located at 3.11 m from the top, and a remaining embedment of 0.40 m below the excavation level.

- Earth Pressure Distribution

The lateral soil pressure increases linearly with depth, which is typical for retained soils under active pressure conditions.

- Bending Moment Diagram

The bending moment diagram, located in the middle, illustrates the internal moment distribution along the wall height due to the applied soil pressure. The maximum bending moment is 78.55 kNm/m, occurring near the support level. It is important for determining the required section modulus of the sheet pile to ensure it can resist the moment without yielding.

- Shear Force Diagram

The shear force distribution is shown on the right. The maximum shear force reaches 112.83 kN/m. These forces must be resisted by the wall section and transferred safely to the support and soil.

4.6 Detailed Construction Procedure

4.6.1. Site Preparation Process

Site preparation is the process involving different steps and should be carried out before the rest of the construction work, hence it is a necessary element for the construction. This site preparation makes the on-site activities to be performed well. It may take different amounts of time depending on the size and location of the project.

It will normally take 1 to 2 months for the construction of residential buildings. But if site preparation is not carried out correctly, it may result in huge challenges regarding the budget and the timeline. There are different steps in site preparation to finish the remaining work on site.

4.6.2. Site Clearing

Site clearing is the initial step towards site preparation, and it entails clearing trees and existing structures and underground barriers. Site clearing plays a very crucial role to play in subsequent construction steps (Best Management Practices (BMPs)..., 2009).

4.6.3.Site Surveying

Surveys are carried out by surveyors before the commencement of construction in order to specify where the building and other structures will be constructed. Pegs or colored spots decide where to construct or not (Site Preparation of Building Construction Project, 2022). This is a discretionary process but for zoning the place and permission processes and for further site plan.

4.6.4. Soil testing

This must be carried out before the site procurement. One must analyze soil properties and test its water absorption rate and construction resistance before one starts any work that is conducted on-site. Engineers who appear on site when soil testing might find that the ground is not ideal for their project, so other options have to be looked into. Soil is classified into classes that influence the determination of the building foundation type (Site Preparation of Building Construction Project, 2022).

4.6.5. Site plan design

After the soil testing process has been completed, all the drainage and septic systems required must be installed. They are shown in site plan design at the correct location. Further, site plan design illustrates the placement of water tanks and structures like usable roads, 20 construction equipment, offices, storage sheds for material, and the building. Worth appreciation is that unlike the rest of site preparation stages, the step is, at times, adjusted due to a change in on-site conditions.

4.6.6. Site investigation

It is critical in structure design depending on received geotechnical information. Site investigation includes rock, soil, and water site conditions, and in most cases includes three phases: preliminary, detailed, and additional. The initial aim here is to define the substructure and foundation parameters, and secondarily to recognize potential risks and dangers during construction

4.6.7. Grading

The ground is to be graded to obtain level flatness (Site Preparation of Building Construction Project, 2022).

4.6.8. Excavation Works

Excavation will be conducted according to construction drawings for proper pile and footing area alignment. Benchmarks are established and ground is surveyed prior to excavation in order to mark existing and necessary levels. Depth of excavation is based on geotechnical investigation and approved by site engineers. As mentioned by Das and Sobhan (2017), alignment control and depth control are necessary for load transfer and foundation integrity.

Where sheet piles, or temporary retaining walls, are installed, they can be removed by a vibratory hammer, with less ground vibration and soil movement upon removal than installation. Athanasopoulos, Vlachakis, and Pelekis (2011) found that vibratory methods reduced induced vibrations, thereby mitigating risks to adjacent structures. As per sustainable building practices, excavated material where possible will be recycled to landscape or utilized for backfilling to reduce waste and save project cost (Zuo and Zhao, 2014).

4.6.9. Sheet pile installation

Cantilever sheet pile is chosen to be driven. The designed 6.08m length sheet pile is to be driven in the medium moist, soft soil mainly made up of well-graded sand. According to the Steel Sheet Pile Installation Manual by ArcelorMittal, the impact vibratory driving method is chosen for pile installation because of a number of benefits. The major concern in an urban area is pollution due to noise, and thus vibratory continuous non-impulsive sound is used instead of the sharp sounds produced by impact driving. Round-grain sand and soft ground conditions are ideal for the vibratory driving method. (Steel sheet pile installation, 2004).

The width of a pile section depends upon the soil properties and driving force. The larger sections are economical but need more force to be used. Hence it is more appropriate to rely on the soil resistivity defined by SPT N value rather than on economic factors. Sheet pile will be hammered passing through 3 layers of soil with SPT N values between 13-16 blows. According to Steel Sheet Pile Installation Manual by ArcelorMittal, the equivalent modulus of wall is 500 cm^3 for high yield steel (2004).

A 1800 RPM normal vibratory pile driver providing centrifugal force of 5000kN would be employed for pile installation. Installation rate is 50 cm/minute.

Reduction of the noise would be obtained through the use of sheet pile presses. Press jack independent of the crane must be employed, which is more favorable to provide mobility from panel to panel.Utilization of cranes would entail the positioning of a crane at each position, which takes time. Pile press jacks rely on the mechanism of using the weight to push the pile to the full depth and the remaining piles are pushed by the aid of the reaction force of the previous piles (Steel sheet pile installation 2004).

With regards to how installation is to be done, panel driving method suits best medium-dense soil so that it has good verticality and good alignment. Piles are considered as a block of panels, but the installation begins with the side piles and proceeds with the inner piles. The second panel is driven and pithed after the alignment of the first panel as seen in Figure 4.6.9.1 below:

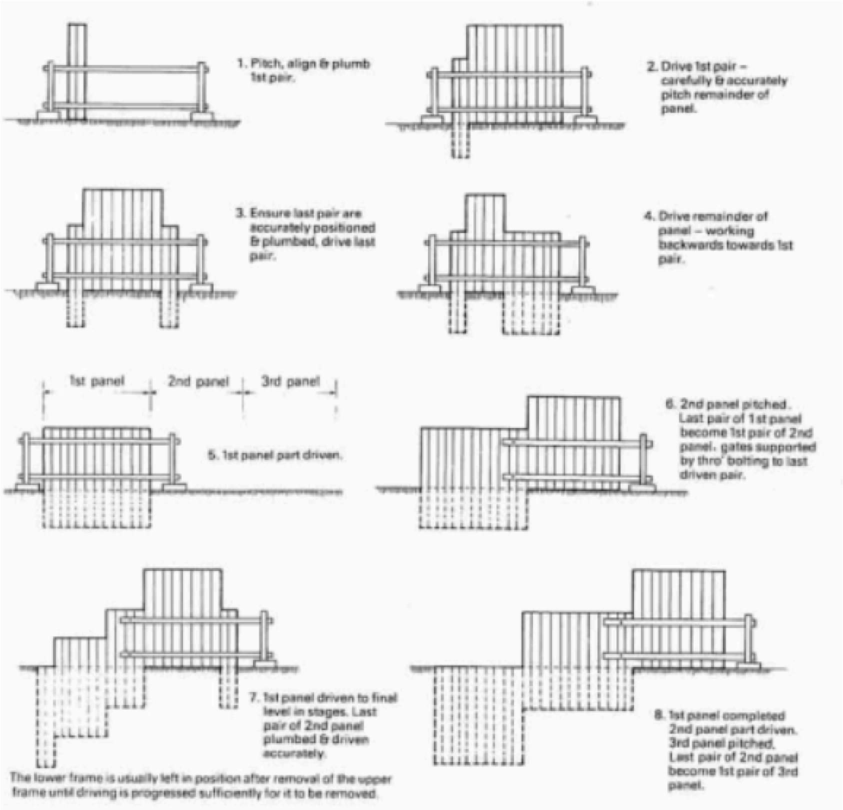


Figure 4.6.9.1. Panel Driving method for sheet pile installation (Steel sheet pile installation, 2004)

4.6.10. Steel Pile Driving Procedure

Driven steel pile driving is done by erecting piles in standing position and driving them into the ground using an impact or vibratory hammer. The choice of hammer is determined by soil type and pile size. Rollins and Olsen (2011) discuss that the wrong choice of a hammer can lead to poor penetration or a pile head damage.

In order to protect the pile head during driving, synthetic pads or wood packing is used for reducing stress concentration (Liao et al., 2019). Driving is watched closely in order to drive every pile to design depth or desired set, usually expressed as a blow count per inch of penetration, a value of bearing capacity.

A drivability analysis, typically with wave equation analysis (WEAP), must be conducted prior to pile driving to establish expected resistance and stress during installation (Paikowsky et al., 2004).

4.6.11. Further monitoring

Piling and excavation necessitate proper monitoring for safety of the excavation, as well as surrounding buildings. Aside from monitoring systems, they are used to validate the design conditions. Site conditions can be other than the analysis results. Thus it crucial to pull fresh data and ensure the design specifications

5. Stormwater System Design

5.1. Topography and inlets

Overall topography of the area is quite flat with around 6-13 ft elevation from the sea level, and the only significant elevation can be noticed in the neighboring area (10 ft mark is where our site is located). This might have been a problem to mitigate in a case if this area would be in a storm risk zone, but as the Topographic Map information on terrain, the area is quite safe regarding flooding.

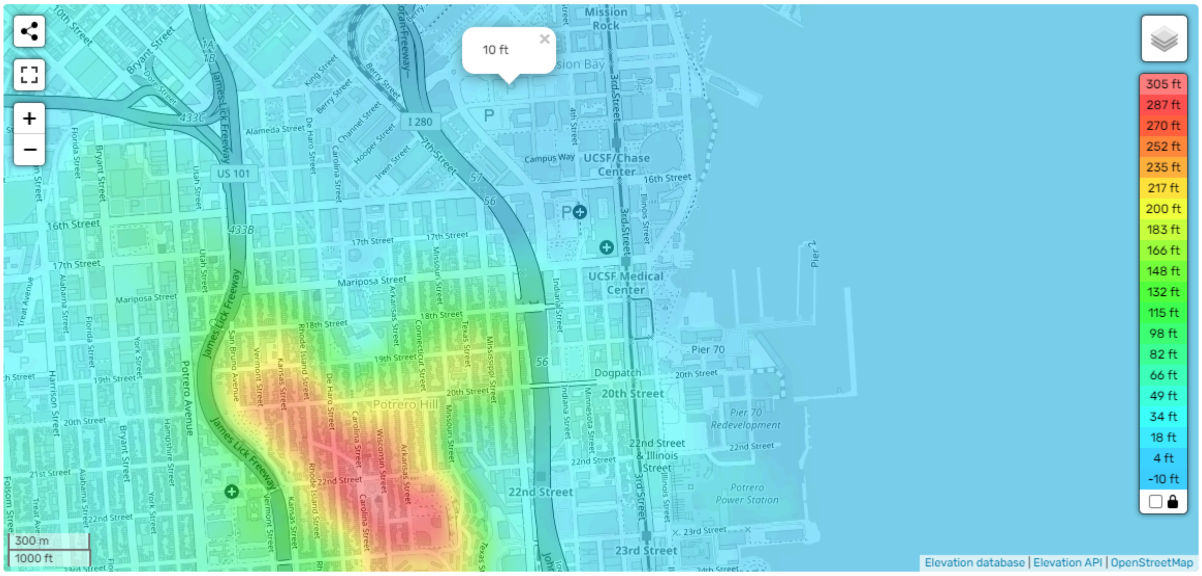


Figure 5.1. Overall topography of a larger area

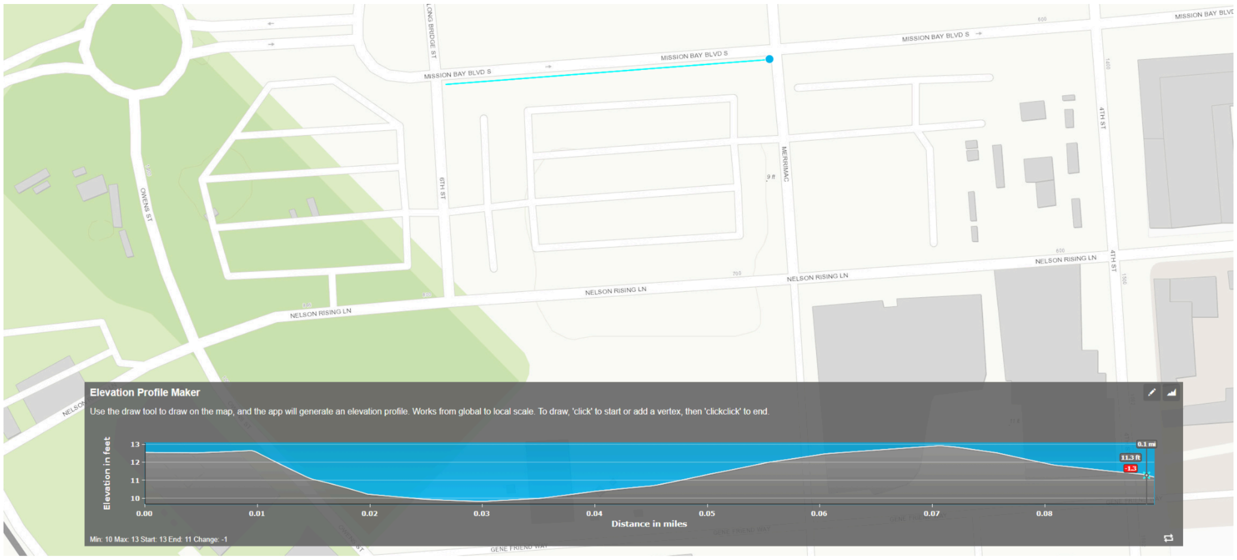
Figure 5.2. Locations of inlets on the site

The present inlets are Stormwater inlet drain 259504 (right corner) and Drop inlet 259556 (near the center) and they are both of MS4 type which means that they are connected to rainwater catchment and disposal to oceanside system (separate from municipal wastewater system) as the area is located near the shore (City and County of San Francisco, n.d.). The whole area uses this system as it is a sustainable way to deal with stormwater.

Specific terrain analysis showed us that the water is more likely to flow from LHS (13 ft) to RHS (7 ft) and from bottom (13 ft) to top (11 ft) of the site as you can see from the following Figure 5.3:

Figure 5.3. Flow direction preferences from topographical analysis.

Elevation along the line is intended for the artificial slope to move rainwater from the area towards catchment basins located at the top Right Hand Side corner of the site (where lines meet). We can see that the natural terrain of the site allows for such positioning and it would be sustainable for the system to use its natural advantages.


Elevation Profile Maker

Use the draw tool to draw on the map, and the app will generate an elevation profile. Works from global to local scale. To draw, 'click' to start or add a vertex, then 'clickclick' to end.

Min: 10 Max: 13 Start: 13 End: 11 Change: -1

Figure 5.4. Elevation profile for horizontal direction

Elevation Profile Maker

Use the draw tool to draw on the map, and the app will generate an elevation profile. Works from global to local scale. To draw, 'click' to start or add a vertex, then 'clickclick' to end.

Min: 11 Max: 15 Start: 14 End: 11 Change: -3

Figure 5.5. Elevation profile for vertical direction

As we have those 2 inlets, the storm water system was designed around them. Because the playground for children and basketball court area has vegetation, only the parking area and roof were provided with extensive outlets and drainage pipes/gutters. Roof catch basins go through balconies, and along with parking area basins they are connected to Storm water inlet drain № 259504 (nearest to building). Topographic parameters were considered to determine the flow direction, the site is currently a parking lot which has a slope of close to 0, so the surfaces (roof and parking area) will be developed with 1-2% slope towards basins. And consequently, the pipeline system too will be inclined in the direction of flow. Following figures illustrate the preliminary system, consider all figures as they complete each other:

5.2. Design of the main pipeline

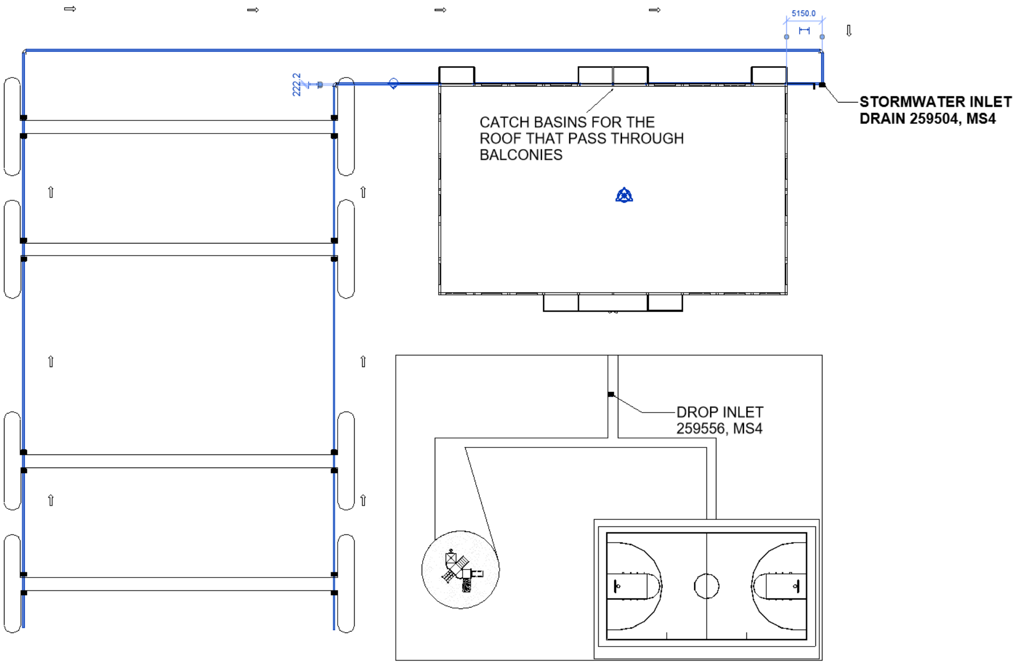


Figure 5.6. 2D plan of the underground system with flow directions

So the design of underground pipes looks like this: they catch the water from the parking area through basins and deliver the water to inlet 259504. Along the way, one of the pipes also catches the rooftop stormwater which is collected along the 3-balcony side of the building and the pipes connecting basins to pipes go through balconies. They will be covered for safety and aesthetic reasons, each balcony has 2 basins and they have separate pipes.

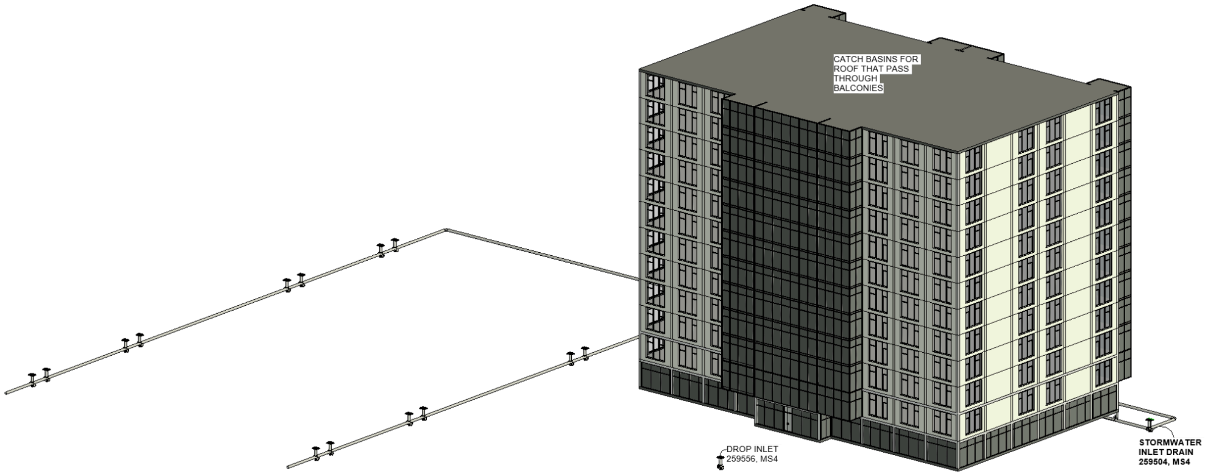


Figure 5.7. 3D plan of the underground system

5.3. Flood risk mitigation

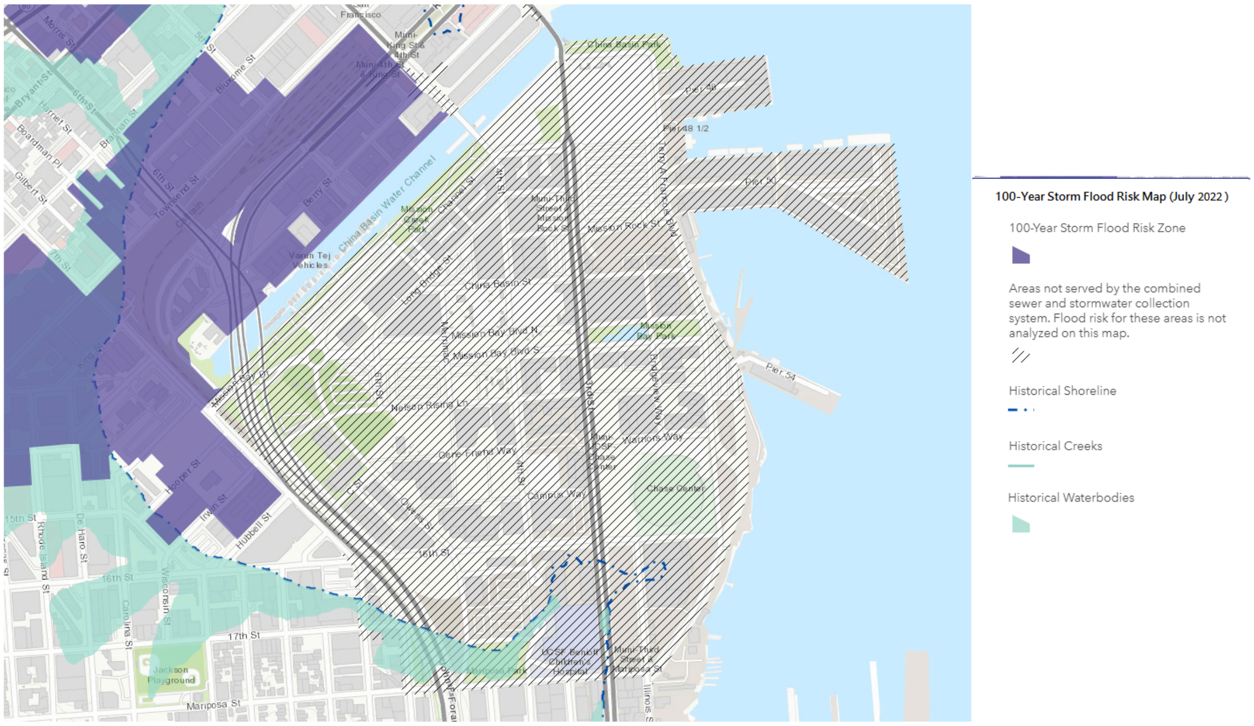


Figure 5.8. Flood map of the whole district

Unfortunately, we can see that the flood risk analysis for the area is missing. Hopefully from Figure 5.7 above we can see that the area has a specific type of inlet and drainage system different from the city system. Both inlets are of MS4 type which means that they are connected to rainwater catchment and disposal to the oceanside system (separate from municipal wastewater system). Which means that it is designed to withstand flood danger even though the location is near 100-Year Storm flood risk zone. Also the proximity of the shore and low elevation from the sea level makes it a hazardous zone. However the topographic analysis below shows that the elevation of the area would not allow for a flood risk as the shoreside seems to be even lower standing at 1m:

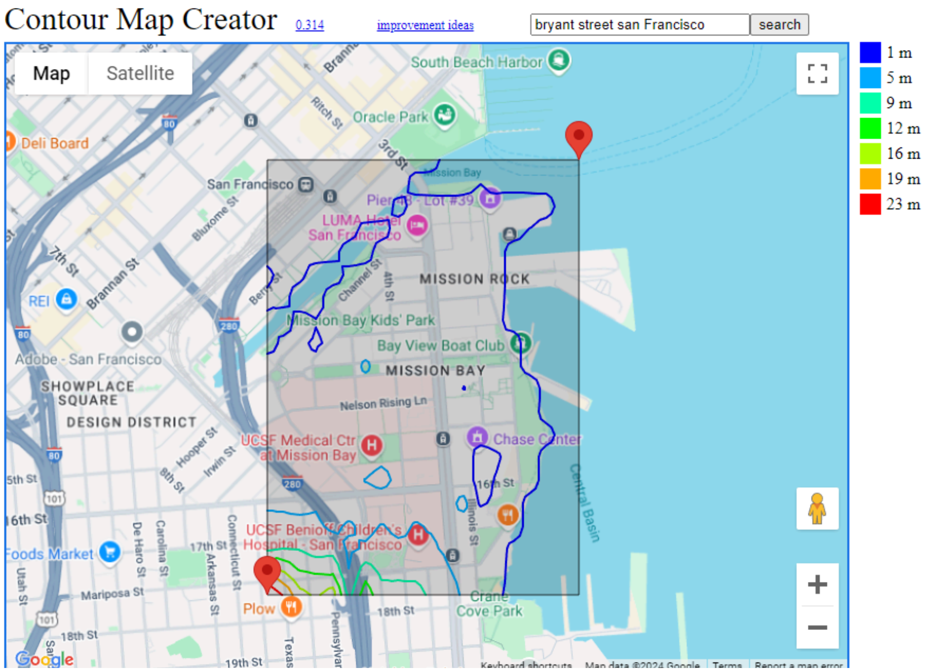


Figure 5.9. Elevation map of the district in respect to the shore

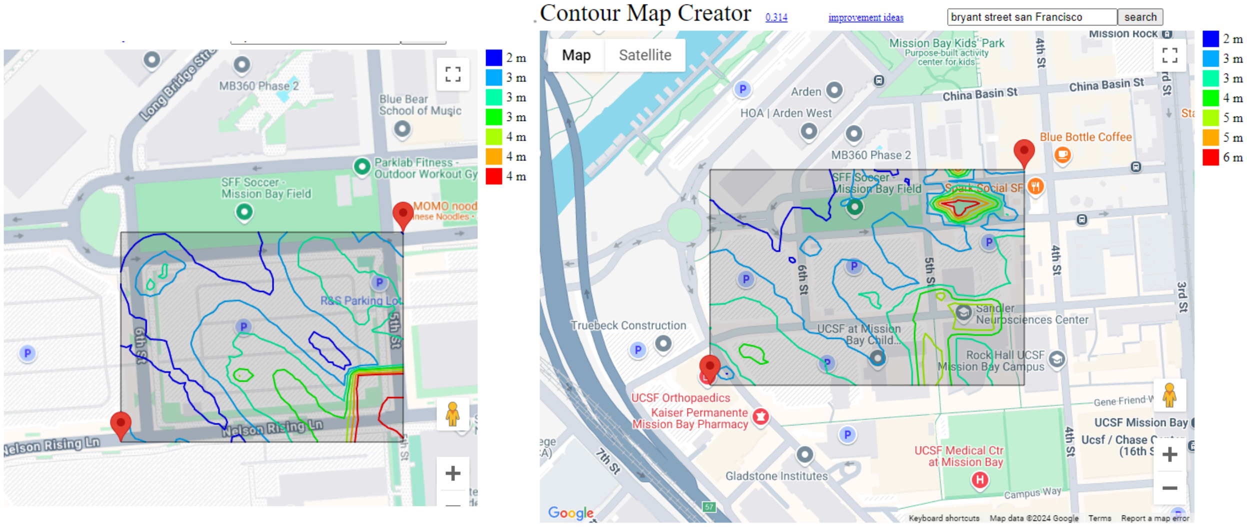


Figure 5.10. Contour map of the site and surrounding area

5.4. Terrain of the site

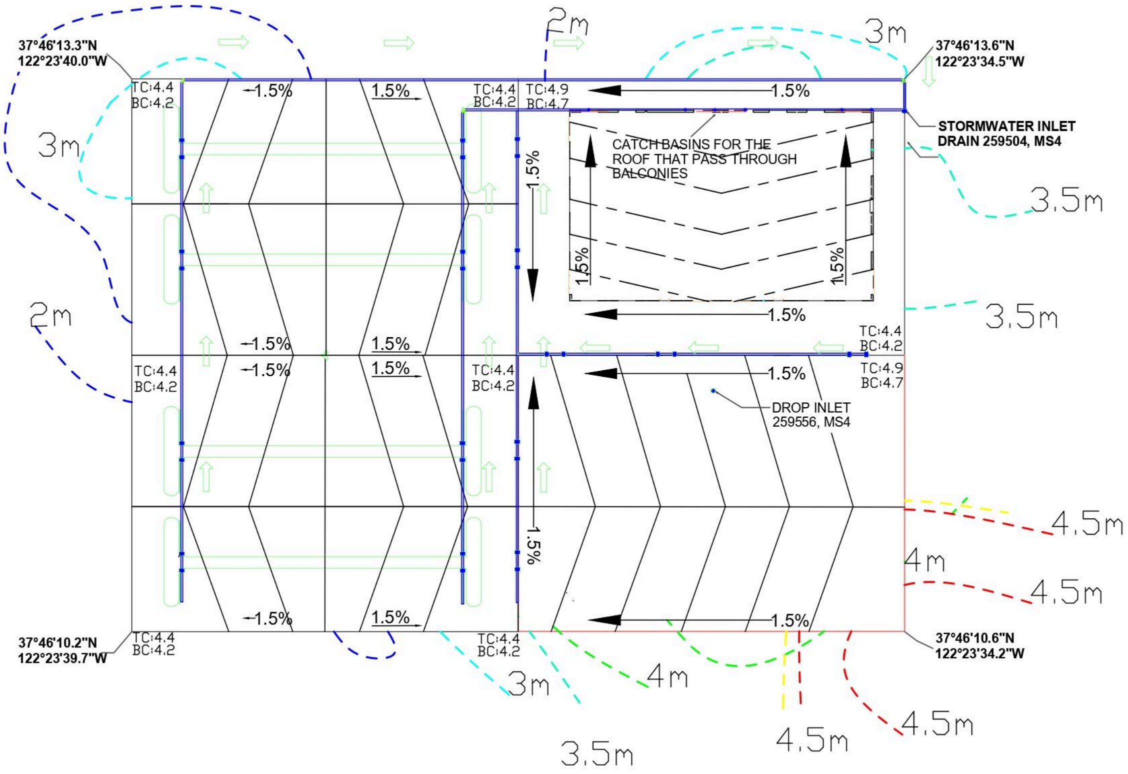


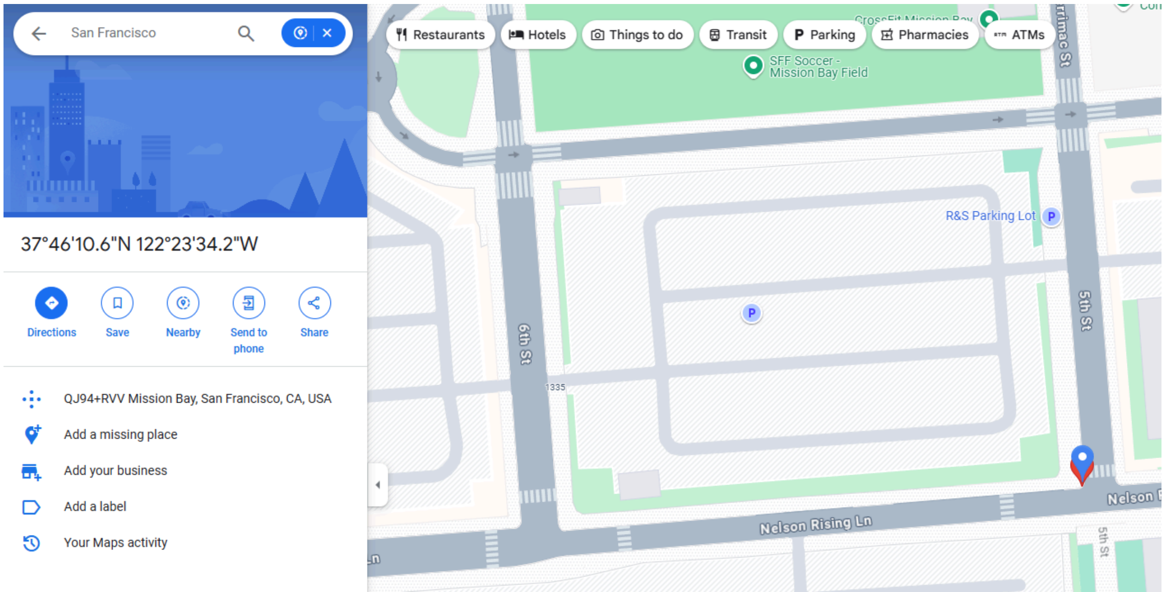
Figure 5.11. Terrain of the site before construction

As you can see, the surface water runoff direction is given by black arrows along with the respective slopes. They represent artificial slopes and are used for the roof of the building and in the areas where cut and fill operations are planned to be conducted. Green arrows represent the underground directions of the stormwater and follow the natural terrain of the site given in Figures 5.4-5.5 above. Also the overall stormwater is collected at the Storm water inlet drain № 259504, MS4, which has much better drainage and bigger capacity for accommodating the columns of water rather than the drop inlet around the middle of the site. So the decision was made to design the system to transfer the surface runoff to the main underground pipes, which were designed for the midterm presentation before (Figure 5.6). Also the Figure above represents the changes in elevation as the previous layout had elevations all around the site. This final design has the elevation only for around the site area, while maintaining the flattened areas for building roof and the other parts such as parking lot, backyard and walking area.

The terrain of the site that is given above is redrawn from the contour map of the site. It is given above in Figure 5.11 and has elevation values rounded to integers, however the actual elevation is a little different and was used in AutoCad, so the values we have put on the design are more accurate.

Arrow directions represent the higher ground (according to the slope-arrow designation) and in this case it is an artificial 1.5% slope all along the site, including the roof slope. Please note that the TC and BC stands for Top Curb and Bottom Curb. The ground level is constructed to be 0.2 meters above the highest elevation for the parts of a site, e.g. backyard and parking. And the height of curbs are uniform, 0.2 meters for the border and for central parking curbs too. So the backyard is positioned a little higher than the rest of the area as the highest elevation of 4.5 meters is present at the RHS corner of the site. Probably some soil filling will be performed to create lawns around the backyard.

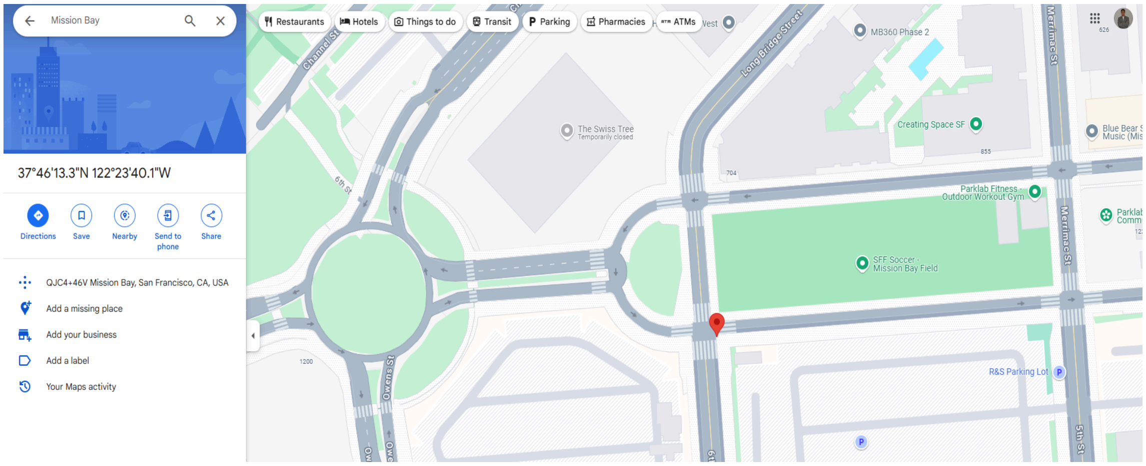
Following Figure 5.12 is an example of a coordinate retrieval from Google Maps and this was done to 4 of the corners of the site. They are provided in the Final drawing and you can find them in Figure 5.11, too.



37°46'10.6"N 122°23'34.2"W

QJ94+RVV Mission Bay, San Francisco, CA, USA

Figure 5.12. Coordinates of RHS bottom corner of the site



37°46'13.3"N 122°23'40.1"W

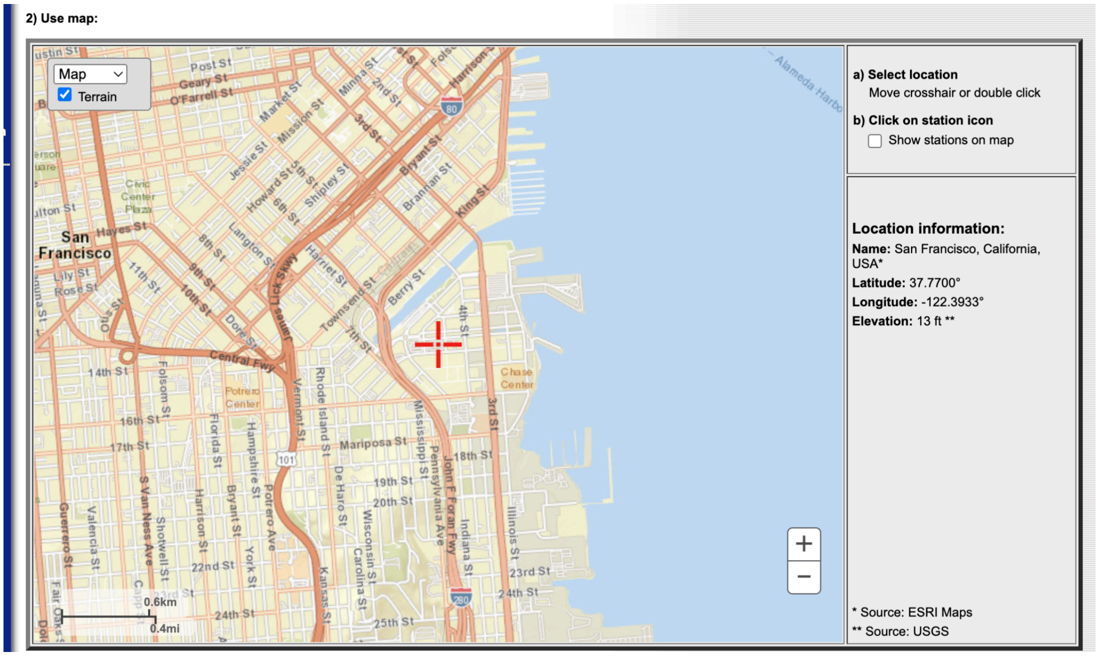
QJC4+46V Mission Bay, San Francisco, CA, USA

Figure 5.13. Coordinates of LHS bottom corner of the site

Overall, the design of the stormwater system turned out to be comprehensive and understandable for the intended audience as it follows the guidelines of engineering designation, such as slope symbols, curb values and the terrain drawings too. It also meets the requirements for sustainability as no electropower is used in maintenance of the system as it takes advantage of the natural terrain. At the same time, it is protected from floods due to relatively similar elevation with the surrounding area, and also has inlets all around to make sure the runoff is discharged properly.

5.5. Rainfall data

We have looked at some calculation of peak runoff using the Rational Formula from our previous class and calculated initial values as well total runoff coefficient based on our area. The division of the site into 3 categories is preliminary as in the future some detailed calculations will be provided as the backyard, for example may vary in terms of the material there (football or basketball court, or some paved walking routes and so on). Also the material for the roof and around the building may be different, as well as some vegetation in the parking lot will definitely change its runoff coefficient. Below is the work done so far on determining the values needed and some findings from official weather broadcasting reports.


2) Use map:

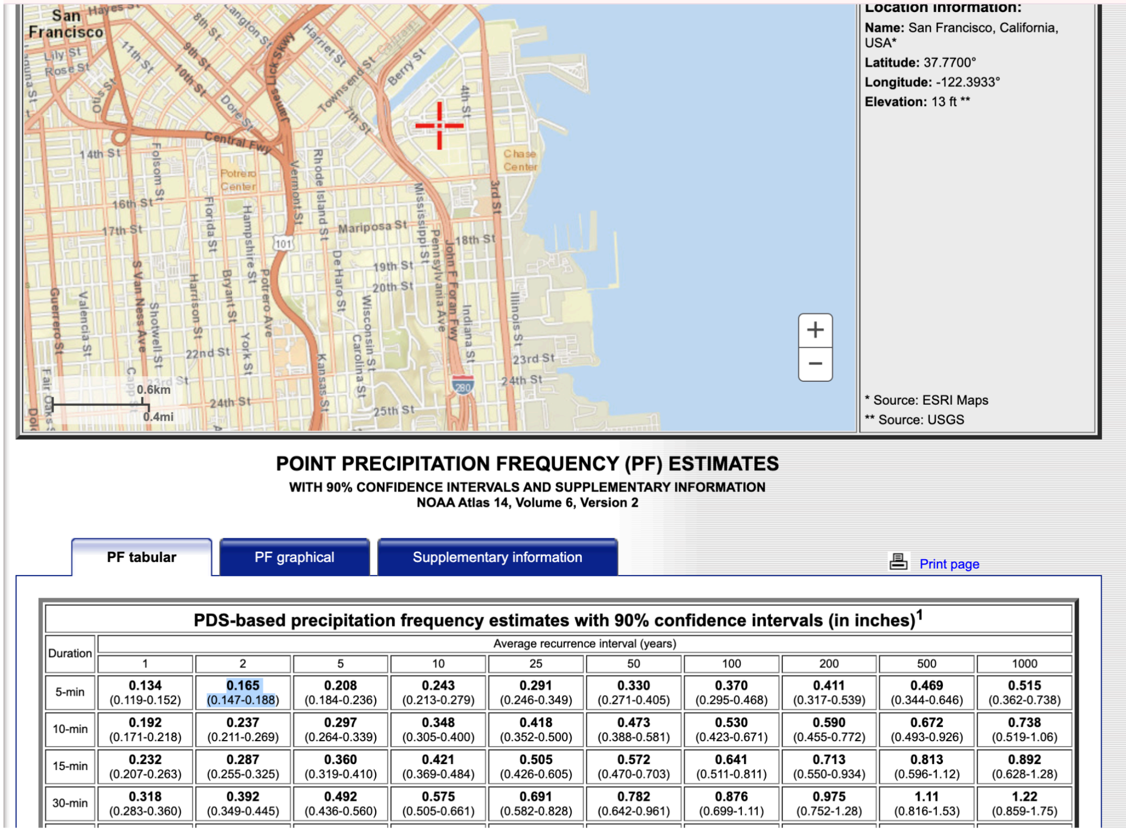
a) Select location
Move crosshair or double click

b) Click on station icon
☐ Show stations on map

Location information:
Name: San Francisco, California, USA*
Latitude: 37.7700°
Longitude: -122.3933°
Elevation: 13 ft **

* Source: ESRI Maps
** Source: USGS

Figure 5.14. Exact location of our site on the website for rainfall data.



Location information:
Name: San Francisco, California, USA*
Latitude: 37.7700°
Longitude: -122.3933°
Elevation: 13 ft **

* Source: ESRI Maps
** Source: USGS

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES
WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 6, Version 2

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.134** (0.119-0.152)	**0.165** (0.147-0.188)	**0.208** (0.184-0.236)	**0.243** (0.213-0.279)	**0.291** (0.246-0.349)	**0.330** (0.271-0.405)	**0.370** (0.295-0.468)	**0.411** (0.317-0.539)	**0.469** (0.344-0.646)	**0.515** (0.362-0.738)
10-min	**0.192** (0.171-0.218)	**0.237** (0.211-0.269)	**0.297** (0.264-0.339)	**0.348** (0.305-0.400)	**0.418** (0.352-0.500)	**0.473** (0.388-0.581)	**0.530** (0.423-0.671)	**0.590** (0.455-0.772)	**0.672** (0.493-0.926)	**0.738** (0.519-1.06)
15-min	**0.232** (0.207-0.263)	**0.287** (0.255-0.325)	**0.360** (0.319-0.410)	**0.421** (0.369-0.484)	**0.505** (0.426-0.605)	**0.572** (0.470-0.703)	**0.641** (0.511-0.811)	**0.713** (0.550-0.934)	**0.813** (0.596-1.12)	**0.892** (0.628-1.28)
30-min	**0.318** (0.283-0.360)	**0.392** (0.349-0.445)	**0.492** (0.436-0.560)	**0.575** (0.505-0.661)	**0.691** (0.582-0.828)	**0.782** (0.642-0.961)	**0.876** (0.699-1.11)	**0.975** (0.752-1.28)	**1.11** (0.816-1.53)	**1.22** (0.859-1.75)

Figure 5.15. Desired rainfall data for 5 min storm with 2 years of recurrence interval.

To determine the total runoff coefficient for our site, we need to use different values for various types of surfaces along the area, such as pavement and grass. Here is how we calculated the total runoff:

5.6. Data of the site

Total Land Area = 115,368 sq ft

Breakdown of areas:

- Parking Lot: 57,391.26 sq ft (49.76%)
- Backyard (Grass): 41,153.94 sq ft (35.68%)
- Building: 29,139.42 sq ft (25.25%)

Land Use Fractions:

- Parking Lot = 0.4976
- Backyard (Playground with Grass) = 0.35679
- Building = 0.2525

Runoff Coefficients:

- Parking Lot = 0.9
- Backyard (Grass) = 0.2
- Building = 0.9

5.7 Weighted Runoff Coefficients

Now we Compute Weighted Runoff Coefficient

The total weighted runoff coefficient is calculated as:

$$C_{total} = (A_1 \times C_1 + A_2 \times C_2 + A_3 \times C_3) / A_{total} \tag{5.1}$$

$$\mathrm{C_{total}} = (57,391.26 \times 0.9 + 41,153.94 \times 0.2 + 29,139.42 \times 0.9) / 115,368$$

$$\mathrm{C_{total}} = (51,652.13 + 8,230.79 + 26,225.48) / 115,368$$

$$\mathrm{C_{total}} = 86,108.4 / 115,368 = 0.7466$$

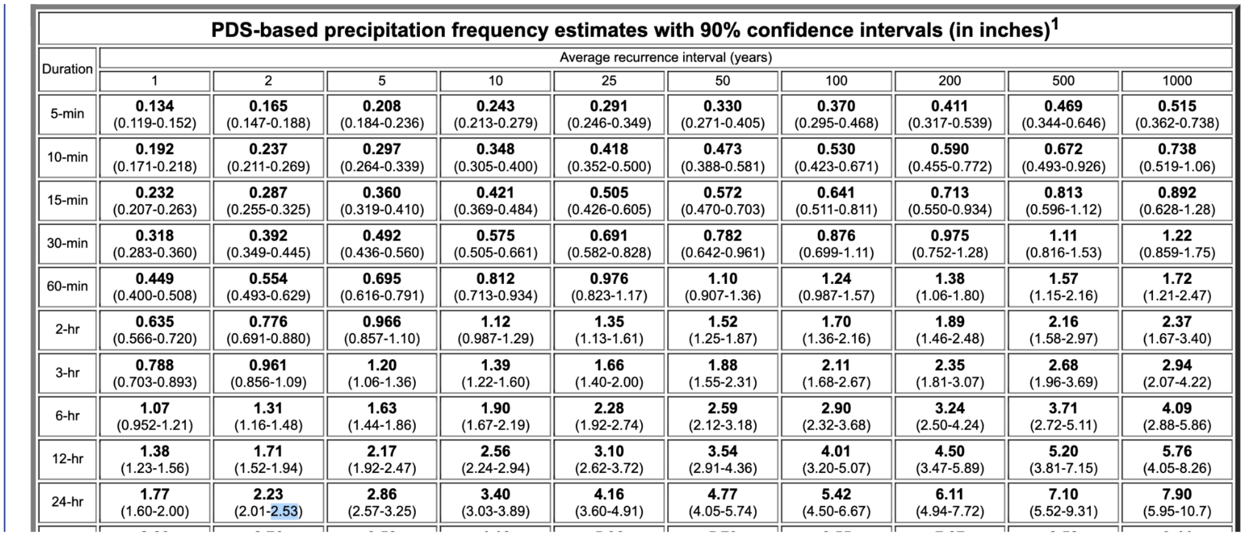
Total weighted runoff coefficient (C) ≈ 0.75. Here we proceed to calculate the runoff flow rate (peak runoff) using the Rational Method:

$$Q = C * I * A \tag{5.2}$$

- C = Runoff coefficient (calculated as 0.75 from previous step)
- I = Rainfall intensity (0.165 in/hr, from Figure 5.15 above)
- A = Drainage area in acres (115,368 sq ft = 2.65 acres)

The runoff flow rate Q is 0.33 cubic feet per second (cfs)

The calculations above are inaccurate because of the incorrect regulation that we used for the duration of the rainfall. We used the values of 5 minutes and also did not convert that value into in/hr unit. The regulation was for Washington, DC, and we took the correct value of 24 hours for the duration of the rainfall as that is required from stormwater drainage systems constructed in the city of San Francisco, California.



PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.134** (0.119-0.152)	**0.165** (0.147-0.188)	**0.208** (0.184-0.236)	**0.243** (0.213-0.279)	**0.291** (0.246-0.349)	**0.330** (0.271-0.405)	**0.370** (0.295-0.468)	**0.411** (0.317-0.539)	**0.469** (0.344-0.646)	**0.515** (0.362-0.738)
10-min	**0.192** (0.171-0.218)	**0.237** (0.211-0.269)	**0.297** (0.264-0.339)	**0.348** (0.305-0.400)	**0.418** (0.352-0.500)	**0.473** (0.388-0.581)	**0.530** (0.423-0.671)	**0.590** (0.455-0.772)	**0.672** (0.493-0.926)	**0.738** (0.519-1.06)
15-min	**0.232** (0.207-0.263)	**0.287** (0.255-0.325)	**0.360** (0.319-0.410)	**0.421** (0.369-0.484)	**0.505** (0.426-0.605)	**0.572** (0.470-0.703)	**0.641** (0.511-0.811)	**0.713** (0.550-0.934)	**0.813** (0.596-1.12)	**0.892** (0.628-1.28)
30-min	**0.318** (0.283-0.360)	**0.392** (0.349-0.445)	**0.492** (0.436-0.560)	**0.575** (0.505-0.661)	**0.691** (0.582-0.828)	**0.782** (0.642-0.961)	**0.876** (0.699-1.11)	**0.975** (0.752-1.28)	**1.11** (0.816-1.53)	**1.22** (0.859-1.75)
60-min	**0.449** (0.400-0.508)	**0.554** (0.493-0.629)	**0.695** (0.616-0.791)	**0.812** (0.713-0.934)	**0.976** (0.823-1.17)	**1.10** (0.907-1.36)	**1.24** (0.987-1.57)	**1.38** (1.06-1.80)	**1.57** (1.15-2.16)	**1.72** (1.21-2.47)
2-hr	**0.635** (0.566-0.720)	**0.776** (0.691-0.880)	**0.966** (0.857-1.10)	**1.12** (0.987-1.29)	**1.35** (1.13-1.61)	**1.52** (1.25-1.87)	**1.70** (1.36-2.16)	**1.89** (1.46-2.48)	**2.16** (1.58-2.97)	**2.37** (1.67-3.40)
3-hr	**0.788** (0.703-0.893)	**0.961** (0.856-1.09)	**1.20** (1.06-1.36)	**1.39** (1.22-1.60)	**1.66** (1.40-2.00)	**1.88** (1.55-2.31)	**2.11** (1.68-2.67)	**2.35** (1.81-3.07)	**2.68** (1.96-3.69)	**2.94** (2.07-4.22)
6-hr	**1.07** (0.952-1.21)	**1.31** (1.16-1.48)	**1.63** (1.44-1.86)	**1.90** (1.67-2.19)	**2.28** (1.92-2.74)	**2.59** (2.12-3.18)	**2.90** (2.32-3.68)	**3.24** (2.50-4.24)	**3.71** (2.72-5.11)	**4.09** (2.88-5.86)
12-hr	**1.38** (1.23-1.56)	**1.71** (1.52-1.94)	**2.17** (1.92-2.47)	**2.56** (2.24-2.94)	**3.10** (2.62-3.72)	**3.54** (2.91-4.36)	**4.01** (3.20-5.07)	**4.50** (3.47-5.89)	**5.20** (3.81-7.15)	**5.76** (4.05-8.26)
24-hr	**1.77** (1.60-2.00)	**2.23** (2.01-2.53)	**2.86** (2.57-3.25)	**3.40** (3.03-3.89)	**4.16** (3.60-4.91)	**4.77** (4.05-5.74)	**5.42** (4.50-6.67)	**6.11** (4.94-7.72)	**7.10** (5.52-9.31)	**7.90** (5.95-10.7)

Figure 5.16. Desired rainfall data for 24 hour storm with 2 years of recurrence interval.

As you can see from the figure, the data is for a 90% confidence interval, and we also took the value of an upper bound just to make sure and be confident in the results. If we look at the regulation for the city, "Separate Sewer Areas (MS4 Areas):

- For projects in Port of San Francisco's jurisdiction, calculations shall demonstrate that the system is sized to capture and treat all runoff from the 85th percentile, 24- hour design storm (0.63-inch total depth, constant intensity of 0.2 in/hr).
- For projects in SFPUC jurisdiction, calculations shall demonstrate how the system is sized to capture and treat all runoff from the 90th percentile, 24-hr storm (0.75-inch total depth, constant intensity of 0.24 in/hr)." (San Francisco Public Utilities Commission, 2016, p. 1)

The drainage system we have is exactly the MS4 type (please refer to the previous reports where it is highlighted for both of the inlets), so even though we could not understand about if it is for the port or for the SFPUS, we went for the 90th percentile to make it safer and even for the upper bound. And the 2 year period was taken because of the quite frequent recurrence of intense rainfalls in the area as it was also found in the report from last semester.

Another thing was to calculate the actual time of concentration for every part of the site, and the previous division had the building part as a whole, but this time we decided to calculate for the roof only, as that is much more representative of the maximum rainfall amount for the sector. As the water around the building would probably dissipate in 2 directions and would not cause a lot of runoff volumes, and the roof would definitely accumulate rainfall of its own. So the actual area of the roof was taken into account from 1:1 AutoCad model of our site, and you can see it from the pictures below:

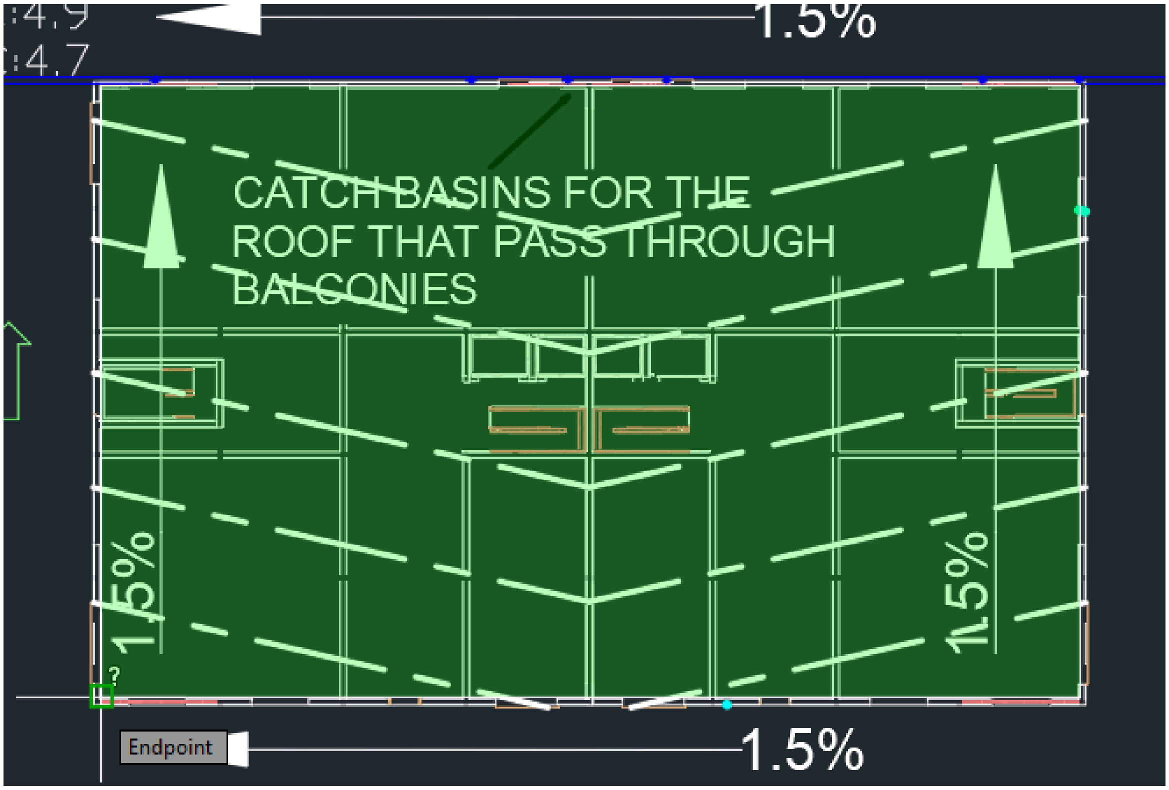


Figure 5.17. AutoCad function of “Area” in use and the values are in mm^2.

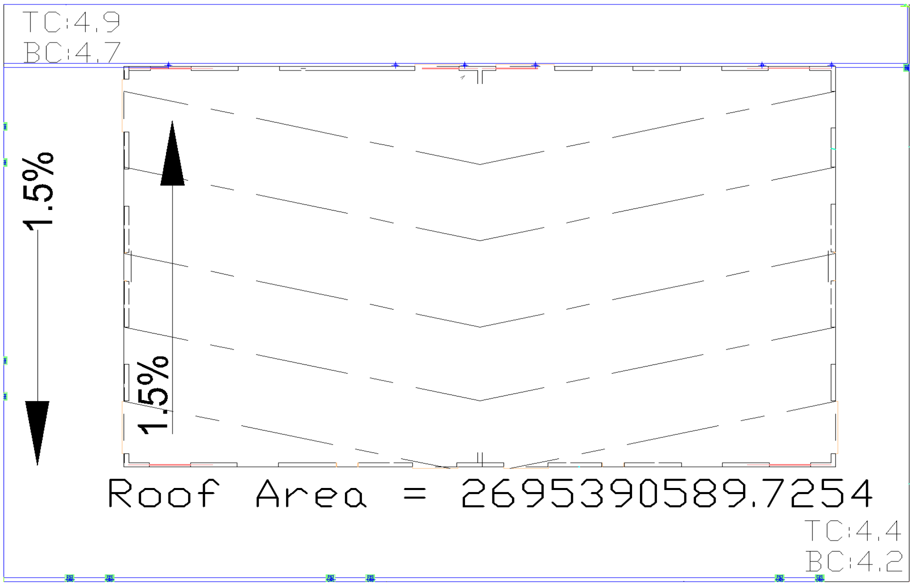
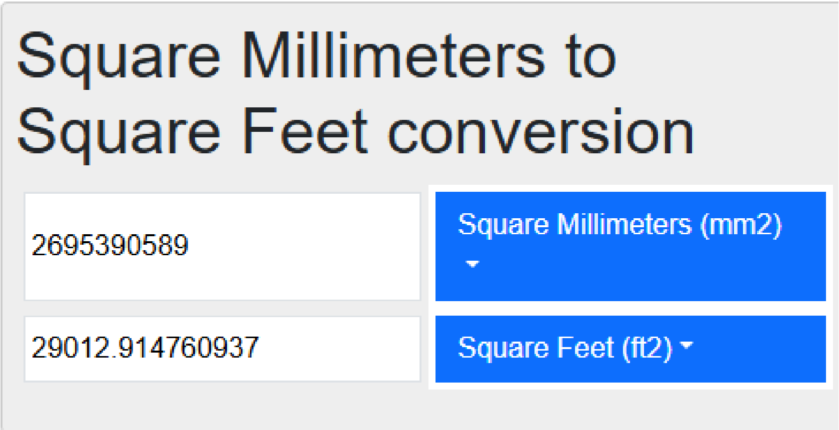


Figure 5.18. AutoCad plan of the building sector with the roof as the main area.



Square Millimeters to Square Feet conversion

2695390589 Square Millimeters (mm2)

29012.914760937 Square Feet (ft2)

Figure 5.19. Area of the roof conversion to sqft.

Same was done for the parking lot and the backyard too, their exact areas are given the final table of this section, Table 5.3.

Before that, as the time of concentration was needed, we calculated 2 of them for each using the formulas 1-3 below:

1. Sheet Flow Equation:

$$T_{ti} = \frac{0.933}{I^{0.4}} = [\frac{nL}{\sqrt{S}}]^{0.6} \qquad (5.3)$$

2. Shallow Concentrated Flow Equation

$$T_{t2} = \frac{L}{V} \tag{5.4}$$

3. Velocity for Shallow Flow:

$$V = 3.28 * k * S_p^{0.5} \tag{5.5}$$

4. Rational Method for Peak Runoff:

$$V = C * I * A \tag{5.6}$$

5. Total Peak Runoff:

$$Q_{total} = Q_{parking\ lot} + Q_{backyard} + Q_{roof} \tag{5.7}$$

5.8 Parameters and calculations

The values for L were taken as the whole width and length for the roof only, for backyard and parking lot, L values were the farthest points from the edge till the drop inlets. Here are the parameters for time of concentration calculations.

Table 5.1. Parameters for the Time of Concentration Calculations

Surface Type	n (Sheet Flow)	k (Shallow Flow)	L (Sheet Flow)	L (Shallow Flow)	Slope (S)	Intensity (I)
Parking Lot	0.011	0.619	14.275 m (46.84 ft)	23.423 m (76.84 ft)	0.015	0.104 in/hr
Backyard (Grass)	0.15	0.457	44.017 m (144.43 ft)	61.525 m (201.85 ft)	0.015	0.104 in/hr
Roof	0.011	0.619	29.7 m (97.44 ft)	47.685 m (156.47 ft)	0.015	0.104 in/hr

Here is an example of time concentration calculation for the parking lot and the different L values in sheet and shallow flows are given in ft already, converted from mm values from AutoCad model.

Sheet flow calculation:

$$T_{ti} = \frac{0.933}{(0.104)^{0.4}} \left[\frac{0.011 \times 46.84}{\sqrt{0.015}}\right]^{0.6}$$

$$T_{ti} = \frac{0.933}{0.323} \left[\frac{0.011 \times 46.84}{0.122}\right]^{0.6} = 2.89 \times (4.22)^{0.6} = 2.89 \times 2.47 = 7.14 \text{ min}$$

Shallow Concentrated Flow:

$$V = 3.28 \times 0.619 \times (1.5)^{0.5} = 3.28 \times 0.619 \times 1.225 = 2.49 \text{ ft/s}$$

$$Tt2 = \frac{76.84}{2.49} = 30.85 \text{ sec} = 0.51 \text{ min}$$

Total Time of Concentration:

$$T_c = T_{ti} + Tt2 = 7.14 + 0.51 = 7.65 \text{ min}$$

Here is the table for all sectors and their respectives times of concentrations, for them to be used in finding the actual rain intensity, they need to be interpolated and adjusted, so the following table is to demonstrate the interpolation results.

Table 5.2. Time of Concentration Results for 3 Sectors

Surface Type	T_{t1} (Sheet Flow)	T_{t2} (Shallow Flow)	Total T (min)
Parking Lot	7.14	0.51	7.65
Backyard (Grass)	88.04	1.83	89.87
Roof	11.23	1.05	12.28

Table 5.3. Interpolations for the Time of Concentration Values

Surface Type	Tc1 (min)	Tc2 (min)	Used Tc (min)
Parking Lot	7.14	0.51	7.14

Backyard (Grass)	88.04	1.83	44.017
Roof	29.7	47.685	29.7

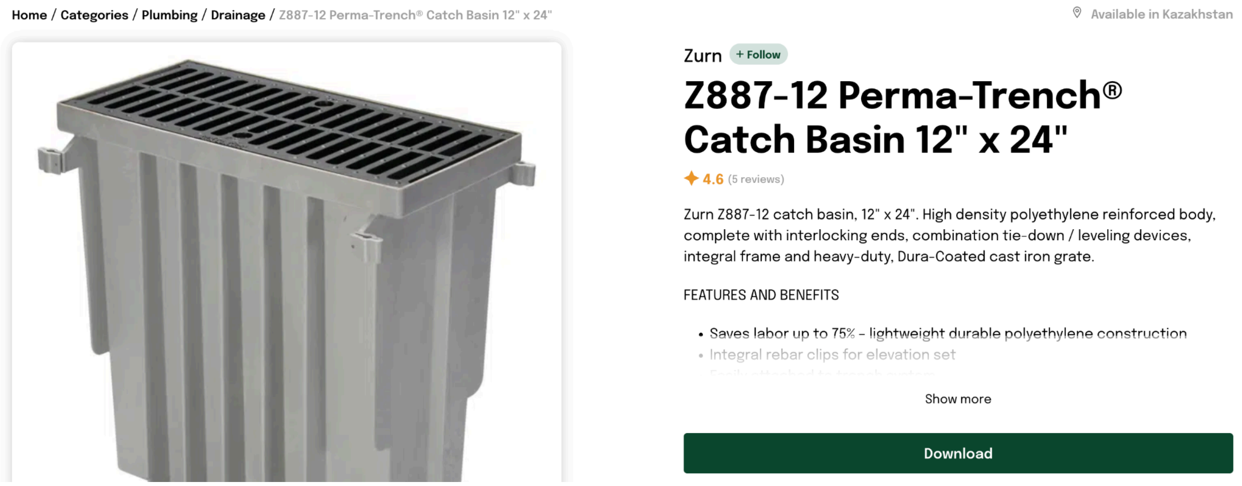
And the last table illustrates the intensity values converted to in/hr and the C values according to the type of surface, impervious ones having a higher value meaning that pavements and roof surfaces increase the runoff volume as they contribute to the water flow and the grass surface retains rainfall and causes less volume to accumulate. And of course, it has the total runoff values at the bottom, as the sum of values of 3 different sectors of our site.

Table 5.4. Final calculations of the total runoff volume

Surface Type	Area (sq ft)	Area (acres)	Pre-Intensity	Rainfall Intensity (in/hr)	Runoff Coefficient (C)	Peak Runoff (cfs)
Parking Lot	58131.85	1.334	0.203	1.59216	0.90	1.911
Backyard (Grass)	29012.94	0.666	0.2598	1.26977	0.25	0.211
Roof/Building	15236.63	0.35	0.664	0.44353	0.85	0.132
Total Runoff	-	-	-	-	-	2.254

5.9. Number of basins

Now we have to calculate the number of catch basins we need for each area separately. For that we need to find the dimensions of the basin that we used for the Revit model of the system. And this below is what we used as catch basins for our site:



Zurn

Z887-12 Perma-Trench® Catch Basin 12" x 24"

4.6 (5 reviews)

Zurn Z887-12 catch basin, 12" x 24". High density polyethylene reinforced body, complete with interlocking ends, combination tie-down / leveling devices, integral frame and heavy-duty, Dura-Coated cast iron grate.

FEATURES AND BENEFITS

- Saves labor up to 75% – lightweight durable polyethylene construction
- Integral rebar clips for elevation set

Figure 5.20. Catch basin type used for our site.

Further we would calculate the capacity of the basins and divide that number to the runoff volumes to see how many of them we need for our system. Per US standards and drainage design manuals (e.g. FHWA HEC-22), a 1 ft × 2 ft curb inlet under typical conditions (flat grade, curb flow) handles approximately 0.75 cfs (and that is a conservative value, considering minimal ponding). The final calculations are made simply by dividing the peak runoff by basin capacity and rounding it to the upper integer. The results are illustrated in Table 5.5.:

Table 5.5. Require basin amount calculations

Area	Peak Runoff (cfs)	Basin Capacity (cfs)	Required Basins
Parking Lot	1.911	0.75	3
Backyard (Grass)	0.211	0.75	1
Roof/Building	0.132	0.75	1
Total	2.254	-	5

Placement of these basins was followed by the slopes and where it is efficient to put them. Obviously, the design is set to have more than a minimum number of basins, so our design has more of them in each of the areas.

5.10. Pipe sizing

Now we need to determine the pipe size for our pipes underneath, for that we will need the Manning's Equation:

$$Q = (\frac{1.49}{n}) \times A \times R^{2/3} \times S^{1/2}$$

Where:

Q = Flow rate (cubic feet per second, cfs)
n = Manning's roughness coefficient (0.013 for concrete and 0.009 for HDPE)
A = Cross-sectional area of pipe (square feet), A = (pi × D^2) / 4
R = Hydraulic radius (feet), R = D / 4 (for full circular pipe)
S = Slope of pipe (unitless), taken here as 0.015 (1.5%)

Here is the comparison of the 2 primary materials for pipelines in San Francisco, concrete and HDPE:

Table 5.5. Comparison of HDPE and Concrete for the pipeline

Criteria	Concrete (RCP)	HDPE (High-Density Polyethylene)
Manning's n	0.013 (smoother than old assumptions)	0.009–0.012 (smoother than RCP)
Strength	High compressive strength	Requires burial support, not self-structural
Durability	Long-lasting (>50 years), corrosion resistant	Resistant to many chemicals, but UV-sensitive
Cost	More expensive to install (labor, equipment)	Cheaper, especially in smaller diameters
Best Use	Roads, heavy loads, deep burial	Residential, flexible alignments, shallow bury

Judging by the important factors of best use and cost, HDPE seems like the best option for our site as the residential purpose is indicated. And that is even though strength criteria is in favor of the concrete, our peak runoff is not as overwhelming to cause any problems and that also applies to durability, concrete pipes would be too much in terms of both material and sufficiency. Here are tables with results from using Manning's equation to compare the sizes of each given the parameters of our site:

Table 5.6. Required diameter in case of a concrete pipe

Area	Flow per Pipe (cfs)	Min Diameter (in)	Recommended Pipe Size
Parking Lot	0.956	6.82	8-inch Concrete
Backyard	0.105	2.98	6-inch Concrete
Roof	0.033	1.93	6-inch Concrete

Table 5.7. Required diameter in case of a HDPE pipe

Area	Flow per Pipe (cfs)	Min Diameter (in)	Recommended Pipe Size
Parking Lot	0.956	6.82	8-inch Concrete
Backyard	0.105	2.98	6-inch Concrete
Roof	0.033	1.93	6-inch Concrete

If we consider the criterias and the economical solution would be HDPE pipes as less material will be used and that seems sufficient, so now we need to look into the regulations for the material. Table 5.8 below shows us different minimum pipe diameters for different use and the appropriate one for our project is the first one, as the ground level pipes will be a "lateral connection" rather than main storm drain line. This means that 6 inches of minimum diameter will be suitable in terms of both the regulation and the peak runoff capacity.

Table 5.8. Required diameter in case of a HDPE pipe

Condition	Minimum Pipe Diameter
Lateral or roof drain connections	**6 inches**
Main storm drain lines	**12 inches**
Curb inlets or shallow flows	**8–12 inches**

5.11. Final technical drawings

Below you will find our cross sectional “cuts” and they demonstrate to us how the water is collected and how the system looks underground. Pay attention to the gravel implementation as an efficient drainage system and it also gets less densely graded deeper you get. This is done to ensure a smooth draining of any excess stormwater. 150 here is in mms and it is 6 inches converted into a unit that is used in AutoCad and the unit that makes sense to us when we think of any lengths. Figures 5.21. -5.22. Illustrate both views and not to scale slope demonstration too:

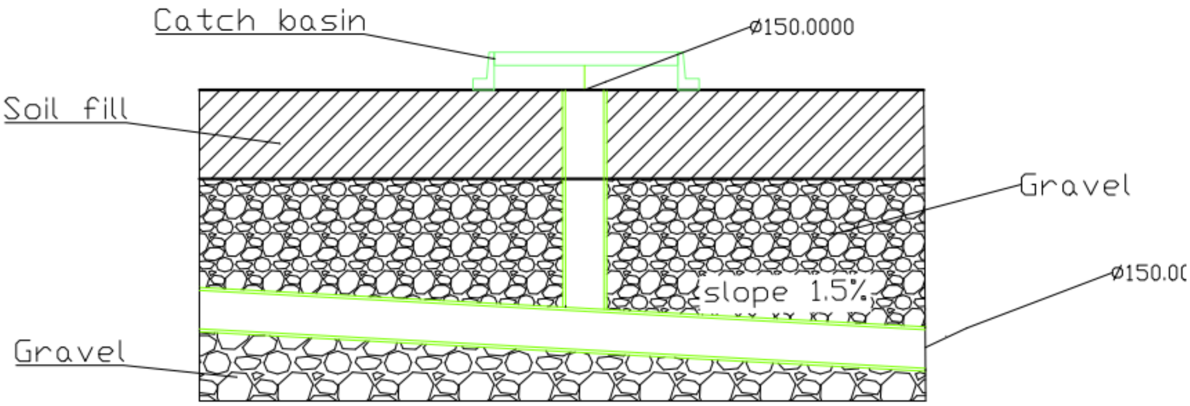


Figure 5.21. Cross section along the pipeline with a slope (not to scale)

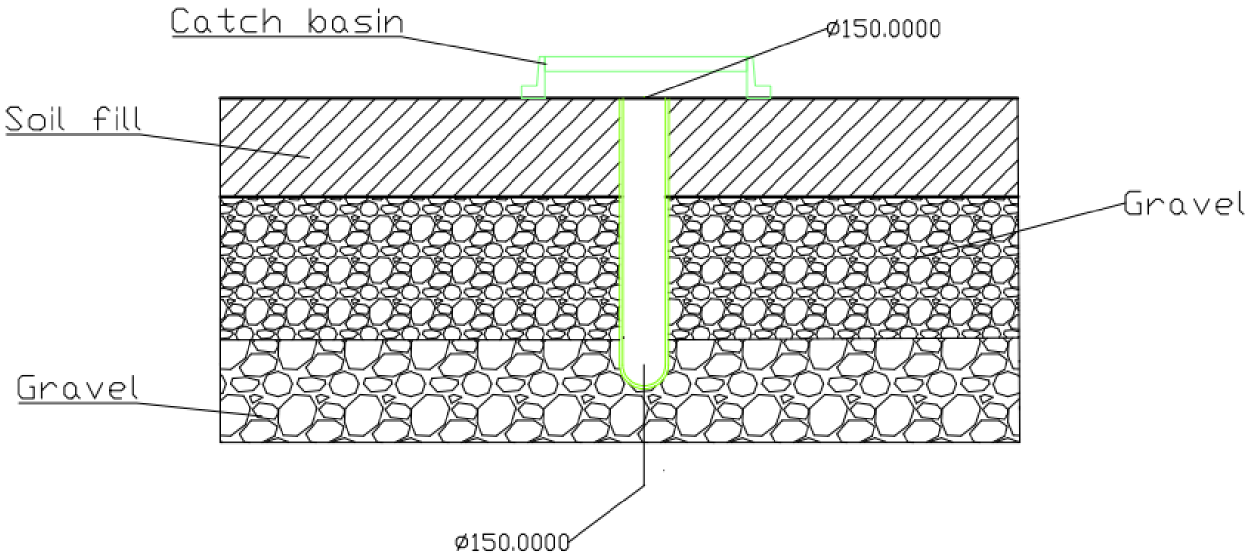


Figure 5.22. Cross section across the pipeline

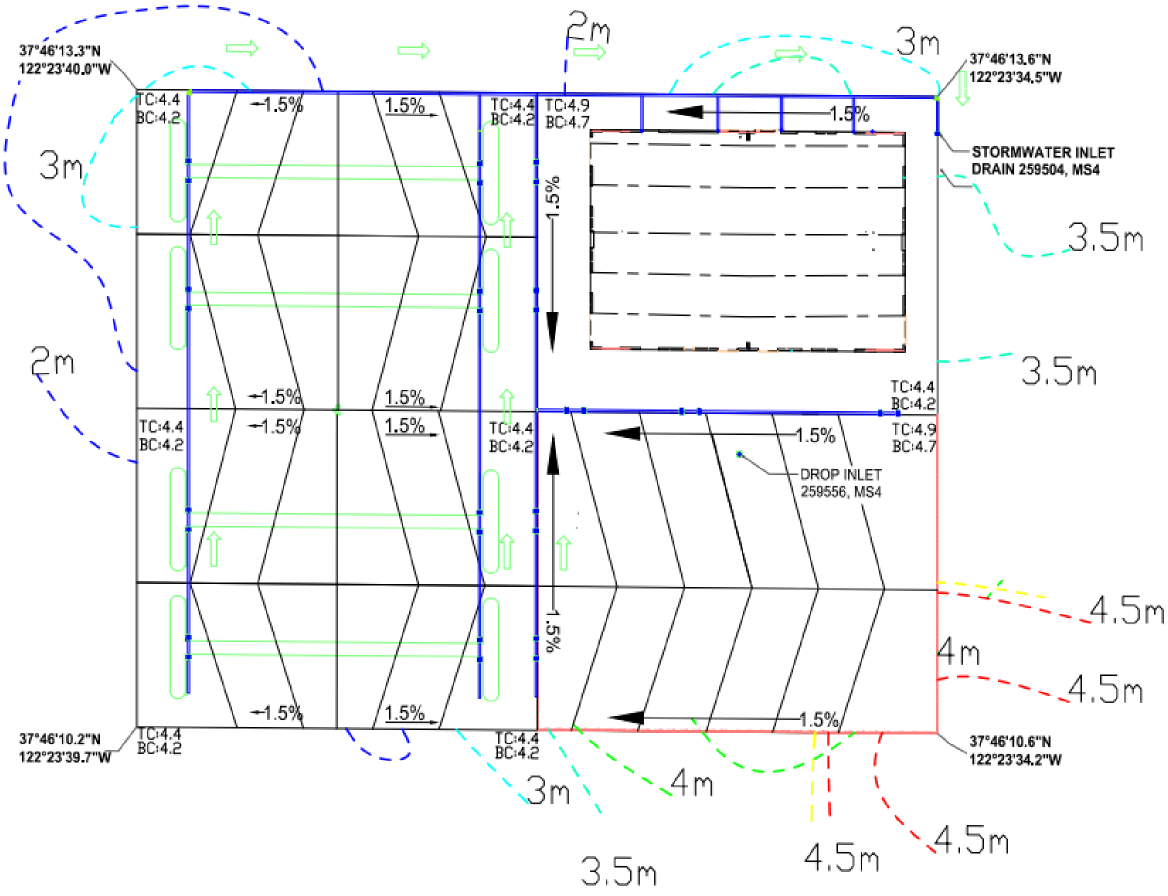


Figure 5.23. Final design of the system

Overall, all the feedback was considered in the Figure 5.23 above and you can see a fully complete final design, better resolution picture is available in technical drawings submitted. It is advised to look there so that all the catch basins are visible, but in general the terrain, pipeline system, slopes, coordinates, and curbs are all seen in the figure.

In conclusion, we believe that the presented work is decent enough to be considered an engineering work, with all its details, calculations and considerations. So we think that the original purpose was achieved by both the group and the professor.

6. Construction management

6.1 Project charter

PROJECT NAME	PROJECT MANAGER	PROJECT SPONSOR
12-Story High-Rise Residential Building	Arsen Yelubayev	Nazarbayev University

EMAIL	PHONE	ORGANIZATIONAL UNIT(S)
arsen@shakehappens.co	777-777-3377	Full Scope Construction Engineering

PROJECT OVERVIEW

PROJECT	This project is aimed to build 12-story joint residential and commercial property to accommodate more people through the governmental program.
PURPOSE OF PROJECT	To construct a 12-story mixed-use building in San Francisco, addressing the city's critical housing shortage while promoting economic vitality. It will provide 110 affordable housing units and ground-floor commercial space.
BUSINESS CASE	San Francisco faces a housing crisis with an 82,000 affordable unit shortfall. This project delivers 110 affordable units and commercial space, reducing per-unit costs by 10%, generating $2.2 million annual tax revenue, and creating 250 jobs.
METRICS	Construct 110 residential units and 10,451 sq ft commercial space in 28 months. Achieve 98% residential and 85% commercial occupancy within 6 months. Obtain LEED Platinum certification. Maintain costs within 7% of $48 million budget. Reduce energy use by 30% versus standard buildings. Increase local foot traffic by 20%.

EXPECTED DELIVERABLES	12-story mixed-use building featuring 110 residential units, ground-floor commercial space, playground, park and parking facility. Incorporated safe seismic protection, energy-efficient systems, community areas, and all accessibility standards.

PROJECT SCOPE	1. 12-story mixed-use building with 110 residential units and 10,451 sq ft commercial space 2. Parking area, playground, and park area 3. LEED Platinum certification and seismic protection systems 4. Compliance with local codes, including permits and certificates 5. Site management from start to finish 6. Building management plan and marketing strategies

KEY MILESTONE	**START**	**FINISH**
Planning, Design, and Permitting	24/08/2024	24/11/2024
Site Preparation and Foundation	26/11/2024	10/03/2025
Building Construction	15/03/2025	20/05/2026
Interior Finishes and Systems Installation	22/05/2026	25/11/2026
Landscaping and Exterior Works	30/07/2026	30/10/2026
Final Inspections, Certifications, and Handover	01/11/2026	01/01/2027

PROJECT TEAM	Aliyar Kadyrov - Structural Engineer Aldiyar Sagidolla - Structural Engineer Ramir Gumarov - Structural Engineer Arsen Yelubayev - Construction Management	Zhaniya Sagyndykova - Geotechnical Engineer Ayaulym Baktiyarkyzy - Geotechnical Engineer Bek Yergeldinov - Environmental Engineer

6.2 Feasibility study

Before the actual construction began on the site for this 12-story high-rise residential building, a comprehensive feasibility study had been conducted. This includes assessing the viability of the project with respect to various parameters; whether the design is compatible with location needs, sustainability, and financial goals.

*Site Analysis:*The site chosen is situated in the Mission Bay area of San Francisco, which is known for ease of access and state-of-the-art infrastructure. Proximity to parks, public transportation lines, and essential services makes this location especially suitable both for residential and commercial purposes. The site allows for sufficient area development activity, storage of materials, and construction of parking and recreation zones. Furthermore, the even terrain decreases difficulties with grading and foundation work, thus allowing for easier construction.

Design and Development Plan: The building's design complies with the San Francisco and California Building Code on the LEED program for energy efficiency and sustainability. Features such as roof gardens, solar panels, and high-performance insulation lead to long-term environmental and cost advantages. The building's design and use of shading devices have maximized natural lighting while minimized energy usage. Accessibility and usability are given equal importance by the development plan, as appropriate in the case of emergency exits, proper parking, and business areas.

Environmental Impact Assessment: Consistent with the California Environmental Quality Act, the environmental consequences of construction have been carefully evaluated. Prevention of associated risks, including stormwater runoff urban heat island (UHI), has been incorporated into the design. The application of eco-friendly and non-toxic materials will reduce the ecological impact of construction, and in particular, waste management systems will provide a sustainable way of managing waste during the whole construction and operational life cycle.

Legal Analysis: The project conforms to all license requirements and construction codes, thereby streamlining approval procedures. Permits for zoning, environmental compliance and safety standards have been received. The construction crew further takes into account the rights and safety of workers by complying with the labor laws and safety regulations of the state of California to provide a safe working environment.

Economic Impact Analysis: This study is anticipated to deliver a lot of economic value through hiring construction workers, engineers, and technical personnel. Residential units are designed to deliver superior housing for over 110 families, with commercial facilities providing further profitability (Projected apartment prices for sale vary between $18,450 and $67,500 depending upon unit size and proposed commercial rental rates are $780 per square foot (annual) of space. The estimated payback period of the project is in the first decade of operation.

Project Schedule and Timeline: A comprehensive work plan was created to provide timeliness of the project and was taken into account snow, holidays, and any other possible schedule interruptions. Scheduled to begin August 2024, the project is clearly defined in terms of key milestones for each stage from discovery work to completion. The estimated completion is set for the start of 2027.

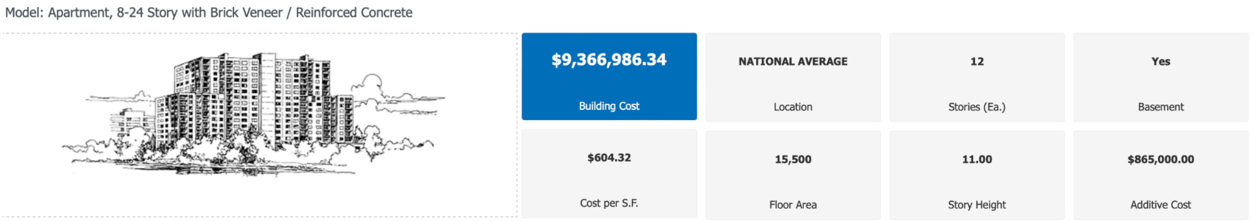
Overall the feasibility study confirms that the 12-storey residential scheme at Mission Bay is practical. Because it is thoughtful, strategically located, and has met the environmental and regulatory requirements it needs to be successful in the division and delivery of high-quality housing and retail-related opportunities, it has everything it needs to succeed.

6.3 Cost/Benefit analysis

6.3.1 RSMeans calculation

Estimation of costs in the construction projects has been one of the primary concerns of project management. It normally happens that there would be a cost estimate which is not accurate during the project building and this leads to financial imbalance and wastage of project resources. In order to meet such requirements, the RSMeans (RSM) software has been made an important tool in construction projects, which gives professionals cost details concerning various regions. This integrates the labor, materials, and equipment data to develop reliable budgets and productivity benchmarks (Gordian, 2023).

According to the Gordian Group, RSMeans provides contractors, architects, and engineers sufficient information on how the construction costs and productivity rates from different geographical areas vary with time. This cost data allows better decision-making based on such geographic and market variance, enabling the interested parties to make their budget more valid against what is real (Gordian, 2023). Also, this software can give a bottom-line analysis of cost-benefit savings with understanding profitability and overhead rates, which would be very important for large residential and commercial developments (1Estimate LLC, 2024).



Model: Apartment, 8-24 Story with Brick Veneer / Reinforced Concrete

$9,366,986.34
Building Cost

NATIONAL AVERAGE
Location

12
Stories (Ea.)

Yes
Basement

$604.32
Cost per S.F.

15,500
Floor Area

11.00
Story Height

$865,000.00
Additive Cost

Figure 6.1. RSMeans output of cost estimation

The RSMeans was used in this study to evaluate the cost projection of a 12-story residential building construction with a basement, where the estimated total cost is $9,366,986 at $604.32 per square foot. However, to calculate this cost RSMeans used only floor area and number of stories and other details were assumed by the software. This is why the estimated cost is not accurate and needed additional manual calculations.

Table 6.1. Cost estimation from RSMeans

		Quantity	% of Total	Cost per S.F.	Cost
A	Substructure		6.45%	$29.42	$455,946.14
A1010	Standard Foundations			$0.70	$10,839.31

	Pile caps, 6 piles, 8' - 6" x 5' - 6' x 37", 40 ton capacity, 14" column size, 458 K column	2.130		$0.25	$3,944.48
	Pile caps, 12 piles, 11' - 6" x 8 - 6" x 49", 40 ton capacity, 19" column size, 900 K column	1.600		$0.44	$6,894.83
A1020	Special Foundations			$19.11	$296,128.83
	Steel H piles, 100' long, 400K load, end bearing, 6 pile cluster	2.130		$5.34	$82,737.93
	Steel H piles, 100' long, 800K load, end bearing, 12 pile cluster	1.600		$8.01	$124,106.90
	Grade beam, 30' span, 52" deep, 14" wide, 12 KLF load	442		$5.76	$89,284.00
A1030	Slab on Grade			$0.48	$7,491.67
	Slab on grade, 4" thick, non industrial, reinforced	1291.66		$0.48	$7,491.67
A2010	Basement Excavation			$0.32	$4,908.33
	Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on site storage	1291.66		$0.32	$4,908.33
A2020	Basement Walls			$8.81	$136,578.00
	Foundation wall, CIP, 12' wall height, pumped, .591 CY/LF, 28.79 PLF, 16" thick	442		$8.81	$136,578.00
B	Shell		46.54%	$212.27	$3,290,201.46
B1010	Floor Construction			$55.50	$860,192.49
	Cast-in-place concrete column, 24" square, tied, 900K load, 12' story height, 567 lbs/LF, 4000PS1	812.38		$10.22	$158,414.92

	Cast-in-place concrete column, 12", square, tied, minimum reinforcing, 150K load, 10'-14' story height, 135 lbs/LF, 4000PS1	1401.14		$6.29	$97,519.34
	Cast-in-place concrete column, 16", square, tied, minimum reinforcing, 300K load, 10'-14' story height, 240 lbs/LF, 4000PS1	1401.14		$8.63	$133,808.87
	Cast-in-place concrete column, 20", square, tied, minimum reinforcing, 500K load, 10'-14' story height, 375 lbs/LF, 4000PS1	1401.14		$12.34	$191,255.61

	Cast-in-place concrete beam and slab, 7.5" slab, two way, 12" column, 25'x25' bay, 40 PSF superimposed load, 149 PSF total load	14208.33		$16.77	$260,012.50
	Flat slab, concrete, with drop panels, 6" slab/2.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 PSF total load	1291.66		$1.24	$19,181.25
B1020	Roof Construction			$1.38	$21,441.67
B1020	Roof Construction			$1.38	$21,441.67
	Roof, concrete, beam and slab, 25')(25' bay, 40 PSF superimposed load, 20" deep beam, 9" slab, 152 PSF	1291.66		$1.38	$21,441.67
	total load				
B2010	Exterior Walls			$124.07	$1,923,018.24

	Brick wall, composite double Wythe, standard face/CMU back-up, 8" thick, perlite core fill, 3" XPS	46675.2		$124.07	$1,923,018.24
B2020	Exterior Windows			$26.55	$411,519.68
	Windows, aluminum, sliding, standard glass, 5' x 3'	777.92		$26.55	$411,519.68
B2030	Exterior Doors			$2.97	$46,058.52
	Door, aluminum & glass, without transom, wide stile, hardware, 3'-0" x 7'-0" opening	0.42		$0.11	$1,699.66
	Door, aluminum & glass, without transom, non-standard, double door, hardware, 6'-0" x 7'-0" opening	0.21		$0.11	$1,758.45
	Door, aluminum & glass, sliding patio, tempered glass, premium, 6'-0" x 7'-0" opening	13.14		$2.75	$42,600.41
B3010	Roof Coverings			$1.80	$27,970.86
	Roofing, single ply membrane, EPDM, 60 mils, loosely laid, stone ballast	1291.66		$0.14	$2,170.00
	Insulation, rigid, roof deck, extruded polystyrene, 40 PSI compressive strength, 4" thick, R20	1291.66		$0.35	$5,437.92
	Roof edges, aluminum, duranodic, .050" thick, 6" face	442		$0.80	$12,464.40
	Flashing, aluminum, no backing sides, .019"	442		$0.16	$2,484.04
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick	442		$0.35	$5,414.50
c	Interiors		12.56%	$57.28	$887,857.05

C1010	Partitions			$24.84	$384,971.67

	Concrete block (CMU) partition, light weight, hollow, 6" thick, no finish	4133.33		$2.77	$42,986.67
	Metal partition, 5/8"fire rated gypsum board face, 1/4" sound deadening gypsum board, 2-1/2" @ 24", same opposite face, no insulation	9644.44		$4.21	$65,292.89
	Furring 1 side only, steel channels, 3/4", 16" OC	8266.66		$1.38	$21,328.00
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 1/2	8266.66		$0.55	$8,514.67
	Add for the following: taping and finishing	8266.66		$0.36	$5,538.67
	1/2" fire rated gypsum board, taped & finished, painted on metal furring	46675.2		$15.57	$241,310.78
C1020	Interior Doors			$8.55	$132,454.55
	Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"	108.39		$8.55	$132,454.55
C1030	Fittings			$8.12	$125,882.05
	Cabinets, residential, base, hardwood, 1 top drawer & 1 door below x 24" W	78.88		$2.60	$40,273.17
	Cabinets, residential, wall, two doors x 48" wide	39.44		$1.76	$27,295.82
	Cabinets, residential, counter top-laminated plastic, stock, economy	1476.28		$3.76	$58,313.06
C2010	Stair Construction			$3.74	$57,916.55
	Stairs, steel, pan tread for conc in-fill, picket rail,12 risers w/ landing	4.590		$3.74	$57,916.55

C3010	Wall Finishes			$2.22	$34,485.78
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	26177.77		$1.50	$23,298.22
	Kitchen sink w/trim, countertop, PE on CI, 24" x 21", single bowl	13.14		$1.29	$19,919.64
	Laundry sink w/trim, PE on CI, black iron frame, 24" x 20", single compt	1.280		$0.15	$2,321.79
	Service sink w/trim, PE on CI, corner floor, 28" x 28", w/rim guard	1.600		$0.34	$5,251.29
	Bathroom, three fixture, 2 wall plumbing, lavatory, water closet & bathtub, stand alone	13.14		$4.90	$75,931.29
D2020	Domestic Water Distribution			$8.42	$130,496.64
	Electric water heater, commercial, 100< F rise, 50 gallon tank, 9 KW 37 GPH	13.14		$8.42	$130,496.64
D2040	Rain Water Drainage			$0.25	$3,920.43
	Roof drain, DWV PVC, 4" diam, diam, 10' high	0.53		$0.05	$777.67
	Roof drain, DWV PVC, 4" diam, for each additional foot add	80.17		$0.20	$3,142.76
D3010	Energy Supply			$8.24	$127,720.00
	Apartment building heating system, fin tube radiation, forced hot water, 30,000 SF area,300,OOO CF vol	15500		$8.24	$127,720.00
D3030	Cooling Generating Systems			$9.49	$147,095.00

	Packaged chiller, air cooled, with fan coil unit, medical centers, 40,000 SF, 93.33 ton	15500		$9.49	$147,095.00
D4010	Sprinklers			$3.02	$46,856.98
	Wet pipe sprinkler systems, steel, light hazard, 1 floor, 10,000 SF	1023		$0.25	$3,877.17
	Wet pipe sprinkler systems, steel, light hazard, each additional floor, 10,000 SF	14461.5		$2.60	$40,347.59
	Standard High Rise Accessory Package 16 story	0.10		$0.17	$2,632.22
D4020	Standpipes			$17.78	$275,550.00
	Wet standpipe risers, class 111, steel, black, sch 40, 6" diam pipe, 1 floor	15		$15.53	$240,750.00
	Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM	1		$2.00	$31,000.00
	Fire pump, electric, for jockey pump system, add	1		$0.25	$3,800.00
D5010	Electrical Service/Distribution			$18.74	$290,450.00
	Underground service installation, includes excavation, backfill, and compaction, 100' length, 4' depth, 3 phase, 4 wire, 277/480 volts, 2000 A	2		$6.70	$103,800.00
	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A	200		$6.34	$98,200.00
	Switchgear installation, incl switchboard, panels & circuit breaker, 120/208 V, 3 phase, 2000 A	2		$5.71	$88,450.00
D5020	Lighting and Branch Wiring			$9.15	$141,818.00

	Receptacles incl plate, box, conduit, wire, 10 per 1000 SF, 1.2 W per SF, with transformer	15500		$3.81	$59,055.00
	Wall switches, 2.5 per 1000 SF	15500		$0.59	$9,145.00
	Miscellaneous power, 2 watts	15500		$0.56	$8,680.00
	Central air conditioning power, 3 watts	15500		$0.62	$9,610.00
	Motor installation, three phase, 460 V, 15 HP motor size	4		$0.68	$10,480.00
	Motor feeder systems, three phase, feed to 200 V 5 HIP, 230 V 7.5 HP, 460 V 15 HIP, 575 V 20 HP	400		$0.29	$4,548.00
	Incandescent fixtures recess mounted, type A, 1 watt per SF, 8 FC, 6 fixtures per 1000 SF	15500		$2.60	$40,300.00
D5030	Communications and Security			$4.01	$62,122.60
	Communication and alarm systems, fire detection, addressable, 100 detectors, includes outlets, boxes, conduit and wire	0.19		$0.97	$15,085.24
	Fire alarm command center, addressable with voice, excl. wire & conduit	1		$0.80	$12,425.00
	Communication and alarm systems, includes outlets, boxes, conduit and wire, intercom systems, 100 stations	0.14		$1.26	$19,481.90
	Communication and alarm systems, includes outlets, boxes, conduit and	0.14		$0.39	$6,062.96

	wire, master TV antenna systems, 30 outlets				
	Internet wiring, 2 data/voice outlets per 1000 S.F.	13.95		$0.58	$9,067.50
E	Equipment & Furnishings		12.59%	$57.42	$890,034.32
E1090	Other Equipment			$57.42	$890,034.32
E1090 D10101 503800	2.00-Traction gearless elevators, passenger, 5000 1b, 10 floors, 200 FPM	2.00		$55.81	$865,000.00
	Architectural equipment, appliances, range, 30" free standing, 1 oven, gas, average	13.14		$1.03	$16,027.75
	Architectural equipment, appliances, dish washer, built-in, 2 cycles, economy	13.14		$0.58	$9,006.57
F	Special Construction		0%		
G	Building Sitework		0%		
	SubTotal		100%	$456.09	$7,069,423.65
	Contractor Fees (GC,Overhead,Profit)		25.0%	$114.02	$1,767,355.91
	Architectural Fees		6.0%	$34.21	$530,206.77
	User Fees		0.0%	$0.00	$0.00
	Total Building Cost			$604.32	$9,366,986.34

6.3.2 Manual calculation of construction

Calculating the volume of concrete for whole building (The dimensions were taken from the Revit):

- **Number of columns per floor**: 54
- **Dimensions and heights** of columns vary between floors. Also, the columns are square shaped so all sides are equal to each other.

