



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

DESIGN OF A 12-STORY “THE TRUSS RESIDENCES” HIGH-RISE RESIDENTIAL BUILDING IN CUPERTINO, CALIFORNIA, USA

Capstone Project II

Group #10: The Truss Fund

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DECLARATION

We hereby declare that this report entitled “Design of a 12-story “The Truss Residences” high-rise residential building in Cupertino, California, USA” is the result of our own project work except for quotations and citations which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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ABSTRACT

The project presents design and planning for the 12 story reinforced concrete residential tower, The Truss Residences, at 10101 N Wolfe Road, Cupertino, California. It contains a one story basement along with the ground commercial level and 11 floors of 110 apartment units. The main objectives of the project were to design a structurally safe residential building, with an appropriate foundation and a stormwater drainage system, all of which would meet building code requirements, safety requirements and be economically feasible. As a result of the work, the most economical gravity load-resisting system selected for design was a two-way slab combined with minor beams. The designed frames incorporated seismic and wind load resistance capabilities along with torsional effect computations during calculations. Iterations to the load distribution were needed during the sizing process according to ACI 318-19 standards for all members including slabs and beams and columns. Stability assessment using SAP2000 software models followed to verify stability alongside hand calculations of internal forces and lateral drifts. The soil response analysis of geotechnical design led to Class C soil identification which affected foundation design standards. Group pile foundations were preferred over single piles and shallow foundations because their performance and capacity for bearing load surpassed both options. The chosen softwares PLAXIS and GEO5 confirmed that foundation bearing capacity and settlement met all safety requirements. For environmental engineering, drainage systems were designed which included pipes within a network built for environmental purposes. A software-based elevation chart development enabled us to determine the topography and create appropriate pipe installations and flow paths. The process of cost-benefit evaluation took place within construction management. Analysis confirmed how durable materials produce extended cost savings.

1. Introduction

The Truss Residences is a 12 story reinforced concrete apartment building located in Cupertino, California, and this capstone project includes the design and planning of the building. It has a basement floor, one-story ground level commercial floor and 11 residential floors with a total of 110 apartment units. The building is characterized by being seismic resistant and incorporates architectural, structural, geotechnical, environmental and construction management aspects.

Detailed architectural designs including code complying floor plans and fire safety plans, structural load calculations as per ASCE 7-16 standards, geotechnical design deriving from site soil layer characteristics and earthquake accelerogram data, are also included in the project scope. Environmental engineering considerations for designing a drainage system with considerations of the site elevations and natural topography. Construction management chapter explains cost calculation and schedule planning as well as safety risk procedures applied to site operations. The design also integrated parking, accessibility, and was fully integrated into LEED requirements.

Main objectives:

- Design a structurally safe 12-story residential building with one-story basement in a zone of high seismic activity according to the building codes;
- Complete geotechnical design analysis by selecting the suitable foundation type with an appropriate factor of safety and retaining wall design;
- Design stormwater drainage system to meet the rainfall intensity and time of concentration in the area;
- Analyze the costs and benefits of the project as well as the construction procedure and duration;
- Design the building in accordance with LEED certification.

1.1 Problem Statement

Although highly known as an innovator city and the hub of major technology firms such as Apple - Cupertino city is facing issues of inadequate housing supply and affordability. The region has experienced steady increase in population over the past decade but the residential infrastructure development has been tardy. For example, a lot of the city's rental stock is undersized and overcrowded and further strains local facilities, thereby also negatively affecting livelihood standards. Additionally, sustainable redevelopment of underused spaces, such as the recently razed Cupertino Square shopping mall, further leads us to recycle this space.

The key issues addressed by this project are: housing density, low sustainability, safety. These issues were all balanced within a high density, mixed use residential complex - The Truss Residences. With the goal of matching the derivative of housing needs and improving urban living standards while delivering the least environmental impact, The Truss Residences strives to achieve this objective by adopting durable material, the optimized building orientation of natural light, and integration of public transport access and well structured construction management.

2. Architectural Design

2.1 Overview of the building

It will be a 12-story residential complex, with the first story for commercial use and open lobbies for residents, while the other floors will be housing apartments, each having a sum of 110 apartments, averaging 10 apartments per floor. The proposed structure will be at 10101 N Wolfe Road, Cupertino, California, Western United States, and it is to be called "The Truss Residents." According to the International Building Code, ICC 2018, the occupancy classification will fall under Residential group R2. It will combine the residents' needs with the commercial necessities through functionality and aesthetics in this modern building.

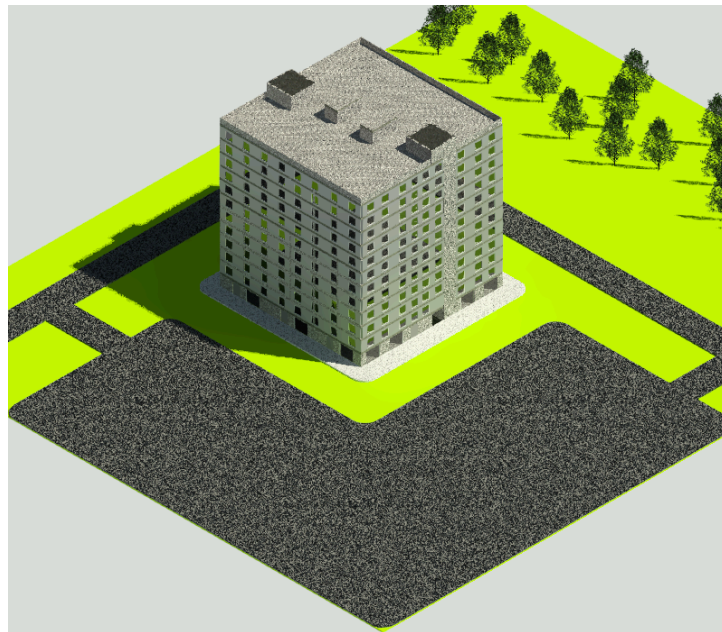


Figure 2.1. 3D view of building

2.2 Site selection and analysis

Cupertino is one of the cities of the Silicon Valley in the US, making it one of the IT centers of the United States. The city is home to Apple Inc. headquarters with Apple offices and an Apple Park (Figure 2.2) which is located in a 3 minute driving distance from the site. The area was previously occupied by an abandoned 2-story shopping center with a parking lot, which was already demolished, as of September of 2024.



Figure 2.2. Satellite image of the area (Google Maps, 2024).

The area has access to bus stops, one located south of the building, another east of the building, making it accessible for people not owning cars. Moreover, there are bike lanes across the street (Figure 2.3), making the building comfortable for bike owners and public transport users. Next, the location is great for families with children: there is a childcare, preschool, elementary school in a 600m radius; middle and high school in a 1km radius; two public parks in a 600m radius.



Figure 2.3. Up-close satellite image of the area (Google Maps, 2024)

According to the State of California Department of Finance (2021), the population growth of Cupertino in 2010-2020 was 5%, while the housing unit growth over these years was

0.1% (Table E-5). These unequal rates of growth lead to longer driving and traveling time, growing house and rent prices, and even can cause an increase in homeless people. These issues create a need for new housing options in the city. However, the overall trend for housing shows that there is a growing trend in favor of multi-family housing units, rather than single-family units (DOFCA, 2023). Moreover, the statistics show that 12% of all homes in Cupertino are overcrowded rented homes (ACS, 2022), which causes an unnecessary pressure on the building's capacity and infrastructure. Therefore, the reasons stated above make this project even more in demand, aligning it with the needs of the city.

The Cupertino area of California is 75 km away from San Jose. Cupertino is one of the cities of the Silicon Valley and is the hometown of Steve Jobs (Levy & Montevirgen, 2024). The site area has business centers, commercial buildings, and 5-6 story residential properties. The area also has an Apple office nearby, with grocery stores, high school and middle school nearby.

In terms of population, the city has 59,000 residents (DOFCA, 2023) with 70.9% of population being Asian, 22.9% being White, 3.16% Latin, and 1.18% African American (World Population, 2024).

Cupertino square is an abandoned shopping center with a parking lot, which was demolished as of September 2024 (Figure 2.4). The shopping mall area is surrounded by trees along the perimeter of the building that existed before.