Table 6.2. Concrete volume calculation for columns

Floors	Square column width, m	Height, m	Quantity	Volume, m^3
0-1	0.75	3.5	108	212.625
2-3	0.7	3.3	108	174.636
4-5	0.65	3.3	108	150.579
6-7	0.6	3.3	108	128.304
8	0.55	3.3	54	53.9045
9	0.5	3.3	54	44.55
10	0.45	3.3	54	36.084
11	0.4	3.3	54	28.512
12	0.35	3.3	54	21.83
Total	851.024 m^3			

Then, the concrete volumes for other structural parts of the building were calculated.

Table 6.3. Concrete volume calculation for whole building

Concrete structure	**Volume,** m^3
Columns	851.024
Beams	1305.72
Slabs	3744.12
Basement walls	221.676
Staircases	33.813
Elevator shafts	207.84
Foundation	1221.458
Total Concrete	7585.531

After identifying the needed units of materials, the manual calculation of cost was calculated regarding the average prices in San Francisco (The prices taken from the RSMeans).

Table 6.4. Manual cost estimation of the building construction

Item	**Unit**	**Quantity**	**Unit cost, $**	**Total cost, $**
Concrete (C40)	m^3	7585.531	200	1,517,106
Reinforced steel	kg	780,263.4	2.5	1,950,657
Site work (excavation, etc)	m^2	1440	500	720,000

Formwork	m^2	43200	80	3,456,000
Elevators		2	200,000	400,000
MEP, fire	m^2	17280	190	3,283,200
Flooring	m^2	17280	60	1,036,800
Wall, Ceiling	m^2	58,752	40	2,350,080
Total cost	14,713,843 $			

As in the manual calculations more detailed data about the structural part of the building the total cost from there is chosen for further calculations.

Table 6.5. The total cost for the project

COSTS

COST TYPE	**RATE**	**Sq Ft**	**AMOUNT**
Land Acquisition Cost	$210.00	115,368	$24,227,280.00
Building Construction	$1272.60	15,500	$20,725,254.00
Architectural Fee	$39.00	15,500	$876,340.00
Insurance	$400,000.00		$400,000.00
		TOTAL COSTS	$46,228,874.00

Next, we calculated revenue. A real estate platform called “Redfin” was used to get a valid median sale price per square foot in Mission Bay which is $928 (Redfin, 2024). However, our building is in just roughly finished condition, this is why the cost per square ft is 450$. The price of commercial area renting was taken from the report of Commercial Cafe (2024) as $70.

Table 6.6. The total revenue for the first year

REVENUE

COST TYPE	RATE	Sq Ft	AMOUNT
Residential Area	$450.00	134,979	$60,740,550.00
Commercial Area Renting	$70.00	10,451	$731,570.00
		TOTAL REVENUE (First Year)	$61,472,120.00

Finally, after obtaining total costs and revenue, net profit (for first year) was calculated:

Table 6.7. Net profit for the first year

NET PROFIT

COST TYPE	AMOUNT
Net Profit	$21,408,337.00

6.4 Work breakdown structure

Effective project scope management will ensure the completion of a project within the required time and budget. Work Breakdown Structure is a basic tool for project scope management, as it involves hierarchical decomposition of the total work scope to be accomplished by the project team. The resulting decomposition will clearly highlight the deliverables and work necessary to make a deliverable, while enhancing mechanisms for planning and control (Project Management Institute, 2006).

To create the WBS for this project, the PMBOK Guide's guidelines were followed which helped to ensure a comprehensive and structured approach. The project was divided into key phases as planning, execution, construction, monitoring and controlling, and finishing. Each phase was further decomposed into specific deliverables and associated tasks with activities, facilitating detailed planning and effective monitoring throughout the project lifecycle (Project Management Institute, 2021).

The Primavera P6 Professional was used to visualize and construct this WBS. This software helped to create clear, detailed diagrams that foster great communication among stakeholders and serve as a point of reference in tracking changes and progress. Using such tools ensures the team maintains a constant view of the scope and structure of the project, hence its overall success.

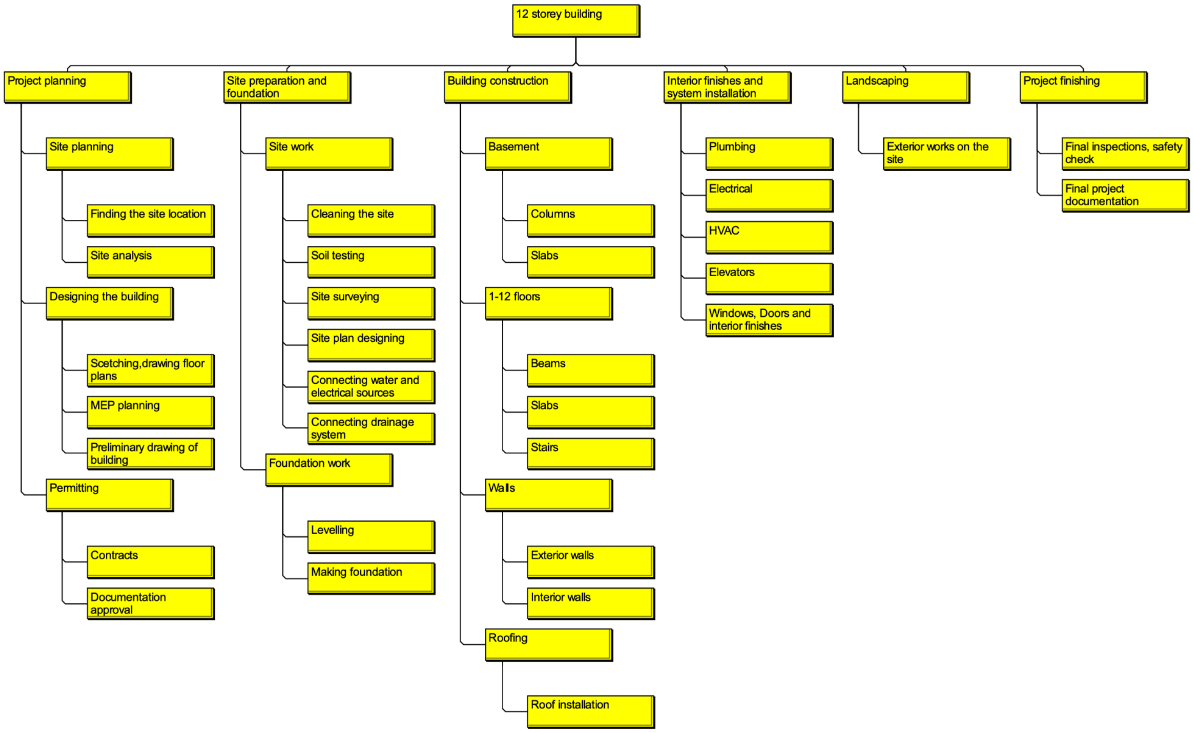


Figure 6.2. Work breakdown structure of project

6.5 Scheduling

Effective scheduling in construction project management is the basis for completing tasks within a certain timeframe and using resources efficiently. A well-structured schedule helps to act properly and guide the team through the different phases of the project. It also facilitates recognizing potential bottlenecks and offers room for taking proactive measures to avoid delays in order to enhance the performance of the project as a whole.

The Primavera P6 Professional software was used in this project development to create a detailed schedule. This tool allowed creating a Gantt chart that is really handy in showing the activities in sequence and the estimated duration. The tool allowed incorporating non-working days, official holidays, and other region-specific constraints for developing a realistic schedule. With this, the critical path was determined and is representative of those tasks that drive the project completion date. The total project duration was calculated as 615 days, and the critical tasks were distinctly marked on the Gantt chart for clear visibility.

Utilizing such scheduling tools not only streamlines the planning process but also enhances communication among stakeholders by providing a clear visual representation of the project's progress. This approach aligns with best practices in construction management, where detailed scheduling is essential for coordinating complex activities and managing resources effectively (ProjectManager, 2024).

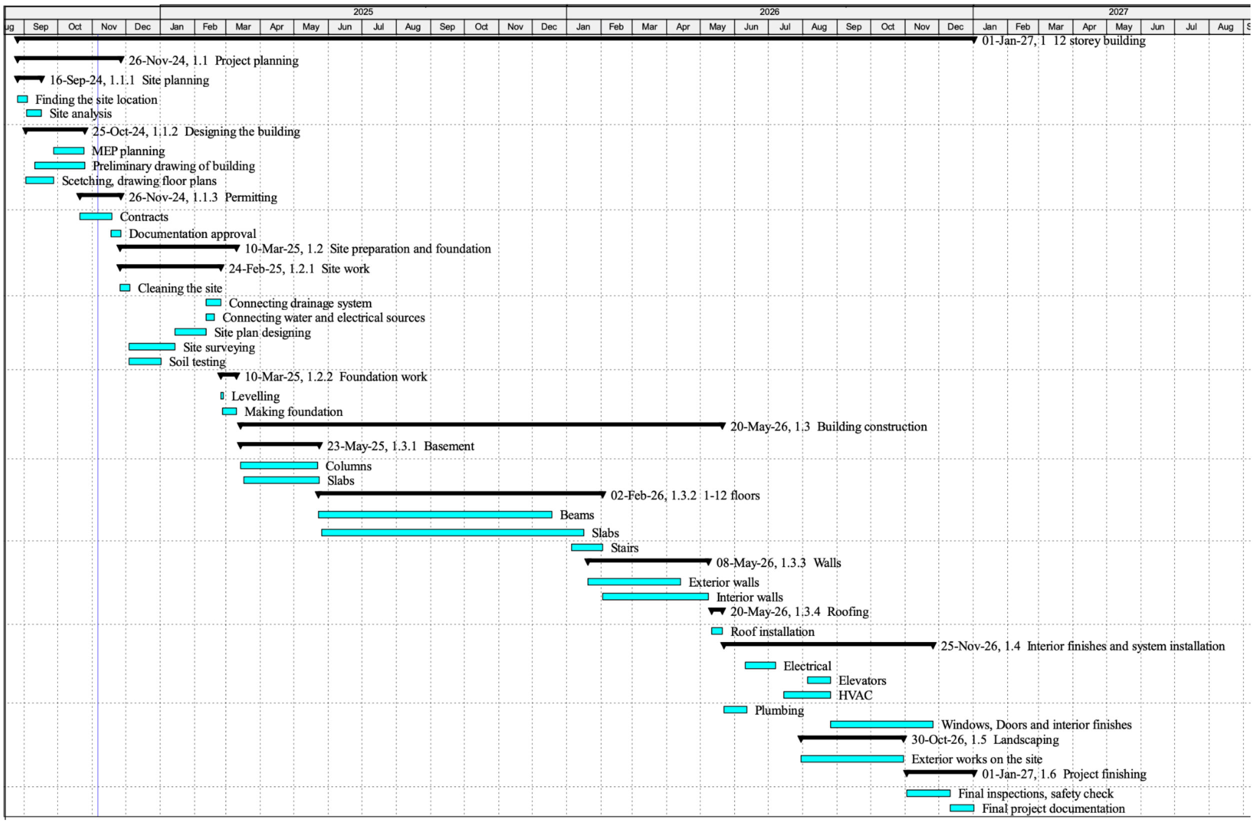
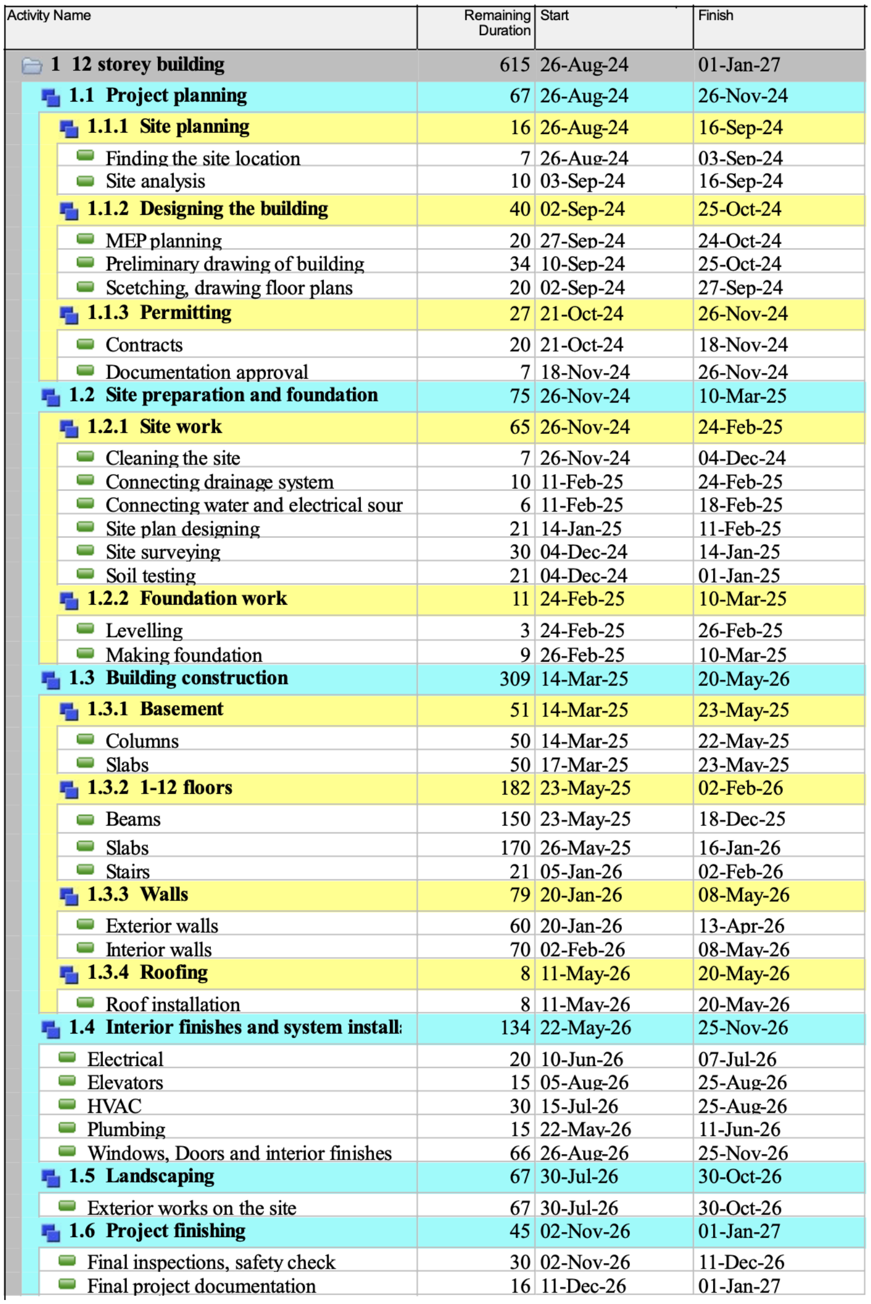


Figure 6.3. Gantt Chart of the project (Primavera P6)



Activity Name	Remaining Duration	Start	Finish
1 12 storey building	615	26-Aug-24	01-Jan-27
1.1 Project planning	67	26-Aug-24	26-Nov-24
1.1.1 Site planning	16	26-Aug-24	16-Sep-24
Finding the site location	7	26-Aug-24	03-Sep-24
Site analysis	10	03-Sep-24	16-Sep-24
1.1.2 Designing the building	40	02-Sep-24	25-Oct-24
MEP planning	20	27-Sep-24	24-Oct-24
Preliminary drawing of building	34	10-Sep-24	25-Oct-24
Scetching, drawing floor plans	20	02-Sep-24	27-Sep-24
1.1.3 Permitting	27	21-Oct-24	26-Nov-24
Contracts	20	21-Oct-24	18-Nov-24
Documentation approval	7	18-Nov-24	26-Nov-24
1.2 Site preparation and foundation	75	26-Nov-24	10-Mar-25
1.2.1 Site work	65	26-Nov-24	24-Feb-25
Cleaning the site	7	26-Nov-24	04-Dec-24
Connecting drainage system	10	11-Feb-25	24-Feb-25
Connecting water and electrical sour	6	11-Feb-25	18-Feb-25
Site plan designing	21	14-Jan-25	11-Feb-25
Site surveying	30	04-Dec-24	14-Jan-25
Soil testing	21	04-Dec-24	01-Jan-25
1.2.2 Foundation work	11	24-Feb-25	10-Mar-25
Levelling	3	24-Feb-25	26-Feb-25
Making foundation	9	26-Feb-25	10-Mar-25
1.3 Building construction	309	14-Mar-25	20-May-26
1.3.1 Basement	51	14-Mar-25	23-May-25
Columns	50	14-Mar-25	22-May-25
Slabs	50	17-Mar-25	23-May-25
1.3.2 1-12 floors	182	23-May-25	02-Feb-26
Beams	150	23-May-25	18-Dec-25
Slabs	170	26-May-25	16-Jan-26
Stairs	21	05-Jan-26	02-Feb-26
1.3.3 Walls	79	20-Jan-26	08-May-26
Exterior walls	60	20-Jan-26	13-Apr-26
Interior walls	70	02-Feb-26	08-May-26
1.3.4 Roofing	8	11-May-26	20-May-26
Roof installation	8	11-May-26	20-May-26
1.4 Interior finishes and system install	134	22-May-26	25-Nov-26
Electrical	20	10-Jun-26	07-Jul-26
Elevators	15	05-Aug-26	25-Aug-26
HVAC	30	15-Jul-26	25-Aug-26
Plumbing	15	22-May-26	11-Jun-26
Windows, Doors and interior finishes	66	26-Aug-26	25-Nov-26
1.5 Landscaping	67	30-Jul-26	30-Oct-26
Exterior works on the site	67	30-Jul-26	30-Oct-26
1.6 Project finishing	45	02-Nov-26	01-Jan-27
Final inspections, safety check	30	02-Nov-26	11-Dec-26
Final project documentation	16	11-Dec-26	01-Jan-27

Figure 6.4. Table of the activities of the project (Primavera P6)

6.6 Risk management

With effective risk management processes, construction projects can identify, evaluate, and mitigate challenges that may affect the outcome of a project. A risk management plan embodies systematic activities such as forecasting risks, assessing the implications of such eventualities, and instituting means of minimizing their impact. In this sense, project resilience is built-in and thus contributes more towards successful project delivery (Zou, Zhang, & Wang, 2007).

In formulating the risk management plan for this project, a thorough assessment of risk has been carried out in order to identify possible threats, such as financial uncertainties, safety problems, or those caused by the environment. All the recognized risks have been assessed concerning their probability and impact, thus enabling prioritization of those requiring immediate action. Mitigation strategies like contingency planning, safety training, and adherence to environmental regulations have been formed to address the risks identified .

The table below shows the completed risk assessment for this project. Further, regular monitoring and review procedures have been installed to effectively adopt any changes that may arise within the project environment keeping the plan relevant throughout the life of the project.

Table 6.8. Risk assessment for the project

Types of Risks	Description of the Risks	Probability	Severity	Mitigation Measures
Schedule				
Sc1	Delays in acquiring city permits and approvals	Medium	High	Expedite permitting process by coordinating closely with agencies
Sc2	Labor shortages due to high demand in the local construction market	Medium	High	Engage workforce planning early and hire additional labor sources
Sc3	Scope changes requested by the client during construction	Medium	High	Establish strict change control procedures
Safety				
S1	On-site accidents due to machinery or height-related risks	Medium	High	Conduct comprehensive safety training and enforce PPE usage
S2	Non-compliance with safety codes	Medium	High	Implement frequent safety audits

S3	Fire hazards in residential and commercial units	Low	High	Install advanced fire detection and suppression systems
Geotechnical				
G1	Unexpected soil composition affecting foundation stability	Medium	Medium	Conduct thorough soil testing and ground surveys
G2	Water table fluctuations impacting foundation integrity	High	High	Design with waterproofing and drainage systems
G3	High earthquake vulnerability due to site location	High	High	Apply seismic-resistant design techniques
Labor				
L1	Labor disputes or union strikes	Medium	Medium	Ensure positive labor relations and contract negotiations
L2	Shortage of skilled construction workers	Medium	Medium	Partner with skilled labor agencies
L3	Decreased productivity due to prolonged working hours	Low	Medium	Manage shifts to avoid fatigue
Political & External				
P1	New government regulations affecting project requirements	Medium	High	Monitor regulatory updates and adjust project plans accordingly
P2	Supply chain delays due to international sourcing	Medium	High	Prioritize local suppliers and increase inventory reserves
Change Management				
C1	Client-initiated changes in building layout	Medium	Medium	Clarify project scope at onset and negotiate scope changes

C2	Design adjustments due to unexpected site conditions	Medium	Medium	Increase design flexibility to allow for modifications
C3	Lack of efficient communication in managing changes	Medium	Medium	Implement clear change management communication channels
Financial				
F1	Budget overruns due to unplanned expenses	Medium	Medium	Conduct regular budget reviews and establish a contingency fund
F2	Fluctuations in material and labor costs	Medium	Medium	Secure fixed-price contracts and consider alternative suppliers
F3	Delays in securing financing	Medium	Medium	Maintain a strong relationship with financial partners
Environmental				
E1	Adverse weather conditions causing delays	Medium	Medium	Plan buffer time for weather-related delays
E2	Impact of building activities on surrounding environment	Low	High	Use eco-friendly materials and implement dust control measures
E3	Unforeseen environmental hazards on-site	Low	High	Conduct an environmental impact assessment before breaking ground
Design				
D1	Inaccurate architectural and engineering designs	Medium	High	Enhance collaboration between architects, engineers, and clients
D2	Design changes during construction	Medium	High	Implement interdisciplinary design reviews

D3	Non-compliance with local building codes	Low	Medium	Conduct regular compliance checks during design phases

After finishing the risk assessment the identified risks were then assessed and prioritized against their likelihood and potential impact using a risk management matrix. This matrix depicts visually the potential risks that may occur, thereby showing better understanding and management for each category of risk.

The following figure illustrates all the identified risks, plotted against probability and severity, and binned into three categories of low, medium, and high. Most risks are related to medium impacts, which mean a moderate likelihood and manageable severity. However, two risks in the geotechnical category were assessed as high impact, namely G3 and G2, relating to seismic risk due to the building sitting in a seismic zone. Because of the potentials, these risks have to cause significant disruptions, immediate mitigation strategies that are focused become tantamount.

Severity

Probability

	Low	Medium	High
High			G3, G2
Medium		G1, L1, L2, C1, C2, C3, F1, F2, F3, E1	Sc1, Sc2, Sc3, S1, S2, P1, P2, D1, D2
Low		L3, D3	S3, E2, E3

Figure 6.5. Risk control matrix

6.7 Quality management

Construction of the 12-story high-rise residential building in San Francisco guarantees that every process satisfies the necessary standards, performance criteria, and safety regulations. Quality management's main objective is to ensure compliance with pertinent building codes and rules (ISO 9001, ASCE, IBC), so preventing defects, improving efficiency, and attaining customer satisfaction. Good quality control reduces hazards and project interruptions, hence increasing the dependability and sustainability of the building project. This covers extensive quality assurance planning, material quality control, on-site practices, environmental and safety standards, testing, and careful record keeping. With a Project Quality Plan (PQP) created to outline the required actions to preserve construction quality, quality assurance planning guarantees that construction processes comply with the International Building Code (IBC), ASCE 7-16, ACI 318-19, and California Green Building Standards (CALGreen). With supplier reviews carried before purchase and testing methods carried in accordance with ASTM criteria, material quality control guarantees that all materials satisfy durability and sustainability criteria. To ensure compliance with approved design plans, on-site practices include the training and certification of construction workers, frequent inspections, and the use of a quality checklist system. By means of stringent compliance with OSHA rules, the use of eco-friendly materials, and the adoption of stormwater management systems, environmental sustainability and worker safety are given top priority. Testing is done to verify LEED certifications and fire safety standards, the structural integrity of the building, and the correct operation of mechanical, electrical, and plumbing systems. Maintaining legal papers including daily site inspection reports, material and structural test results, and final quality certification depends strongly on record keeping, which also helps to track construction progress and guarantee compliance. The long-term durability and functioning of the structure will be guaranteed by the application of these thorough quality management techniques, which will see its construction meet all legislative, environmental, and safety criteria.

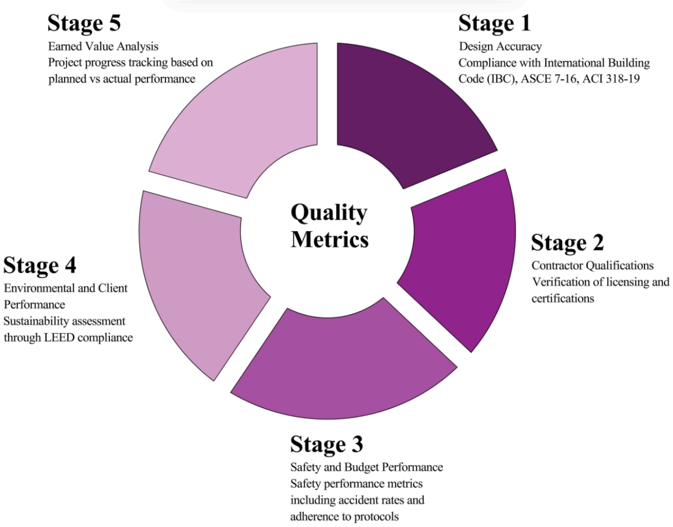


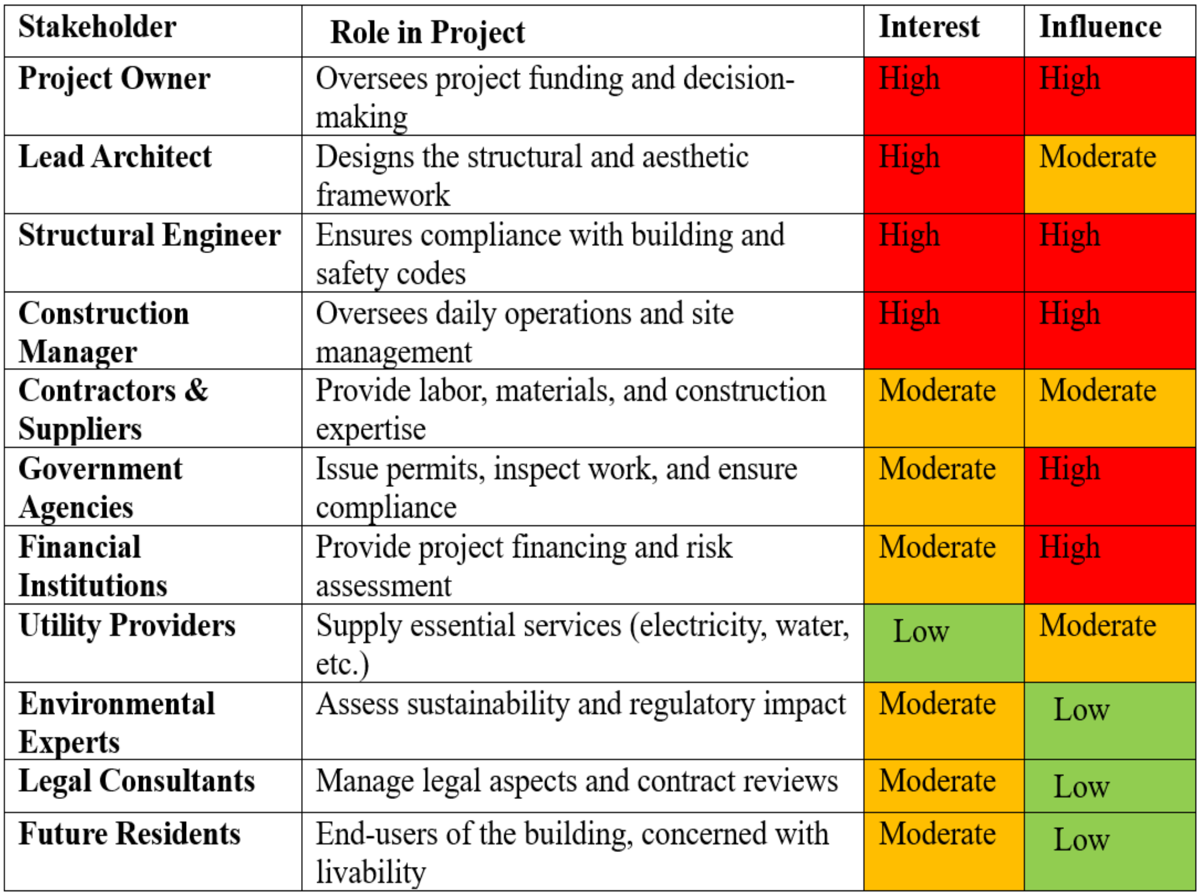
Figure 6.6. Quality metrics

6.8 Procurement planning

Stakeholders list

Understanding the many parts and impacts in a construction project starts with stakeholder analysis. The project directly or indirectly involves or affects its members, individuals or groups. Their level of interest and impact will help you group them. The major stakeholders in this building initiative and their interests are listed in the table above. Decision-making as well as compliance with regulations greatly depend on high-influence stakeholders such the project owner, head architect, and structural engineer. Additionally strong in controlling operations and legal compliance are government agencies and the construction manager. Important for project execution are moderate-influence stakeholders including contractors, finance institutions, and suppliers who offer vital materials and monetary support. Important for sustainability, legal compliance, and project success in the long run, future residents, environmental specialists, and legal counsel have moderate or low influence though. A well constructed stakeholder engagement strategy guarantees clear communication, alignment of interests, and risk management all through the project lifespan. Good decision making and a thriving, sustainable construction project depend on stakeholder dynamics.

Table 6.9. Stakeholders list



Stakeholder	Role in Project	Interest	Influence
Project Owner	Oversees project funding and decision-making	High	High
Lead Architect	Designs the structural and aesthetic framework	High	Moderate
Structural Engineer	Ensures compliance with building and safety codes	High	High
Construction Manager	Oversees daily operations and site management	High	High
Contractors & Suppliers	Provide labor, materials, and construction expertise	Moderate	Moderate
Government Agencies	Issue permits, inspect work, and ensure compliance	Moderate	High
Financial Institutions	Provide project financing and risk assessment	Moderate	High
Utility Providers	Supply essential services (electricity, water, etc.)	Low	Moderate
Environmental Experts	Assess sustainability and regulatory impact	Moderate	Low
Legal Consultants	Manage legal aspects and contract reviews	Moderate	Low
Future Residents	End-users of the building, concerned with livability	Moderate	Low

Table 6.10. Risk identification

Risk Identification	Likelihood (1-5)	Severity (1-5)	Risk Rating
Seismic Activity	3	5	15
Foundation Instability	4	4	16
Material Supply Delays	3	3	9
Stormwater Drainage Failure	2	4	8
Structural Integrity Issues	3	5	15
Worker Safety Violations	4	3	12
Non-Compliance with LEED	2	3	6
Budget Overruns	4	3	12
Electrical System Failures	3	4	12
Noise Pollution	2	2	4
Environmental Impact	3	4	12
Delays in Permit Approvals	3	3	9

A methodical risk identification process was performed to guarantee the effective execution of the 12-story tall high-rise residential construction project in San Francisco. This procedure evaluated possible hazards depending on their severity and probability of occurrence, thereby classifying each noted danger.

Among the main dangers listed in the "Red Zone" are earthquake-prone locations (rated 15), foundation instability (rated 16), and structural integrity issues (rated 15). The project's geographic position raises the possibility of earthquakes, hence requiring structural strengthening to withstand seismic pressures. Through comprehensive soil testing and prudent foundation design to reduce settlement or failure, a key hazard foundation instability will be handled. To prevent building flaws, strict adherence to safety guidelines and regular inspections will closely watch structural integrity.

Rated 9, moderate risks in the "Yellow Zone" include material supply delays; rated 8, stormwater drainage failure; rated 12, worker safety violations; rated 12 budget overruns; rated 12 electrical system failure; rated 12 environmental concerns; and rated 9 permission approval delays. Alternative sourcing and supply chain management techniques will help to handle material supply delays. Good design and sustainable planning will be applied to avoid stormwater drainage problems. Employee training, rigorous compliance to OSHA rules, and safe work practices will help to give top priority to worker safety. Expecting unexpected expenses, contingency budgeting will help to control actively monitored budget overages. Regular inspections and adherence to electrical safety rules will help to reduce electrical system failure. Sustainable design choices and LEED compliance will help to resolve environmental issues. Working with regulators and making sure building applications are submitted on time will help to avoid delays in permit approval.

Rated 6, LEED noncompliance and noise pollution (rated 4) are low-risk factors found in the "Green Zone. By guaranteeing the project satisfies all environmental criteria, the danger of noncompliance with LEED standards will be reduced. Soundproofing and coordinated building planning will help to reduce noise pollution by means of avoiding disturbance to the local population.

To properly manage these hazards, an active risk management strategy will be in place. The project seeks to avoid or reduce these risks by putting in place the requisite controls, including contingency planning and stakeholder involvement, therefore guaranteeing higher success and goal attainment.

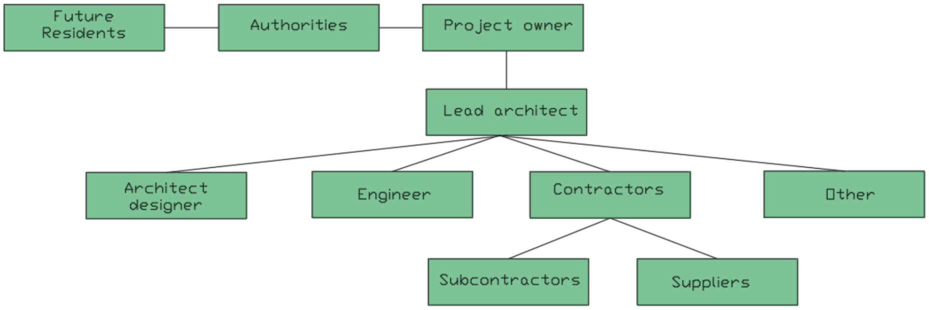


Figure 6.7. Stakeholders in Construction Procurement

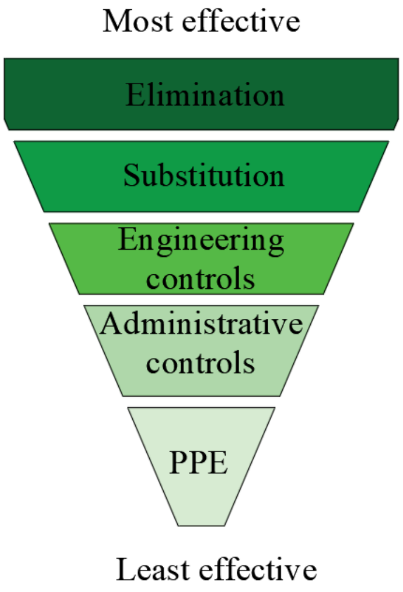


Figure 6.8. Hierarchy of Controls

6.9 Construction safety

Construction safety management is very important to both the workforce and the operational integrity of construction activities and, as such, needs to be accorded the highest level of priority by all stakeholders in a construction project. In building construction, paying attention and following safety standards and regulations, such as those issued by OSHA, is compulsory. Firstly, a group of professionals ought to come up with a comprehensive safety plan, and simultaneously, workers need to be well informed about all potential hazards, risks, and safety precautions. Secondly, there should be regular updates on the safety plan throughout the various phases of the construction. It is also necessary for workers on the site to be equipped with the personal protective equipment and instructed on the control and prevention of the potential hazards during the construction. Apart from protecting workers' safety, all the visitors to a site need to adhere to established safety regulations. Further, operations conducted at construction sites must not compromise public safety, such as the transportation of materials, storage, high noise levels, and other associated factors.

6.10 Construction site planning

Planning the construction site for the 12-story apartment building in San Francisco calls for a carefully organized design maximizing safety and efficiency. Key areas inside the building site are clearly defined in the layout, each one supporting the construction process in a particular way. The main aspect of the site is the "vehicle movement" zone, which is vital for guaranteeing that construction materials, tools, and labor flows freely to and from the site. This area is carefully located to prevent traffic and provide safe movement for all cars on site.

Several important material storage locations next to the vehicle movement zone are the "excavated ground," where soil taken out during the excavation process is kept, and the "material stockpile," which houses other construction supplies including cement, steel, and Designated for storing reinforcing materials like rebar and other parts critical for the structural integrity of the building is the "reinforcement stockpile" zone.

Another important part of the site design is the "waste material" zone where construction debris is handled. Maintaining a neat and orderly workplace which is essential for both safety and environmental compliance depends on correct treatment and disposal of waste materials.

Designated on the periphery of the site for the storage and maintenance of construction equipment and machinery is the "equipment and motor park" area, thus guaranteeing that tools and vehicles are in optimum operational condition and readily available when required.

At last, office buildings are located on the site to act as administrative hubs for the project, accommodating project managers and staff members engaged in daily construction work.

This design of the building site guarantees effective material handling, simple worker and equipment access, and obvious zone separation to foster project safety and production by means of clear separation.

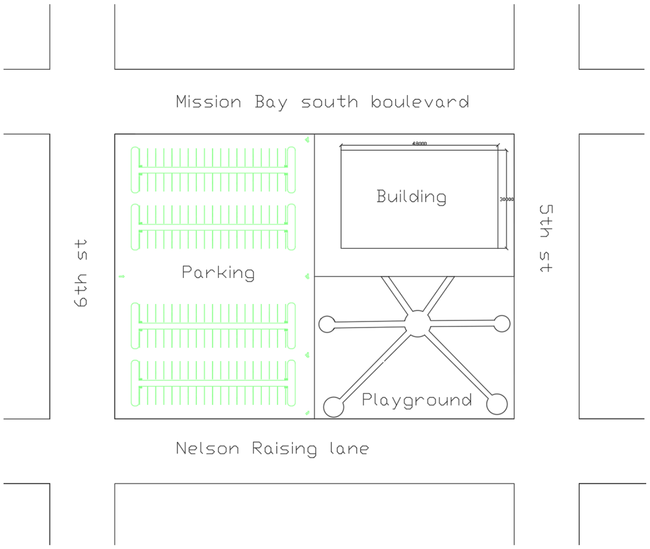


Figure 6.9. Future site planning for residents

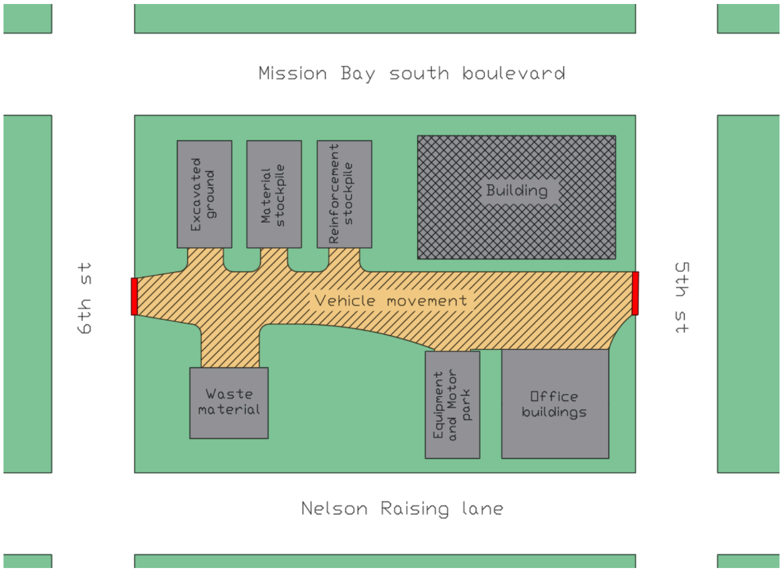


Figure 6.10. Construction site planning

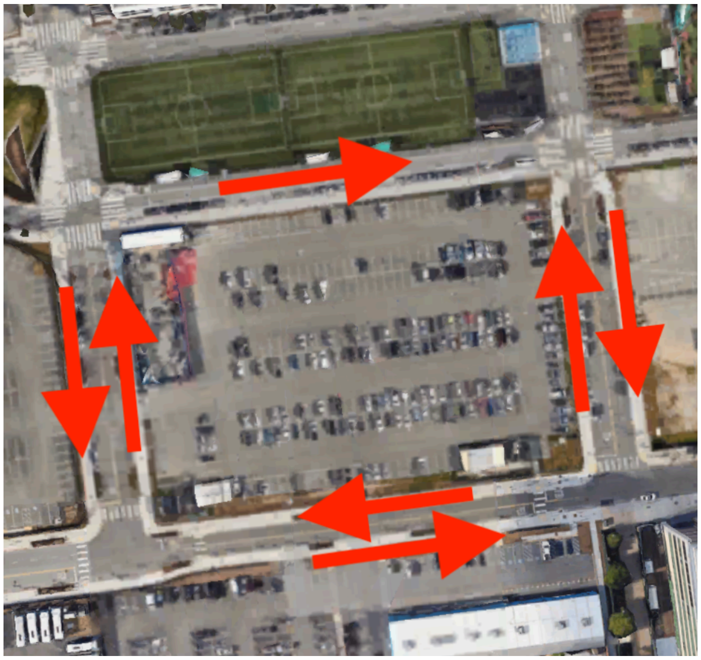


Figure 6.11. Traffic direction on the site

7. Conclusion

A high-rise residential building was designed at 1335 6th St, San Francisco, California, USA. All aspects of the construction project were put into consideration in the design, including architectural, structural, geotechnical, environmental, and construction management aspects.

The architectural design outcomes incorporated material selection, partition layouts, and structural arrangements. Emphasis was put on compliance with International Building Code, San Francisco and California building code standards and with sustainable solutions that include the green roofs, solar panels, and the stormwater collection system. USGBC, 2022. In order to make the building environmentally safe, a LEED rating system was used. Another solution was to use Life-365 software to estimate concrete serviceability cost and life-cycle costs. Also, the floor layouts and designs were optimized in order to provide maximum utility and aesthetics, accommodating a variety of residential and commercial needs.

The internal force was analyzed using SAP2000: gravity, and lateral analysis. The structure's design followed the provisions of American Concrete Institute, ACI 318-19, and American Society of Civil Engineers, ASCE, for seismic safety and making the structure resilient in case of such hazards in the high seismic zone of San Francisco. Reinforced concrete was chosen because it is durable, resists fire, and withstands marine climates; therefore, it satisfies project requirements and site specifics (Kodur et al., 2007). The entire structural design and analysis procedures were conducted following the ACI 318-19 (ACI, 2019).

Geotechnical assessments recommended a pile foundation as the best option, for which the designs included axial and lateral bearing capacity, and settlement analyses. Soil-structure interaction was modeled by software like Plaxis 3D to ensure that the design was safe and met all the requirements of geotechnical standards. The sheet pile was designed and the design parameters were confirmed using GEO5. Additionally, pile cap and single pile reinforcements were introduced and confirmed through the software.

In stormwater management design, it used sustainable drainage solutions that reduced the risk of flooding-a critical consideration given the flat urban terrain of Mission Bay. The design of a stormwater system includes the elevation profile as well as terrain of the area, and the technical drawing contains slopes and curbs.

Construction management was geared toward maintaining project scope, optimizing cost by applying RSMeans, and proposing methods that were to be used in evaluating risk. Scheduling and risk management were executed to satisfy the OSHA standards and LEED sustainability standards on safety and sustainability throughout the entire project life cycle. Site plan management, procurement planning, and safety were incorporated, construction management aspects were expanded, and the cost estimation was revised with the scheduling accordingly.

Eventually, the proposed design will be integrated with resilience, sustainability, and functionality in coming up with solutions to urban housing challenges and shortages while meeting environmental and ethical imperatives. The design also will be further validated in the future for its real-world applicability through detailed simulations and consultations with stakeholders.

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Appendix

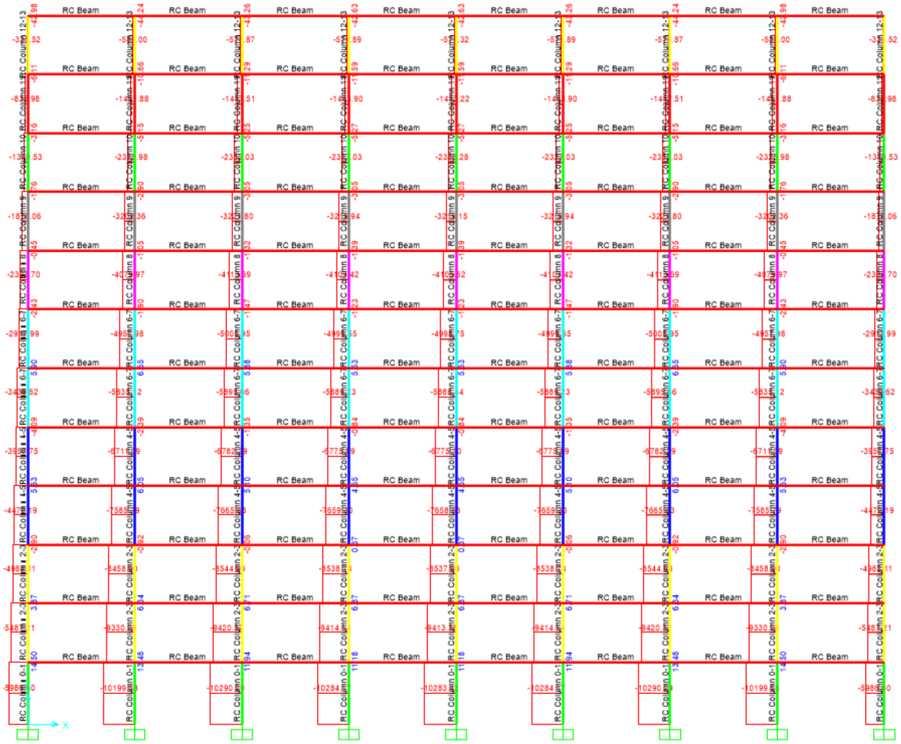


Figure 9.1 Axial Load Analysis (GLRS)

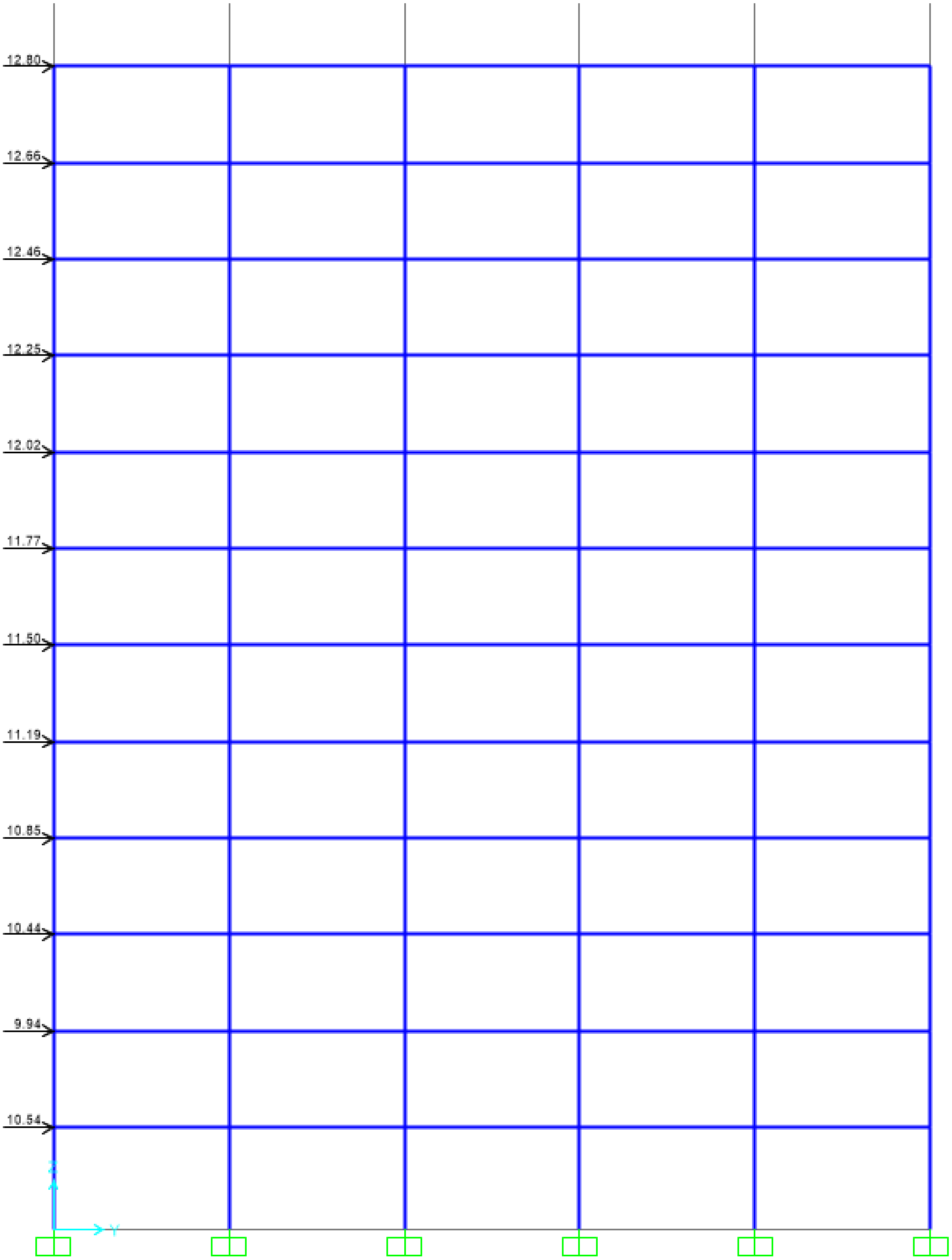


Figure 9.2 Load Assignment for Case II Wind Load (Frame C)

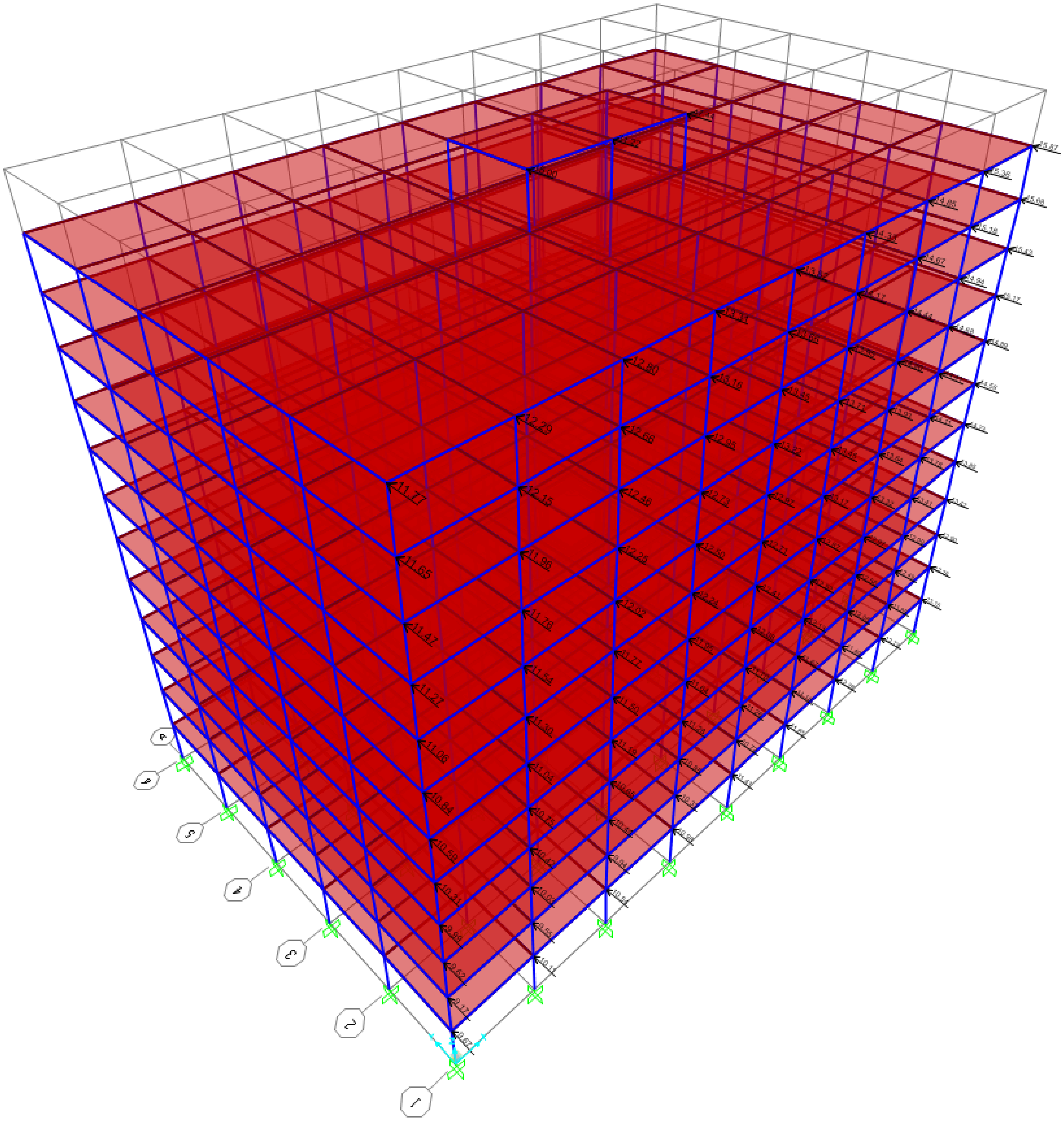


Figure 9.3 3D Load Assingment for Case II Wind Load

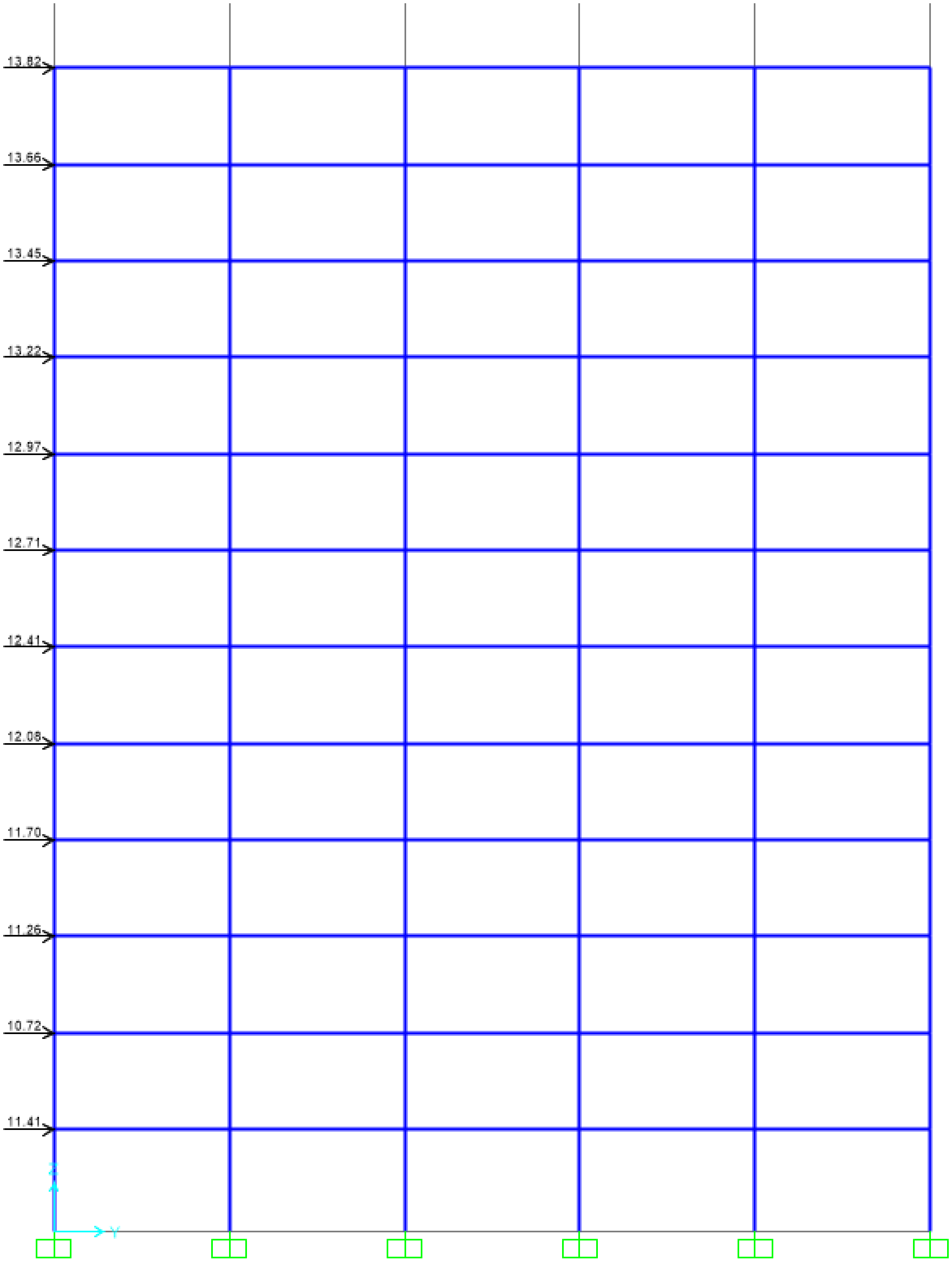


Figure 9.4 Load Assignment for Case III Wind Load (Frame C)

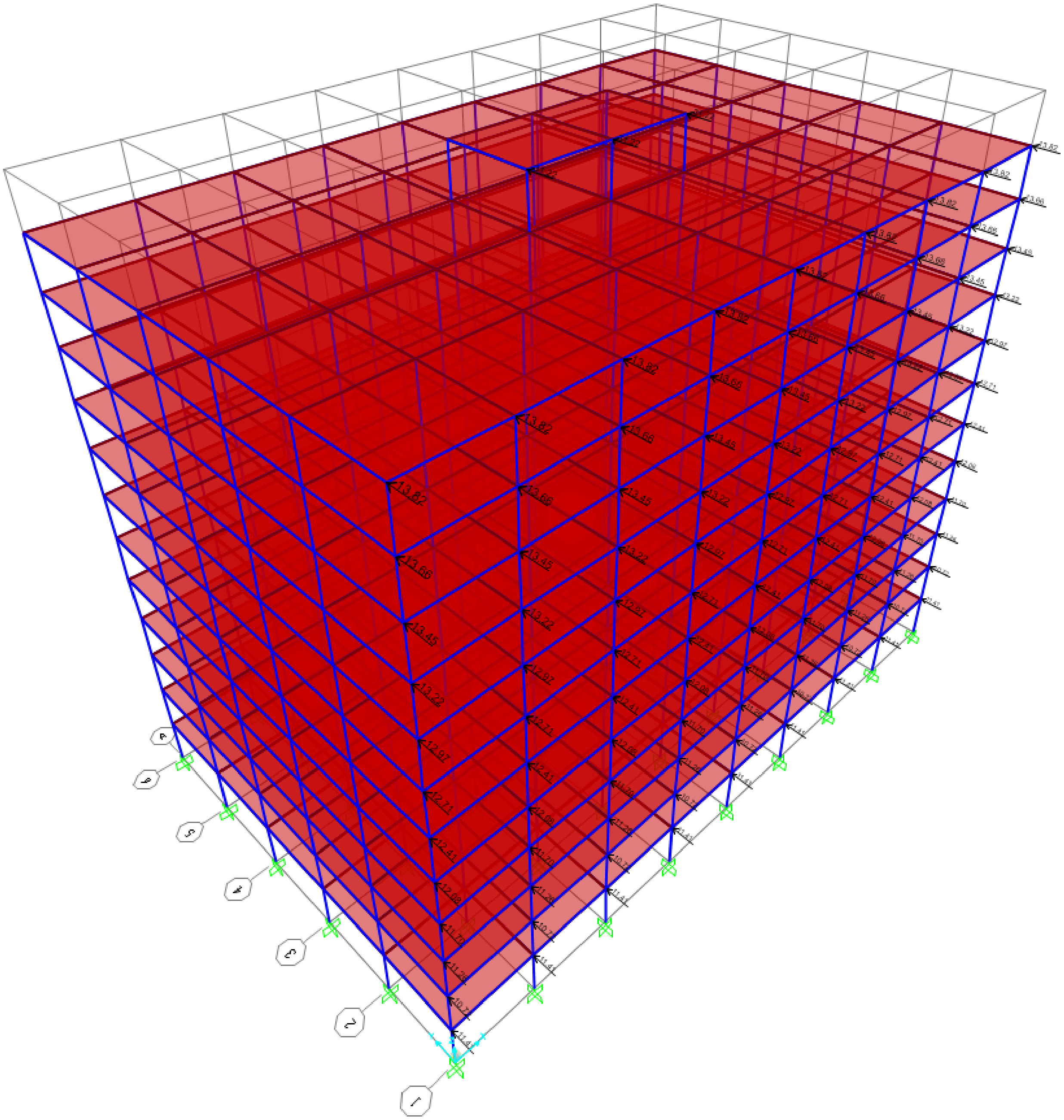


Figure 9.5 3D Load Assignment for Case III Wind Load

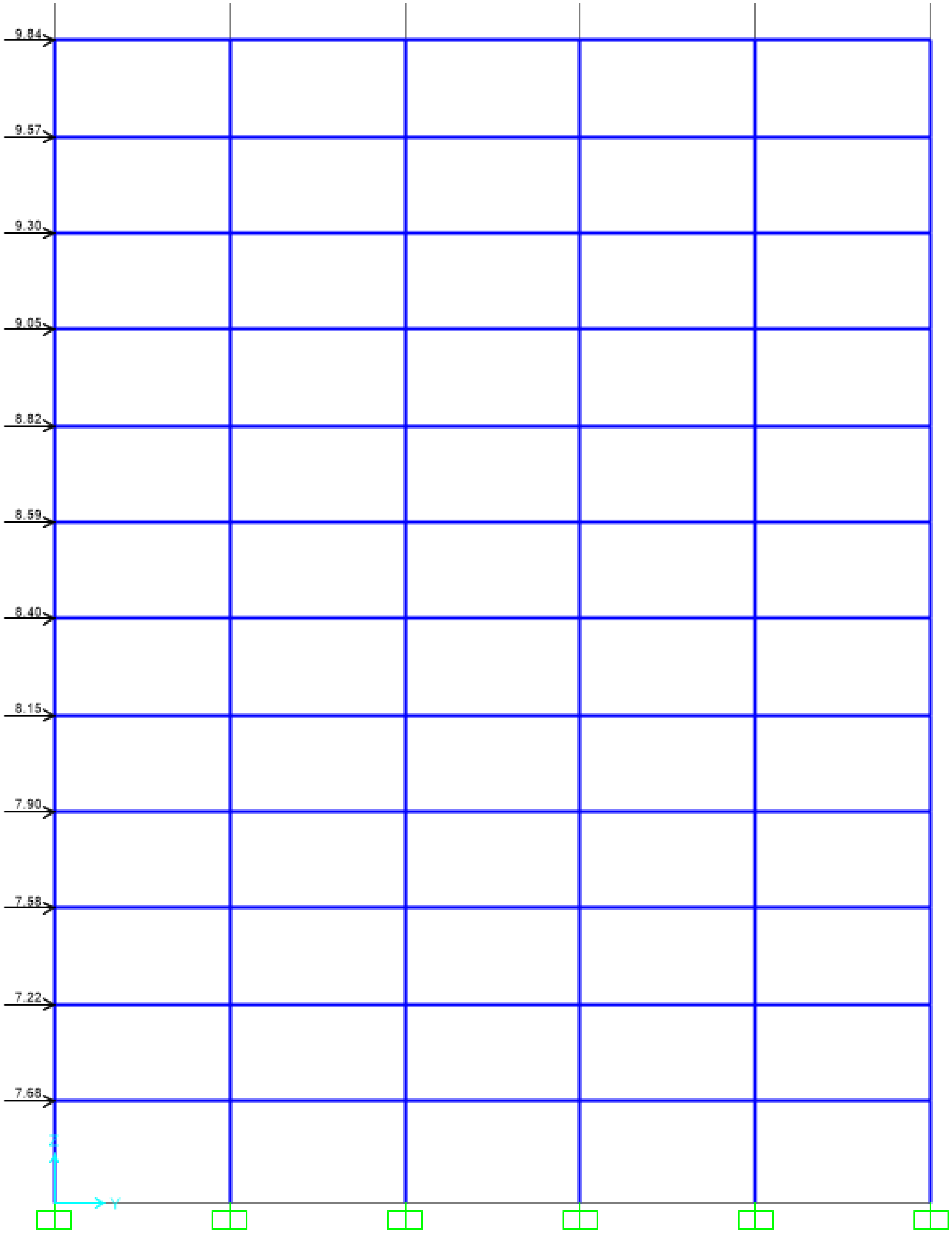


Figure 9.6 Load Assignment for Case IV Wind Load (Frame C)

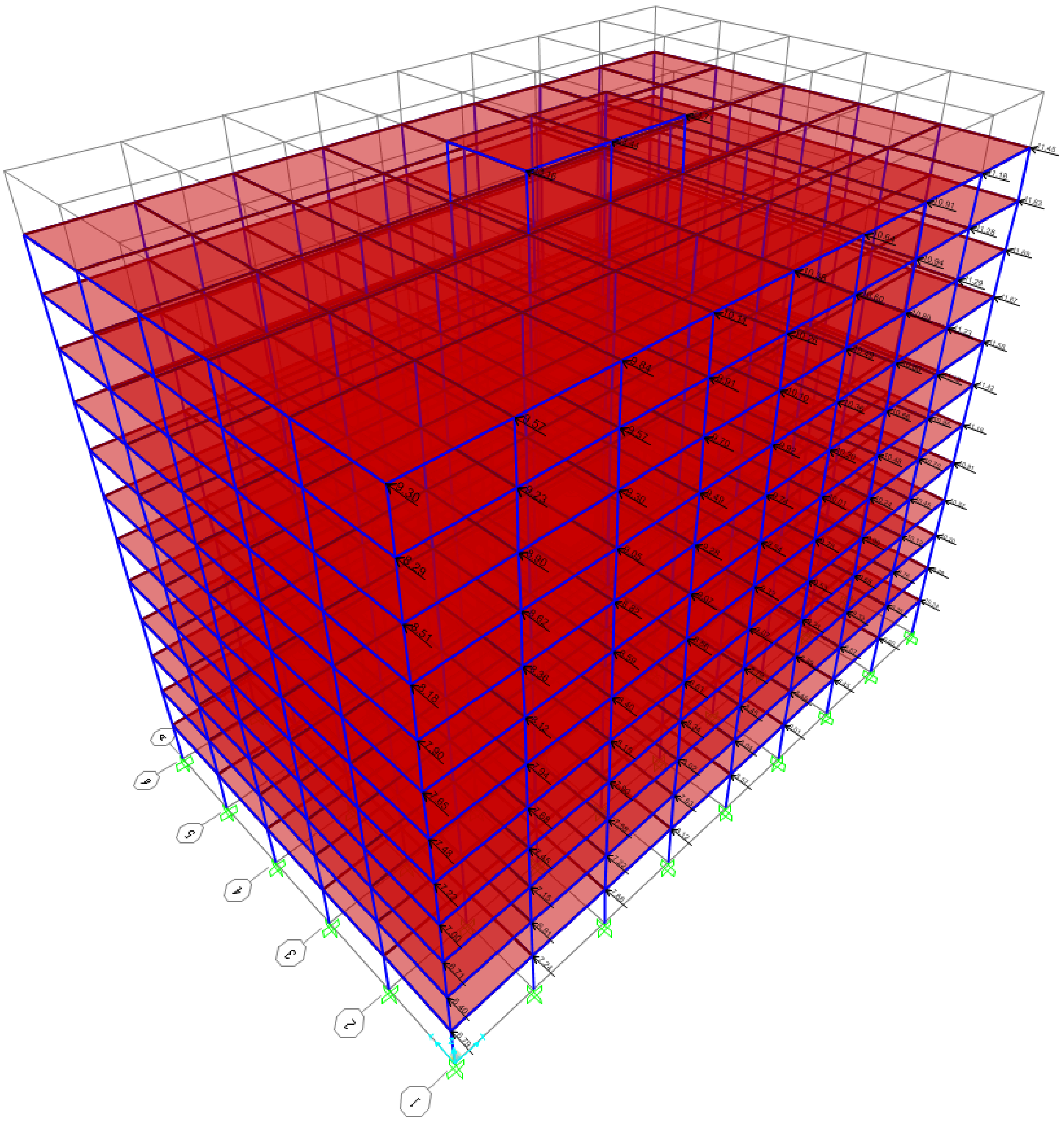


Figure 9.7. 3D Load Assignment for Case IV Wind Load

Table 9-1. Wind Force Calculations for first floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.6234	10.10	10.72	1	-1.7414	11.41	9.67
B	0.3741	10.10	10.47	2	-1.3060	11.41	10.11
C	0.1247	10.10	10.22	3	-0.8707	11.41	10.54
D	-0.1247	10.10	9.97	4	-0.4353	11.41	10.98
E	-0.3741	10.10	9.72	5	0.0000	11.41	11.41
F	-0.6234	10.10	9.47	6	0.4353	11.41	11.85
				7	0.8707	11.41	12.28
				8	1.3060	11.41	12.72
				9	1.7414	11.41	13.15

Table 9-2. Wind Force Calculations for second floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.5459	9.53	10.07	1	-1.5485	10.72	9.17
B	0.3276	9.53	9.86	2	-1.1613	10.72	9.55
C	0.1092	9.53	9.64	3	-0.7742	10.72	9.94
D	-0.1092	9.53	9.42	4	-0.3871	10.72	10.33
E	-0.3276	9.53	9.20	5	0.0000	10.72	10.72
F	-0.5459	9.53	8.98	6	0.3871	10.72	11.10
				7	0.7742	10.72	11.49
				8	1.1613	10.72	11.88
				9	1.5485	10.72	12.26

Table 9-3. Wind Force Calculations for fourth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.6113	10.06	10.67	1	-1.7111	11.70	9.99
B	0.3668	10.06	10.42	2	-1.2834	11.70	10.42
C	0.1223	10.06	10.18	3	-0.8556	11.70	10.85
D	-0.1223	10.06	9.93	4	-0.4278	11.70	11.28
E	-0.3668	10.06	9.69	5	0.0000	11.70	11.70
F	-0.6113	10.06	9.44	6	0.4278	11.70	12.13
				7	0.8556	11.70	12.56
				8	1.2834	11.70	12.99
				9	1.7111	11.70	13.42

Table 9-4. Wind Force Calculations for fifth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.6354	10.42	11.05	1	-1.7711	12.08	10.31
B	0.3812	10.42	10.80	2	-1.3283	12.08	10.75
C	0.1271	10.42	10.55	3	-0.8856	12.08	11.19
D	-0.1271	10.42	10.29	4	-0.4428	12.08	11.64
E	-0.3812	10.42	10.04	5	0.0000	12.08	12.08
F	-0.6354	10.42	9.78	6	0.4428	12.08	12.52
				7	0.8856	12.08	12.97
				8	1.3283	12.08	13.41
				9	1.7711	12.08	13.85

Table 9-5. Wind Force Calculations for sixth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.6562	10.74	11.39	1	-1.8231	12.41	10.59
B	0.3937	10.74	11.13	2	-1.3673	12.41	11.04
C	0.1312	10.74	10.87	3	-0.9115	12.41	11.50
D	-0.1312	10.74	10.61	4	-0.4558	12.41	11.95
E	-0.3937	10.74	10.34	5	0.0000	12.41	12.41
F	-0.6562	10.74	10.08	6	0.4558	12.41	12.87
				7	0.9115	12.41	13.32
				8	1.3673	12.41	13.78
				9	1.8231	12.41	14.23

Table 9-6. Wind Force Calculations for seventh floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.6748	11.02	11.70	1	-1.8693	12.71	10.84
B	0.4049	11.02	11.43	2	-1.4020	12.71	11.30
C	0.1350	11.02	11.16	3	-0.9346	12.71	11.77
D	-0.1350	11.02	10.89	4	-0.4673	12.71	12.24
E	-0.4049	11.02	10.62	5	0.0000	12.71	12.71
F	-0.6748	11.02	10.35	6	0.4673	12.71	13.17
				7	0.9346	12.71	13.64
				8	1.4020	12.71	14.11
				9	1.8693	12.71	14.58

Table 9-7. Wind Force Calculations for eighth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	0.6915	11.28	11.97	1	-1.9110	12.97	11.06
B	0.4149	11.28	11.70	2	-1.4332	12.97	11.54
C	0.1383	11.28	11.42	3	-0.9555	12.97	12.02
D	-0.1383	11.28	11.14	4	-0.4777	12.97	12.50
E	-0.4149	11.28	10.87	5	0.0000	12.97	12.97
F	-0.6915	11.28	10.59	6	0.4777	12.97	13.45
				7	0.9555	12.97	13.93
				8	1.4332	12.97	14.41
				9	1.9110	12.97	14.89

Table 9-8. Wind Force Calculations for ninth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	0.7069	11.52	12.23	1	-1.9492	13.22	11.27
B	0.4241	11.52	11.94	2	-1.4619	13.22	11.76
C	0.1414	11.52	11.66	3	-0.9746	13.22	12.25
D	-0.1414	11.52	11.38	4	-0.4873	13.22	12.73
E	-0.4241	11.52	11.09	5	0.0000	13.22	13.22
F	-0.7069	11.52	10.81	6	0.4873	13.22	13.71
				7	0.9746	13.22	14.20
				8	1.4619	13.22	14.68

				9	1.9492	13.22	15.17

Table 9-9. Wind Force Calculations for tenth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.7210	11.74	12.46	1	-1.9844	13.45	11.47
B	0.4326	11.74	12.17	2	-1.4883	13.45	11.96
C	0.1442	11.74	11.88	3	-0.9922	13.45	12.46
D	-0.1442	11.74	11.59	4	-0.4961	13.45	12.95
E	-0.4326	11.74	11.31	5	0.0000	13.45	13.45
F	-0.7210	11.74	11.02	6	0.4961	13.45	13.95
				7	0.9922	13.45	14.44
				8	1.4883	13.45	14.94
				9	1.9844	13.45	15.43

Table 9-10. Wind Force Calculations for eleventh floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	0.7342	11.95	12.68	1	-2.0173	13.66	11.65
B	0.4405	11.95	12.39	2	-1.5129	13.66	12.15
C	0.1468	11.95	12.09	3	-1.0086	13.66	12.66
D	-0.1468	11.95	11.80	4	-0.5043	13.66	13.16
E	-0.4405	11.95	11.50	5	0.0000	13.66	13.66
F	-0.7342	11.95	11.21	6	0.5043	13.66	14.17
				7	1.0086	13.66	14.67

				8	1.5129	13.66	15.18
				9	2.0173	13.66	15.68

Table 9-11. Wind Force Calculations for twelfth floor in both directions including the torsional effect (Case 2)

Case 2 (X-axis)				Case 2 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	0.7466	12.10	12.84	1	-2.0481	13.82	11.77
B	0.4479	12.10	12.54	2	-1.5360	13.82	12.29
C	0.1493	12.10	12.25	3	-1.0240	13.82	12.80
D	-0.1493	12.10	11.95	4	-0.5120	13.82	13.31
E	-0.4479	12.10	11.65	5	0.0000	13.82	13.82
F	-0.7466	12.10	11.35	6	0.5120	13.82	14.33
				7	1.0240	13.82	14.85
				8	1.5360	13.82	15.36
				9	2.0481	13.82	15.87

Table 9-12. Wind Force Calculations for first floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.7752	7.58	9.36	1	-1.7752	8.57	6.79
B	1.0651	7.58	8.65	2	-1.3314	8.57	7.24
C	0.3550	7.58	7.94	3	-0.8876	8.57	7.68
D	-0.3550	7.58	7.23	4	-0.4438	8.57	8.12
E	-1.0651	7.58	6.52	5	0.0000	8.57	8.57
F	-1.7752	7.58	5.81	6	0.4438	8.57	9.01

				7	0.8876	8.57	9.45
				8	1.3314	8.57	9.90
				9	1.7752	8.57	10.34

Table 9-13. Wind Force Calculations for second floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.6450	7.15	8.80	1	-1.6450	8.04	6.40
B	0.9870	7.15	8.14	2	-1.2337	8.04	6.81
C	0.3290	7.15	7.48	3	-0.8225	8.04	7.22
D	-0.3290	7.15	6.82	4	-0.4112	8.04	7.63
E	-0.9870	7.15	6.17	5	0.0000	8.04	8.04
F	-1.6450	7.15	5.51	6	0.4112	8.04	8.45
				7	0.8225	8.04	8.87
				8	1.2337	8.04	9.28
				9	1.6450	8.04	9.69

Table 9-14. Wind Force Calculations for fourth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.7816	7.55	9.33	1	-1.7816	8.79	7.00
B	1.0690	7.55	8.62	2	-1.3362	8.79	7.45
C	0.3563	7.55	7.91	3	-0.8908	8.79	7.90
D	-0.3563	7.55	7.19	4	-0.4454	8.79	8.34
E	-1.0690	7.55	6.48	5	0.0000	8.79	8.79

F	-1.7816	7.55	5.77	6	0.4454	8.79	9.23
				7	0.8908	8.79	9.68
				8	1.3362	8.79	10.12
				9	1.7816	8.79	10.57

Table 9-15. Wind Force Calculations for fifth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	1.8461	7.82	9.67	1	-1.8461	9.07	7.22
B	1.1076	7.82	8.93	2	-1.3846	9.07	7.68
C	0.3692	7.82	8.19	3	-0.9230	9.07	8.15
D	-0.3692	7.82	7.45	4	-0.4615	9.07	8.61
E	-1.1076	7.82	6.71	5	0.0000	9.07	9.07
F	-1.8461	7.82	5.97	6	0.4615	9.07	9.53
				7	0.9230	9.07	9.99
				8	1.3846	9.07	10.45
				9	1.8461	9.07	10.91

Table 9-16. Wind Force Calculations for sixth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	1.8390	8.06	9.90	1	-1.8390	9.32	7.48
B	1.1034	8.06	9.16	2	-1.3792	9.32	7.94
C	0.3678	8.06	8.43	3	-0.9195	9.32	8.40
D	-0.3678	8.06	7.69	4	-0.4597	9.32	8.86

E	-1.1034	8.06	6.96	5	0.0000	9.32	9.32
F	-1.8390	8.06	6.22	6	0.4597	9.32	9.78
				7	0.9195	9.32	10.24
				8	1.3792	9.32	10.70
				9	1.8390	9.32	11.16

Table 9-17. Wind Force Calculations for seventh floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	1.8870	8.27	10.16	1	-1.8870	9.54	7.65
B	1.1322	8.27	9.41	2	-1.4153	9.54	8.12
C	0.3774	8.27	8.65	3	-0.9435	9.54	8.59
D	-0.3774	8.27	7.90	4	-0.4718	9.54	9.07
E	-1.1322	8.27	7.14	5	0.0000	9.54	9.54
F	-1.8870	8.27	6.39	6	0.4718	9.54	10.01
				7	0.9435	9.54	10.48
				8	1.4153	9.54	10.95
				9	1.8870	9.54	11.42

Table 9-18. Wind Force Calculations for eighth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
A	1.8379	8.47	10.31	1	-1.8379	9.74	7.90
B	1.1027	8.47	9.57	2	-1.3784	9.74	8.36
C	0.3676	8.47	8.84	3	-0.9189	9.74	8.82

D	-0.3676	8.47	8.10	4	-0.4595	9.74	9.28
E	-1.1027	8.47	7.37	5	0.0000	9.74	9.74
F	-1.8379	8.47	6.63	6	0.4595	9.74	10.20
				7	0.9189	9.74	10.66
				8	1.3784	9.74	11.12
				9	1.8379	9.74	11.58

Table 9-19. Wind Force Calculations for ninth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.7438	8.65	10.39	1	-1.7438	9.92	8.18
B	1.0463	8.65	9.69	2	-1.3078	9.92	8.62
C	0.3488	8.65	9.00	3	-0.8719	9.92	9.05
D	-0.3488	8.65	8.30	4	-0.4359	9.92	9.49
E	-1.0463	8.65	7.60	5	0.0000	9.92	9.92
F	-1.7438	8.65	6.90	6	0.4359	9.92	10.36
				7	0.8719	9.92	10.80
				8	1.3078	9.92	11.23
				9	1.7438	9.92	11.67

Table 9-20. Wind Force Calculations for tenth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.5912	8.81	10.40	1	-1.5912	10.10	8.51
B	0.9547	8.81	9.77	2	-1.1934	10.10	8.90
C	0.3182	8.81	9.13	3	-0.7956	10.10	9.30

D	-0.3182	8.81	8.49	4	-0.3978	10.10	9.70
E	-0.9547	8.81	7.86	5	0.0000	10.10	10.10
F	-1.5912	8.81	7.22	6	0.3978	10.10	10.49
				7	0.7956	10.10	10.89
				8	1.1934	10.10	11.29
				9	1.5912	10.10	11.69

Table 9-21. Wind Force Calculations for eleventh floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.3687	8.97	10.34	1	-1.3687	10.26	8.89
B	0.8212	8.97	9.79	2	-1.0265	10.26	9.23
C	0.2737	8.97	9.24	3	-0.6844	10.26	9.57
D	-0.2737	8.97	8.69	4	-0.3422	10.26	9.91
E	-0.8212	8.97	8.15	5	0.0000	10.26	10.26
F	-1.3687	8.97	7.60	6	0.3422	10.26	10.60
				7	0.6844	10.26	10.94
				8	1.0265	10.26	11.28
				9	1.3687	10.26	11.63

Table 9-22. Wind Force Calculations for twelfth floor in both directions including the torsional effect (Case 4)

Case 4 (X-axis)				Case 4 (Y-axis)			
Frame	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**	**Frame**	**Ftorsion (kN)**	**Fdirect (kN)**	**Ftotal (kN)**
A	1.0779	9.08	10.16	1	-1.0779	10.38	9.30

B	0.6467	9.08	9.73	2	-0.8084	10.38	9.57
C	0.2156	9.08	9.30	3	-0.5389	10.38	9.84
D	-0.2156	9.08	8.87	4	-0.2695	10.38	10.11
E	-0.6467	9.08	8.43	5	0.0000	10.38	10.38
F	-1.0779	9.08	8.00	6	0.2695	10.38	10.64
				7	0.5389	10.38	10.91
				8	0.8084	10.38	11.18
				9	1.0779	10.38	11.45

Table 9-23. Stiffness Center Calculations

							Transverse	**Longitudinal**
Floor	**Column size**	**Ic,cr, m4**	**Ib,cr, m4**	**Minor b, Ib,cr**	**D, kN/m**	**h, m**	**Cf, kN/rad**	**Cf, kN/rad**
1	0.75	1.85E-02	1.89E-03	0	8655	3.5	242351	151469
2	0.7	1.40E-02	1.89E-03	0	9605	3.3	253573	158483
3	0.7	1.40E-02	1.89E-03	0	9605	3.3	253573	158483
4	0.65	1.04E-02	1.89E-03	0	9381	3.3	247668	154793
5	0.65	1.04E-02	1.89E-03	0	9381	3.3	247668	154793
6	0.6	7.56E-03	1.89E-03	0	9071	3.3	239466	149666
7	0.6	7.56E-03	1.89E-03	0	9071	3.3	239466	149666
8	0.55	5.34E-03	1.89E-03	0	8636	3.3	227993	142496
9	0.5	3.65E-03	1.89E-03	0	8029	3.3	211959	132474
10	0.45	2.39E-03	1.89E-03	0	7192	3.3	189878	118674

11	0.4	1.49E-03	1.89E-03	0	6083	3.3	160600	100375
12	0.35	8.75E-04	1.89E-03	0	4717	3.3	124522	77826
13	0.35	8.75E-04	1.89E-03	0	4717	3.3	124522	77826

Table 9-24. Seismic Load Calculations for first floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	0.1719	7.38	7.55	1	-0.8054	4.92	4.12
B	0.1032	7.38	7.49	2	-0.6040	4.92	4.32
C	0.0344	7.38	7.42	3	-0.4027	4.92	4.52
D	-0.0344	7.38	7.35	4	-0.2013	4.92	4.72
E	-0.1032	7.38	7.28	5	0.0000	4.92	4.92
F	-0.1719	7.38	7.21	6	0.2013	4.92	5.12
				7	0.4027	4.92	5.32
				8	0.6040	4.92	5.53
				9	0.8054	4.92	5.73

Table 9-25. Seismic Load Calculations for second floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	0.4357	18.71	19.15	1	-2.0410	12.47	10.43
B	0.2614	18.71	18.97	2	-1.5308	12.47	10.94

C	0.0871	18.71	18.80	3	-1.0205	12.47	11.45
D	-0.0871	18.71	18.62	4	-0.5103	12.47	11.96
E	-0.2614	18.71	18.45	5	0.0000	12.47	12.47
F	-0.4357	18.71	18.27	6	0.5103	12.47	12.98
				7	1.0205	12.47	13.49
				8	1.5308	12.47	14.00
				9	2.0410	12.47	14.51

Table 9-26. Seismic Load Calculations for fourth floor in both directions including the torsional effect

Transverse				**Longitudinal**			
Frame	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**	**Frame**	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**
A	1.1374	48.84	49.98	1	-5.3278	32.56	27.23
B	0.6824	48.84	49.52	2	-3.9958	32.56	28.56
C	0.2275	48.84	49.07	3	-2.6639	32.56	29.89
D	-0.2275	48.84	48.61	4	-1.3319	32.56	31.23
E	-0.6824	48.84	48.16	5	0.0000	32.56	32.56
F	-1.1374	48.84	47.70	6	1.3319	32.56	33.89
				7	2.6639	32.56	35.22
				8	3.9958	32.56	36.55
				9	5.3278	32.56	37.89

Table 9-27. Seismic Load Calculations for fifth floor in both directions including the torsional effect

Transverse				**Longitudinal**			
Frame	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**	**Frame**	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**
A	1.5614	67.04	68.61	1	-7.3140	44.70	37.38

B	0.9368	67.04	67.98	2	-5.4855	44.70	39.21
C	0.3123	67.04	67.36	3	-3.6570	44.70	41.04
D	-0.3123	67.04	66.73	4	-1.8285	44.70	42.87
E	-0.9368	67.04	66.11	5	0.0000	44.70	44.70
F	-1.5614	67.04	65.48	6	1.8285	44.70	46.53
				7	3.6570	44.70	48.35
				8	5.4855	44.70	50.18
				9	7.3140	44.70	52.01

Table 9-28. Seismic Load Calculations for sixth floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**	**Frame**	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**
A	1.9923	85.55	87.54	1	-9.3324	57.03	47.70
B	1.1954	85.55	86.74	2	-6.9993	57.03	50.03
C	0.3985	85.55	85.95	3	-4.6662	57.03	52.37
D	-0.3985	85.55	85.15	4	-2.3331	57.03	54.70
E	-1.1954	85.55	84.35	5	0.0000	57.03	57.03
F	-1.9923	85.55	83.55	6	2.3331	57.03	59.36
				7	4.6662	57.03	61.70
				8	6.9993	57.03	64.03
				9	9.3324	57.03	66.36

Table 9-29. Seismic Load Calculations for seventh floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**	**Frame**	**$F_{torsion}$, kN**	**F_{direct}, kN**	**F_{total}, kN**

A	2.4821	106.58	109.06	1	-11.6266	71.05	59.42
B	1.4892	106.58	108.07	2	-8.7200	71.05	62.33
C	0.4964	106.58	107.07	3	-5.8133	71.05	65.24
D	-0.4964	106.58	106.08	4	-2.9067	71.05	68.14
E	-1.4892	106.58	105.09	5	0.0000	71.05	71.05
F	-2.4821	106.58	104.10	6	2.9067	71.05	73.96
				7	5.8133	71.05	76.86
				8	8.7200	71.05	79.77
				9	11.6266	71.05	82.68

Table 9-30. Seismic Load Calculations for eighth floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	2.9593	127.07	130.03	1	-13.8620	84.71	70.85
B	1.7756	127.07	128.84	2	-10.3965	84.71	74.32
C	0.5919	127.07	127.66	3	-6.9310	84.71	77.78
D	-0.5919	127.07	126.48	4	-3.4655	84.71	81.25
E	-1.7756	127.07	125.29	5	0.0000	84.71	84.71
F	-2.9593	127.07	124.11	6	3.4655	84.71	88.18
				7	6.9310	84.71	91.64
				8	10.3965	84.71	95.11
				9	13.8620	84.71	98.57

Table 9-31. Seismic Load Calculations for ninth floor in both directions including the torsional effect

Transverse				Longitudinal			

Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	3.4541	148.32	151.77	1	-16.1799	98.88	82.70
B	2.0725	148.32	150.39	2	-12.1349	98.88	86.74
C	0.6908	148.32	149.01	3	-8.0900	98.88	90.79
D	-0.6908	148.32	147.62	4	-4.0450	98.88	94.83
E	-2.0725	148.32	146.24	5	0.0000	98.88	98.88
F	-3.4541	148.32	144.86	6	4.0450	98.88	102.92
				7	8.0900	98.88	106.97
				8	12.1349	98.88	111.01
				9	16.1799	98.88	115.06

Table 9-32. Seismic Load Calculations for tenth floor in both directions including the torsional effect

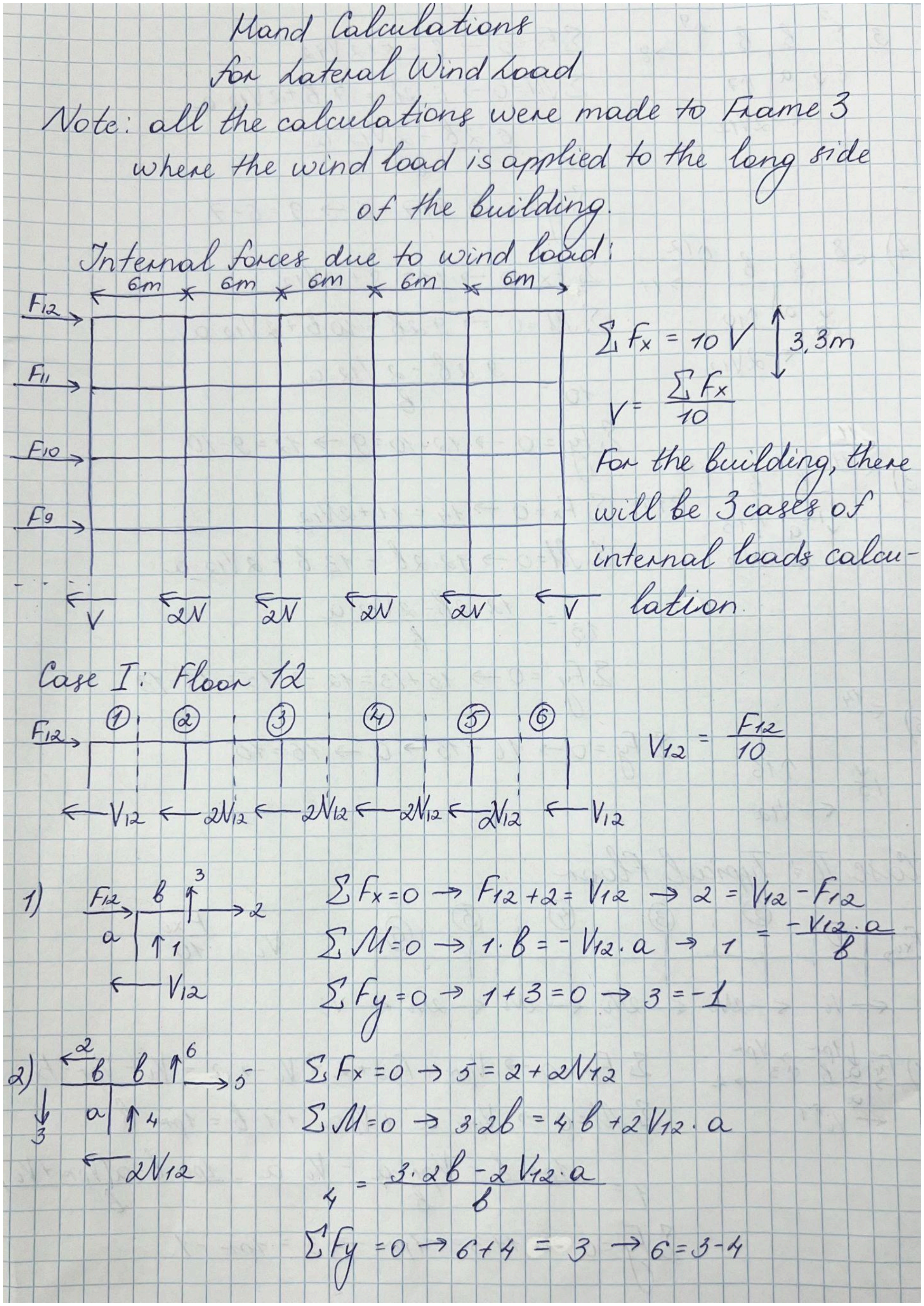
Transverse				Longitudinal			
Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	3.9657	170.28	174.25	1	-18.5764	113.52	94.95
B	2.3794	170.28	172.66	2	-12.4809	113.52	101.04
C	0.7931	170.28	171.08	3	-8.3206	113.52	105.20
D	-0.7931	170.28	169.49	4	-4.1603	113.52	109.36
E	-2.3794	170.28	167.90	5	0.0000	113.52	113.52
F	-3.9657	170.28	166.32	6	4.1603	113.52	117.68
				7	8.3206	113.52	121.84
				8	12.4809	113.52	126.00
				9	16.6412	113.52	130.16

Table 9-33. Seismic Load Calculations for eleventh floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	4.4938	192.96	197.45	1	-21.0502	128.64	107.59
B	2.6963	192.96	195.66	2	-15.7877	128.64	112.85
C	0.8988	192.96	193.86	3	-10.5251	128.64	118.12
D	-0.8988	192.96	192.06	4	-5.2626	128.64	123.38
E	-2.6963	192.96	190.26	5	0.0000	128.64	128.64
F	-4.4938	192.96	188.47	6	5.2626	128.64	133.90
				7	10.5251	128.64	139.17
				8	15.7877	128.64	144.43
				9	21.0502	128.64	149.69

Table 9-34. Seismic Load Calculations for twelfth floor in both directions including the torsional effect

Transverse				Longitudinal			
Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN	Frame	$F_{torsion}$, kN	F_{direct}, kN	F_{total}, kN
A	5.0387	216.35	221.39	1	-23.6023	144.24	120.63
B	3.0232	216.35	219.38	2	-17.7017	144.24	126.53
C	1.0077	216.35	217.36	3	-11.8012	144.24	132.44
D	-1.0077	216.35	215.35	4	-5.9006	144.24	138.34
E	-3.0232	216.35	213.33	5	0.0000	144.24	144.24
F	-5.0387	216.35	211.32	6	5.9006	144.24	150.14
				7	11.8012	144.24	156.04
				8	17.7017	144.24	161.94
				9	23.6023	144.24	167.84



Hand Calculations
for Lateral Wind Load

Note: all the calculations were made to Frame 3 where the wind load is applied to the long side of the building.