Figure 2.4. 3D image of the site area. (Google Earth, 2024)

According to ASCE hazard tool report (2024), the area has a medium wind of 33 m/s with a 50 year occurrence. According to WeatherSpark (2024), the wind direction in the city is predominantly from the West, with the peak wind occurrence being in August (Figure 2.5). The dominant wind direction becomes North during winter season from November to February.

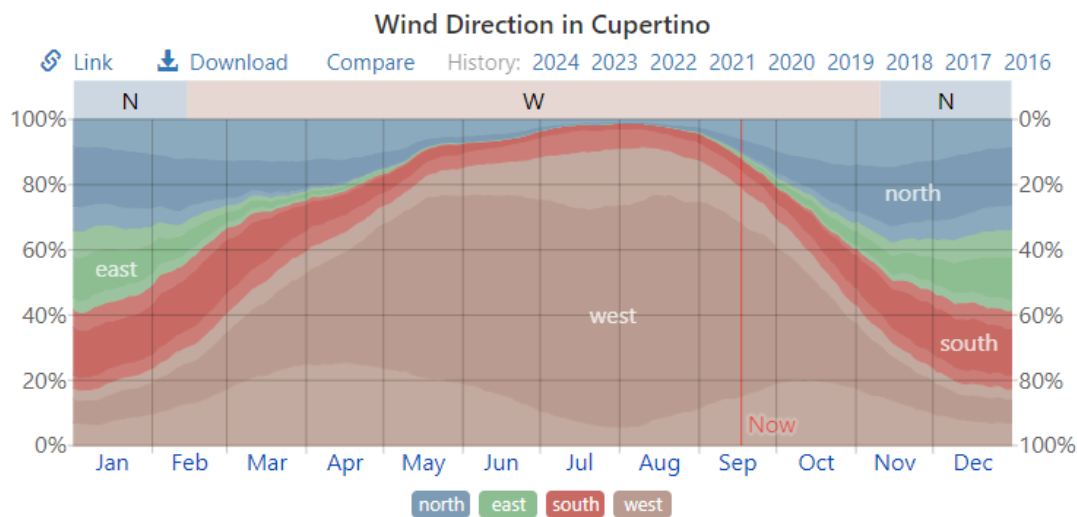


Figure 2.5. Wind direction in Cupertino (WeatherSpark, 2024).

The average monthly temperatures were also obtained (Figure 2.6). The lowest registered temperature was 43°F (6.1°C) in winter, while the highest temperature reached up to 76°F (24.4°C) in the summer. This shows that the temperature differences are not high.

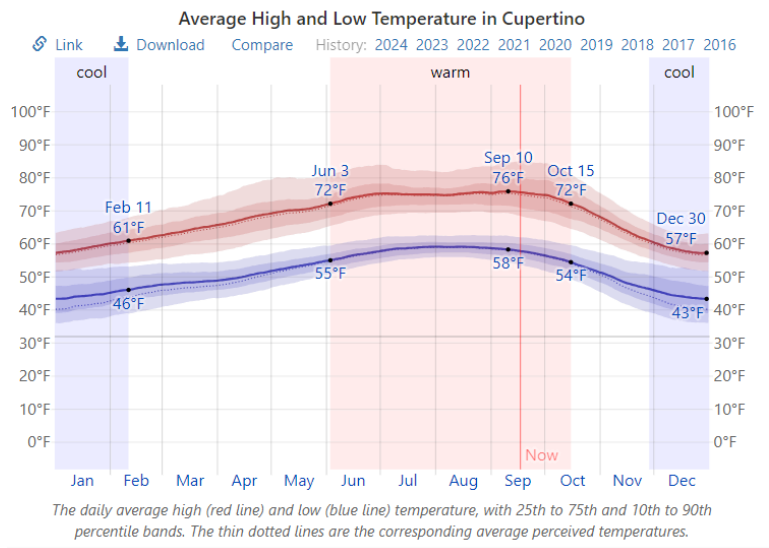


Figure 2.6. Average high and low temperature in Cupertino. (WeatherSpark, 2024).

The average rainfall in inches was also registered. Figure 2.7 shows a table of rainfall in Cupertino by month. In summary, the highest average rainfall happens in February, January, December.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall	3.4"	3.8"	2.7"	1.2"	0.4"	0.1"	0.0"	0.0"	0.2"	0.9"	2.2"	3.4"

Figure 2.7. Average monthly rainfall (in) in Cupertino. (WeatherSpark, 2024).

According to the Sun Direction online tool, the solar access map in the area was obtained. Considering that the buildings nearby are 6 story high, there will be a lot of direct sunlight to the building, creating an opportunity for energy conservation and reducing electricity consumption. This factor will be of advantage for the design concept. Figure 2.8 demonstrates the number of daylight hours by month with the highest being 14hr 44 mins in June and the lowest 9hr 35mins in December. Overall, the city has an extended period of sunlight, creating great conditions for energy conservation in terms of lighting and heating. Going further in the direction of sunlight into the site, Figure 2.9 shows results of sun direction (Suncalc.org, 2024). The results from the tool of the website showed that the shadow length is 0.9 m and the sun duration is 9 h 52 min. As the sun rises from the right corner in the morning and sets to the left corner in the evening, it has already been illustrated that the apartments in the front side will receive more sunlight. Hence, the design of windows should be considered with care for controlling the amount of energy consumption and for making the building efficient.

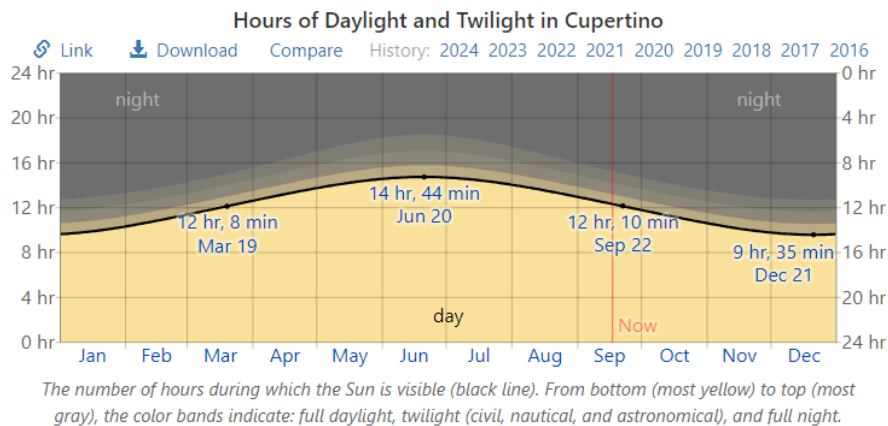


Figure 2.8. Hours of daylight and twilight in Cupertino. (WeatherSpark, 2024).

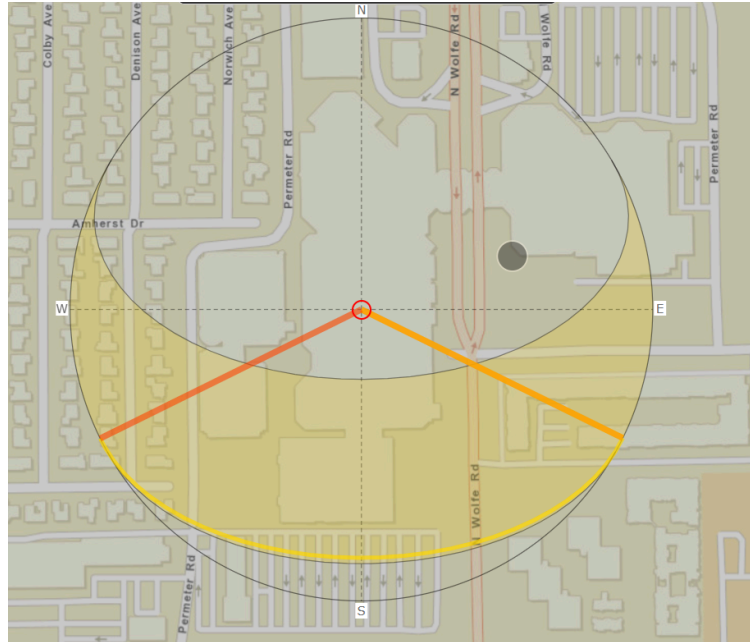


Figure 2.9. Sun direction

There is no slope on the site, based on the geotechnical report.