Internal forces due to wind load:

$$\Sigma F_x = 10V$$

$$V = \frac{\Sigma F_x}{10}$$

For the building, there will be 3 cases of internal loads calculation.

Case I: Floor 12

$$V_{12} = \frac{F_{12}}{10}$$

1)

$$\Sigma F_x = 0 \rightarrow F_{12} + 2 = V_{12} \rightarrow 2 = V_{12} - F_{12}$$

$$\Sigma M = 0 \rightarrow 1 \cdot b = -V_{12} \cdot a \rightarrow 1 = \frac{-V_{12} \cdot a}{b}$$

$$\Sigma F_y = 0 \rightarrow 1 + 3 = 0 \rightarrow 3 = -1$$

2)

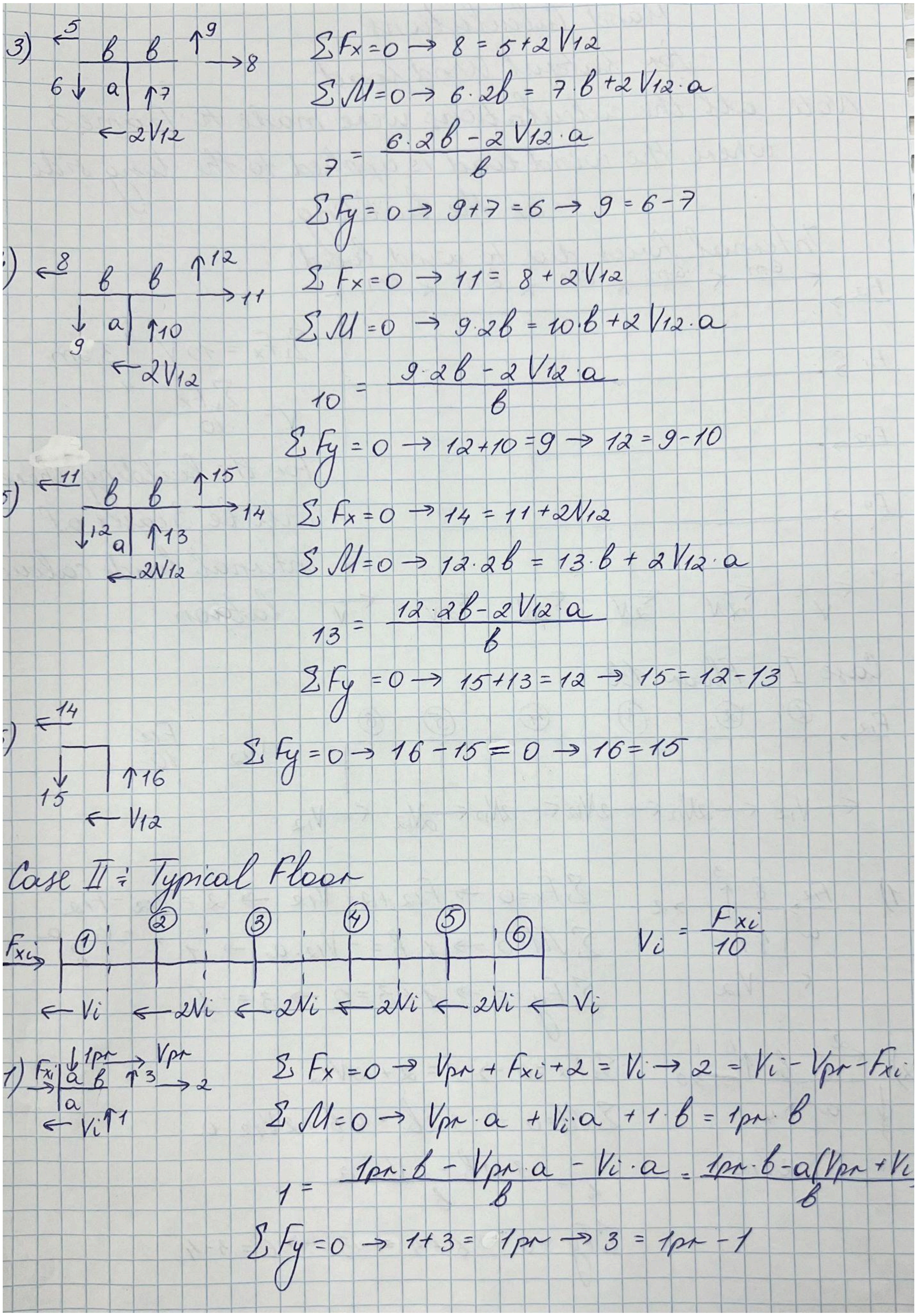
$$\Sigma F_x = 0 \rightarrow 5 = 2 + 2V_{12}$$

$$\Sigma M = 0 \rightarrow 3 \cdot 2b = 4 \cdot b + 2V_{12} \cdot a$$

$$4 = \frac{3 \cdot 2b - 2V_{12} \cdot a}{b}$$

$$\Sigma F_y = 0 \rightarrow 6 + 4 = 3 \rightarrow 6 = 3 - 4$$

Figure 9.8. Hand calculations of the internal forces under wind load Part-1



$\sum F_x = 0 \rightarrow 8 = 5 + 2V_{12}$

$\sum M = 0 \rightarrow 6 \cdot 2b = 7 \cdot b + 2V_{12} \cdot a$

$$7 = \frac{6 \cdot 2b - 2V_{12} \cdot a}{b}$$

$\sum F_y = 0 \rightarrow 9 + 7 = 6 \rightarrow 9 = 6 - 7$

$\sum F_x = 0 \rightarrow 11 = 8 + 2V_{12}$

$\sum M = 0 \rightarrow 9 \cdot 2b = 10 \cdot b + 2V_{12} \cdot a$

$$10 = \frac{9 \cdot 2b - 2V_{12} \cdot a}{b}$$

$\sum F_y = 0 \rightarrow 12 + 10 = 9 \rightarrow 12 = 9 - 10$

$\sum F_x = 0 \rightarrow 14 = 11 + 2N_{12}$

$\sum M = 0 \rightarrow 12 \cdot 2b = 13 \cdot b + 2V_{12} \cdot a$

$$13 = \frac{12 \cdot 2b - 2V_{12} \cdot a}{b}$$

$\sum F_y = 0 \rightarrow 15 + 13 = 12 \rightarrow 15 = 12 - 13$

$\sum F_y = 0 \rightarrow 16 - 15 = 0 \rightarrow 16 = 15$

Case II : Typical Floor

$$V_i = \frac{F_{xi}}{10}$$

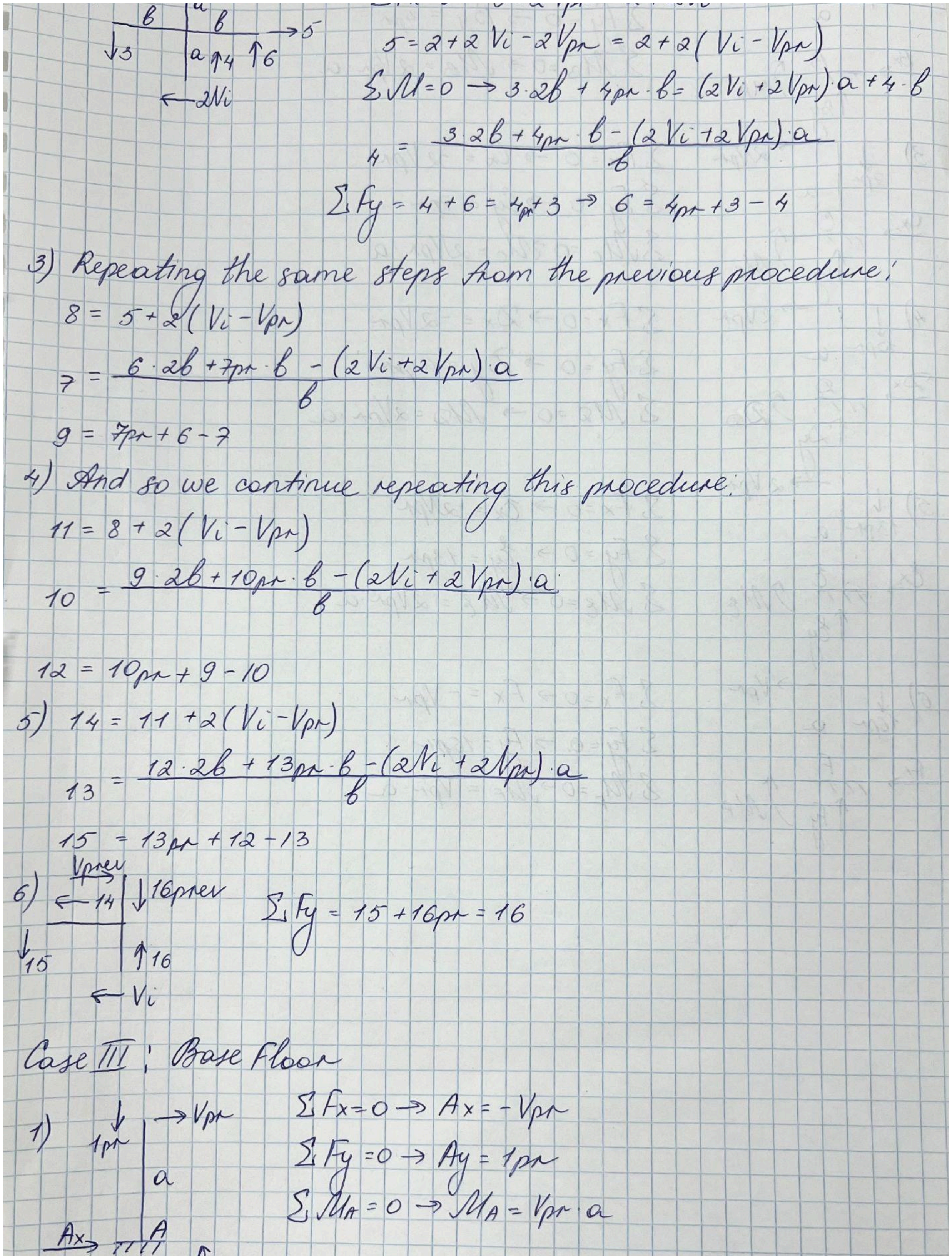
$\sum F_x = 0 \rightarrow V_{pr} + F_{xi} + 2 = V_i \rightarrow 2 = V_i - V_{pr} - F_{xi}$

$\sum M = 0 \rightarrow V_{pr} \cdot a + V_i \cdot a + 1 \cdot b = 1pr \cdot b$

$$1 = \frac{1pr \cdot b - V_{pr} \cdot a - V_i \cdot a}{b} = \frac{1pr \cdot b - a(V_{pr} + V_i}{b}$$

$\sum F_y = 0 \rightarrow 1 + 3 = 1pr \rightarrow 3 = 1pr - 1$

Figure 9.9. Hand calculations of the internal forces under wind load Part-2



$5 = 2 + 2\,V_i - 2V_{pr} = 2 + 2(V_i - V_{pr})$

$\sum M = 0 \rightarrow 3 \cdot 2b + 4_{pr} \cdot b = (2V_i + 2V_{pr}) \cdot a + 4 \cdot b$

$$4 = \frac{3 \cdot 2b + 4_{pr} \cdot b - (2V_i + 2V_{pr}) \cdot a}{b}$$

$\sum F_y = 4 + 6 = 4_{pr} + 3 \rightarrow 6 = 4_{pr} + 3 - 4$

3) Repeating the same steps from the previous procedure:

$8 = 5 + 2(V_i - V_{pr})$

$$7 = \frac{6 \cdot 2b + 7_{pr} \cdot b - (2V_i + 2V_{pr}) \cdot a}{b}$$

$9 = 7_{pr} + 6 - 7$

4) And so we continue repeating this procedure:

$11 = 8 + 2(V_i - V_{pr})$

$$10 = \frac{9 \cdot 2b + 10_{pr} \cdot b - (2V_i + 2V_{pr}) \cdot a}{b}$$

$12 = 10_{pr} + 9 - 10$

5) $14 = 11 + 2(V_i - V_{pr})$

$$13 = \frac{12 \cdot 2b + 13_{pr} \cdot b - (2V_i + 2V_{pr}) \cdot a}{b}$$

$15 = 13_{pr} + 12 - 13$

6)

$\sum F_y = 15 + 16_{pr} = 16$

Case III: Base Floor

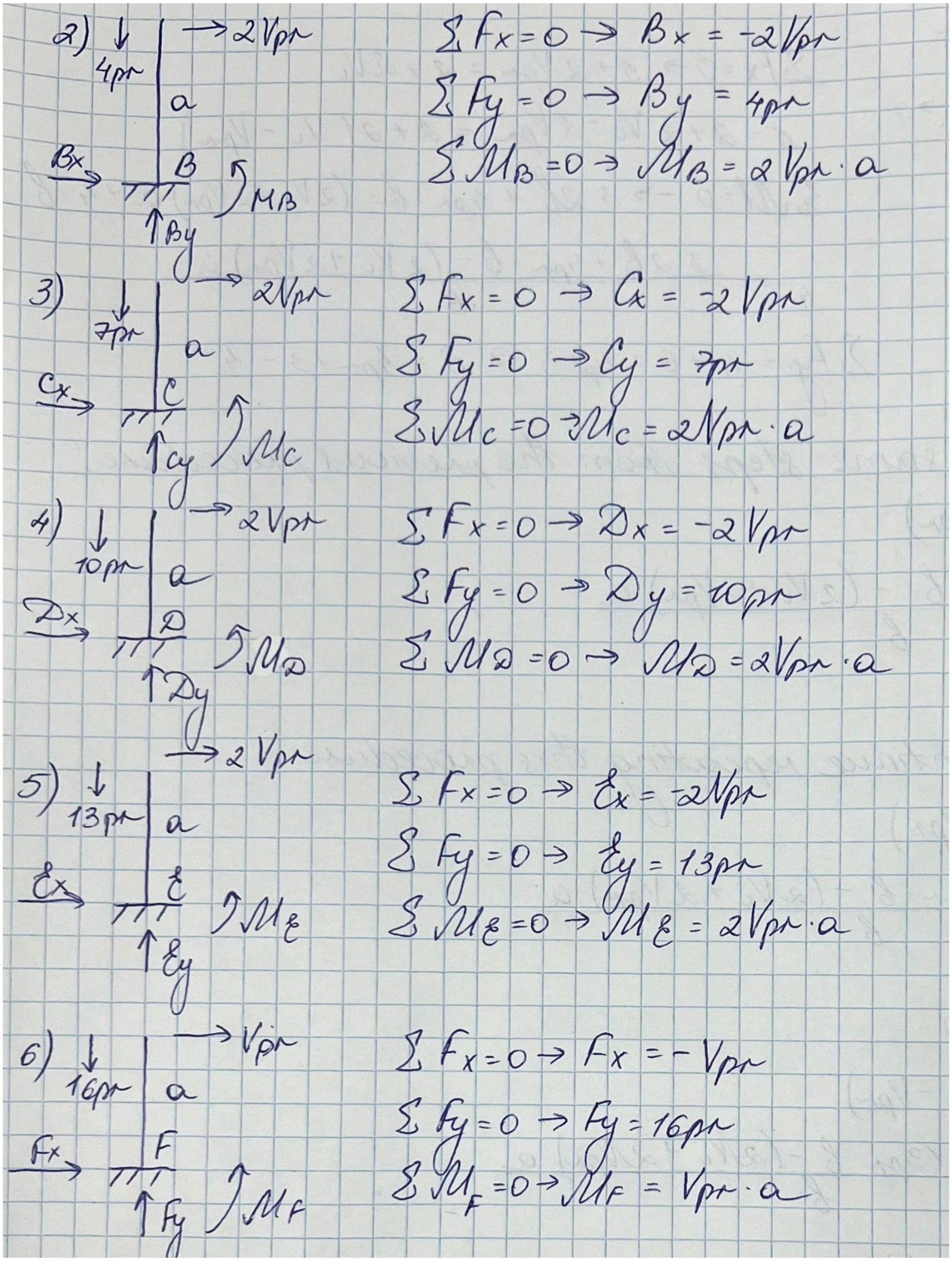
1)

$\sum F_x = 0 \rightarrow A_x = -V_{pr}$

$\sum F_y = 0 \rightarrow A_y = 1_{pr}$

$\sum M_A = 0 \rightarrow M_A = V_{pr} \cdot a$

Figure 9.10. Hand calculations of the internal forces under wind load Part-3



2) $\sum F_x = 0 \rightarrow B_x = -2Vpr$

$\sum F_y = 0 \rightarrow B_y = 4pr$

$\sum M_B = 0 \rightarrow M_B = 2Vpr \cdot a$

3) $\sum F_x = 0 \rightarrow C_x = -2Vpr$

$\sum F_y = 0 \rightarrow C_y = 7pr$

$\sum M_C = 0 \rightarrow M_C = 2Vpr \cdot a$

4) $\sum F_x = 0 \rightarrow D_x = -2Vpr$

$\sum F_y = 0 \rightarrow D_y = 10pr$

$\sum M_D = 0 \rightarrow M_D = 2Vpr \cdot a$

5) $\sum F_x = 0 \rightarrow E_x = -2Vpr$

$\sum F_y = 0 \rightarrow E_y = 13pr$

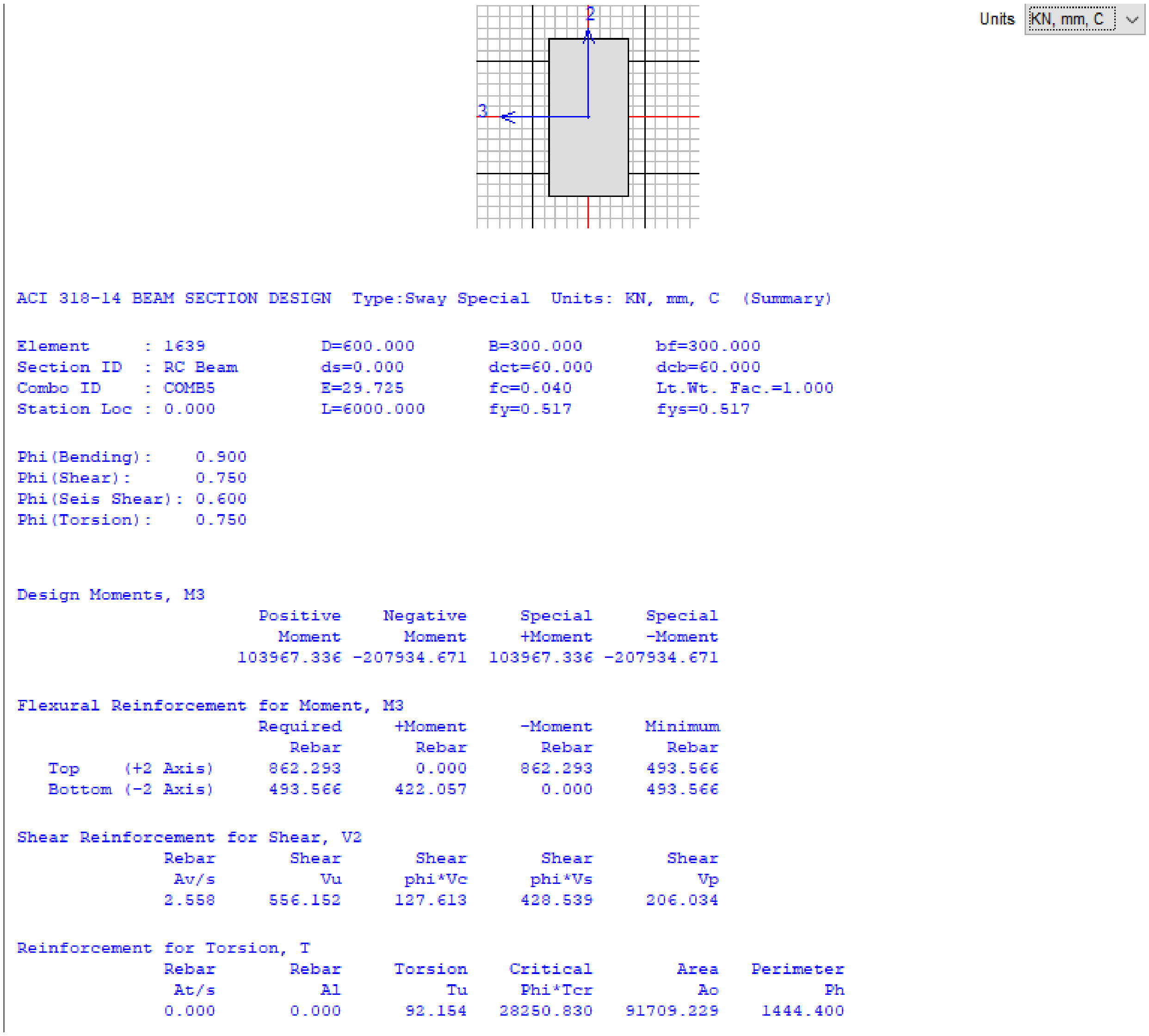
$\sum M_E = 0 \rightarrow M_E = 2Vpr \cdot a$

6) $\sum F_x = 0 \rightarrow F_x = -Vpr$

$\sum F_y = 0 \rightarrow F_y = 16pr$

$\sum M_F = 0 \rightarrow M_F = Vpr \cdot a$

Figure 9.11. Hand calculations of the internal forces under wind load Part-3



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

Element      : 1639          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000         L=6000.000      fy=0.517        fys=0.517

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

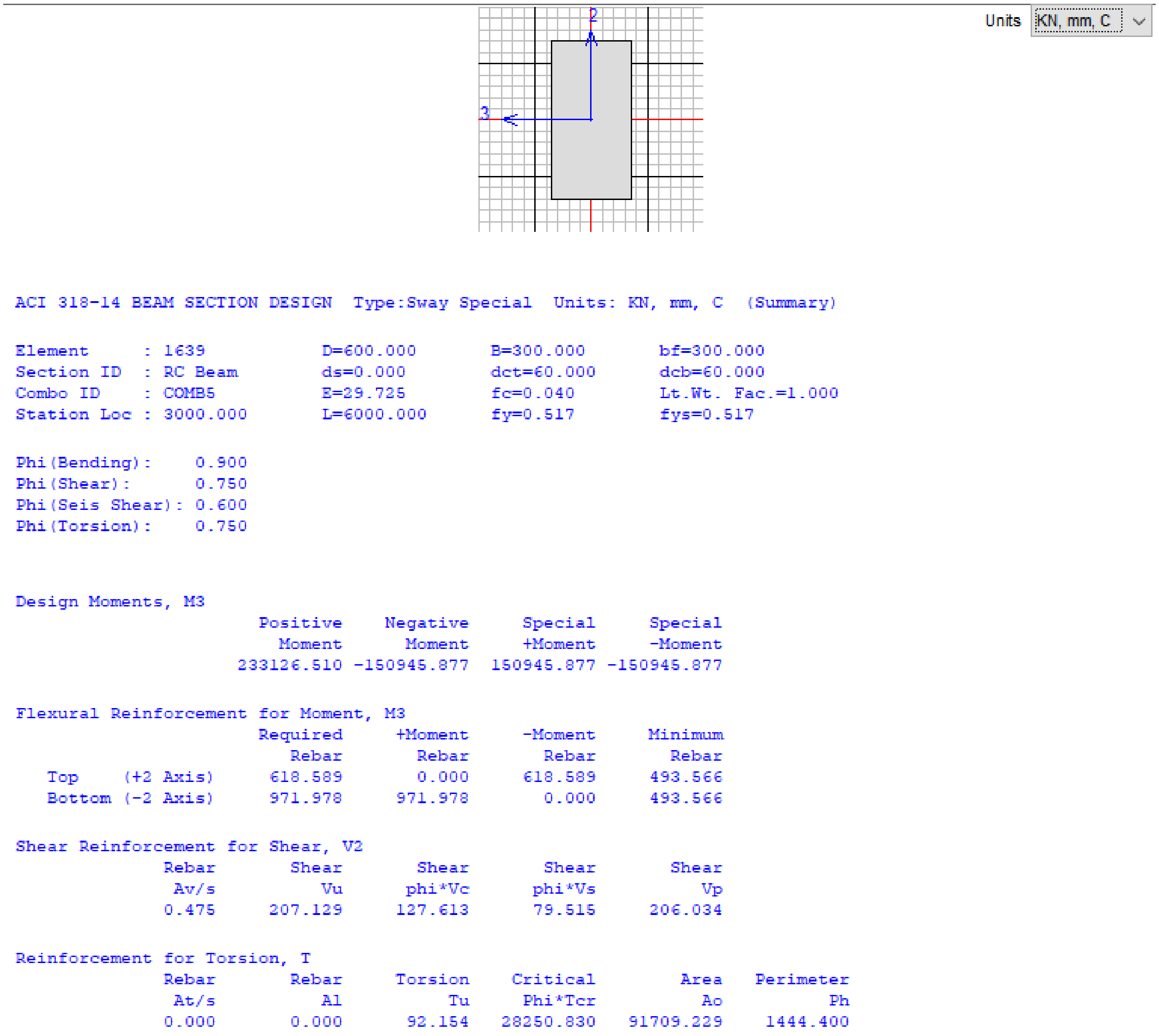

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment      +Moment      -Moment
                     103967.336  -207934.671   103967.336  -207934.671

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)     862.293        0.000      862.293      493.566
   Bottom (-2 Axis)     493.566      422.057        0.000      493.566

Shear Reinforcement for Shear, V2
                 Rebar        Shear        Shear        Shear        Shear
                  Av/s           Vu      phi*Vc       phi*Vs           Vp
                 2.558      556.152      127.613      428.539      206.034

Reinforcement for Torsion, T
                 Rebar        Rebar      Torsion     Critical         Area    Perimeter
                  At/s           Al           Tu      Phi*Tcr           Ao           Ph
                 0.000        0.000       92.154    28250.830    91709.229     1444.400
```

Figure 9.12. SAP 2000 Major Beam Design Summary for the 1st floor (location 0 m)



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

Element      : 1639          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 3000.000      L=6000.000      fy=0.517        fys=0.517

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

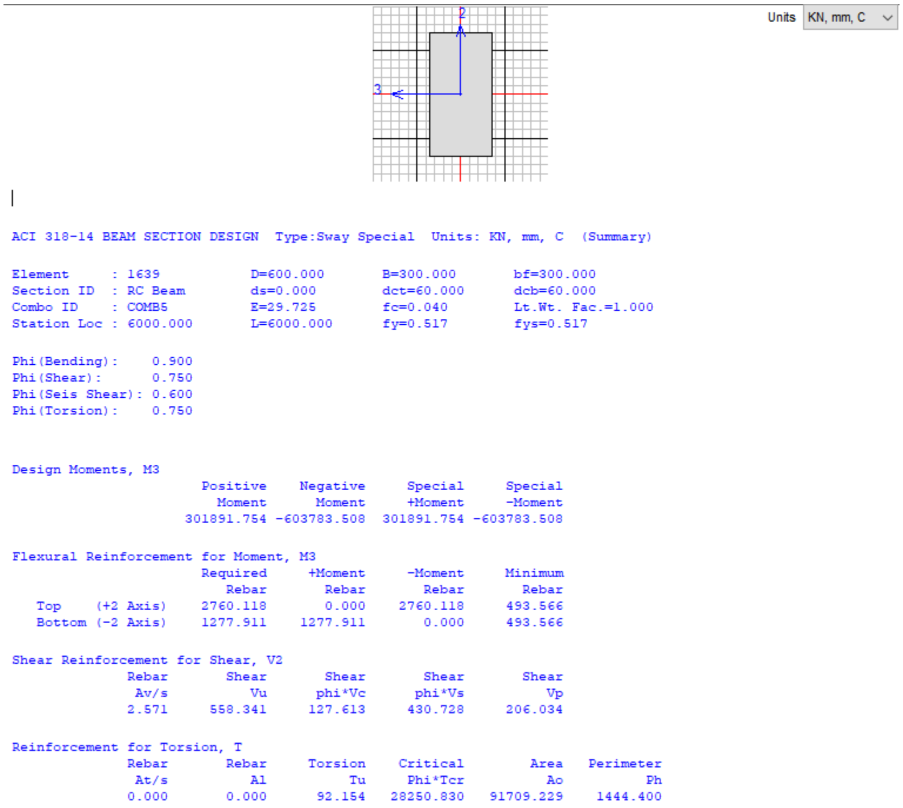

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     233126.510 -150945.877  150945.877 -150945.877

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     618.589       0.000     618.589     493.566
   Bottom (-2 Axis)     971.978     971.978       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu     phi*Vc      phi*Vs          Vp
              0.475     207.129     127.613      79.515     206.034

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000      92.154   28250.830   91709.229    1444.400
```

Figure 9.13. SAP 2000 Major Beam Design Summary for the 1st floor (location 3 m)



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

Element     : 1639          D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 6000.000      L=6000.000      fy=0.517        fys=0.517

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

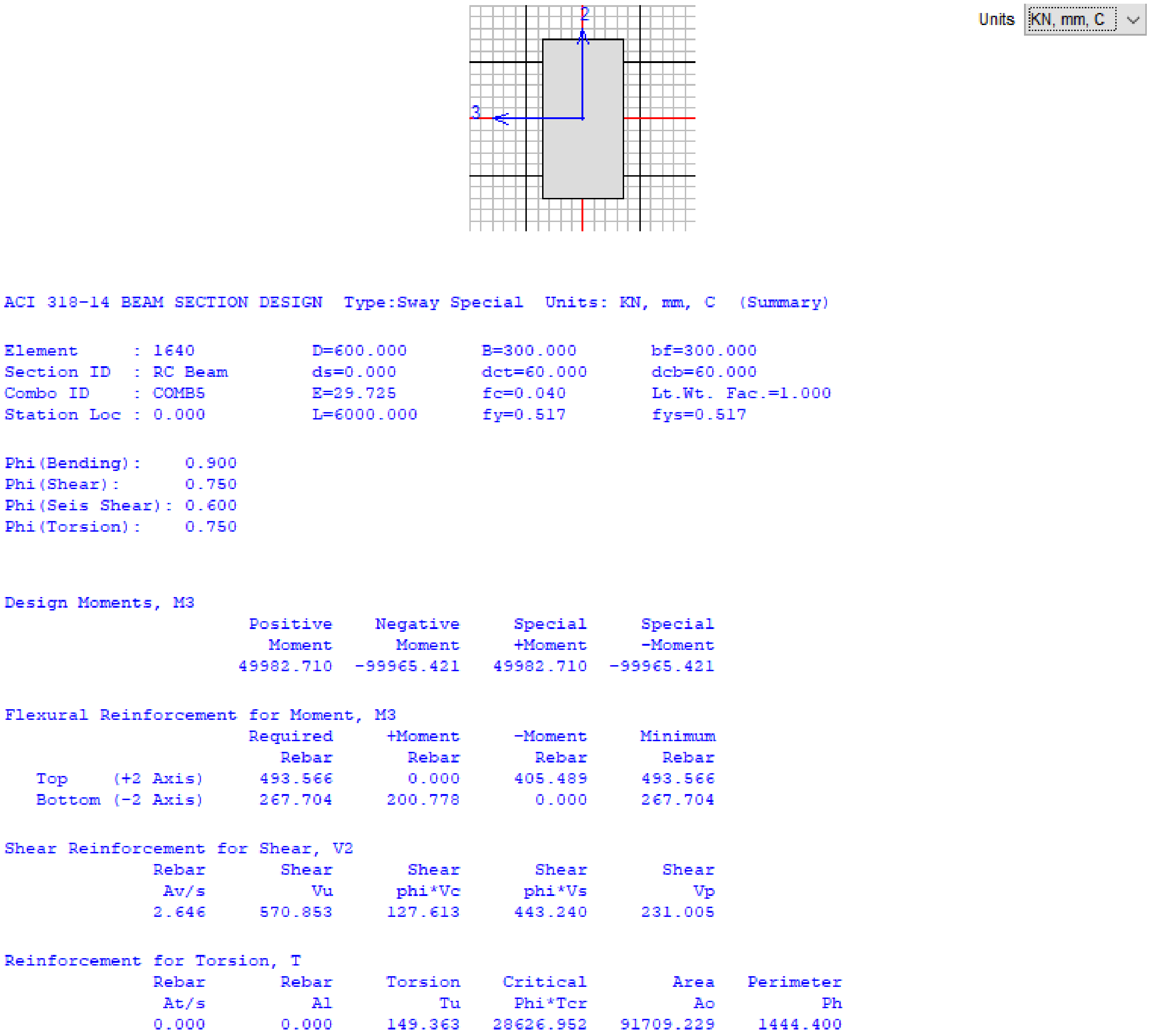

Design Moments, M3
                       Positive    Negative      Special      Special
                         Moment      Moment     +Moment      -Moment
                     301891.754 -603783.508  301891.754 -603783.508

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar       Rebar        Rebar        Rebar
   Top    (+2 Axis)    2760.118       0.000     2760.118      493.566
   Bottom (-2 Axis)    1277.911    1277.911        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear        Shear        Shear
               Av/s          Vu     phi*Vc       phi*Vs           Vp
              2.571     558.341     127.613      430.728      206.034

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion     Critical         Area    Perimeter
               At/s          Al          Tu     Phi*Tcr            Ao           Ph
              0.000       0.000      92.154   28250.830    91709.229     1444.400
```

Figure 9.14. SAP 2000 Major Beam Design Summary for the 1st floor (location 6 m)



Units KN, mm, C

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

Element      : 1640            D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam         ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5           E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000           L=6000.000      fy=0.517        fys=0.517

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

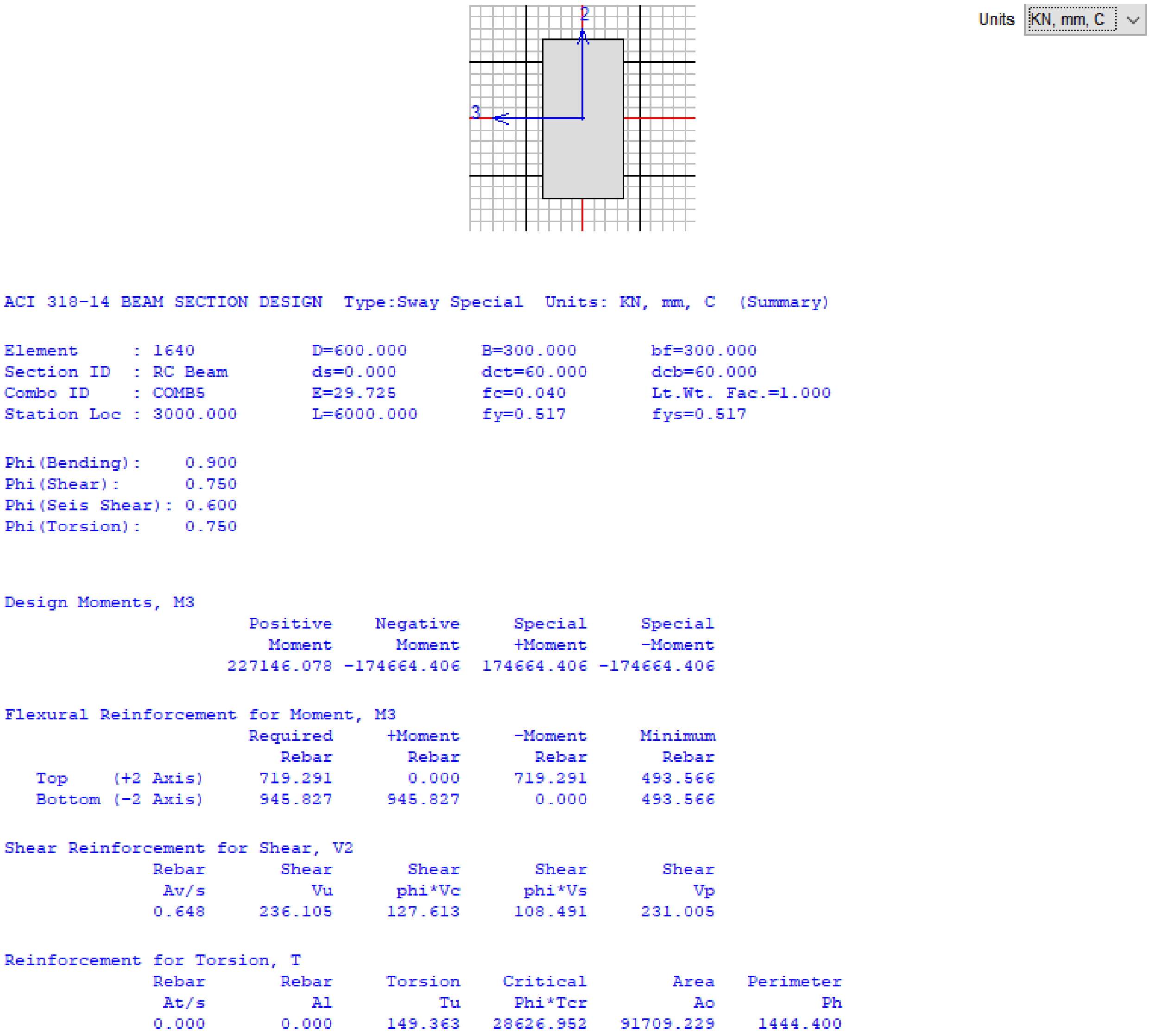

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                      49982.710   -99965.421    49982.710   -99965.421

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)     493.566        0.000      405.489      493.566
   Bottom (-2 Axis)     267.704      200.778        0.000      267.704

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu      phi*Vc       phi*Vs           Vp
              2.646      570.853      127.613      443.240      231.005

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu     Phi*Tcr           Ao           Ph
              0.000        0.000      149.363    28626.952    91709.229     1444.400
```

Figure 9.15. SAP 2000 Major Beam Design Summary for the 2nd floor (location 0 m)



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

Element     : 1640           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 3000.000       L=6000.000      fy=0.517        fys=0.517

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

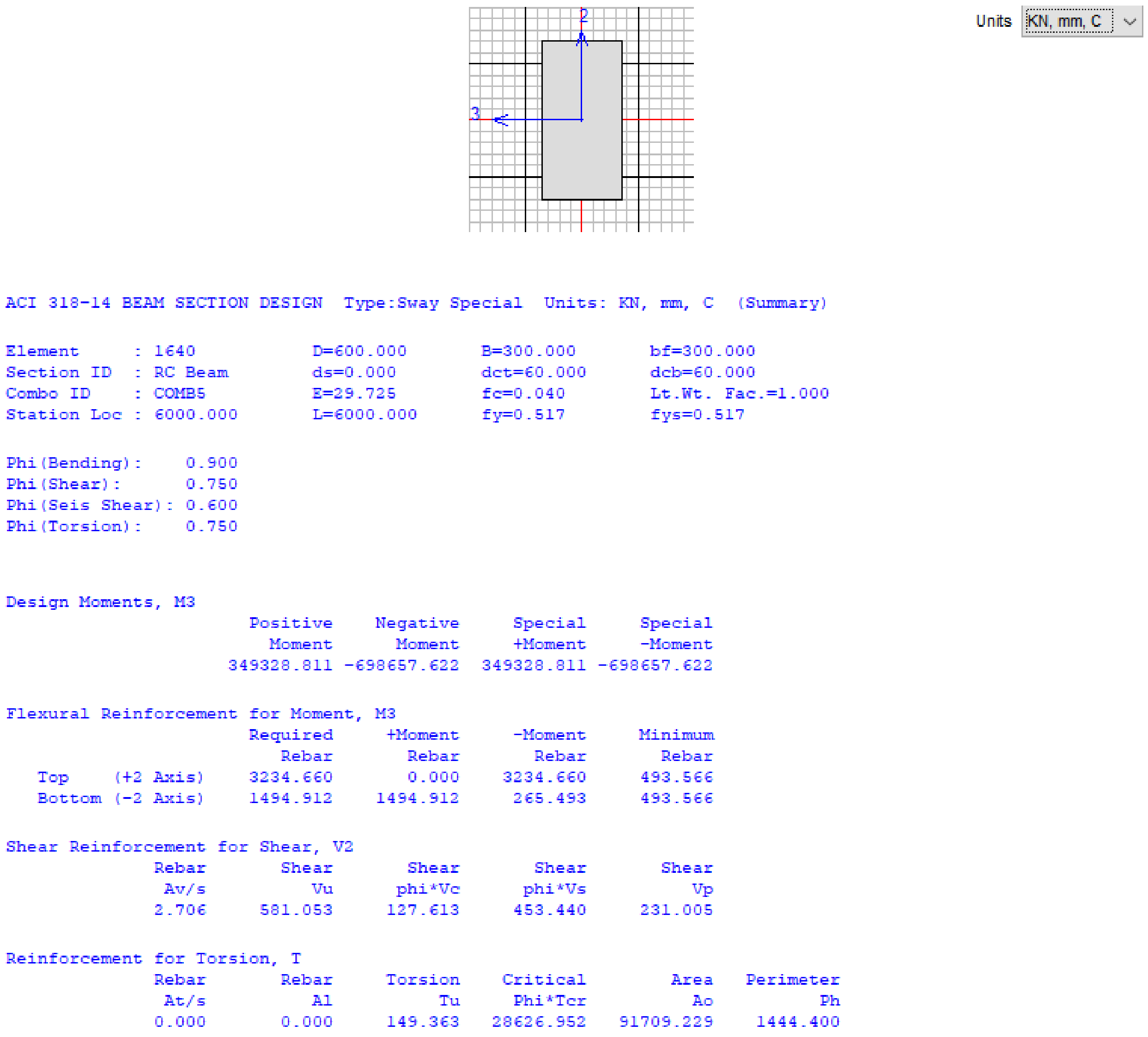

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     227146.078 -174664.406  174664.406 -174664.406

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     719.291       0.000     719.291     493.566
   Bottom (-2 Axis)     945.827     945.827       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.648     236.105     127.613     108.491     231.005

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     149.363   28626.952   91709.229    1444.400
```

Figure 9.16. SAP 2000 Major Beam Design Summary for the 2nd floor (location 3 m)



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

Element      : 1640          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000      L=6000.000      fy=0.517        fys=0.517

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

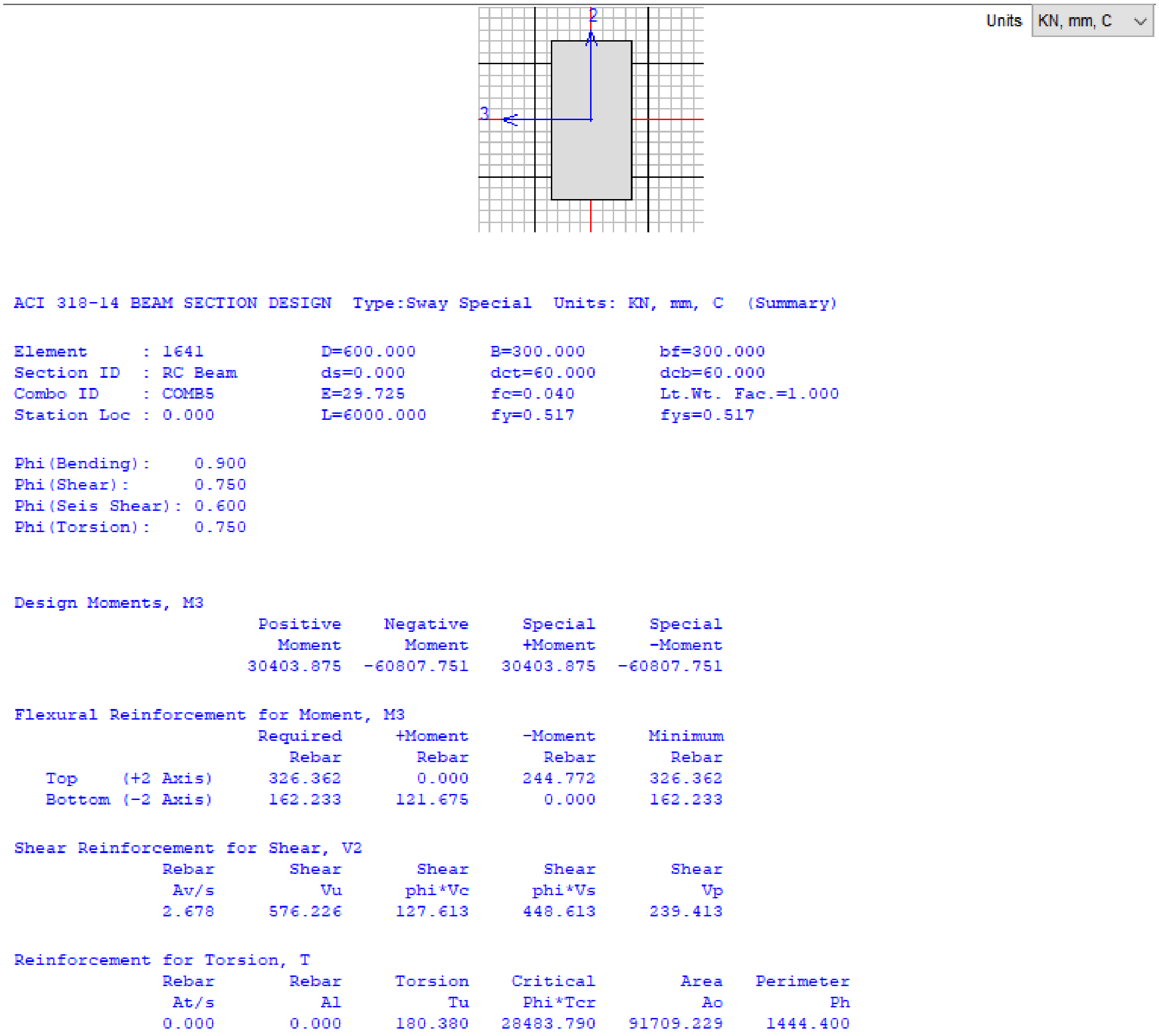

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     349328.811 -698657.622  349328.811 -698657.622

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    3234.660       0.000    3234.660     493.566
   Bottom (-2 Axis)    1494.912    1494.912     265.493     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.706     581.053     127.613     453.440     231.005

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     149.363   28626.952   91709.229    1444.400
```

Figure 9.17. SAP 2000 Major Beam Design Summary for the 2nd floor (location 6 m)



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

Element      : 1641          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000         L=6000.000      fy=0.517        fys=0.517

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

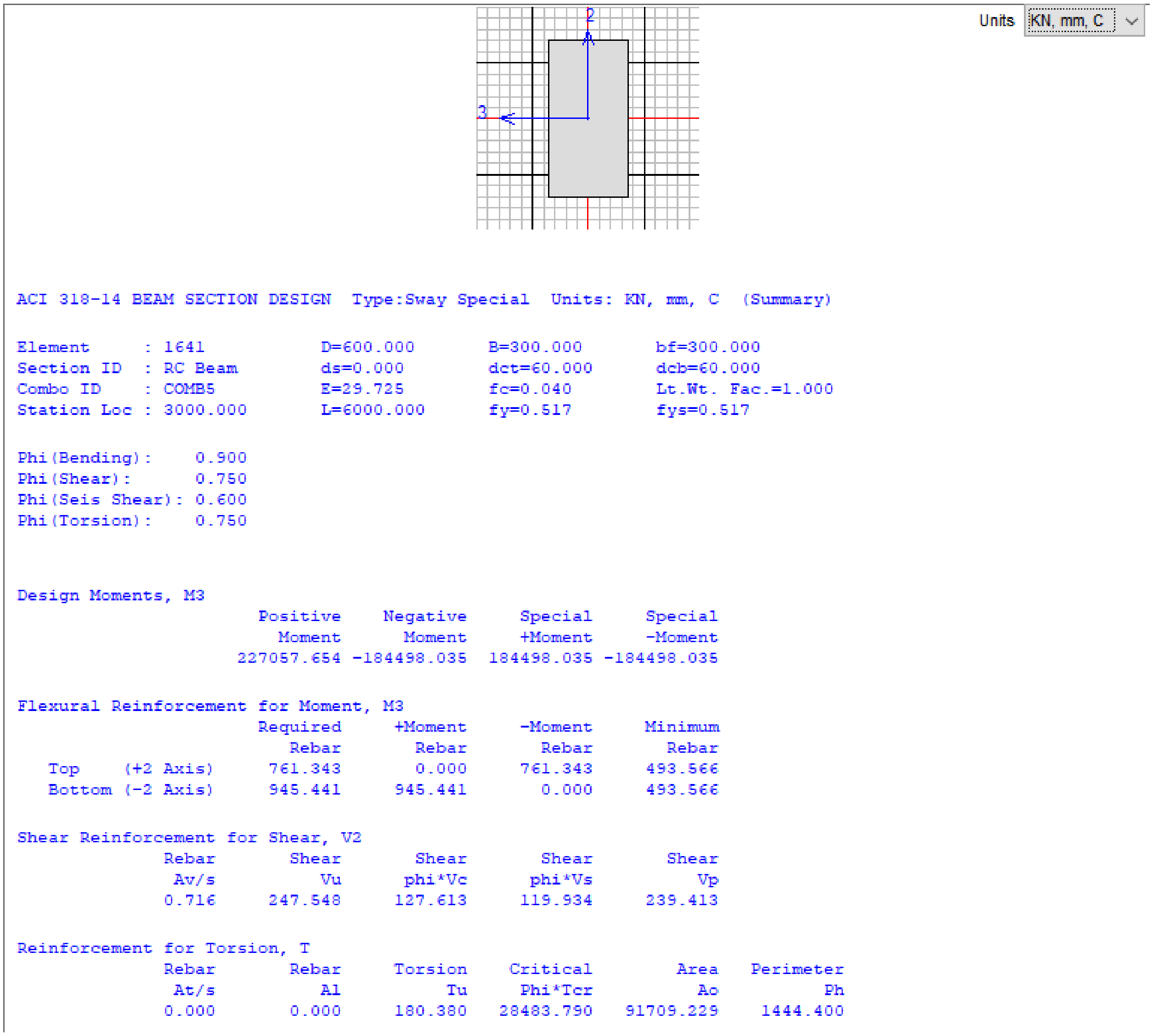

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment      +Moment      -Moment
                      30403.875   -60807.751    30403.875   -60807.751

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)     326.362        0.000      244.772      326.362
   Bottom (-2 Axis)     162.233      121.675        0.000      162.233

Shear Reinforcement for Shear, V2
             Rebar        Shear        Shear        Shear        Shear
              Av/s           Vu      phi*Vc       phi*Vs           Vp
             2.678      576.226      127.613      448.613      239.413

Reinforcement for Torsion, T
             Rebar        Rebar      Torsion     Critical         Area    Perimeter
              At/s           Al           Tu     Phi*Tcr            Ao           Ph
             0.000        0.000      180.380    28483.790    91709.229     1444.400
```

Figure 9.18. SAP 2000 Major Beam Design Summary for the 3rd floor (location 0 m)



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

Element      : 1641           D=600.000        B=300.000        bf=300.000
Section ID   : RC Beam        ds=0.000         dct=60.000       dcb=60.000
Combo ID     : COMB5          E=29.725         fc=0.040         Lt.Wt. Fac.=1.000
Station Loc  : 3000.000       L=6000.000       fy=0.517         fys=0.517

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

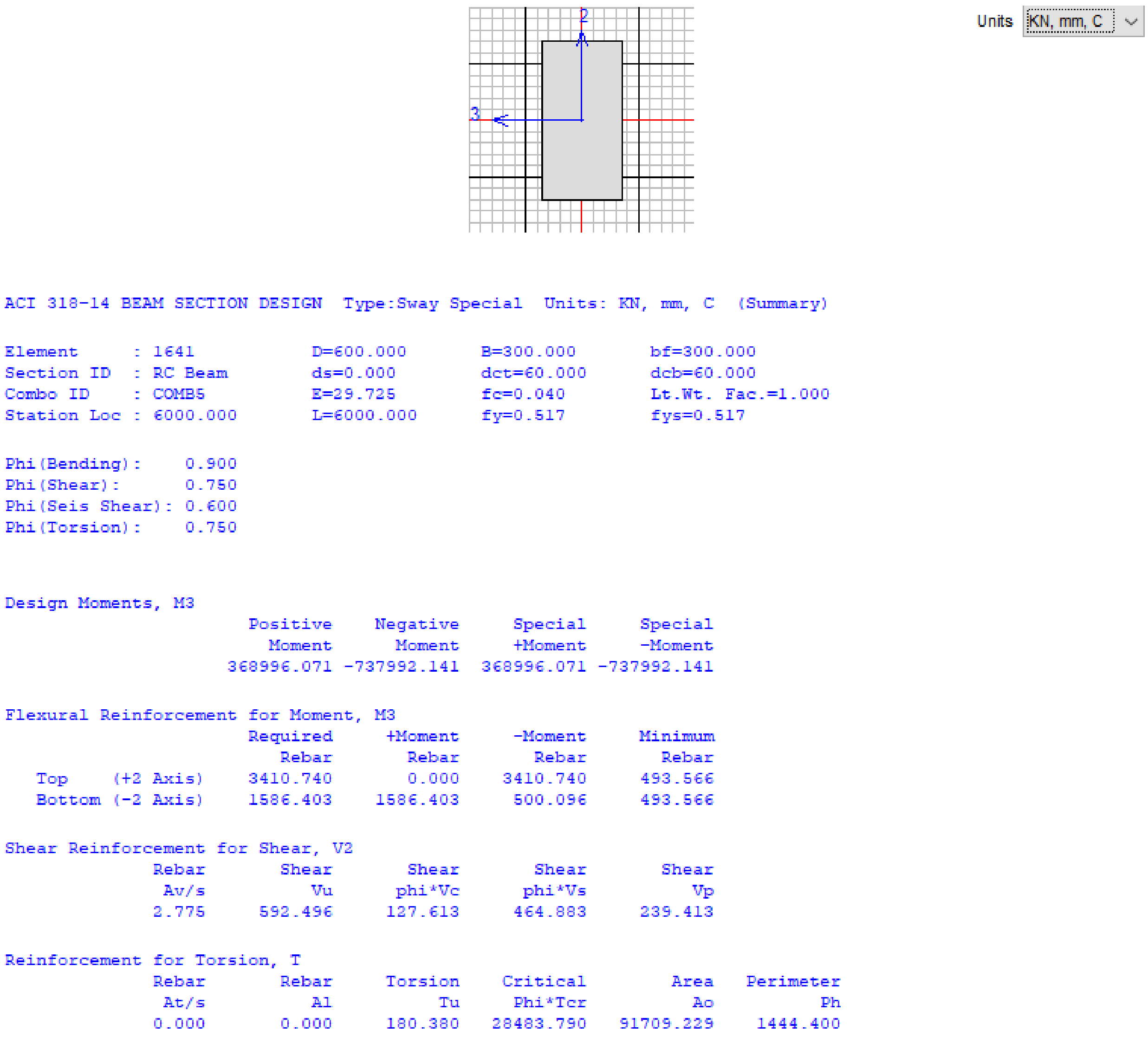

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment      +Moment      -Moment
                     227057.654  -184498.035   184498.035  -184498.035

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)     761.343        0.000      761.343      493.566
   Bottom (-2 Axis)     945.441      945.441        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu       phi*Vc       phi*Vs           Vp
              0.716      247.548      127.613      119.934      239.413

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu      Phi*Tcr           Ao           Ph
              0.000        0.000      180.380    28483.790    91709.229     1444.400
```

Figure 9.19. SAP 2000 Major Beam Design Summary for the 3rd floor (location 3 m)



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

Element     : 1641           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 6000.000       L=6000.000      fy=0.517        fys=0.517

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

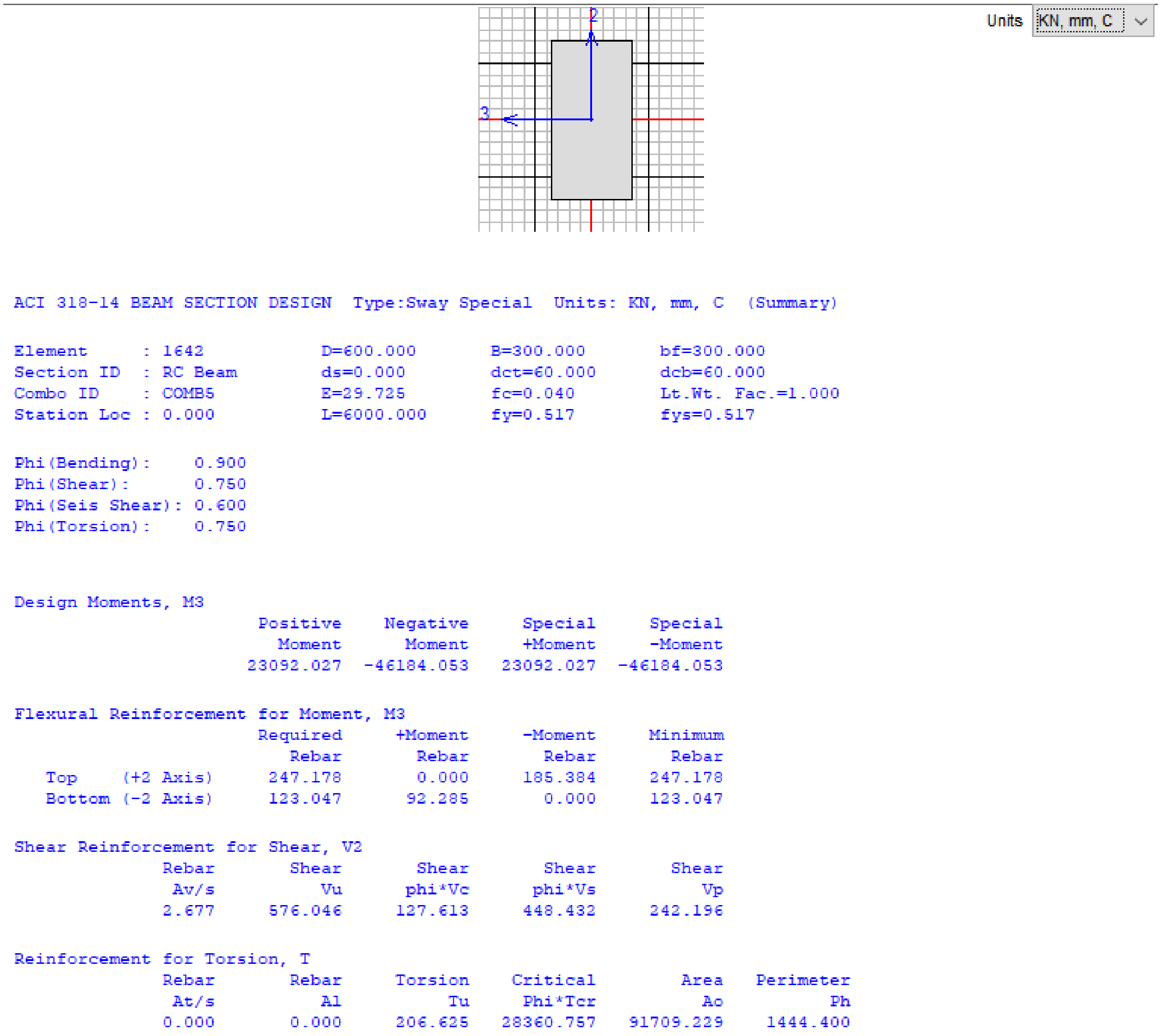

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     368996.071 -737992.141  368996.071 -737992.141

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    3410.740       0.000    3410.740     493.566
   Bottom (-2 Axis)    1586.403    1586.403     500.096     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.775     592.496     127.613     464.883     239.413

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     180.380   28483.790   91709.229    1444.400
```

Figure 9.20. SAP 2000 Major Beam Design Summary for the 3rd floor (location 6 m)



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

Element      : 1642          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000         L=6000.000      fy=0.517        fys=0.517

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

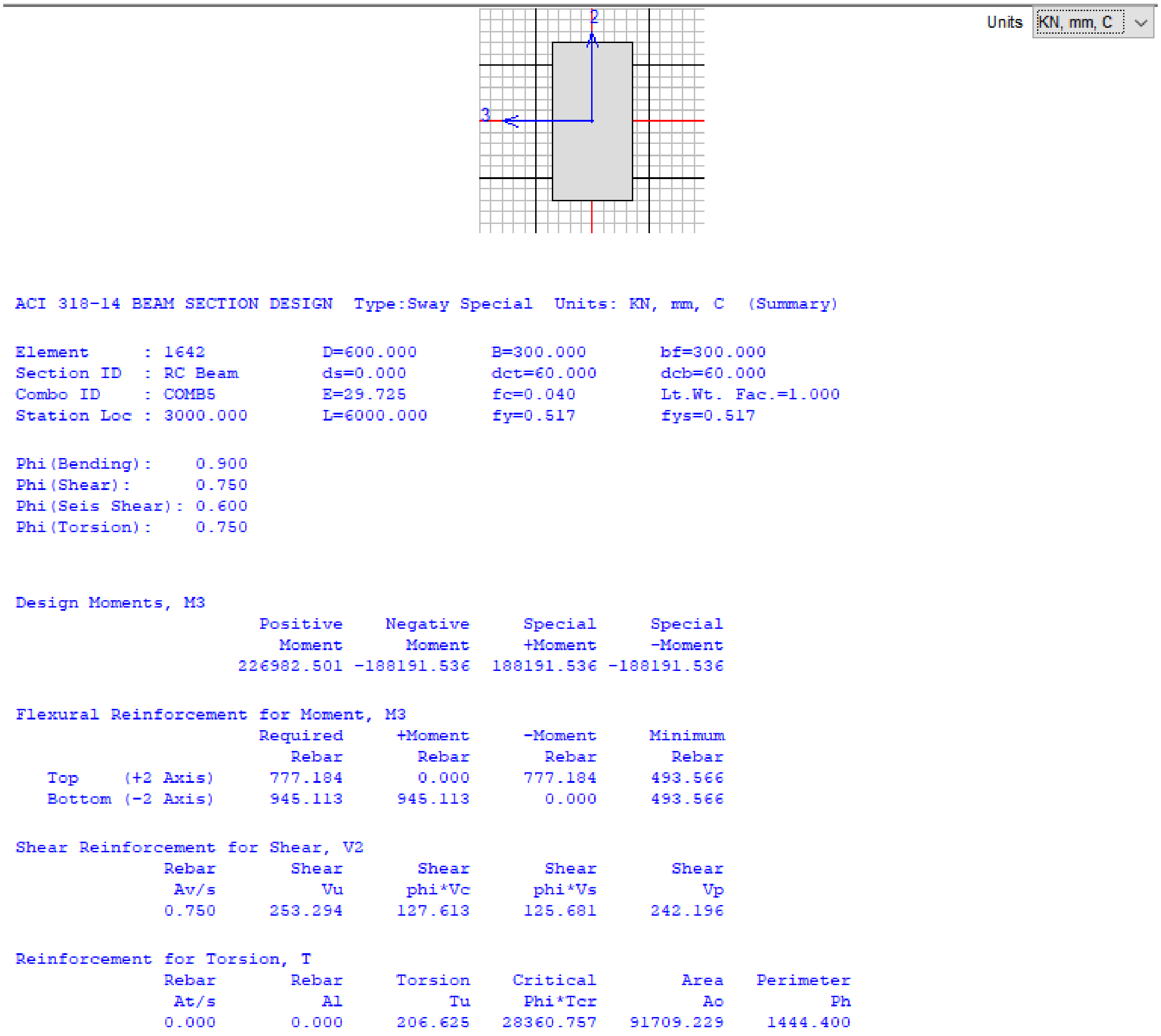

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                      23092.027   -46184.053   23092.027   -46184.053

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)    247.178        0.000      185.384      247.178
   Bottom (-2 Axis)     123.047       92.285        0.000      123.047

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu       phi*Vc       phi*Vs           Vp
              2.677      576.046      127.613      448.432      242.196

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu     Phi*Tcr           Ao           Ph
              0.000        0.000      206.625    28360.757    91709.229     1444.400
```

Figure 9.21. SAP 2000 Major Beam Design Summary for the 4th floor (location 0 m)



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

Element      : 1642           D=600.000        B=300.000        bf=300.000
Section ID   : RC Beam        ds=0.000         dct=60.000       dcb=60.000
Combo ID     : COMB5          E=29.725         fc=0.040         Lt.Wt. Fac.=1.000
Station Loc  : 3000.000       L=6000.000       fy=0.517         fys=0.517

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

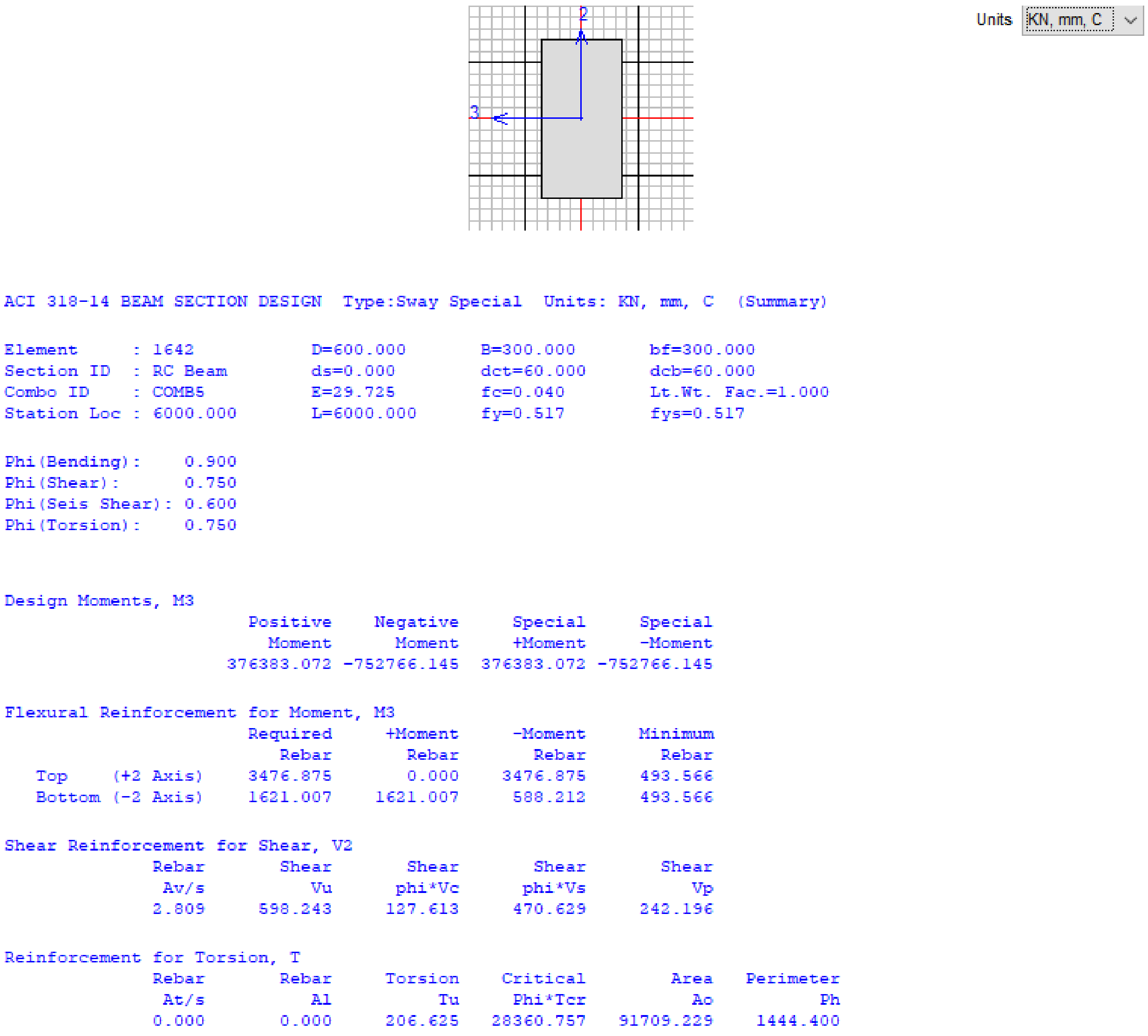

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                     226982.501  -188191.536   188191.536  -188191.536

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)    777.184        0.000      777.184      493.566
   Bottom  (-2 Axis)    945.113      945.113        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu      phi*Vc       phi*Vs           Vp
              0.750      253.294      127.613      125.681      242.196

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu      Phi*Tcr           Ao           Ph
              0.000        0.000      206.625    28360.757    91709.229     1444.400
```

Figure 9.22. SAP 2000 Major Beam Design Summary for the 4th floor (location 3 m)



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

Element      : 1642          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000      L=6000.000      fy=0.517        fys=0.517

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

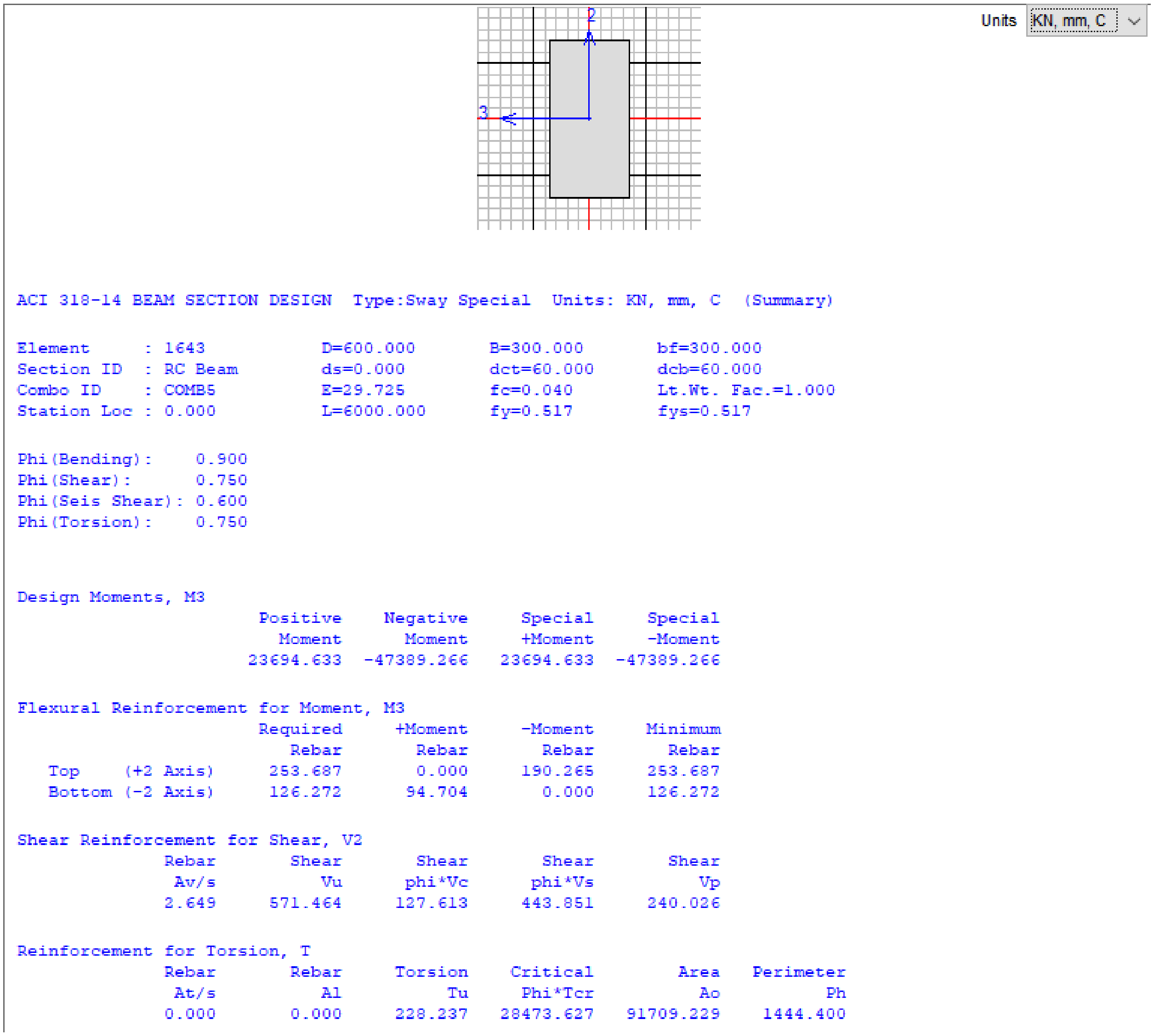

Design Moments, M3
                       Positive    Negative      Special      Special
                         Moment      Moment     +Moment      -Moment
                     376383.072 -752766.145  376383.072 -752766.145

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar       Rebar        Rebar        Rebar
   Top    (+2 Axis)    3476.875       0.000     3476.875      493.566
   Bottom (-2 Axis)    1621.007    1621.007      588.212      493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear        Shear        Shear
               Av/s          Vu     phi*Vc       phi*Vs           Vp
              2.809     598.243     127.613      470.629      242.196

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion     Critical         Area    Perimeter
               At/s          Al          Tu     Phi*Tcr            Ao           Ph
              0.000       0.000     206.625   28360.757     91709.229     1444.400
```

Figure 9.23. SAP 2000 Major Beam Design Summary for the 4th floor (location 6 m)



Units KN, mm, C

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

Element      : 1643           D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000          L=6000.000      fy=0.517        fys=0.517

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

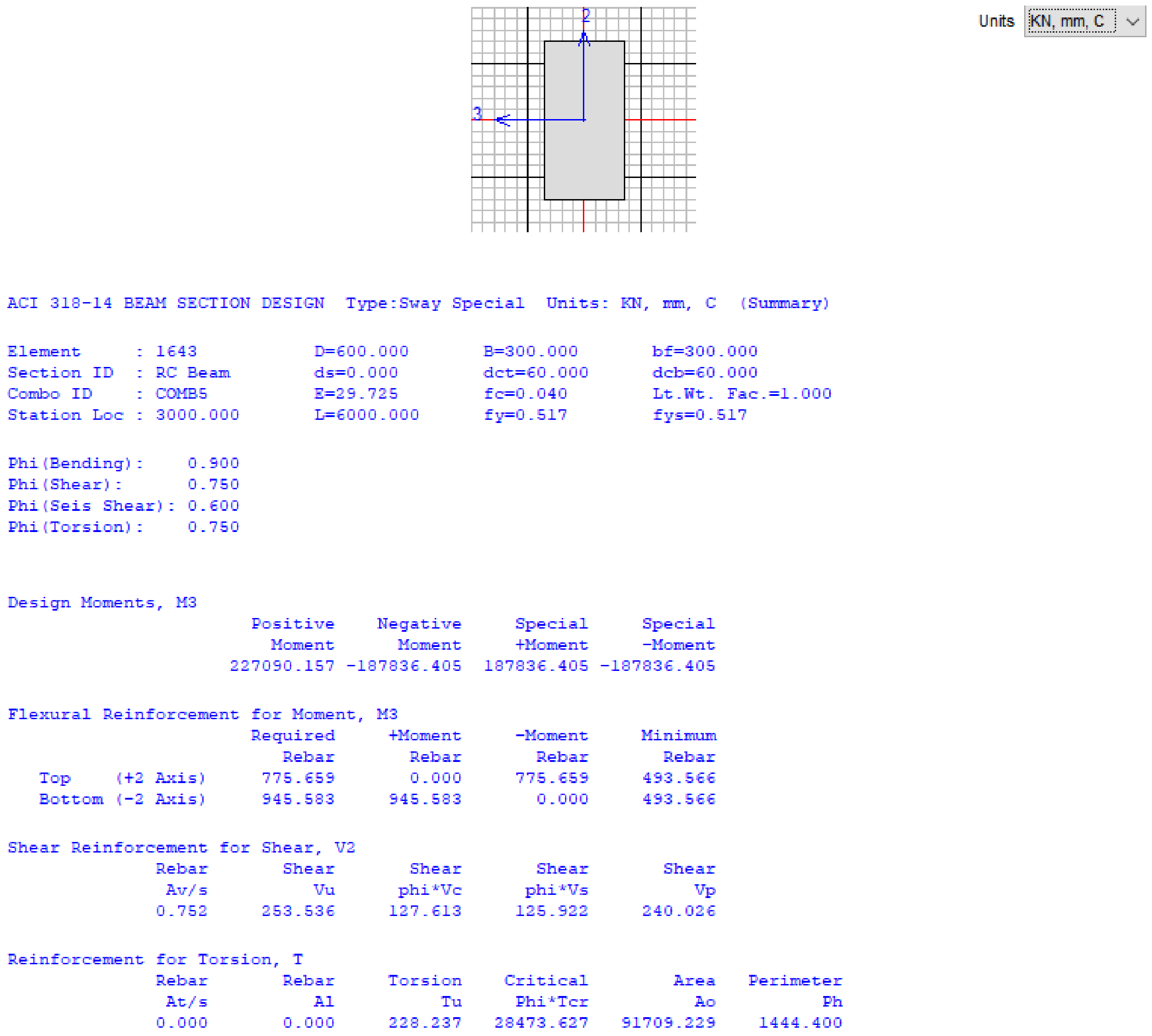

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                      23694.633   -47389.266    23694.633   -47389.266

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)    253.687        0.000      190.265      253.687
   Bottom (-2 Axis)     126.272       94.704        0.000      126.272

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu      phi*Vc       phi*Vs           Vp
              2.649      571.464      127.613      443.851      240.026

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu     Phi*Tcr           Ao           Ph
              0.000        0.000      228.237    28473.627    91709.229     1444.400
```

Figure 9.24. SAP 2000 Major Beam Design Summary for the 5th floor (location 0 m)



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

Element     : 1643           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 3000.000       L=6000.000      fy=0.517        fys=0.517

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

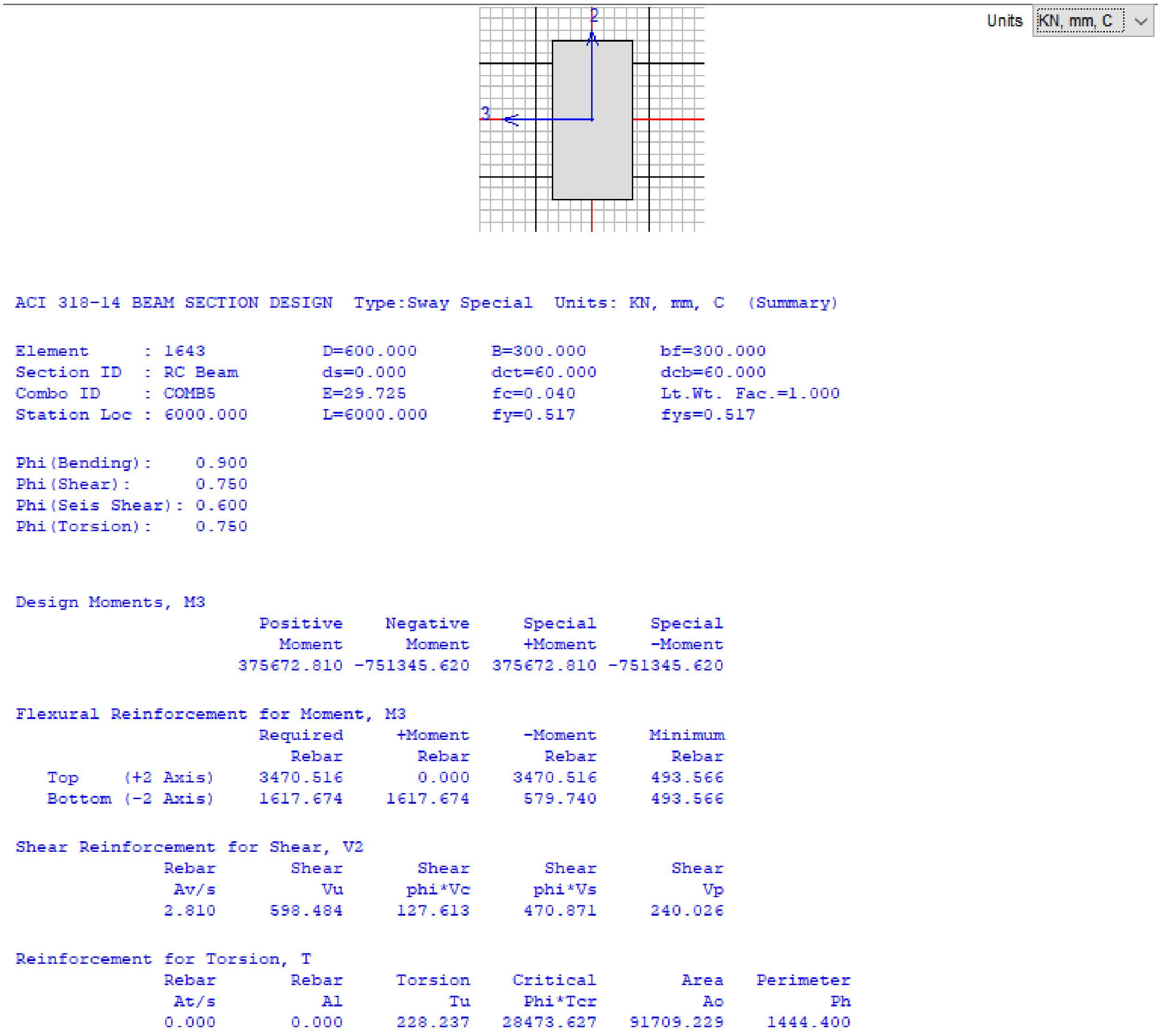

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     227090.157 -187836.405  187836.405 -187836.405

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     775.659       0.000     775.659     493.566
   Bottom (-2 Axis)     945.583     945.583       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.752     253.536     127.613     125.922     240.026

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     228.237   28473.627   91709.229    1444.400
```

Figure 9.25. SAP 2000 Major Beam Design Summary for the 5th floor (location 3 m)



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

Element      : 1643           D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000       L=6000.000      fy=0.517        fys=0.517

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

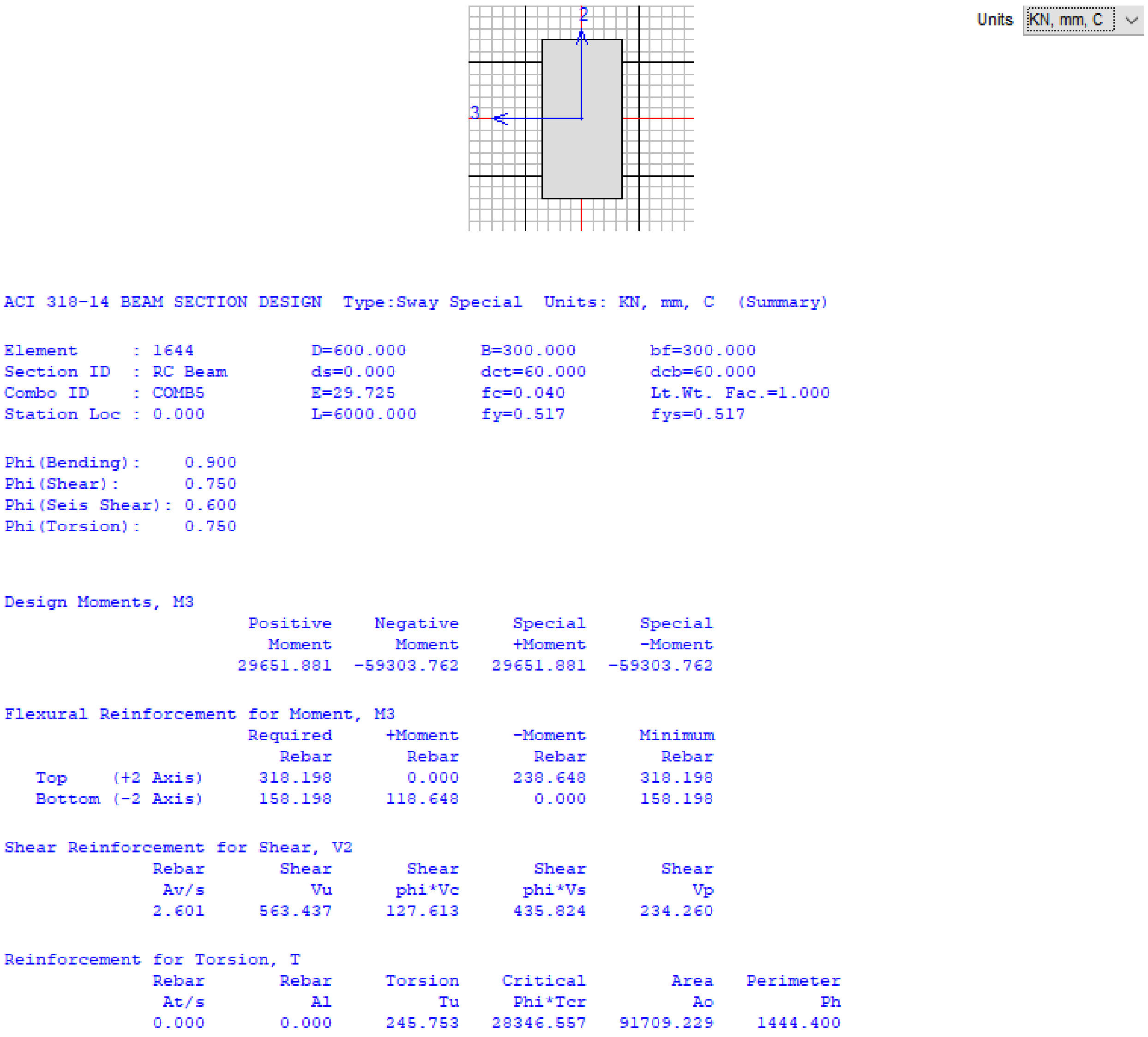

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     375672.810 -751345.620  375672.810 -751345.620

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    3470.516       0.000    3470.516     493.566
   Bottom (-2 Axis)    1617.674    1617.674     579.740     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.810     598.484     127.613     470.871     240.026

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     228.237   28473.627   91709.229    1444.400
```

Figure 9.26. SAP 2000 Major Beam Design Summary for the 5th floor (location 6 m)



Units KN, mm, C

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

Element      : 1644           D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000          L=6000.000      fy=0.517        fys=0.517

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

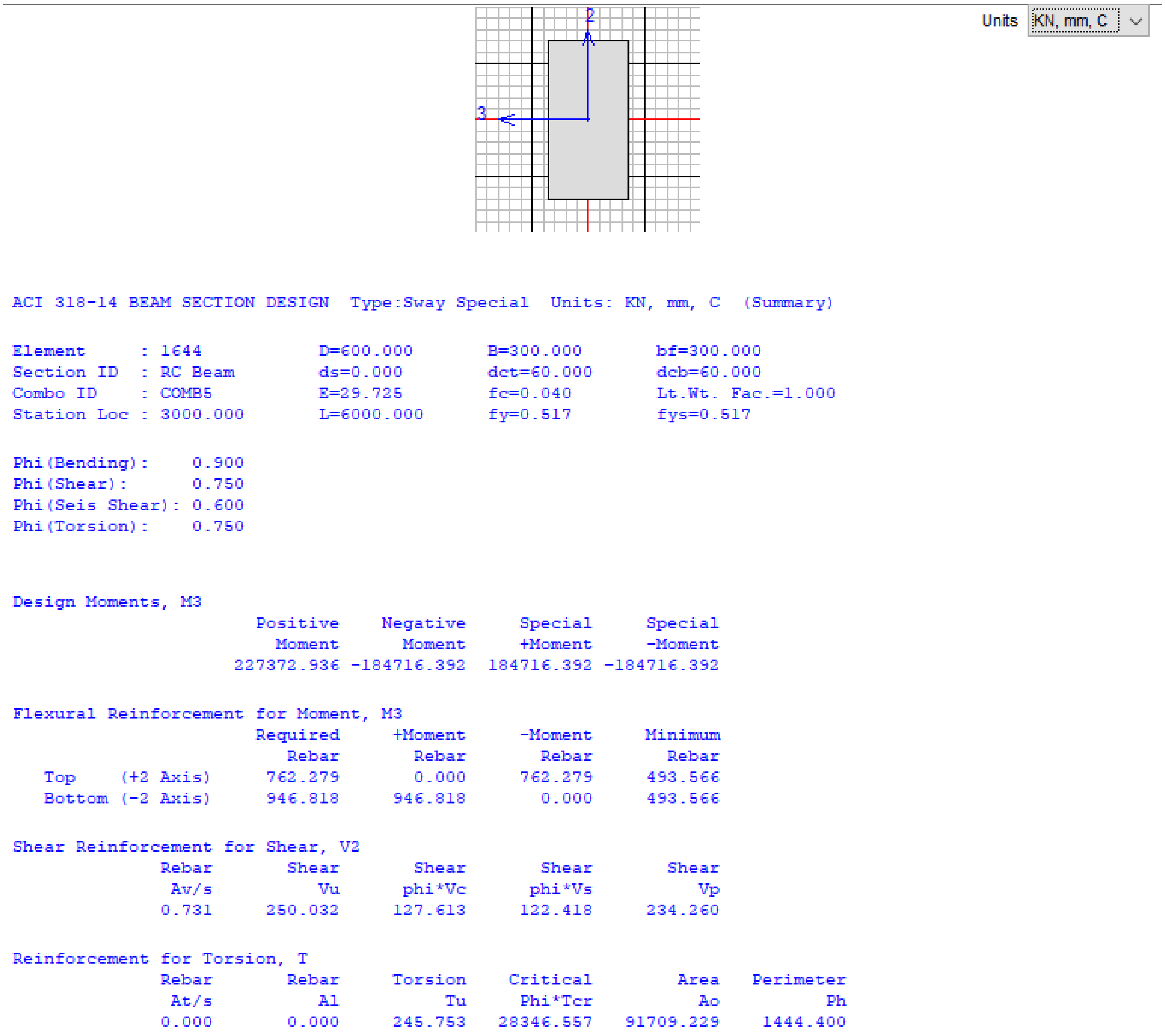

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                      29651.881   -59303.762   29651.881   -59303.762

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)    318.198        0.000      238.648      318.198
   Bottom (-2 Axis)     158.198      118.648        0.000      158.198

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu       phi*Vc       phi*Vs           Vp
              2.601      563.437      127.613      435.824      234.260

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu      Phi*Tcr           Ao           Ph
              0.000        0.000      245.753    28346.557    91709.229     1444.400
```