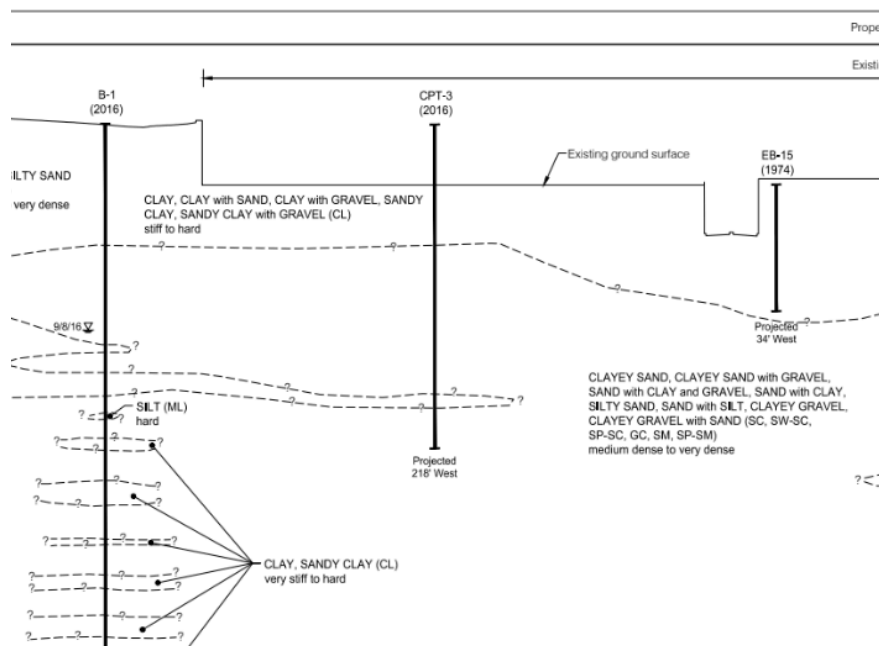


Figure 2.10. Surface slope (Langan, 2023).

There are 2 bus stops in the area: one on the eastern and another on the southern parts of the area. This creates an access to public transport and makes it accessible for people who do not own a private transport.

2.3 Design conditions

Figure 2.11 is a conceptual design of the building at first. But with a deeper understanding of parking space demands as well as on-site conditions, a design approach was adopted in order to reduce overall space of the building. A mass study was made in order to finalize the design of the building in which simple square-shaped mass developed into a final concept of a building with these observations.

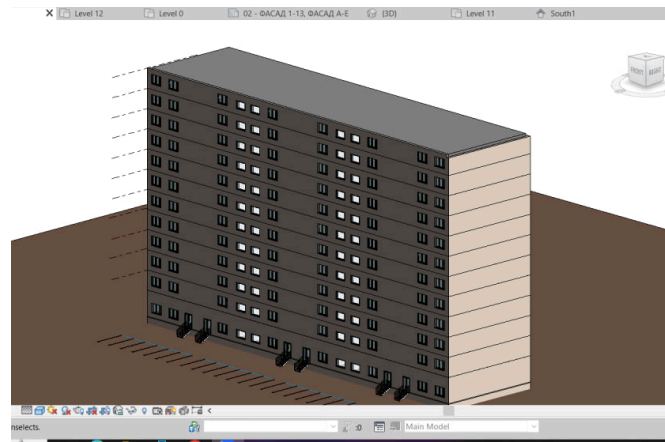


Figure 2.11. Initial design condition

2.4 Dimensions of the building

Regarding the dimensions of the building and apartments, the overall floor area will be 1224 m², with all floors maintaining consistent area dimensions. The layouts for the 1st floor, 2nd floor, basement, roof, and parking, as well as all the detailed dimensions, are presented in Figures 2.12 to 2.15, respectively.

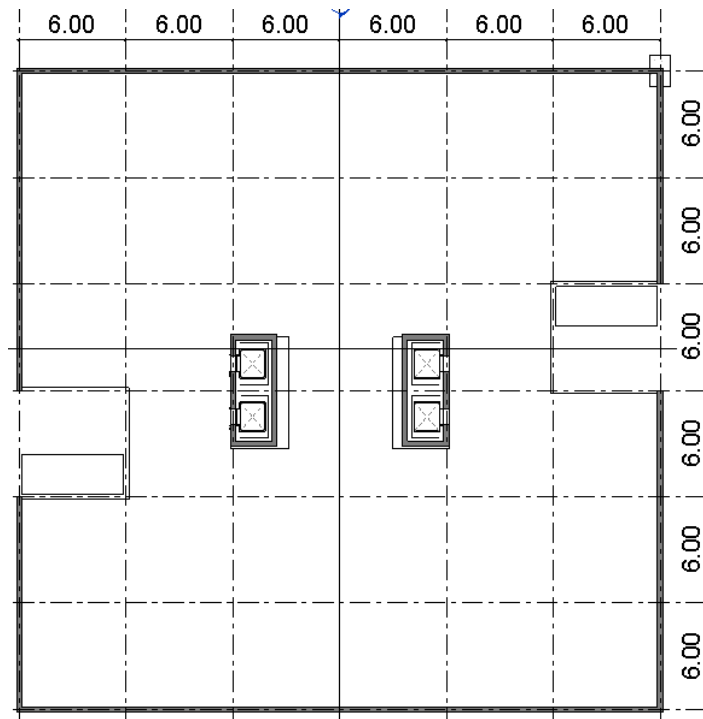


Figure 2.14. Basement floor layout

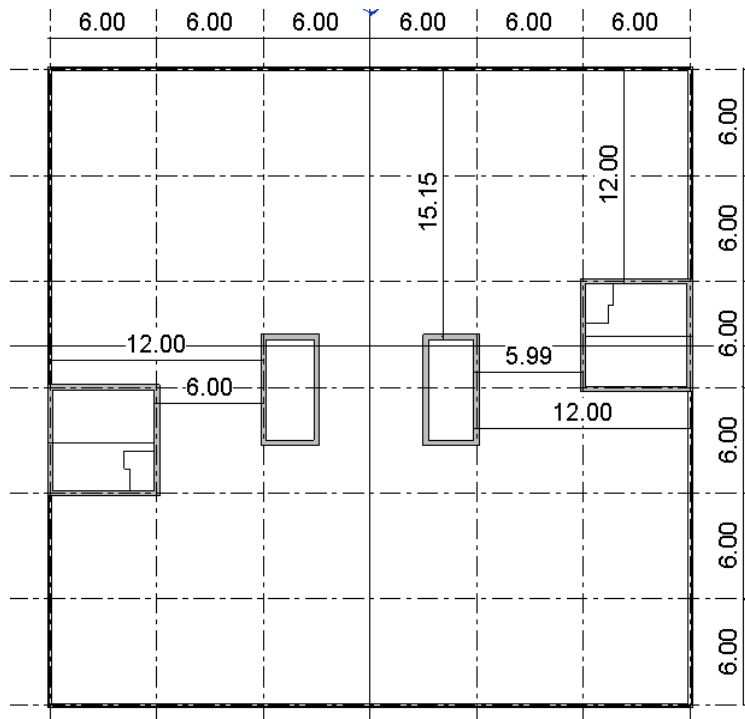


Figure 2.15. Roof layout

2.5. Non-structural members design

2.5.1. Ceiling

The ceiling will consist of two layers and will have a thickness of 133 mm:

- Mechanical duct allowance
- Acoustical fiber board

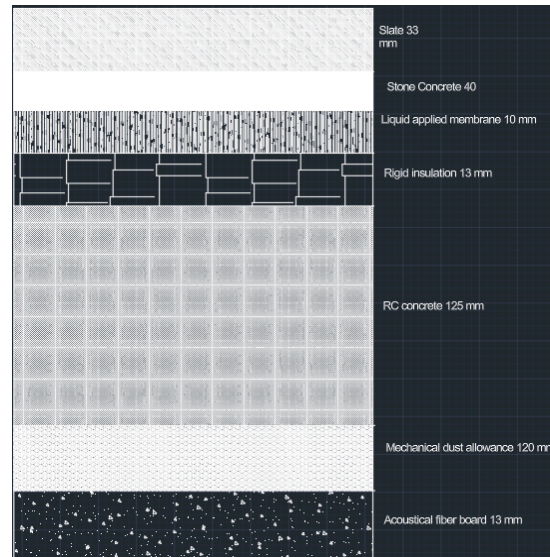


Figure 2.16. Floor finishing components

2.5.2. Floor finishing

The floor finishing materials were selected based on aesthetics, market availability, and suitability for residential buildings. The total thickness of the floor finish without including the structural slab is 96 mm. According to the International Building Code, 2020, from top to bottom, the main layers are:

- Finish: Slate
- Leveling Layer: Stone Concrete
- Waterproofing: Liquid applied membrane
- Insulation: Rigid insulation layer
- Structural Slab: Reinforced concrete

2.5.3. Partition and exterior walls

Hollow concrete masonry was chosen as the best material option for the interior wall design. Its advantages are as follows: it has excellent thermal insulation, is structurally integral, lightweight, can be flexibly designed, and highly fire-resistant (PCA, n.d.). In addition, the interior walls are adequately insulated and finished with plaster on concrete.