Figure 9.27. SAP 2000 Major Beam Design Summary for the 6th floor (location 0 m)



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

Element     : 1644           D=600.000      B=300.000      bf=300.000
Section ID  : RC Beam        ds=0.000       dct=60.000     dcb=60.000
Combo ID    : COMB5          E=29.725       fc=0.040       Lt.Wt. Fac.=1.000
Station Loc : 3000.000       L=6000.000     fy=0.517       fys=0.517

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

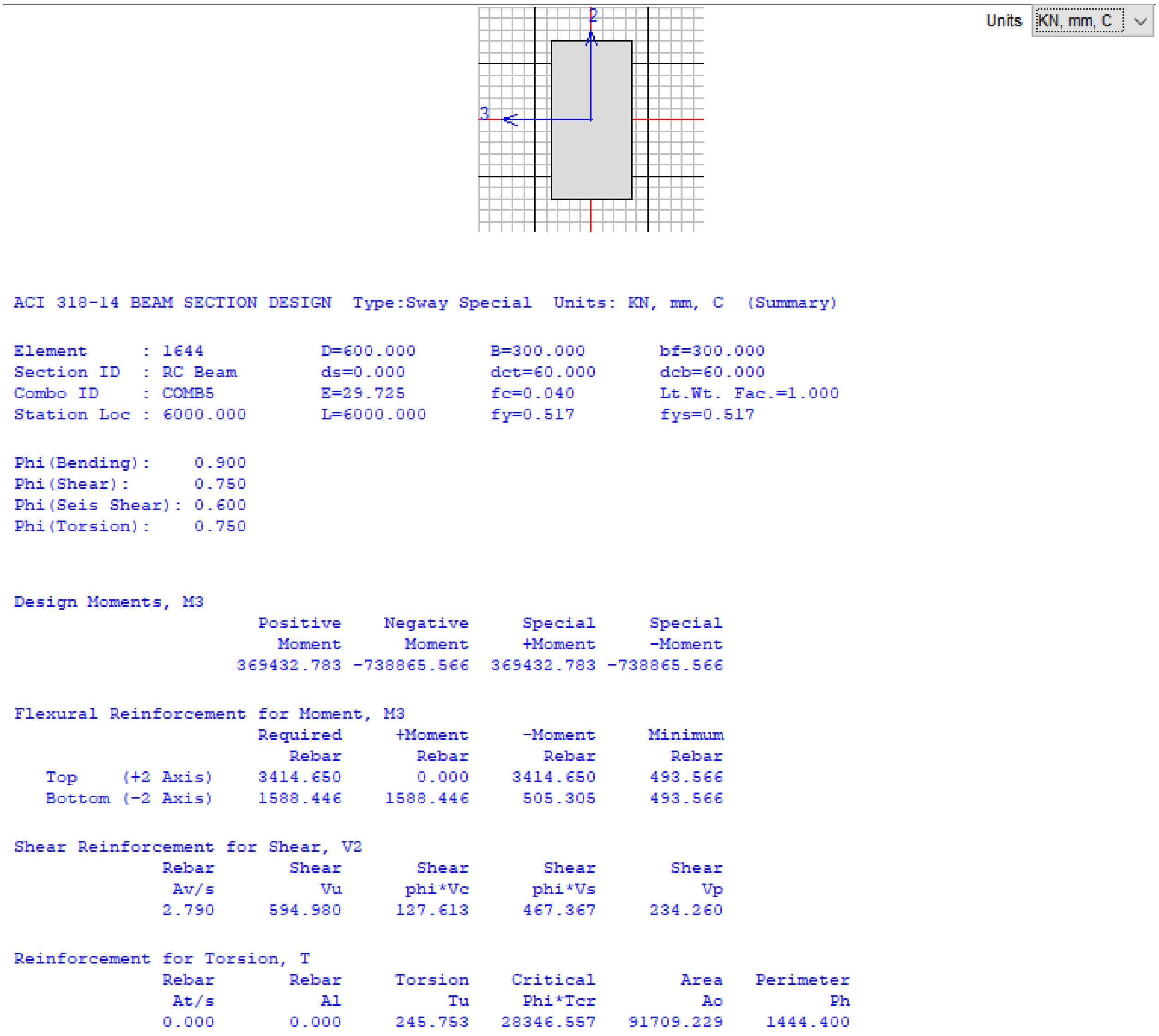

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     227372.936 -184716.392  184716.392 -184716.392

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     762.279       0.000     762.279     493.566
   Bottom (-2 Axis)     946.818     946.818       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.731     250.032     127.613     122.418     234.260

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     245.753   28346.557   91709.229    1444.400
```

Figure 9.28. SAP 2000 Major Beam Design Summary for the 6th floor (location 3 m)



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

Element      : 1644           D=600.000        B=300.000        bf=300.000
Section ID   : RC Beam        ds=0.000         dct=60.000       dcb=60.000
Combo ID     : COMB5          E=29.725         fc=0.040         Lt.Wt. Fac.=1.000
Station Loc  : 6000.000       L=6000.000       fy=0.517         fys=0.517

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

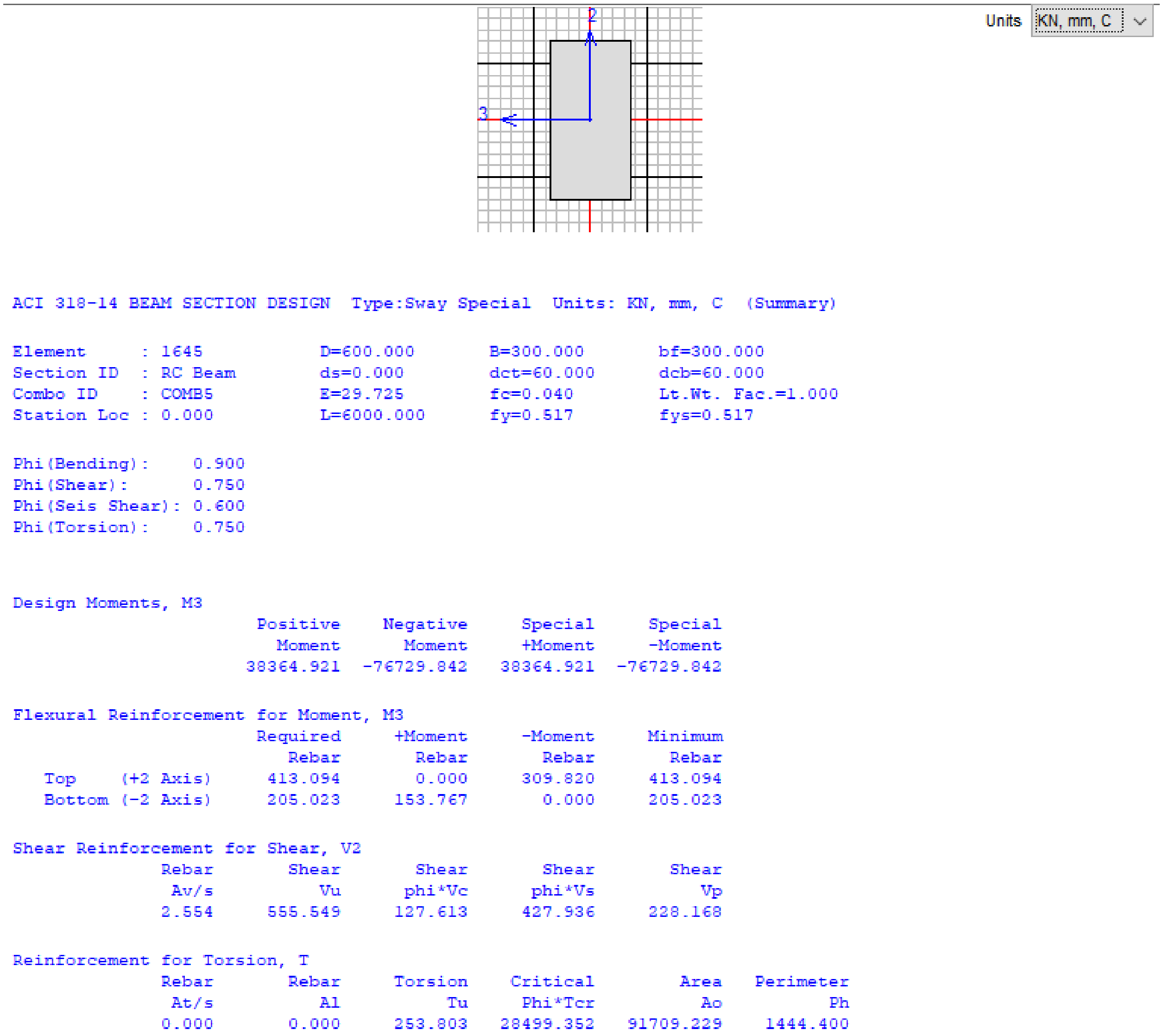

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment      +Moment      -Moment
                     369432.783  -738865.566   369432.783  -738865.566

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)    3414.650        0.000     3414.650      493.566
   Bottom (-2 Axis)    1588.446     1588.446      505.305      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu       phi*Vc       phi*Vs           Vp
              2.790      594.980      127.613      467.367      234.260

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu      Phi*Tcr           Ao           Ph
              0.000        0.000      245.753    28346.557    91709.229     1444.400
```

Figure 9.29. SAP 2000 Major Beam Design Summary for the 6th floor (location 6 m)



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

Element      : 1645          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000         L=6000.000      fy=0.517        fys=0.517

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

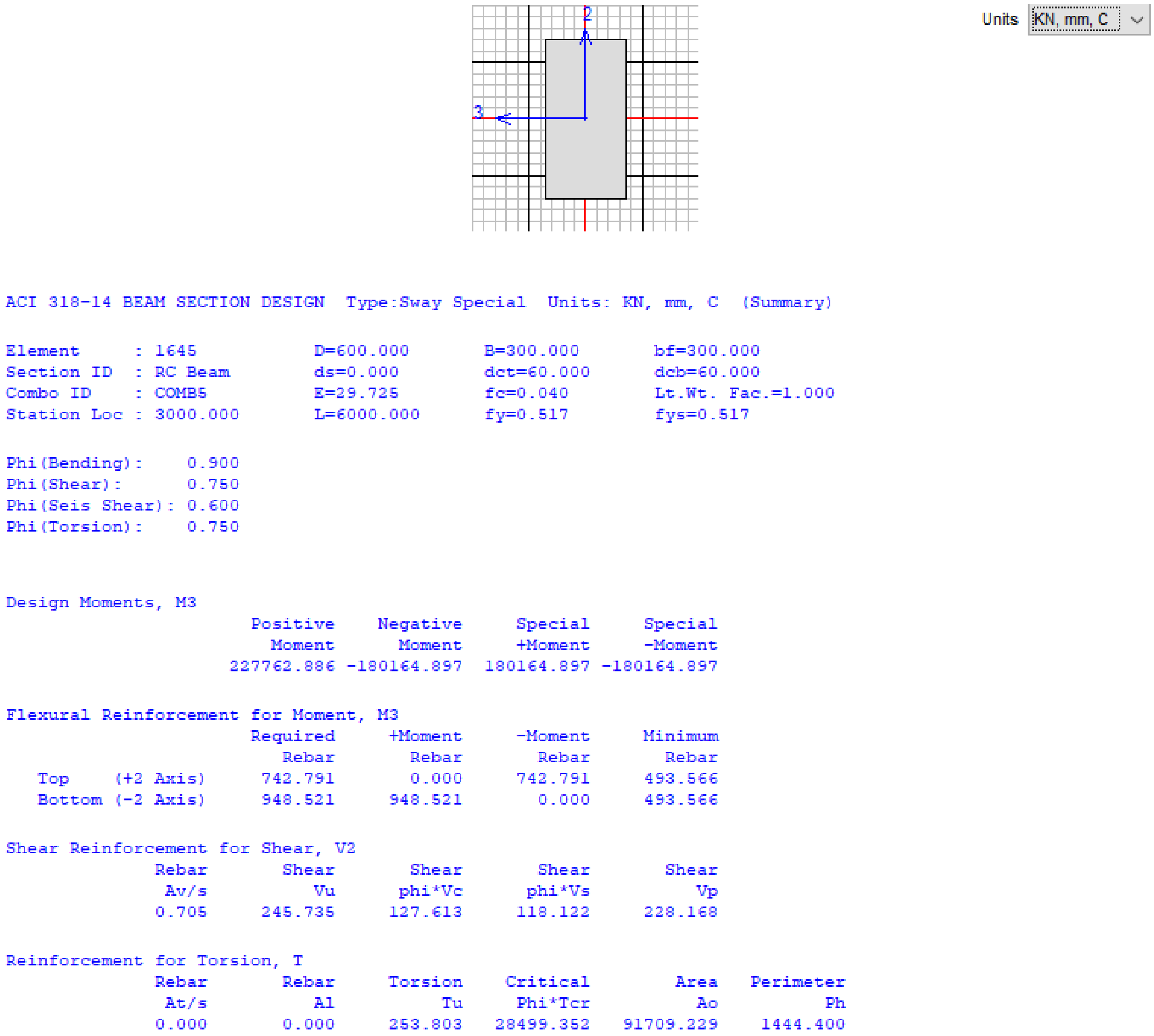

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                      38364.921   -76729.842   38364.921   -76729.842

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)     413.094        0.000      309.820      413.094
   Bottom (-2 Axis)     205.023      153.767        0.000      205.023

Shear Reinforcement for Shear, V2
              Rebar       Shear        Shear        Shear        Shear
               Av/s          Vu       phi*Vc       phi*Vs           Vp
              2.554     555.549      127.613      427.936      228.168

Reinforcement for Torsion, T
              Rebar       Rebar      Torsion     Critical         Area    Perimeter
               At/s          Al           Tu     Phi*Tcr            Ao           Ph
              0.000       0.000      253.803   28499.352    91709.229     1444.400
```

Figure 9.30. SAP 2000 Major Beam Design Summary for the 7th floor (location 0 m)



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

Element      : 1645          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 3000.000      L=6000.000      fy=0.517        fys=0.517

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

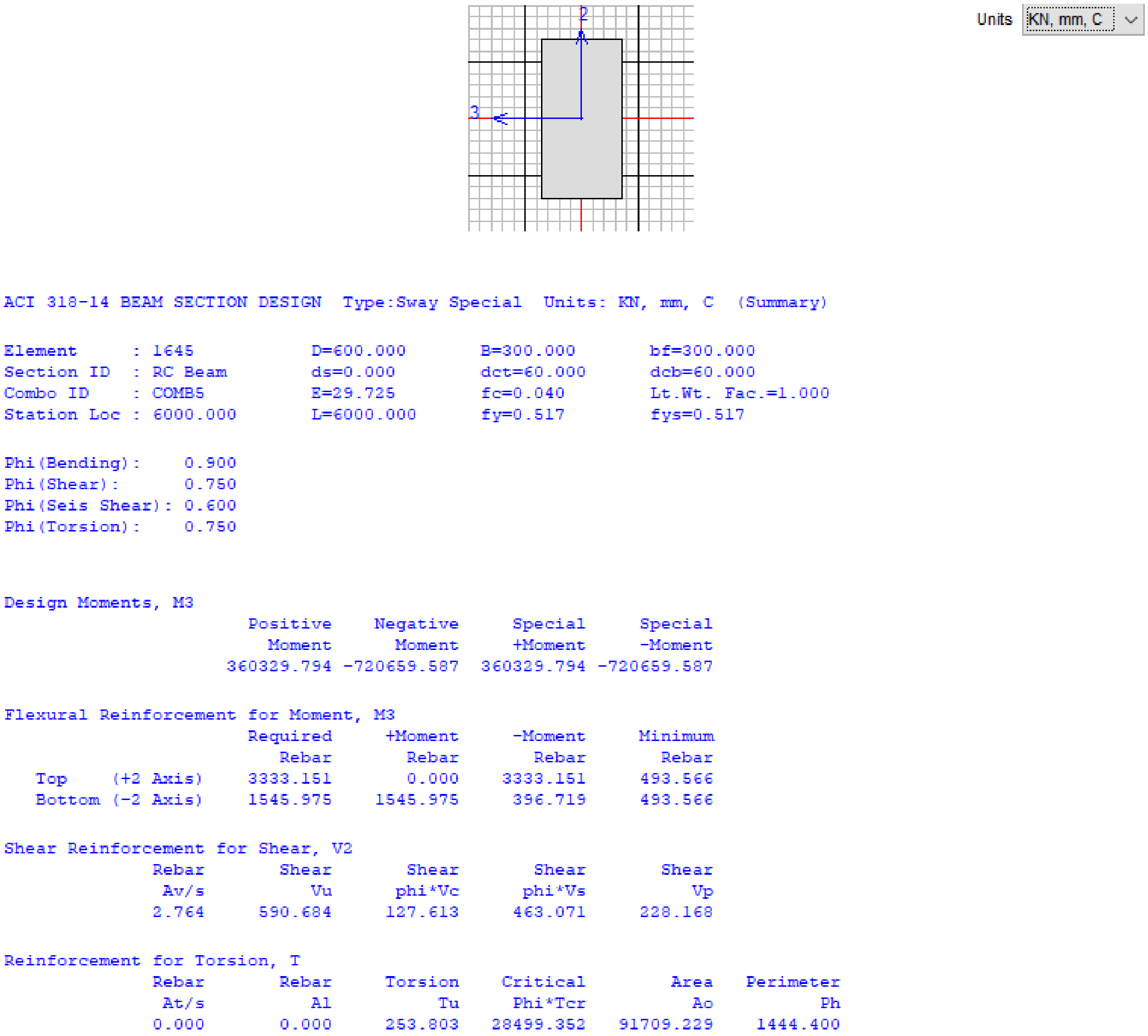

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                     227762.886 -180164.897   180164.897  -180164.897

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)    742.791        0.000      742.791      493.566
   Bottom  (-2 Axis)    948.521      948.521        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu      phi*Vc       phi*Vs           Vp
              0.705      245.735      127.613      118.122      228.168

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu     Phi*Tcr           Ao           Ph
              0.000        0.000      253.803   28499.352    91709.229     1444.400
```

Figure 9.31. SAP 2000 Major Beam Design Summary for the 7th floor (location 3 m)



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

Element     : 1645           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 6000.000       L=6000.000      fy=0.517        fys=0.517

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

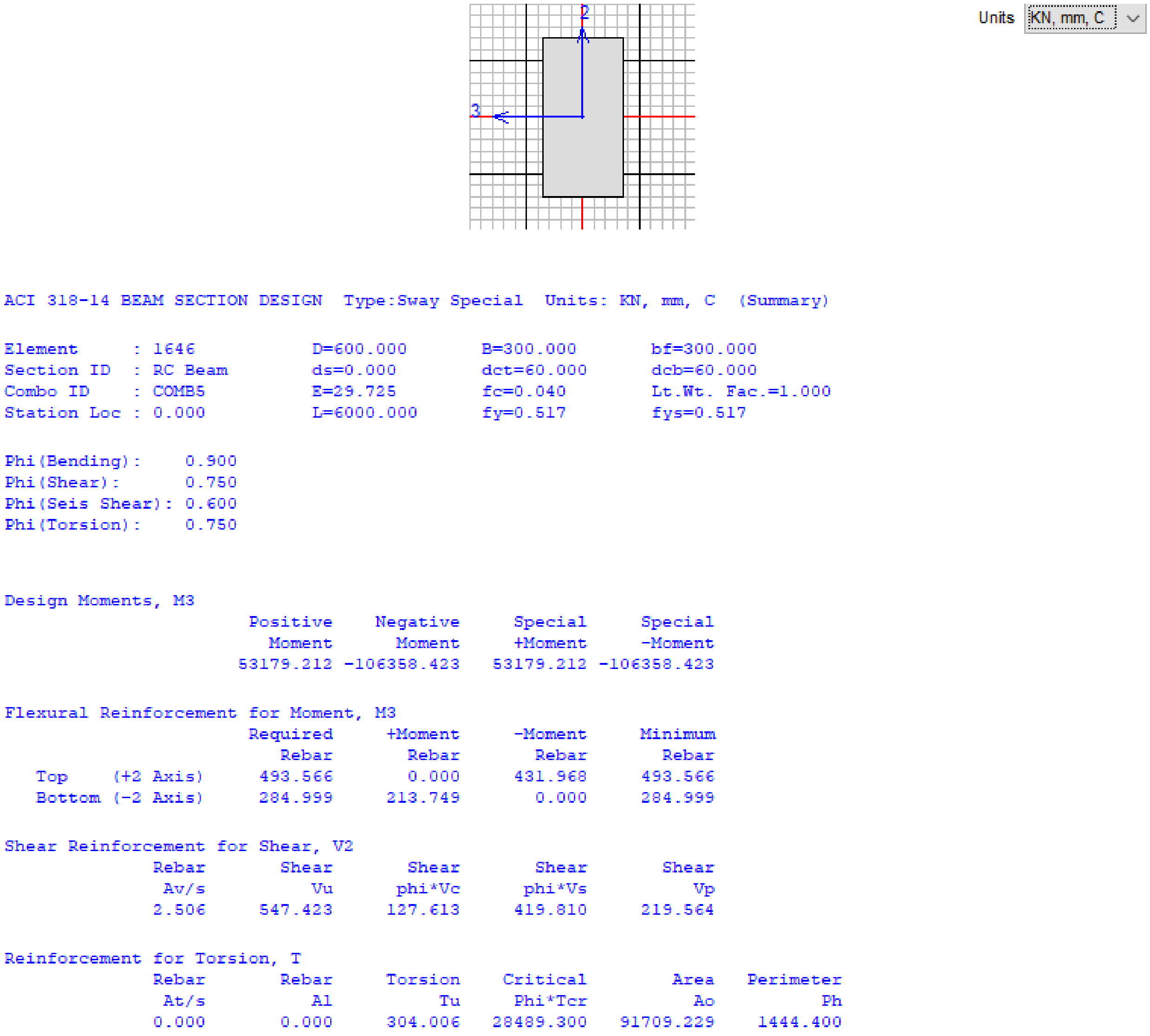

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     360329.794 -720659.587  360329.794 -720659.587

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    3333.151       0.000    3333.151     493.566
   Bottom (-2 Axis)    1545.975    1545.975     396.719     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.764     590.684     127.613     463.071     228.168

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     253.803   28499.352   91709.229    1444.400
```

Figure 9.32. SAP 2000 Major Beam Design Summary for the 7th floor (location 6 m)



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

Element      : 1646           D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000          L=6000.000      fy=0.517        fys=0.517

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

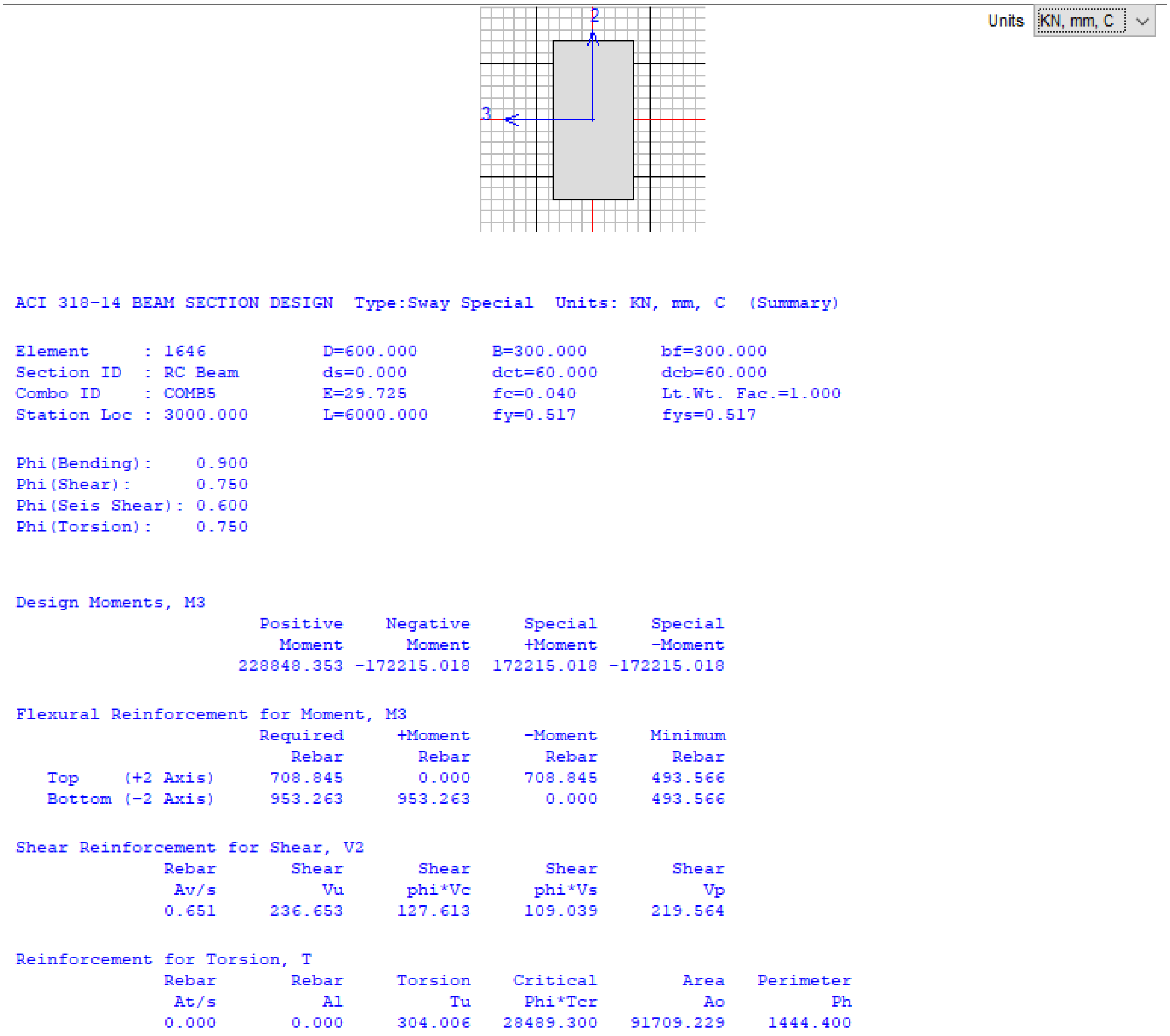

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                      53179.212 -106358.423    53179.212 -106358.423

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top    (+2 Axis)     493.566        0.000      431.968      493.566
   Bottom (-2 Axis)     284.999      213.749        0.000      284.999

Shear Reinforcement for Shear, V2
              Rebar       Shear        Shear        Shear        Shear
               Av/s          Vu       phi*Vc       phi*Vs           Vp
              2.506     547.423      127.613      419.810      219.564

Reinforcement for Torsion, T
              Rebar       Rebar      Torsion     Critical         Area    Perimeter
               At/s          Al           Tu      Phi*Tcr           Ao           Ph
              0.000       0.000      304.006    28489.300    91709.229     1444.400
```

Figure 9.33. SAP 2000 Major Beam Design Summary for the 8th floor (location 0 m)



Units KN, mm, C

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

Element     : 1646           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 3000.000       L=6000.000      fy=0.517        fys=0.517

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

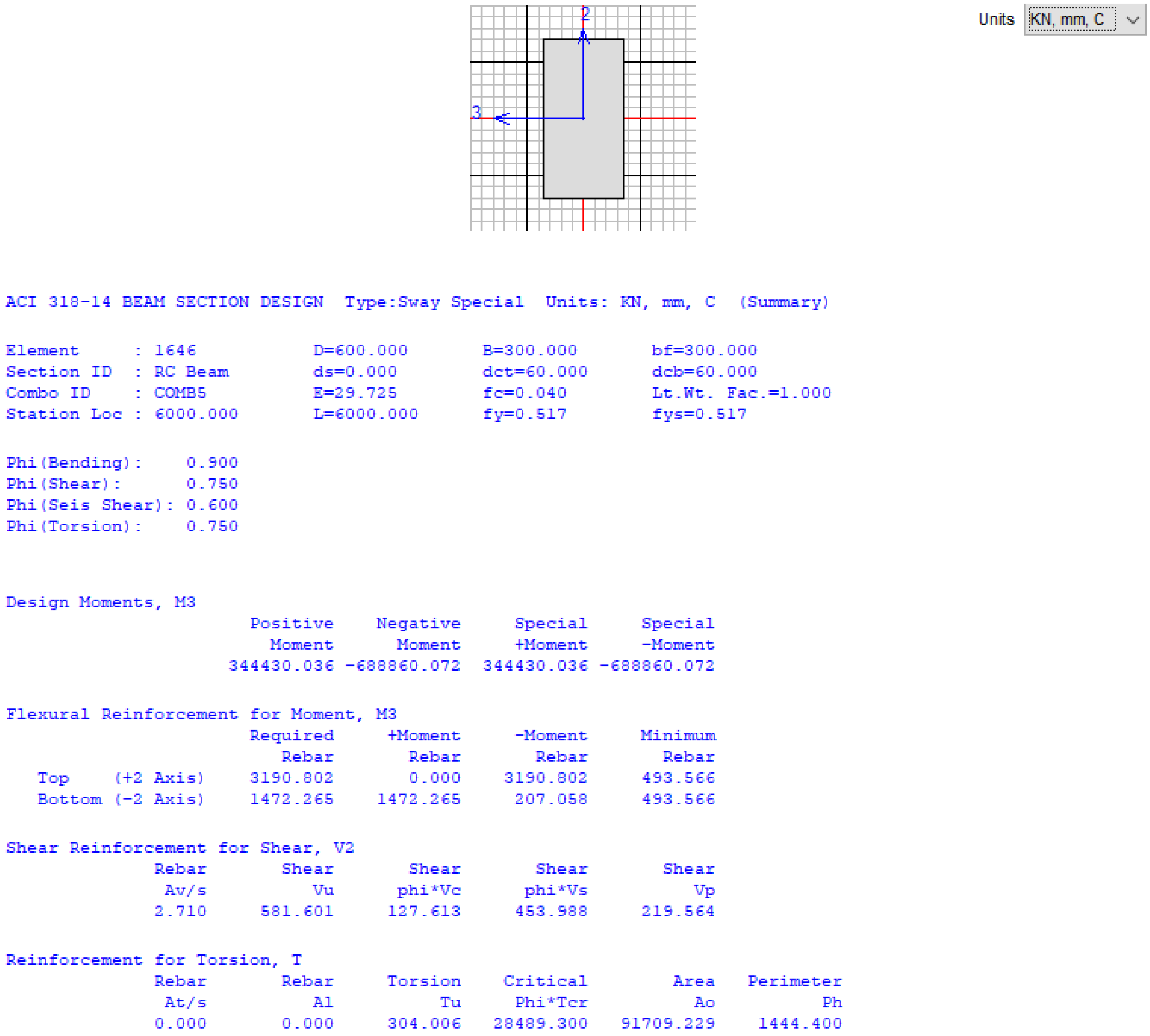

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     228848.353 -172215.018  172215.018 -172215.018

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top     (+2 Axis)    708.845       0.000     708.845     493.566
   Bottom (-2 Axis)     953.263     953.263       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.651     236.653     127.613     109.039     219.564

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     304.006   28489.300   91709.229    1444.400
```

Figure 9.34. SAP 2000 Major Beam Design Summary for the 8th floor (location 3 m)



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

Element      : 1646           D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000       L=6000.000      fy=0.517        fys=0.517

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

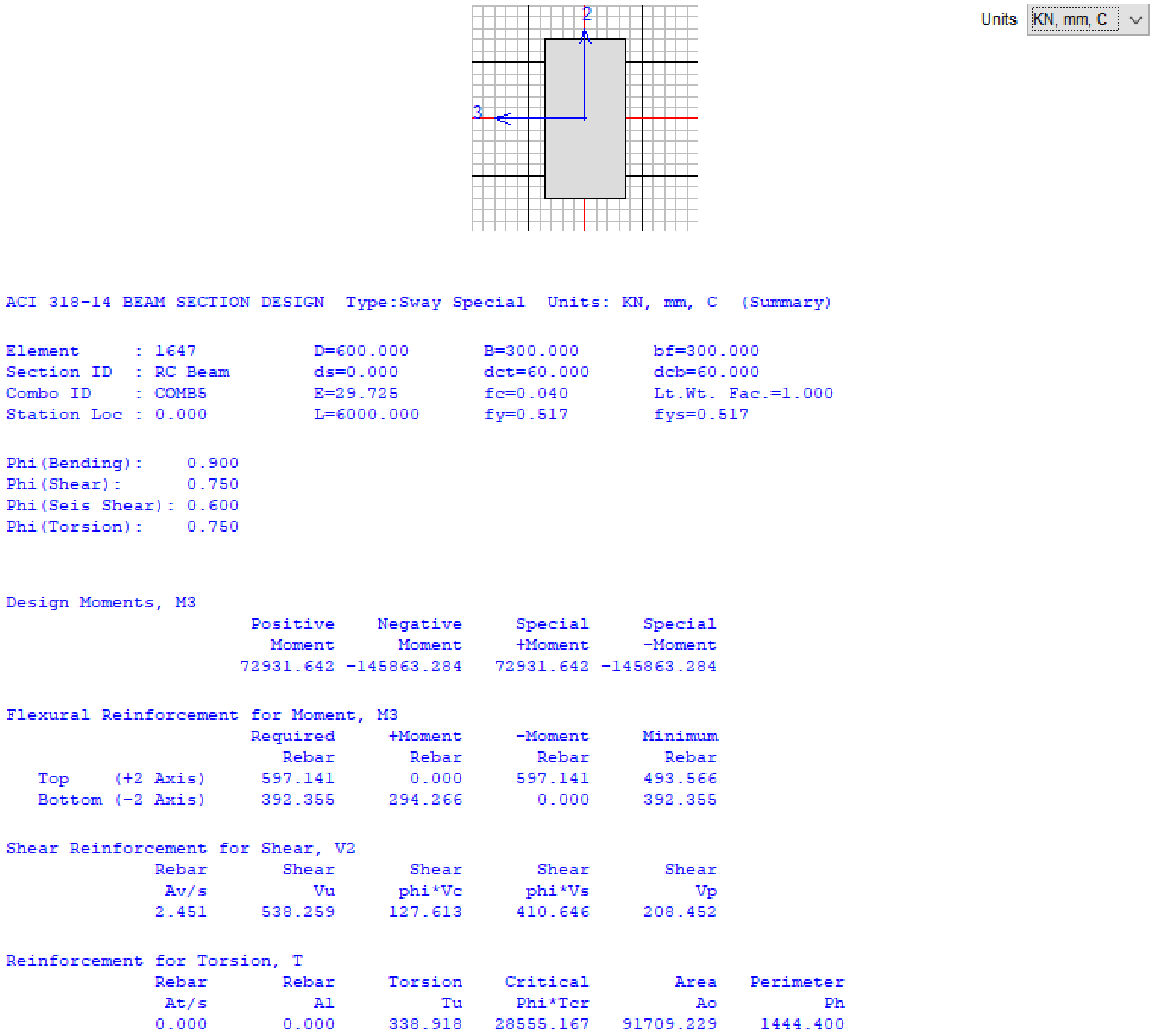

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     344430.036 -688860.072  344430.036 -688860.072

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    3190.802       0.000    3190.802     493.566
   Bottom (-2 Axis)    1472.265    1472.265     207.058     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.710     581.601     127.613     453.988     219.564

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     304.006   28489.300   91709.229    1444.400
```

Figure 9.35. SAP 2000 Major Beam Design Summary for the 8th floor (location 6 m)



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

Element      : 1647          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 0.000         L=6000.000      fy=0.517        fys=0.517

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

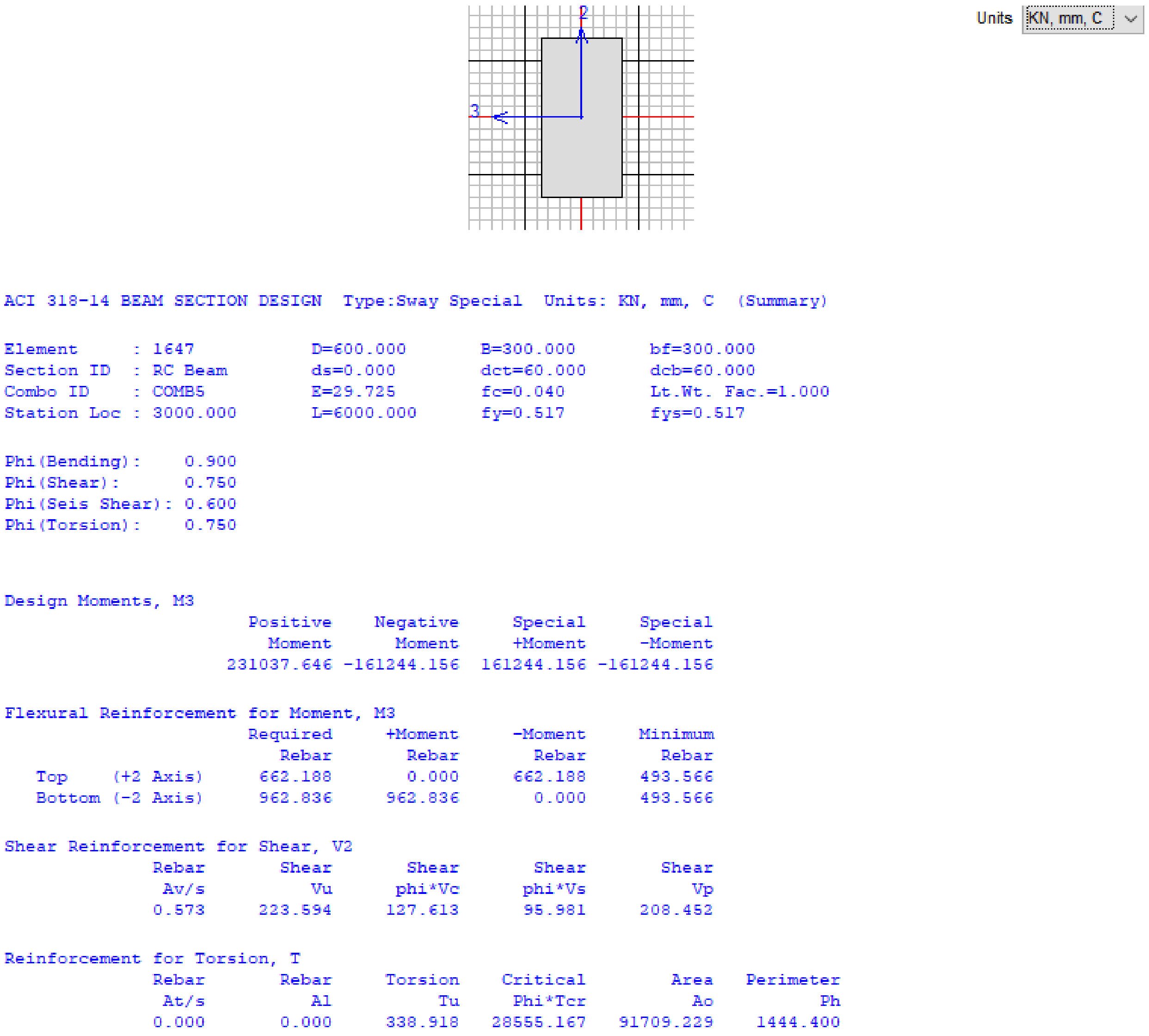

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                      72931.642 -145863.284   72931.642 -145863.284

Flexural Reinforcement for Moment, M3
                       Required    +Moment     -Moment     Minimum
                          Rebar      Rebar       Rebar       Rebar
   Top     (+2 Axis)    597.141      0.000     597.141     493.566
   Bottom (-2 Axis)     392.355    294.266       0.000     392.355

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu     phi*Vc      phi*Vs          Vp
              2.451     538.259     127.613     410.646     208.452

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu    Phi*Tcr           Ao          Ph
              0.000       0.000     338.918   28555.167   91709.229    1444.400
```

Figure 9.36. SAP 2000 Major Beam Design Summary for the 9th floor (location 0 m)



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

Element     : 1647          D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 3000.000      L=6000.000      fy=0.517        fys=0.517

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

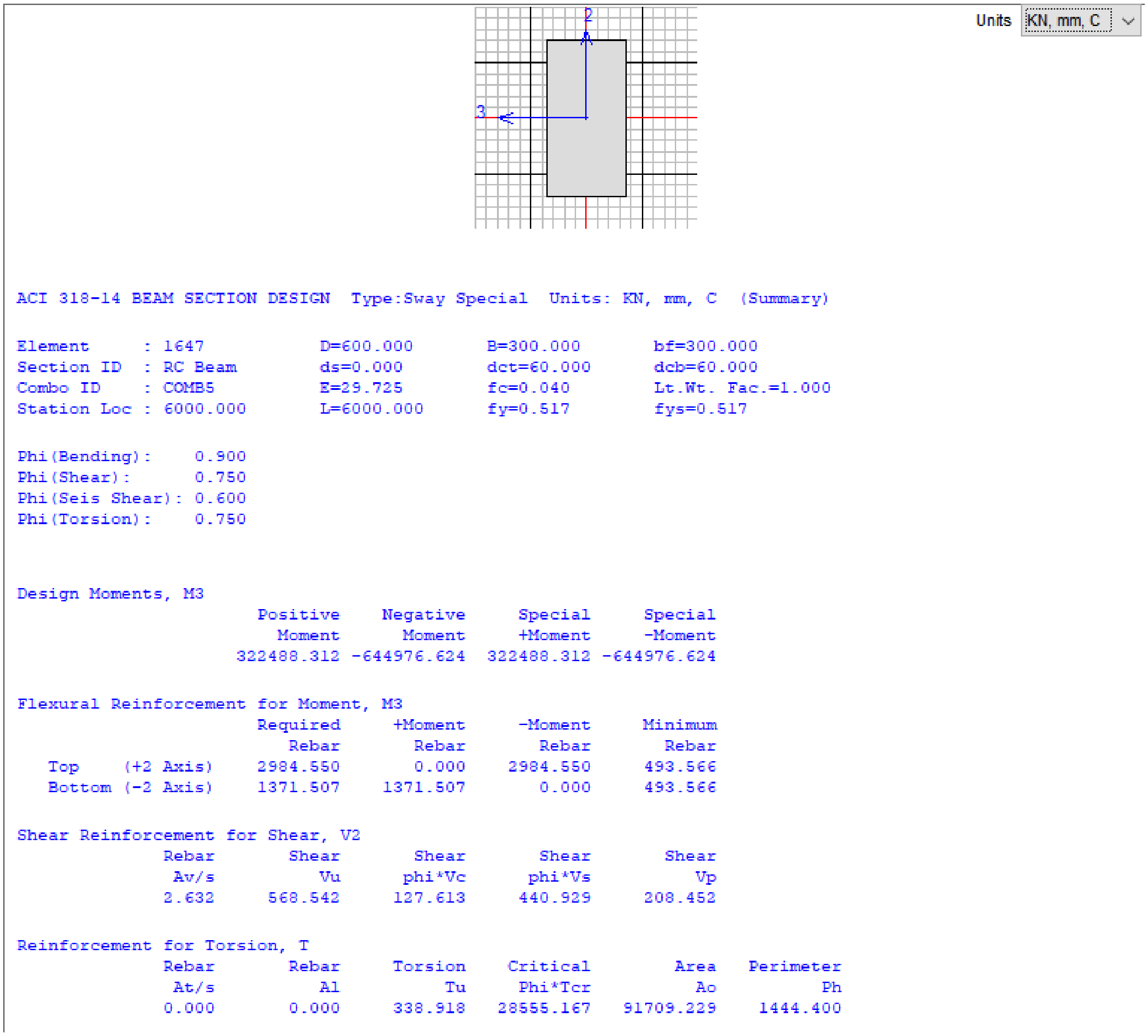

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     231037.646 -161244.156  161244.156 -161244.156

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     662.188       0.000     662.188     493.566
   Bottom (-2 Axis)     962.836     962.836       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.573     223.594     127.613      95.981     208.452

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     338.918   28555.167   91709.229    1444.400
```

Figure 9.37. SAP 2000 Major Beam Design Summary for the 9th floor (location 3 m)



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

Element     : 1647           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 6000.000       L=6000.000      fy=0.517        fys=0.517

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

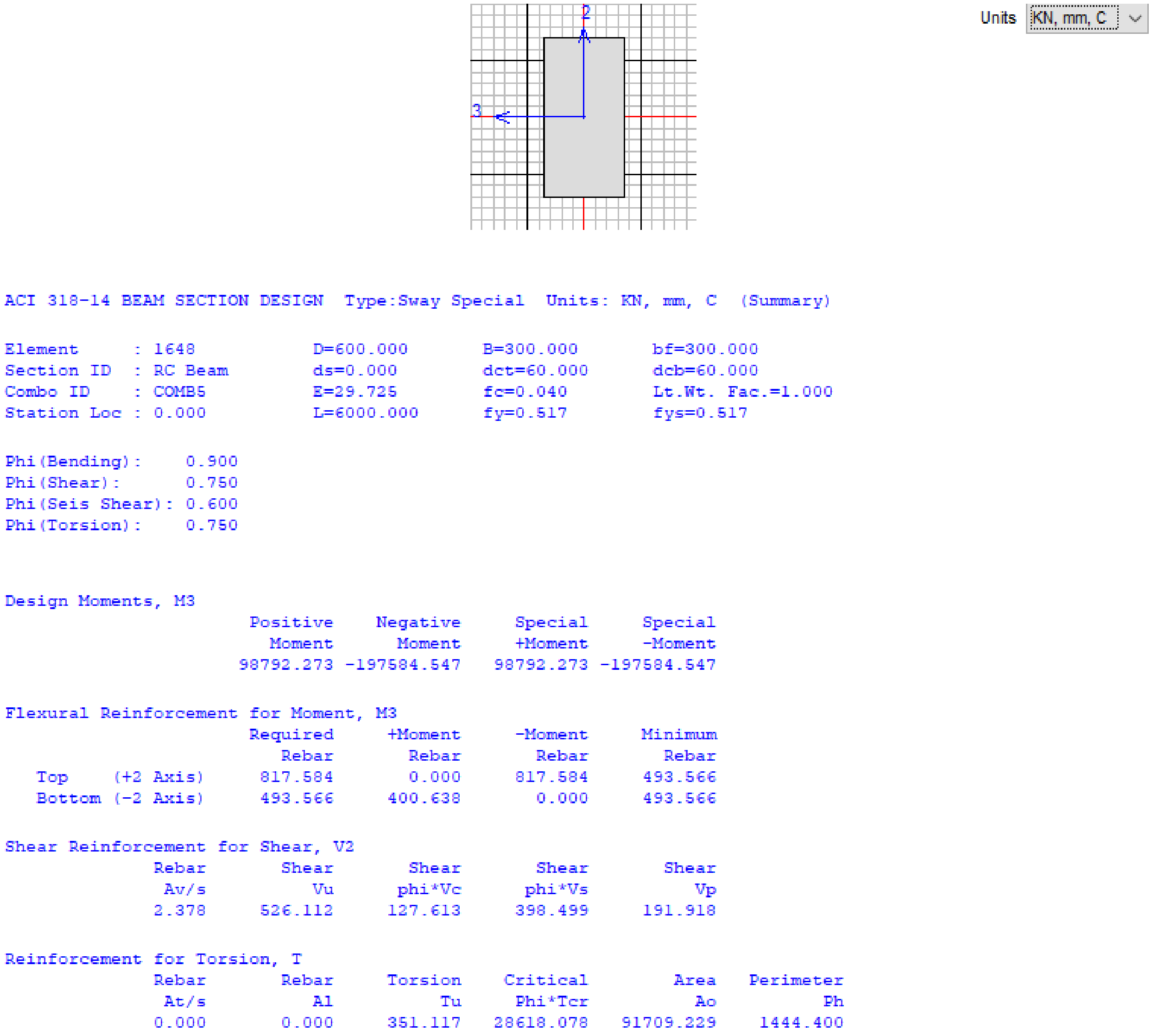

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     322488.312 -644976.624  322488.312 -644976.624