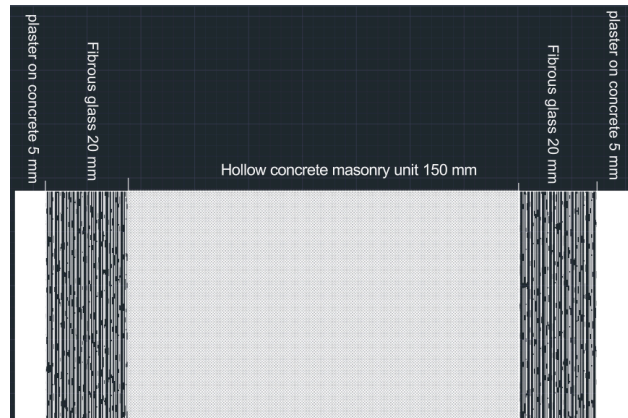


Figure 2.17. Interior wall components

Hollow concrete masonry was also chosen as the primary material for the exterior wall design due to its lightweight characteristics. The exterior surface will be finished with ceramic tile cladding, as previously mentioned. Figures 2.17 and 2.18 display the interior and exterior wall designs, respectively.

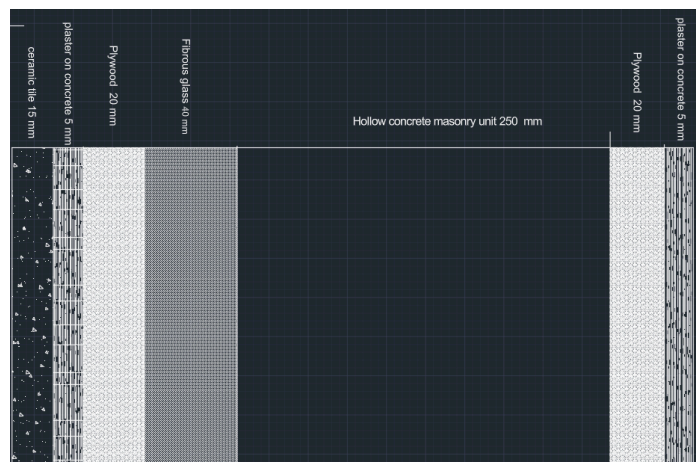


Figure 2.18. Exterior walls components

2.5.4. Parapets

According to the 2020 International Building Code, minimum roof parapet heights are 762 mm; thus, the design will be pursued with the selected height of 1 meter. Roof parapets shall be in hollow concrete masonry, 250 mm thick. 15 plaster on concrete shall be smeared on the internal side, with glazing provided on the exterior with plaster on concrete cladding of the same type. Thus, the total thickness of the parapets is found to be 280 mm.

2.5.5. Roof drainage system

For proper drainage, the rooftop will be designed with a 2% slope to ensure that stormwater flows efficiently toward the drainage system, in accordance with the guidelines set in Section 1507.10.1.

2.5.6. Shaft

There are six shafts in the building; two of them lead to the stairs, while the other four lead to the elevators. Due to the building's two blocks, the shafts will be arranged symmetrically in each block. The elevator shaft is 1800 mm by 2400 mm. Two stair shafts measuring 3000 mm to 1500 mm will function as fire emergency staircases along the building's sides. The elevator shaft walls will be composed of concrete, and they will be fireproof for two hours (IBC, 2020).

2.5.7. Elevators

For elevators from Section 3001:

- Minimum interior dimensions for an accessible elevator cab: 51 inches (1,295 mm) deep by 80 inches (2,032 mm) wide.
- The door opening must be at least 36 inches (914 mm) wide to allow wheelchair access (ADA/IBC).
- Elevator car must accommodate an ambulance stretcher (610 mm by 2134 mm)

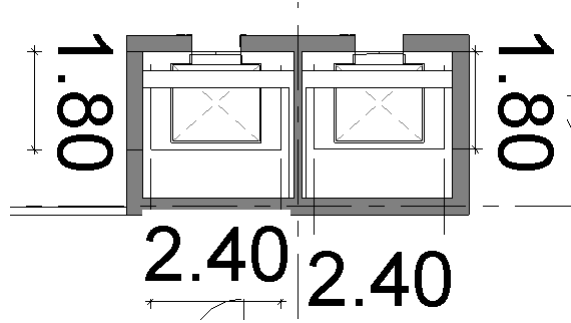


Figure 2.19. Elevator Dimensions

2.5.8. Stairs

Occupancy load is an important factor to determine the minimum number of exits. According to Section 1004.5 of the International Building Code (IBC), the occupant load factor for residential spaces is 200 gross square feet per occupant. To calculate the maximum occupancy load, the area of the largest private dwelling is used—in this case, a 104.3 square meter apartment. The following equations will then be applied for further calculations.

$$1 \text{ square foot} = 0.0929 \text{ m}^2$$

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

$$\text{Occupancy load per room} = \frac{104.3}{18.58} = 5.61$$

$$\text{Occupancy load per story} = 5.61 * 4 = 22.44$$

According to Section 1006.3.3, the occupancy load per story falls within the range of 1-500, which requires a minimum of 2 exits. Moreover, the maximum occupancy load per space, the value of which is 5.61, is less than the 20 specified in the Spaces with One Exit or Exit Access Doorway table in Section 1006.2.1. Therefore, only one entrance door per apartment is permitted, as long as the common path does not exceed 125 feet (38.1 meters). In this project, the maximum common path is 18.01 meters, which complies with this requirement.

Section 1011 of the International Building Code (IBC) provides specific requirements for the design and construction of stairways to ensure safety and accessibility. Key dimensions include: Minimum Stair Width:

- For buildings with an occupant load of 50 or more, the minimum width is 44 inches (1,118 mm). For an occupant load of less than 50, the minimum width is 36 inches (914 mm).

Tread and Riser Dimensions:

- The maximum riser height is 7 inches (178 mm).
- The minimum tread depth is 11 inches (279 mm).
- Risers and treads must have uniform dimensions throughout the stairway.

Headroom Clearance:

- Stairways must have a minimum headroom clearance of 80 inches (2,032 mm) measured vertically from the tread nosing to the ceiling or any obstruction.

Landings:

- Landings are required at the top and bottom of each stairway.
- The depth of landings must be at least as wide as the stairway, with a minimum depth of 36 inches (914 mm).

Handrails:

- Stairways must have handrails on at least one side if serving an occupant load of fewer than 50. For larger occupant loads, handrails are required on both sides.
- Handrails must be placed between 34 inches and 38 inches above the tread nosing.

Stairway Angle:

- The slope of the stairway must remain within a safe angle, typically resulting from the riser and tread dimensions.

Stair dimensions chosen for this building are illustrated in the figure below:

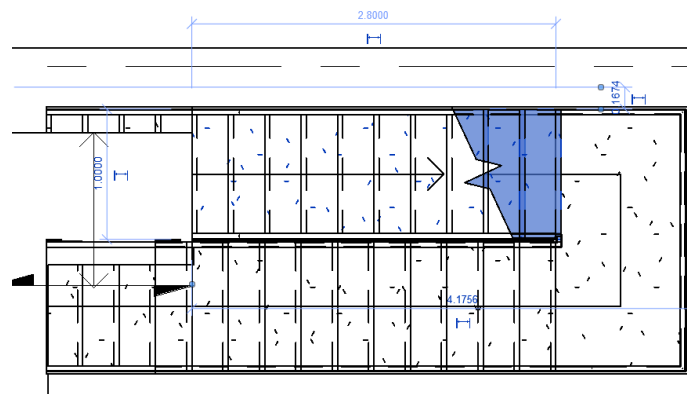


Figure 2.20. Stair Dimensions

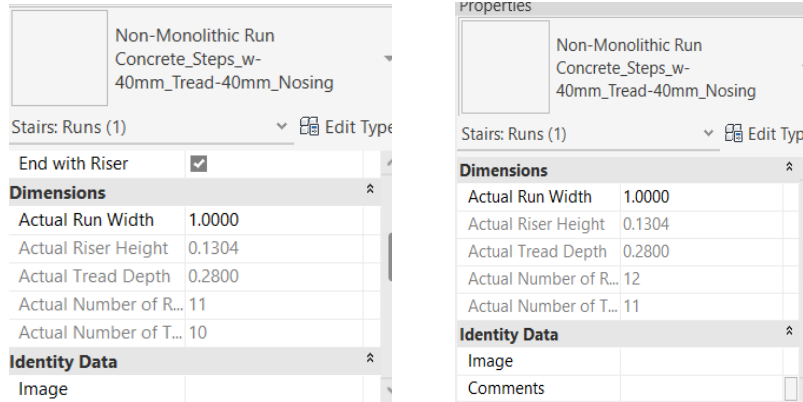


Figure 2.21. Stair Dimensions, Riser & Tread Depth

2.5.9. Doors

As per Section 1004.5 of the International Building Code (IBC), the number of exit doors is determined based on the occupancy load of the space. For specific calculations of the minimum number of exits required per story, refer to Section 1004.5(b).

Regarding door dimensions, Section 1010.1.1 specifies the following minimum requirements:

- Each dwelling unit must have at least one egress door with a minimum clear width of 813 mm and a clear height of 2,032 mm.
- Bedroom and bathroom doors typically require a minimum width of 762 mm and a height of 2,032 mm.
- Minimum spacing between doors in a series: 1,219 mm

Based on these criteria, a door with a clear opening width of 910 mm and a height of 2110 mm was selected. This door meets the required height and spacing specifications.

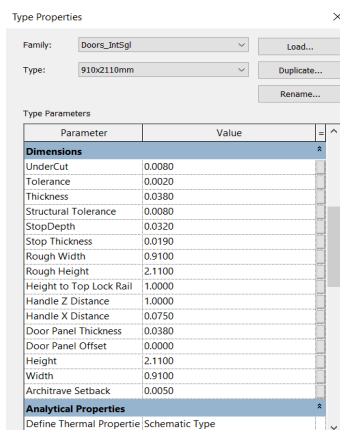


Figure 2.22. Door dimensions

2.5.10. Windows

According to Section 1204.2 of the International Building Code (IBC), every habitable room in a residential building must be provided with natural light, which is typically achieved through windows. The guideline specifies that the total window area should be at least 8% of the room's floor area. This standard has been adhered to in the project, ensuring that each habitable room features windows constituting more than 8% of the total room area.

Minimum Opening Area:

- The minimum net clear opening for emergency escape and rescue windows in habitable spaces is 5.7 square feet (0.53 m²). The window must provide a clear opening of at least 20 inches (508 mm) in width and 24 inches (610 mm) in height (IBC Section 1030.2).

Minimum Sill Height:

- For emergency escape windows, the sill height must not exceed 44 inches (1,118 mm) above the finished floor (IBC Section 1030.2).

The project complies with these specifications for each window type, with most rooms having generously sized windows to maximize natural light, fully satisfying the required criteria.

The screenshot displays a software interface for configuring window properties. At the top, the 'Family' is set to 'Windows_Sgl_Plain' and the 'Type' is '1810x1810mm 2'. Below these are buttons for 'Load...', 'Duplicate...', and 'Rename...'. The main section is titled 'Type Parameters' and contains a table with two columns: 'Parameter' and 'Value'.

Parameter	Value
Material Glass	Glass
Material Frame	Window Frame
Dimensions	
Window Board Projection	0.0250
Window Board Extension	0.0250
Rough Width	1.8100
Rough Height	1.8100
Height	1.8100
Width	1.8100
Glazing Thickness	0.0240
FrameOffset - External	0.0500
Frame Depth	0.0650
Cill Depth	0.0875
Cavity Closer Width	0.0500
Cavity Closer Offset from	0.1025
Cavity Closer Depth	0.0850
Analytical Properties	
Define Thermal Properties	Schematic Type
Visual Light Transmittance	0.900000

Figure 2.23. Window dimensions

2.5.11. Corridors

According to the International Building Code (IBC), corridor dimensions are designed to ensure safety and accessibility. Key requirements include:

- Corridors must be at least 44 inches (1,118 mm) wide for egress (IBC Section 1018.2).
- For occupant loads over 50, corridors may need to be 60 inches (1,524 mm) wide.

Accessible routes must have a minimum width of 36 inches (914 mm) (IBC Section 1104.1).

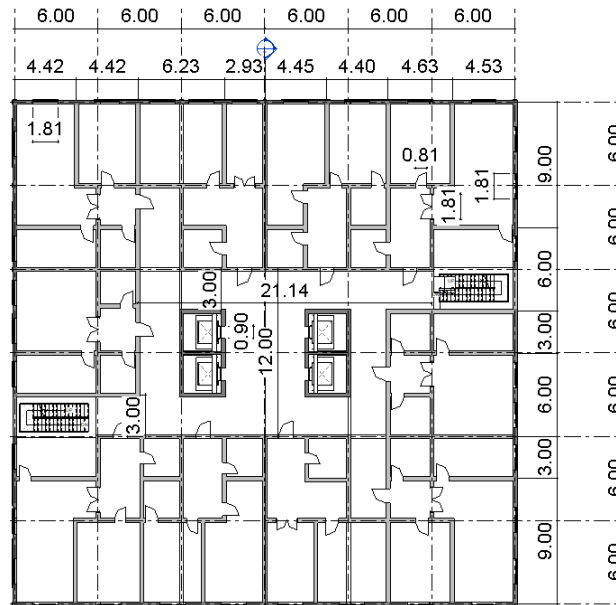


Figure 2.24. Corridors dimensions

2.5.12. Maximum distance from the room to the building entrance

For a high-rise residential building in Cupertino, California, the maximum allowable distance from a room to the nearest building entrance or exit must comply with the International Building Code (IBC) and California Building Standards Code. According to the IBC 2024, the maximum allowable exit travel distance is determined based on the building's occupancy classification and whether an automatic sprinkler system is installed. For this residential building, classified as R-2 under Section 302.1, Section 1017.2 specifies that the maximum travel distance from the farthest room to the building entrance should not exceed 250 feet (76.2 meters), provided that a sprinkler system is in place.