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    2984.550       0.000    2984.550     493.566
   Bottom (-2 Axis)    1371.507    1371.507       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.632     568.542     127.613     440.929     208.452

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     338.918   28555.167   91709.229    1444.400
```

Figure 9.38. SAP 2000 Major Beam Design Summary for the 9th floor (location 6 m)



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

Element     : 1648          D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 0.000         L=6000.000      fy=0.517        fys=0.517

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

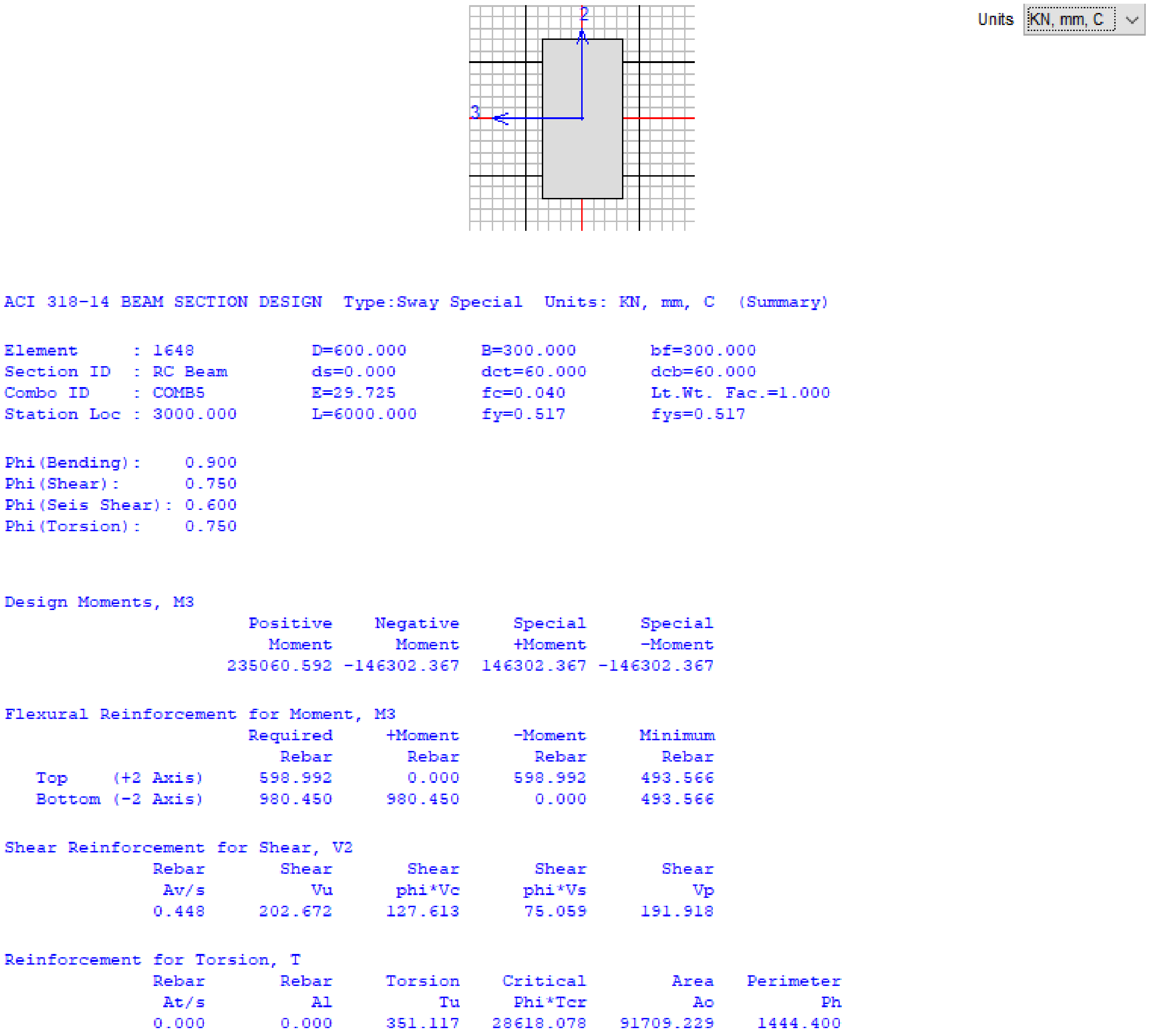

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                      98792.273 -197584.547   98792.273 -197584.547

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     817.584       0.000     817.584     493.566
   Bottom (-2 Axis)     493.566     400.638       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.378     526.112     127.613     398.499     191.918

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     351.117   28618.078   91709.229    1444.400
```

Figure 9.39. SAP 2000 Major Beam Design Summary for the 10th floor (location 0 m)



Units KN, mm, C

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

Element      : 1648          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 3000.000      L=6000.000      fy=0.517        fys=0.517

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

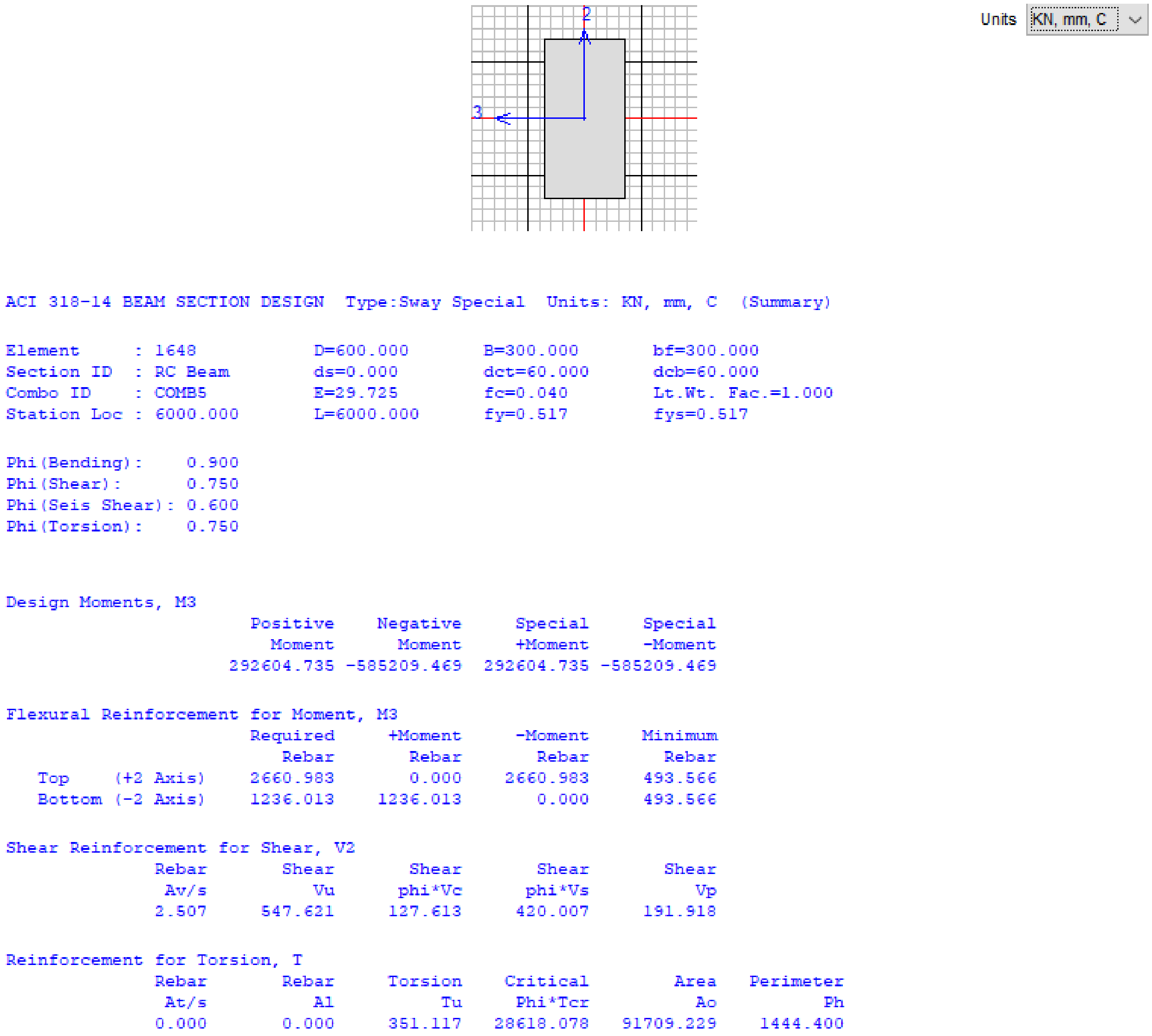

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     235060.592 -146302.367  146302.367 -146302.367

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     598.992       0.000     598.992     493.566
   Bottom (-2 Axis)     980.450     980.450       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.448     202.672     127.613      75.059     191.918

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     351.117   28618.078   91709.229    1444.400
```

Figure 9.40. SAP 2000 Major Beam Design Summary for the 10th floor (location 3 m)



Units KN, mm, C

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

Element      : 1648             D=600.000        B=300.000        bf=300.000
Section ID   : RC Beam          ds=0.000         dct=60.000       dcb=60.000
Combo ID     : COMB5            E=29.725         fc=0.040         Lt.Wt. Fac.=1.000
Station Loc  : 6000.000         L=6000.000       fy=0.517         fys=0.517

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

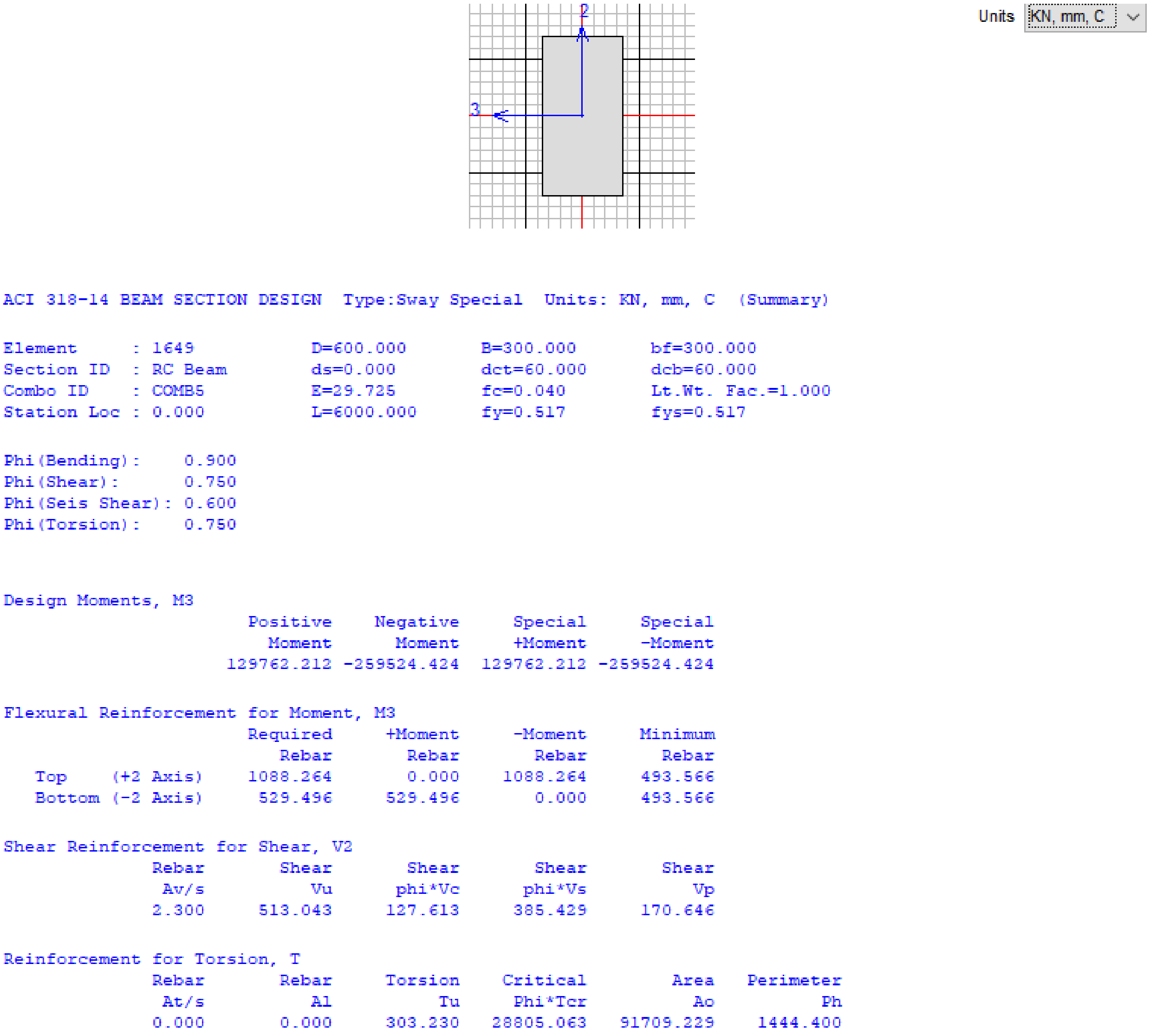

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment      +Moment      -Moment
                     292604.735  -585209.469   292604.735  -585209.469

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)   2660.983        0.000     2660.983      493.566
   Bottom  (-2 Axis)   1236.013     1236.013        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu      phi*Vc       phi*Vs           Vp
              2.507      547.621      127.613      420.007      191.918

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu      Phi*Tcr           Ao           Ph
              0.000        0.000      351.117    28618.078    91709.229     1444.400
```

Figure 9.41. SAP 2000 Major Beam Design Summary for the 10th floor (location 6 m)



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

Element     : 1649           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 0.000          L=6000.000      fy=0.517        fys=0.517

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

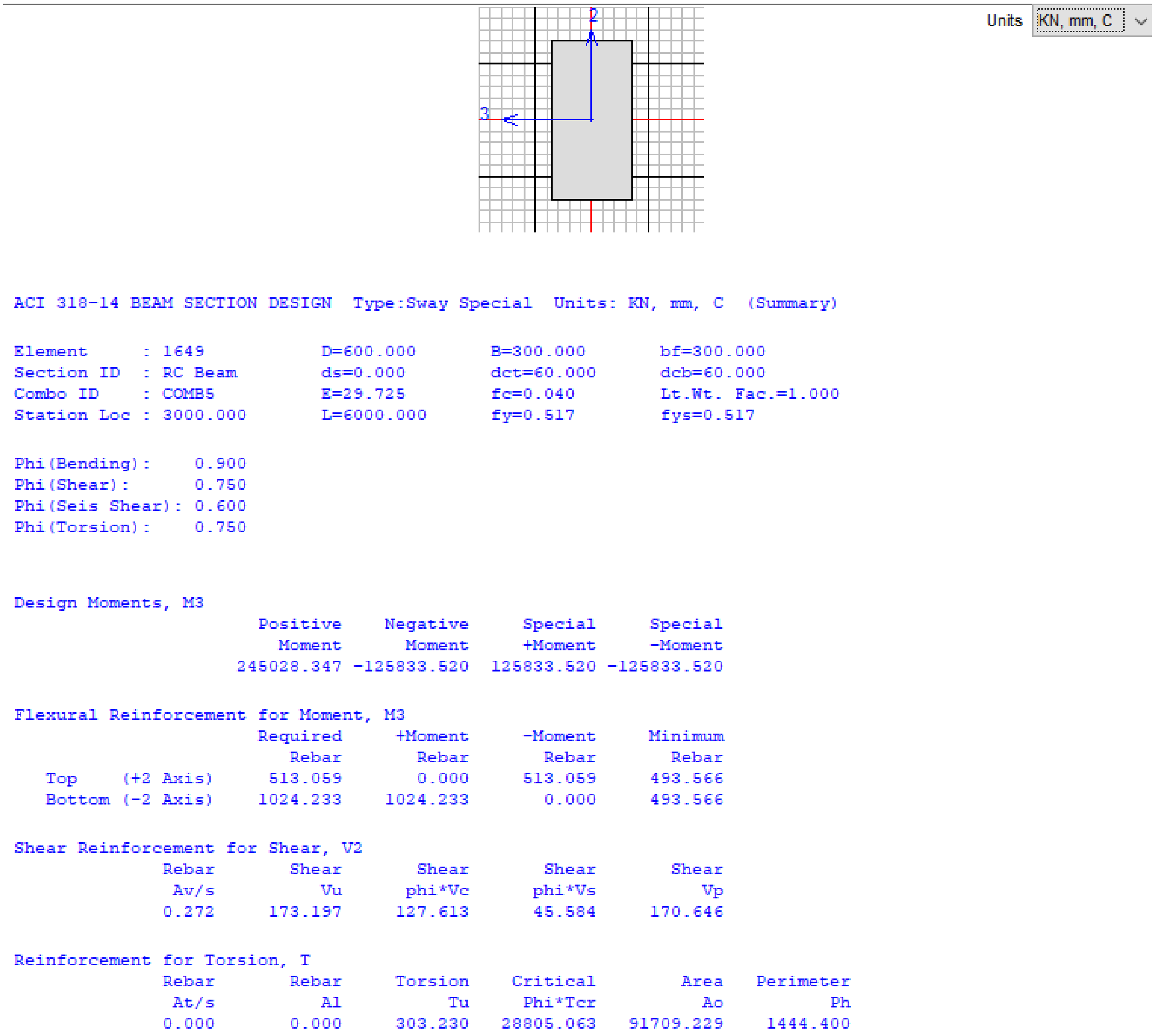

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     129762.212 -259524.424  129762.212 -259524.424

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)    1088.264       0.000    1088.264     493.566
   Bottom (-2 Axis)     529.496     529.496       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              2.300     513.043     127.613     385.429     170.646

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000     303.230   28805.063   91709.229    1444.400
```

Figure 9.42. SAP 2000 Major Beam Design Summary for the 11th floor (location 0 m)



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

Element      : 1649          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 3000.000      L=6000.000      fy=0.517        fys=0.517

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

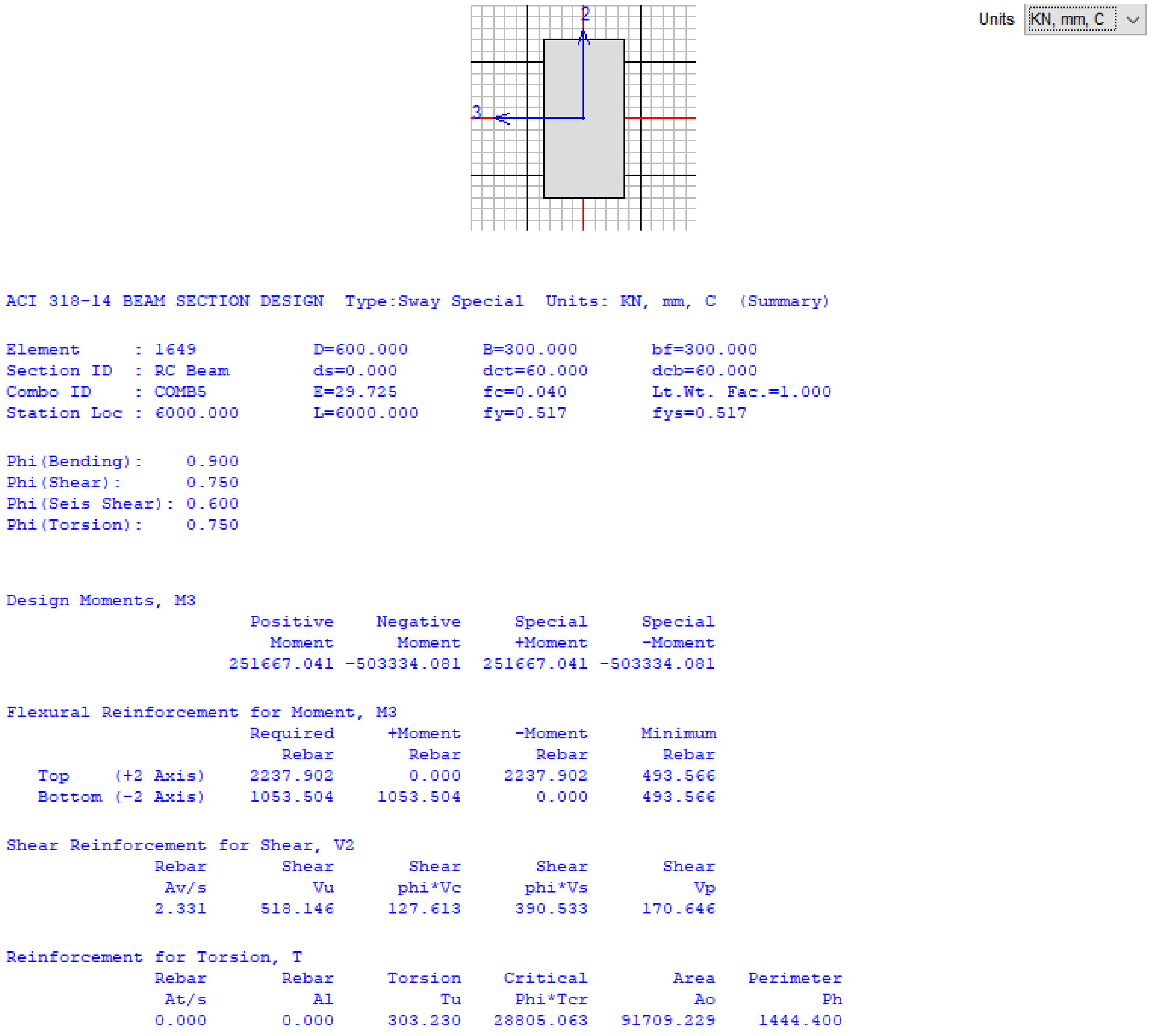

Design Moments, M3
                       Positive    Negative      Special     Special
                         Moment      Moment      +Moment     -Moment
                     245028.347 -125833.520   125833.520 -125833.520

Flexural Reinforcement for Moment, M3
                       Required     +Moment      -Moment     Minimum
                          Rebar       Rebar        Rebar       Rebar
   Top    (+2 Axis)     513.059       0.000      513.059     493.566
   Bottom (-2 Axis)    1024.233    1024.233        0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear        Shear       Shear
               Av/s          Vu     phi*Vc       phi*Vs          Vp
              0.272     173.197     127.613       45.584     170.646

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion     Critical        Area   Perimeter
               At/s          Al          Tu      Phi*Tcr          Ao          Ph
              0.000       0.000     303.230    28805.063   91709.229    1444.400
```

Figure 9.43. SAP 2000 Major Beam Design Summary for the 11th floor (location 3 m)



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

Element      : 1649          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000      L=6000.000      fy=0.517        fys=0.517

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

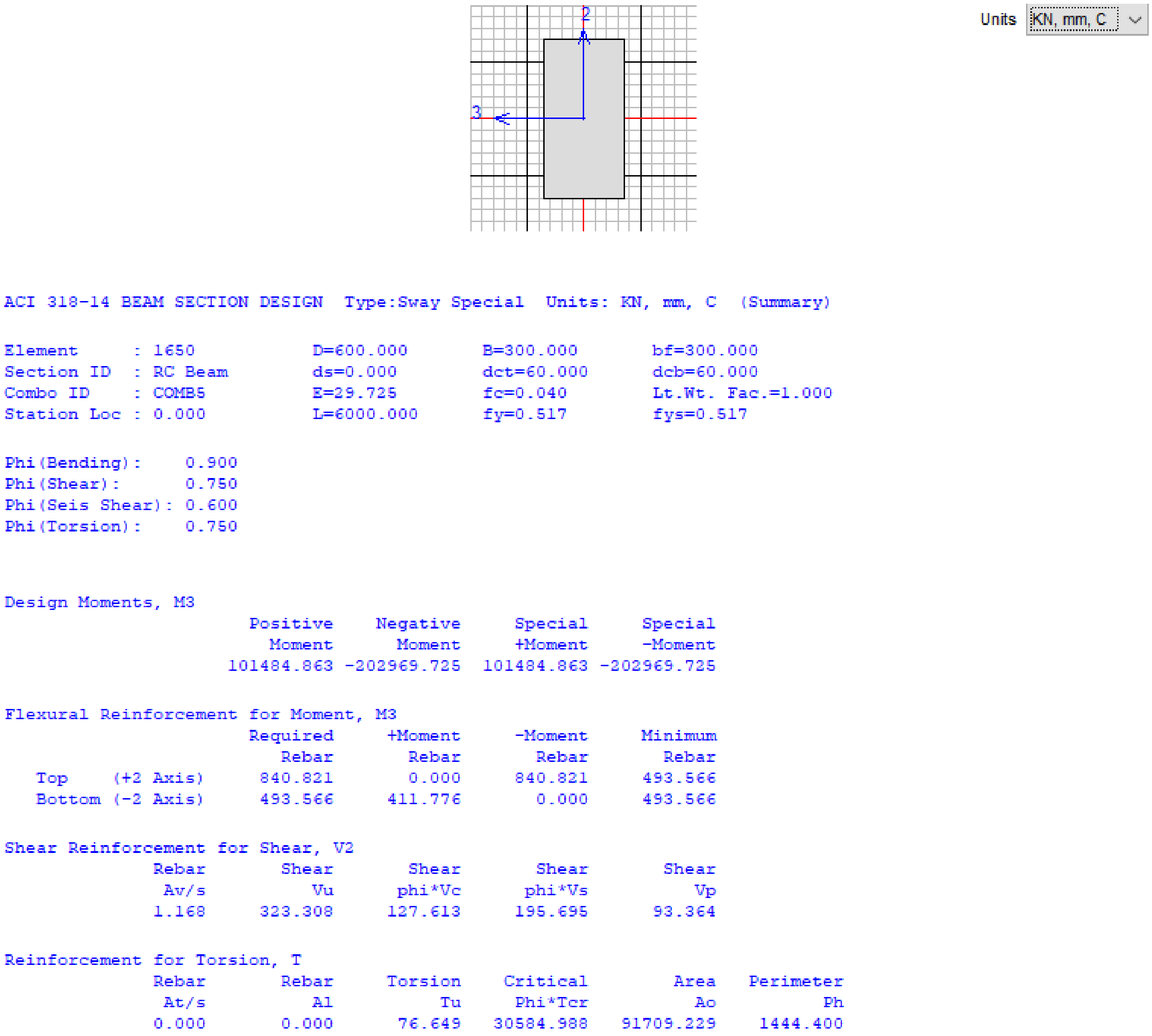

Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment     +Moment      -Moment
                     251667.041 -503334.081  251667.041 -503334.081

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)   2237.902        0.000     2237.902      493.566
   Bottom (-2 Axis)    1053.504     1053.504        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu      phi*Vc      phi*Vs           Vp
              2.331      518.146      127.613      390.533      170.646

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu     Phi*Tcr            Ao           Ph
              0.000        0.000      303.230    28805.063    91709.229     1444.400
```

Figure 9.44. SAP 2000 Major Beam Design Summary for the 11th floor (location 6 m)



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

Element     : 1650          D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 0.000         L=6000.000      fy=0.517        fys=0.517

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

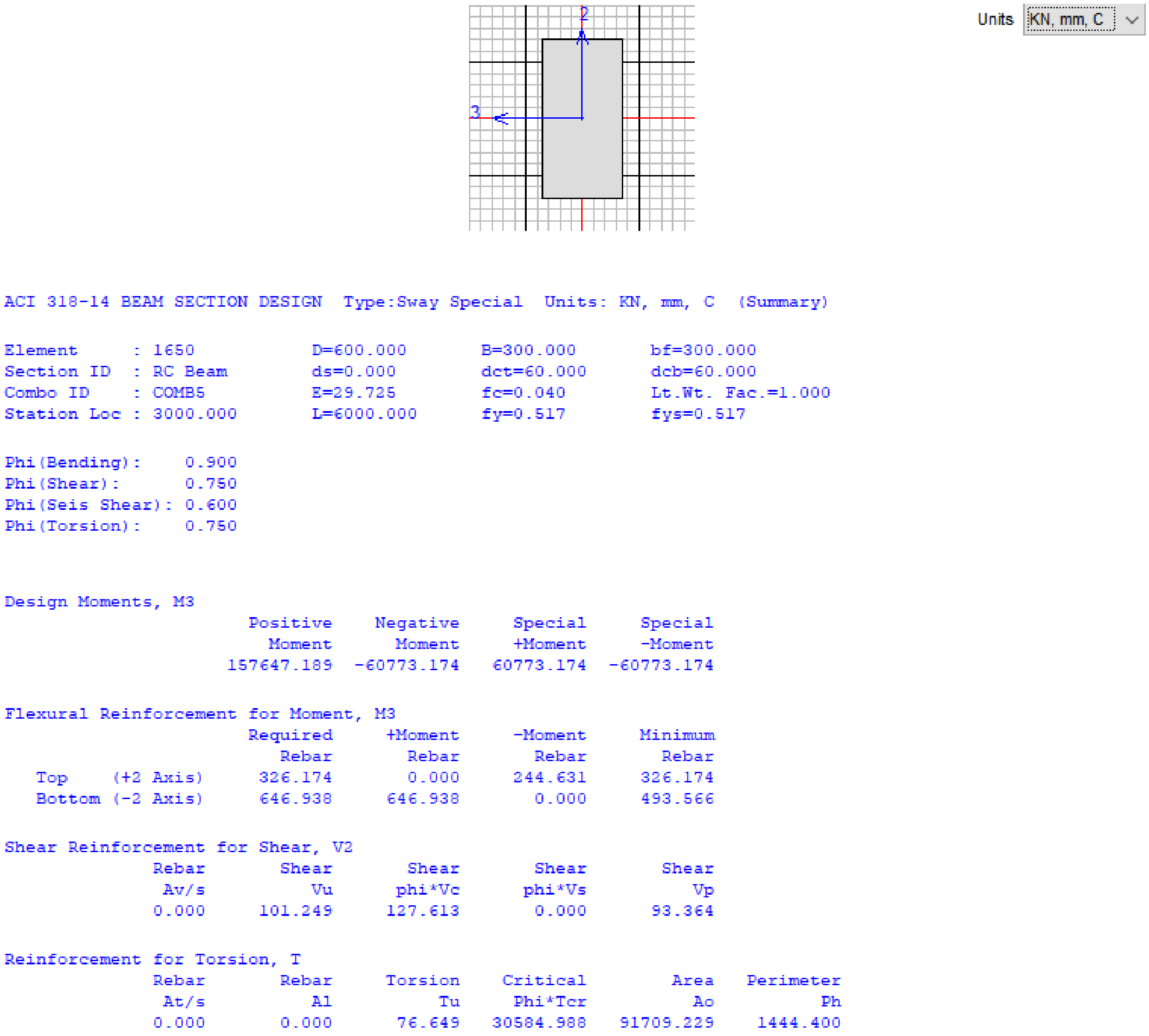

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment     +Moment     -Moment
                     101484.863 -202969.725  101484.863 -202969.725

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     840.821       0.000     840.821     493.566
   Bottom (-2 Axis)     493.566     411.776       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              1.168     323.308     127.613     195.695      93.364

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000      76.649   30584.988   91709.229    1444.400
```

Figure 9.45. SAP 2000 Major Beam Design Summary for the 12th floor (location 0 m)



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

Element     : 1650           D=600.000       B=300.000       bf=300.000
Section ID  : RC Beam        ds=0.000        dct=60.000      dcb=60.000
Combo ID    : COMB5          E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc : 3000.000       L=6000.000      fy=0.517        fys=0.517

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

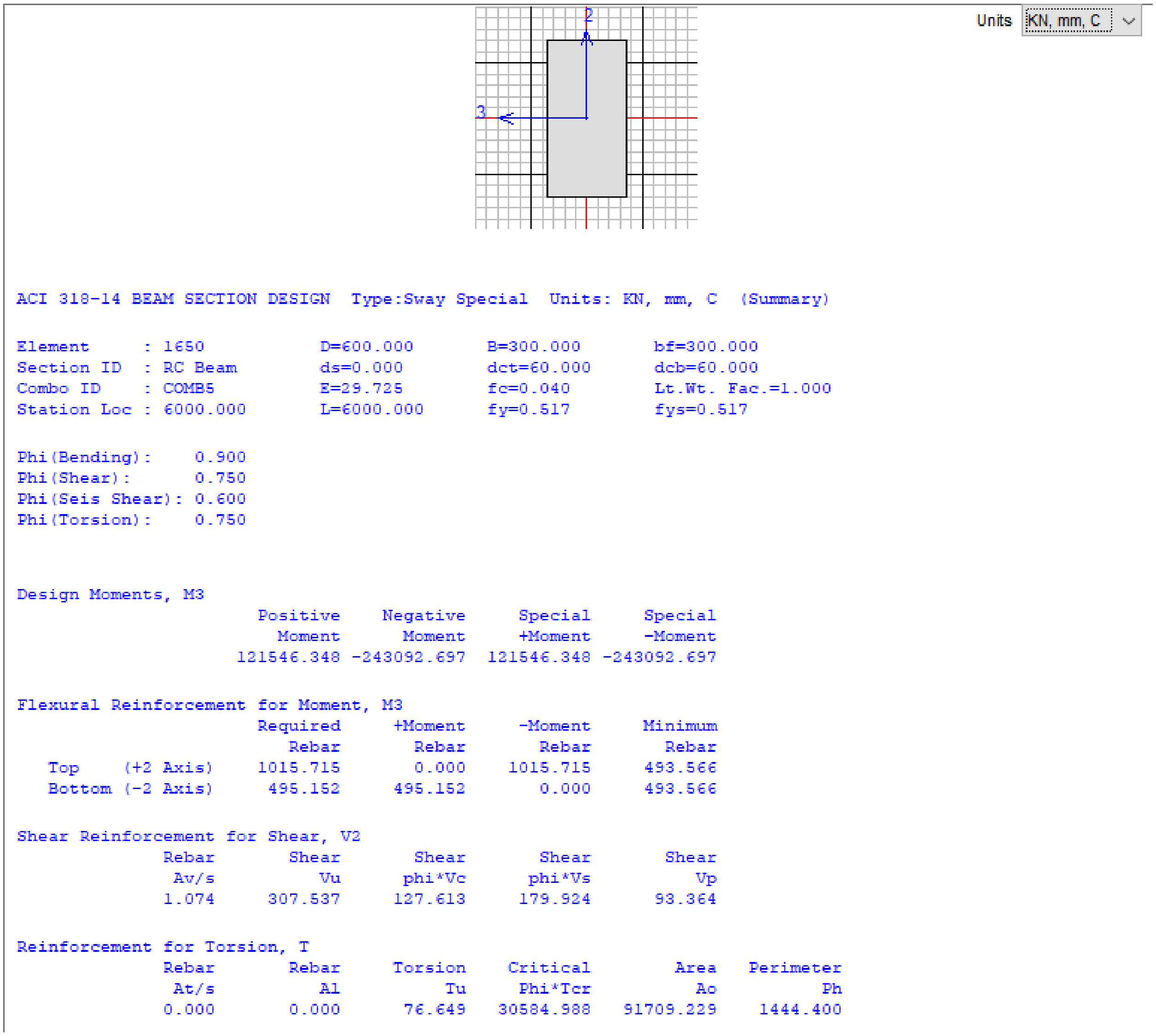

Design Moments, M3
                       Positive    Negative     Special     Special
                         Moment      Moment    +Moment     -Moment
                     157647.189  -60773.174   60773.174  -60773.174

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top    (+2 Axis)     326.174       0.000     244.631     326.174
   Bottom (-2 Axis)     646.938     646.938       0.000     493.566

Shear Reinforcement for Shear, V2
              Rebar       Shear       Shear       Shear       Shear
               Av/s          Vu      phi*Vc      phi*Vs          Vp
              0.000     101.249     127.613       0.000      93.364

Reinforcement for Torsion, T
              Rebar       Rebar     Torsion    Critical        Area   Perimeter
               At/s          Al          Tu     Phi*Tcr          Ao          Ph
              0.000       0.000      76.649   30584.988   91709.229    1444.400
```

Figure 9.46. SAP 2000 Major Beam Design Summary for the 12th floor (location 3 m)



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

Element      : 1650          D=600.000       B=300.000       bf=300.000
Section ID   : RC Beam       ds=0.000        dct=60.000      dcb=60.000
Combo ID     : COMB5         E=29.725        fc=0.040        Lt.Wt. Fac.=1.000
Station Loc  : 6000.000      L=6000.000      fy=0.517        fys=0.517

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


Design Moments, M3
                       Positive     Negative      Special      Special
                         Moment       Moment      +Moment      -Moment
                     121546.348  -243092.697   121546.348  -243092.697

Flexural Reinforcement for Moment, M3
                       Required      +Moment      -Moment      Minimum
                          Rebar        Rebar        Rebar        Rebar
   Top     (+2 Axis)   1015.715        0.000     1015.715      493.566
   Bottom  (-2 Axis)    495.152      495.152        0.000      493.566

Shear Reinforcement for Shear, V2
              Rebar        Shear        Shear        Shear        Shear
               Av/s           Vu       phi*Vc       phi*Vs           Vp
              1.074      307.537      127.613      179.924       93.364

Reinforcement for Torsion, T
              Rebar        Rebar      Torsion     Critical         Area    Perimeter
               At/s           Al           Tu      Phi*Tcr           Ao           Ph
              0.000        0.000       76.649    30584.988    91709.229     1444.400
```

Figure 9.47. SAP 2000 Major Beam Design Summary for the 12th floor (location 6 m)

Table 9.35. Floor 1 Major Beam Reinforcement Distribution

Major Beam for Floor 1			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	494	972	1278
Area, in2	0.77	1.51	1.98
SAP 2000 Compression Area, mm2	862	619	2760
Area, in2	1.34	0.96	4.28
Tension Bars	**4#4**	**2#8**	**2#9**
Compression Bars	**5#5**	**5#4**	**6#8**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.36. Floor 2 Major Beam Reinforcement Distribution

Major Beam for Floor 2			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	268	946	1495
Area, in2	0.42	1.47	2.32
SAP 2000 Compression Area, mm2	494	719	3225
Area, in2	0.77	1.11	5.00
Tension Bars	**4#3**	**2#8**	**3#8**
Compression Bars	**4#4**	**2#7**	**5#9**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.37. Floor 3 Major Beam Reinforcement Distribution

Major Beam for Floor 3			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	162	945	1586
Area, in2	0.25	1.46	2.46
SAP 2000 Compression Area, mm2	326	761	3411
Area, in2	0.51	1.18	5.29
Tension Bars	**3#3**	**2#8**	**6#6**
Compression Bars	**3#4**	**2#7**	**7#8**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.38. Floor 5 Major Beam Reinforcement Distribution

Major Beam for Floor 5			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	126	946	1618
Area, in2	0.20	1.47	2.51
SAP 2000 Compression Area, mm2	254	776	3471
Area, in2	0.39	1.20	5.38
Tension Bars	**2#3**	**2#8**	**3#9**
Compression Bars	**4#3**	**4#5**	**6#9**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.39. Floor 6 Major Beam Reinforcement Distribution

Major Beam for Floor 6			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	158	947	1588
Area, in2	0.24	1.47	2.46
SAP 2000 Compression Area, mm2	318	762	3415
Area, in2	0.49	1.18	5.29
Tension Bars	**3#3**	**2#8**	**3#9**
Compression Bars	**3#4**	**2#7**	**6#9**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.40. Floor 7 Major Beam Reinforcement Distribution

Major Beam for Floor 7			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	205	949	1546
Area, in2	0.32	1.47	2.40
SAP 2000 Compression Area, mm2	413	743	3333
Area, in2	0.64	1.15	5.17
Tension Bars	**3#3**	**2#8**	**4#7**
Compression Bars	**4#4**	**2#7**	**7#8**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.41. Floor 8 Major Beam Reinforcement Distribution

Major Beam for Floor 8			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	285	953	1472
Area, in2	0.44	1.48	2.28
SAP 2000 Compression Area, mm2	494	709	3191
Area, in2	0.77	1.10	4.95
Tension Bars	**5#3**	**2#8**	**3#8**
Compression Bars	**4#4**	**2#7**	**5#9**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.42. Floor 9 Major Beam Reinforcement Distribution

Major Beam for Floor 9			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	392	963	1372
Area, in2	0.61	1.49	2.13
SAP 2000 Compression Area, mm2	597	662	2985
Area, in2	0.93	1.03	4.63
Tension Bars	**2#5**	**2#8**	**3#8**
Compression Bars	**5#4**	**2#7**	**6#8**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.43. Floor 10 Major Beam Reinforcement Distribution

Major Beam for Floor 10			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	494	980	1236
Area, in2	0.77	1.52	1.92
SAP 2000 Compression Area, mm2	818	599	2661
Area, in2	1.27	0.93	4.12
Tension Bars	**4#4**	**2#8**	**2#9**
Compression Bars	**3#6**	**5#4**	**6#8**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.44. Floor 11 Major Beam Reinforcement Distribution

Major Beam for Floor 11			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	529	1024	1054
Area, in2	0.82	1.59	1.63
SAP 2000 Compression Area, mm2	1088	513	2238
Area, in2	1.69	0.80	3.47
Tension Bars	**2#6**	**4#6**	**4#6**
Compression Bars	**4#6**	**2#6**	**6#7**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.45. Floor 12 Major Beam Reinforcement Distribution

Major Beam for Floor 12			
Variables	**End**	**Mid**	**End**
SAP 2000 Tension Area, mm2	494	647	1016
Area, in2	0.77	1.00	1.57
SAP 2000 Compression Area, mm2	841	326	495
Area, in2	1.30	0.51	0.77
Tension Bars	**4#4**	**2#7**	**4#6**
Compression Bars	**3#6**	**3#4**	**4#4**
Shear Reinforcement	#3 at 130 mm	#3 at 150 mm	#3 at 130 mm

Table 9.46. Tensile Reinforcement for Left End of Major Beam, location 0 m

Floor	**Bottom Reinforcement (Tension Bars)**				
	SAP 2000 Area, mm2	**Area, in2**	**Bar Size**	**Area, in2**	**Area, mm2**
1	494	0.77	4#4	0.79	510
2	268	0.42	4#3	0.44	284
3	162	0.25	3#3	0.33	213
4	123	0.19	2#3	0.22	142
5	126	0.20	2#3	0.20	129
6	158	0.24	3#3	0.33	213
7	205	0.32	3#3	0.33	213
8	285	0.44	5#3	0.55	355
9	392	0.61	2#5	0.61	394
10	494	0.77	4#4	0.79	510

11	529	0.82	2#6	0.88	568
12	494	0.77	4#4	0.79	510

Table 9.47. Compressive Reinforcement for Left End of Major Beam, location 0 m

Floor	**Top Reinforcement (Compression Bars)**				
	SAP 2000 Area, mm2	**Area, in2**	**Bar Size**	**Area, in2**	**Area, mm2**
1	862	1.34	5#5	1.53	987
2	494	0.77	4#4	0.79	510
3	326	0.51	3#4	0.59	381
4	247	0.38	2#4	0.39	252
5	254	0.39	4#3	0.44	284
6	318	0.49	3#4	0.59	381
7	413	0.64	4#4	0.79	510
8	494	0.77	4#4	0.79	510
9	597	0.93	5#4	0.98	632
10	818	1.27	3#6	1.33	858
11	1088	1.69	4#6	1.77	1142
12	841	1.30	3#6	1.33	858

Table 9.48. Tensile Reinforcement for Mid-Span of Major Beam, location 3 m

Floor	**Bottom Reinforcement (Tension Bars)**				
	SAP 2000 Area, mm2	**Area, in2**	**Bar Size**	**Area, in2**	**Area, mm2**
1	972	1.51	2#8	1.57	1013
2	946	1.47	2#8	1.57	1013
3	945	1.46	2#8	1.57	1013
4	945	1.46	2#8	1.57	1013
5	946	1.47	2#8	1.57	1013
6	947	1.47	2#8	1.57	1013
7	949	1.47	2#8	1.57	1013
8	953	1.48	2#8	1.57	1013
9	963	1.49	2#8	1.57	1013
10	980	1.52	2#8	1.57	1013
11	1024	1.59	4#6	1.77	1142
12	647	1.00	2#7	1.20	774

Table 9.49. Compressive Reinforcement for Mid-Span of Major Beam, location 3 m

Floor	Top Reinforcement (Compression Bars)				
	SAP 2000 Area, mm2	Area, in2	Bar Size	Area, in2	Area, mm2
1	619	0.96	5#4	0.98	632
2	719	1.11	2#7	1.20	774
3	761	1.18	2#7	1.20	774
4	777	1.20	4#5	1.23	794
5	776	1.20	4#5	1.23	794
6	762	1.18	2#7	1.20	774
7	743	1.15	2#7	1.20	774
8	709	1.10	2#7	1.20	774
9	662	1.03	2#7	1.20	774
10	599	0.93	5#4	0.98	632
11	513	0.80	2#6	0.88	568
12	326	0.51	3#4	0.59	381

Table 9.50. Tensile Reinforcement for Right End of Major Beam, location 6 m

Floor	Bottom Reinforcement (Tension Bars)				
	SAP 2000 Area, mm2	Area, in2	Bar Size	Area, in2	Area, mm2
1	1278	1.98	2#9	2.00	1290
2	1495	2.32	3#8	2.36	1523
3	1586	2.46	6#6	2.65	1710
4	1621	2.51	3#9	3.00	1936
5	1618	2.51	3#9	3.00	1936
6	1588	2.46	3#9	3.00	1936
7	1546	2.40	4#7	2.41	1555
8	1472	2.28	3#8	2.36	1523
9	1372	2.13	3#8	2.36	1523
10	1236	1.92	2#9	2.00	1290
11	1054	1.63	4#6	1.77	1142
12	1016	1.57	4#6	1.77	1142

Table 9.51. Compressive Reinforcement for Right End of Major Beam, location 6 m

Floor	Top Reinforcement (Compression Bars)				
	SAP 2000 Area, mm2	Area, in2	Bar Size	Area, in2	Area, mm2
1	2760	4.28	6#8	4.71	3039
2	3225	5.00	5#9	5.00	3226
3	3411	5.29	7#8	5.50	3549
4	3477	5.39	6#9	6.00	3871
5	3471	5.38	6#9	6.00	3871
6	3415	5.29	6#9	6.00	3871
7	3333	5.17	7#8	5.50	3549
8	3191	4.95	5#9	5.00	3226
9	2985	4.63	6#8	4.71	3039
10	2661	4.12	6#8	4.71	3039
11	2238	3.47	6#7	3.61	2329
12	495	0.77	4#4	0.79	510