Maximum travel distance of the left entrance is 18.01 meters:

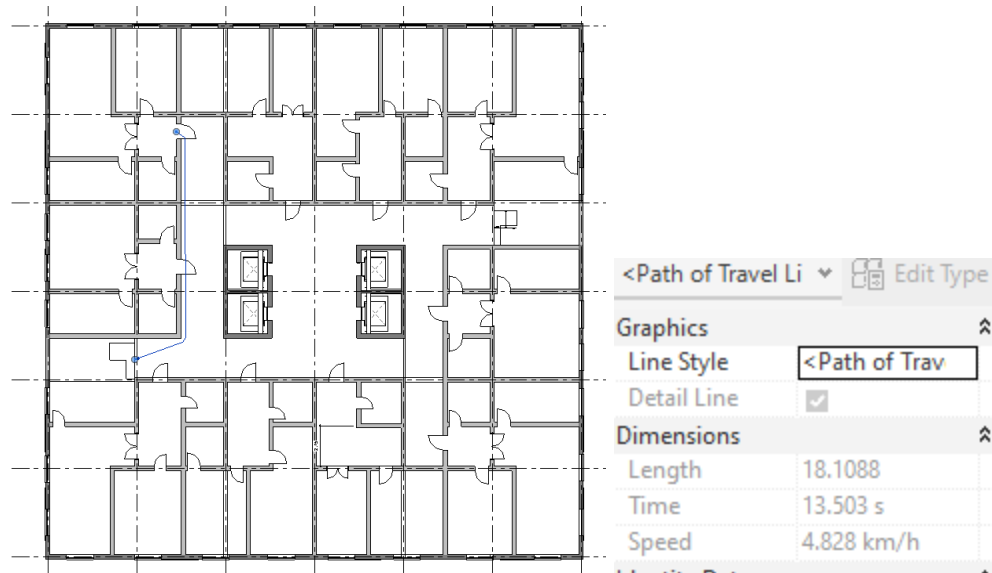


Figure 2.25 . Maximum Travel Distance, Left Entrance

Maximum travel distance of the right entrance is 17.9 meters:

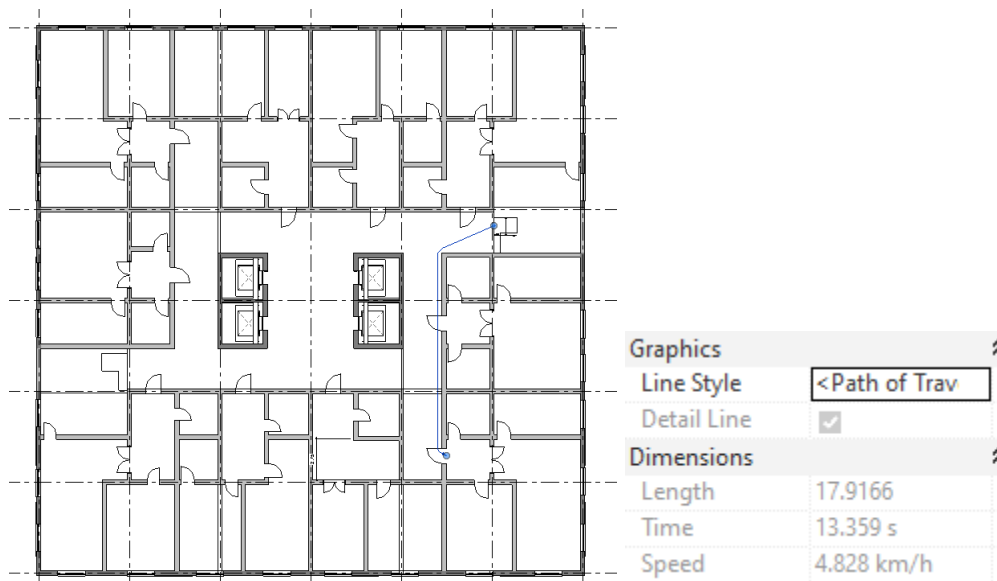


Figure 2.26 . Maximum Travel Distance, Right Entrance

The floor plan of the project adheres to the specified building code, maintaining a maximum exit travel distance of 18.01 meters.

2.5.13. Area for the residential buildings rooms

According to Section 602.2, this residential building qualifies as a Type I construction project since it primarily consists of noncombustible materials. Referring to the Allowable Area Factor table in Section 506.2, R-2 occupancy buildings constructed with Type I materials are not subject to limitations on building area. As a result, this project's building area is considered unlimited under the code.

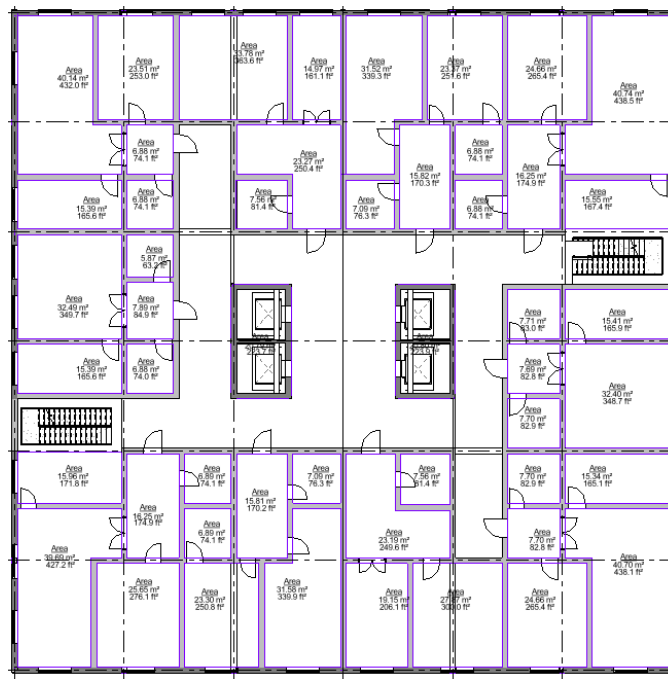


Figure 2.27. Area of building rooms

2.5.14. Allowable clear building height

The allowable building height is entirely determined by the type and presence of the sprinkler system. For this project, the NFPA 13 system, as outlined by the National Fire Protection Association, will be used. According to Section 504.3 of the International Building Code (IBC), this system allows for an unlimited building height. The same condition applies to the minimum number of stories, as specified in Section 504.4—since the NFPA 13 system is implemented, the number of floors is also unrestricted.

2.6. Fire safety design

2.6.1. Fire systems

In accordance with the International Fire Code (2020), our building will be equipped with automatic fire sprinkler systems. In addition, Class A, E, and F fire extinguishers will be placed in order to comply with the IBC's fire safety regulations for residential structures.

2.6.2. Evacuation routes

The emergency exit plan and placement of fire extinguishers was thoroughly thought over and reflected in Figures 2.28 and 2.29. The emergency plan for the 2nd floor would apply uniformly for all residential floors (2 - 12) because their layout is identical.

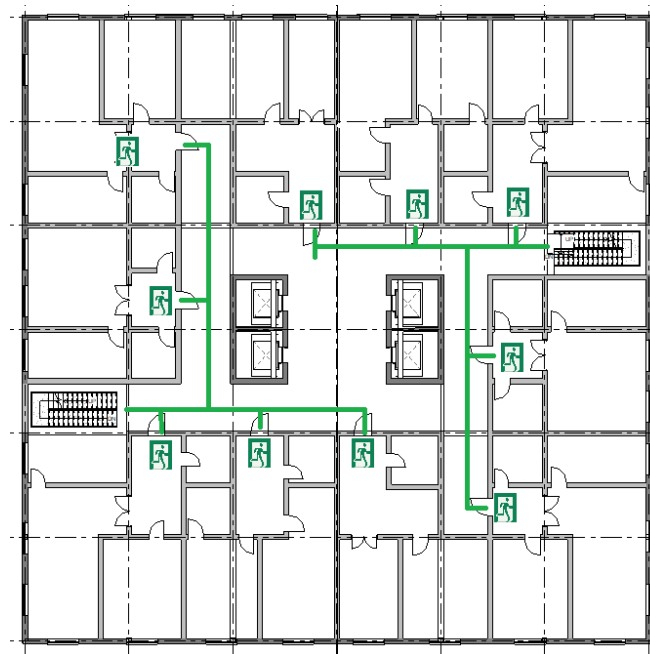


Figure 2.28. 2-12 floor emergency plan

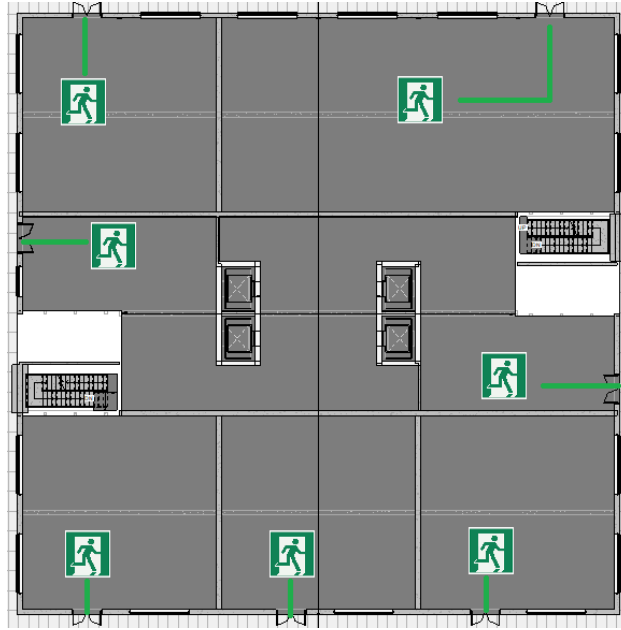


Figure 2.29. 1st floor emergency plan

2.7. Parking and site layout

According to the City of Cupertino, California, Municipal Code, Chapter 19.124, the following regulation were outlined for this project:

- For a building of residential purposes, medium to high-density multiple story condominium, as in our project, it is classified as R-4 category of occupancy.
- Medium to Highest Density Multiple Story Condominium the number of parking spaces will be 2 per dwelling unit.
- The dimensions of parking space dimensions are given in Figure 11.
- “Bicycle parking shall be provided in multi-family residential developments and in commercial districts”
- For bicycle parking, shall be conveniently located and adjacent to on-site bicycle circulation pedestrian routes
- For R-4 type residential development, Class III facility bicycle parking, which are individual bicycle racks without installed chains or locks for the bicycles.
- Class III facility requires a minimum of 1 bicycle stall per 10 residential units.

Table 19.124.040(B)

Type of Parking Stall	Angle (In Degrees)	Stall Width	Aisle Width (One-Way Aisle)	Aisle Width (Two-Way Aisle)	Car Space Depth
		(A)	(B)	(B)	(C)
Uni-Size	0°	8.5	10.0	18.0	22.0
	30°	8.5	10.0	18.0	18.0
	35°	8.5	10.0	18.0	18.0
	40°	8.5	10.0	18.0	18.0
	45°	8.5	10.0	18.0	18.0
	50°	8.5	10.0	18.0	18.0
	55°	8.5	11.5	18.5	18.0
	60°	8.5	13.0	19.0	18.0
	65°	8.5	14.5	19.5	18.0
	70°	8.5	16.0	20.0	18.0
	90°	8.5	N/A	22.0	18.0

NOTES TO TABLE:

* For handicap accessible spaces, please refer to § 1118A.4 of 1994 Uniform Building Code.

* For further information, please refer to the Public Works Department Standard Details.

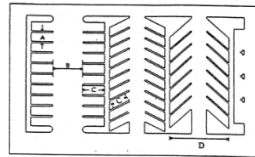


Figure 2.30. Parking spot size requirements. (City of Cupertino Municipal Code)

According to the ADA government website:

- There should be a certain minimum number of accessible parking spaces for disabled. For 101 to 150 parking spots, a minimum of 5 accessible parking spots should be provided, one of which must be for a van accessible space.
- The sizes are provided in the Figure 2.31.

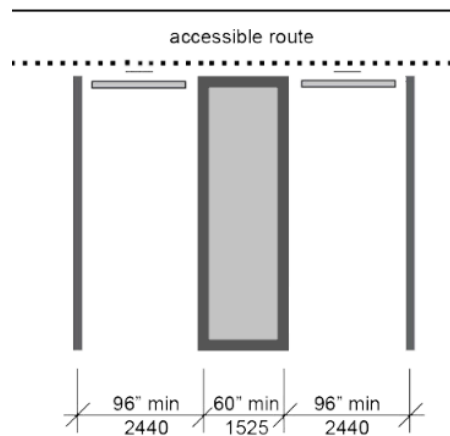


Figure 2.31. Accessible parking space size requirements. (ADA, n.d.)

Based on the regulations presented above, the parking space was designed as following.

The total number of parking spots for the residential development will be 110. For 55 residential in the building, with 2 parking spots per unit, a total number of 110 parking places will be designed. Next, the parking spot sizes will be determined based on Figure 1.8 requirements, with a 0° angle - perpendicular parking to the main Wolfe road. So, the universal size parking space dimensions will be used: individual parking space sizes - 8.5' by 22' with a space of 18' or 2.59m by 6.7m with a space of 5.49m.

For bicycle space regulations, a minimum of 6 bicycle parking spots should be placed to meet the minimum requirements for the total residential development - for 55 residential units, 5.5 bicycle parking spaces. However, because we want to create easier access and promote alternative transportation types for the LEED certification, we propose to install 27 bicycle parking spaces to accommodate for at least half of the dwelling units available.

With this reason, the following 2d view plan of the parking spot was designed (Figure 2.30).

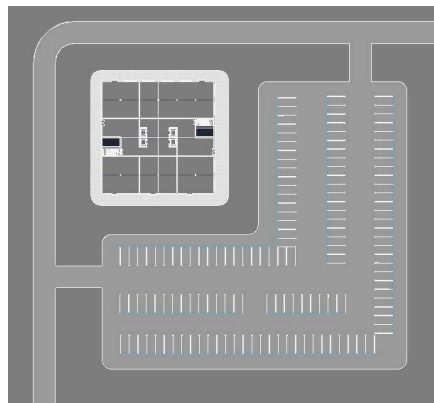


Figure 2.32. Designed parking spot for the project.

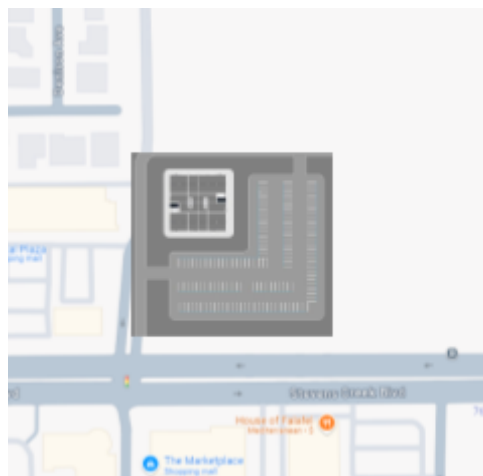


Figure 2.33. Location of the parking facility and the residential complex on the map.

2.8. Concrete mixture

4 alternative binary blended combinations and a baseline case concrete mixture were created in order to conduct a thorough durability study utilizing the Life-365 service life prediction software. Finding the best composition in terms of sustainability and long-term performance is the aim of this comparison.

1. Baseline Mixture (Control)

The baseline mixture represents a conventional concrete mix with no supplementary cementitious materials (SCMs), and it is characterized by the following composition:

- Water-to-cement (w/c) ratio: 0.35
- Fly ash content: 0%
- Silica fume content: 0%
- Slag content: 0%

This mix serves as the reference point for evaluating the effects of SCM inclusion on durability and service life extension.

2. Alternative Mixture 1: Fly Ash Blend

This mixture incorporates Class F fly ash as a partial replacement for Portland cement:

- w/c ratio: 0.35
- Fly ash content: 15%
- Silica fume content: 0%
- Slag content: 0%

Fly ash contributes to improved workability and long-term strength development while reducing the environmental footprint of the cementitious content.

3. Alternative Mixture 2: Silica Fume Blend

This alternative utilizes silica fume to enhance strength and reduce permeability:

- w/c ratio: 0.35

- Fly ash content: 0%
- Silica fume content: 10%
- Slag content: 0%

Due to its ultrafine particle size and pozzolanic reactivity, silica fumes significantly increase durability and compressive strength.

To regulate the setting time and stop calcium nitrite from infiltrating, an additional 5L/m³ calcium nitrite inhibitor was introduced. Additionally, in order to determine their impact on the concrete's life cycle, the membrane and sealers were applied in various mixes.

4. Alternative Mixture 3: Slag Blend

This mix includes slag to improve durability performance:

- w/c ratio: 0.35
- Fly ash content: 0%
- Silica fume content: 0%
- Slag content: 30%

Slag is known for reducing permeability and enhancing resistance to chemical attacks, which contributes to prolonged service life in chloride-exposed environments.

5. Alternative Mixture 4: Fly Ash and Silica Fume Blend

A combined blend of fly ash and silica fume is proposed to achieve balanced improvements in workability, strength, and durability:

- w/c ratio: 0.35
- Fly ash content: 15%
- Silica fume content: 5%
- Slag content: 0%

This hybrid approach leverages the benefits of both SCMs and may offer synergistic effects in terms of mechanical and durability properties.

2.9. Life cycle cost

To calculate the life cycle cost in software, following costs of materials were used.

These prices were taken from the actual market conditions in Cupertino:

Concrete - \$180/m³

Black steel - \$5/kg

Membrane - \$40/m²

Sealer - \$20/m²

Inhibitor - \$2/L

Repair - \$300/m²

Moreover, other factors like temperature and chloride exposure were automatically used in the software taking into account location. Software database doesn't have Cupertino among locations, so nearest city Los Angeles was chosen. Therefore, the chloride exposure was 0.68% wt. concrete and temperature was between 13.8°C and 21.4°C.

Calculated life cycle cost is shown in the table 2.1

Table 2.1. Total life cycle costs.

Type of mixture	Construction cost, \$	Barrier cost, \$	Repair cost, \$	Life-cycle cost, \$
Base case	41500	6280	38750	86530
Alternative 1	46890	6280	32600	85770
Alternative 2	47100	6280	30500	83880
Alternative 3	46500	6280	28900	81680
Alternative 4	48640	6280	25450	80370

The life cycle cost analysis showed that the lowest total cost over 75 years was for the Fly Ash and Silica Fume blend with calcium nitrite inhibitor, although this was the most expensive to material purchase upfront. This is due to the fact that the blend is most durable and causes significant delaying of chloride ingress due to the dense microstructure formed due to the synergistic pozzolanic activity of fly ash and silica fume. Apart from that, the application of calcium nitrite inhibitor even protects the reinforcing steel even more against corrosion initiation in the event of chlorides being present. Thus, the blend proved the longest period for the

corrosion initiation process and delayed repairs. The fewer occurrences of events and delayed timings of maintenance activities, when discounted over time, resulted in the lowest total life cycle cost. This shows that higher initial investment in durable materials can lead to substantial long-term economic benefits.

2.10. Service life

Service life was also estimated in LCC. Table xx demonstrate the service life of each 5 cases. As it can be seen from the table alternative 4 (Fly Ash 15% + Silica Fume 5%) has longest estimated service life. It is due to the fact that Fly ash improves long-term strength and reduces permeability, while silica fume significantly densifies the concrete matrix, making it more resistant to chloride penetration.

Table 2.2. Service life.

Type	Base case	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Service life	40	55.5	65	60.8	78

2.11. Corrosion prevention

For ensuring long-term sustainability of reinforced concrete structures, especially in environments exposed to chlorides, there needs to be an integrated strategy taking into consideration material selection, structural design, and protection systems. The corrosion of the steel reinforcement is mainly induced by chlorides' ingress, which degrades the passive layer encasing the steel, initiating corrosion that causes spalling, cracking, and decreased structural performance. For this reason, as a means to reduce corrosion, supplementary cementitious materials (SCMs) like fly ash, silica fume, and slag have been researched and recommended in abundance. These SCMs have an effect on making the microstructure dense by filling voids and improving the pore network, thus minimizing chlorides' diffusion significantly. Research by Thomas et al. (2012) and Aïtcin (2016) ascertains that 15–30% fly ash and up to 10% silica fume's use can increase concrete's service life by decades because they possess pozzolanic activity and have chlorides-adsorbing qualities.

In addition, corrosion inhibitors such as calcium nitrite also provide added protection by chemically stabilizing the passive film on the steel surface. Berke and Hicks (1992) demonstrated in research that corrosion initiation can be delayed by calcium nitrite even when

chlorides are introduced, particularly when combined with SCMs. Surface protection materials such as waterproof membranes and silane-based sealers have also been shown to effectively repel water and chloride penetration, as established by field tests by Bentz et al. (2011). Structurally, other measures include deeper enclosing concrete, a lower water-to-cement ratio, and adequate drainage. When these measures are integrated, as when in Alternative 4—both fly ash, silica fume, and calcium nitrite are added—the concrete produced possesses greater durability and a longer than 75-year service life, ensuring reduced costs over a lifetime and less repair needs.

2.12. Leadership in Energy and Environmental Design (LEED)

LEED - Leadership in Energy and Environmental Design is the most common green building assessment rating of sustainability developed by the US Green Building Council (USGBC, n.d.). The LEED for Homes is the most suitable for this construction project in terms of the certification scope and their requirements (energy efficiency, water efficiency, indoor environmental quality, and sustainable site development) (Walsh, 2024).

LEED certification ratings are as follows: Certified (40-49 points), Silver (50-59), Gold (60-79), Platinum (80+) (Walsh, 2024). Therefore to obtain the certification, a minimum of 40 points is required. The building passes the minimum requirements (USGBC, 2023):

- The building is located in a permanent land, meaning that it will not be changing its location throughout the years of use. It will be constructed and operated on the same land area.
- The project will use all area dedicated to the construction.
- The building has a floor area of 1296 m², which meets the minimum requirement of 93 m².

The "greenness" of the building is evaluated based on seven different criteria, which are as follows: energy and atmosphere, sustainable sites, indoor environmental quality, materials and resources, water efficiency, innovations in design, and regional importance. **Figure xx** shows the LEED score for this project. The acquired LEED score is 76, indicating Gold certification. These outstanding results indicate that this project is environmentally friendly and sustainable.

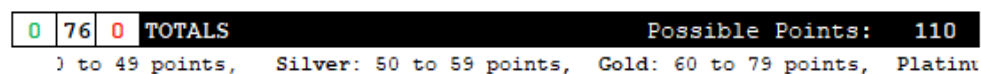


Figure 2.34. LEED scores.

2.12.1. Location and Transportation

This building earned 2 points for Surrounding Density and Diverse Use by creating a dense, pedestrian-oriented community with a mix of residences and office space. The location near VTA bus stops satisfied requirements for Access to Quality Transit (5 points), encouraging public transit use. Sustainable transportation features include a Reduced Parking Footprint (1 point), achieved by focused reductions in parking spaces meeting local code and promoting shared usage. Low-emission transport is also promoted by the inclusion of Electric Vehicle charging stations (1 point), aligned with California's clean energy policies.

Y	?	N			
			Credit	Integrative Process	1
0	9	0		Location and Transportation	16
Y			Credit	LEED for Neighborhood Development Location	16
		N	Credit	Sensitive Land Protection	1
		N	Credit	High Priority Site and Equitable Development	2
Y	2		Credit	Surrounding Density and Diverse Uses	5
Y	5		Credit	Access to Quality Transit	5
		N	Credit	Bicycle Facilities	1
Y	1		Credit	Reduced Parking Footprint	1
Y	1		Credit	Electric Vehicles	1

Figure 2.35. Location and transportation scoreboard.

2.12.2. Sustainable Sites

This project also demonstrates comprehensive sustainable site design as revealed in its rating under the LEED Sustainable Sites category. It earned the Site Assessment credit (1 point) by conducting comprehensive analysis of site conditions prior to development. Environmental protection measures made a significant contribution to the project's sustainability record. Full points were achieved for Protect or Restore Habitat (2 points) through sensitive site design that preserved natural areas and incorporated native landscaping. The Open Space credit (1 point) was achieved by providing accessible green space that enhances occupant health. Stormwater management was specifically targeted, with the project receiving optimum score for Rainwater Management (3 points) through novel design features controlling runoff quantity and quality. Heat Island Reduction strategies (2 points) were attained through reflective roof and pavement materials and extensive vegetation.

Although the project amassed most of the credits possible in this category, the credit for Light Pollution Reduction (1 point) was not pursued. This was a design decision based on site-specific conditions of exterior lighting requirements for the mixed-use development. The project's amassing of 9 out of 10 possible points in the Sustainable Sites category demonstrates outstanding environmental stewardship in site selection and development methods.

0	9	0	Sustainable Sites		10
Y			Prereq	Construction Activity Pollution Prevention	Required
Y	1		Credit	Site Assessment	1
Y	2		Credit	Protect or Restore Habitat	2
Y	1		Credit	Open Space	1
Y	3		Credit	Rainwater Management	3
Y	2		Credit	Heat Island Reduction	2
		N	Credit	Light Pollution Reduction	1

Figure 2.36. Sustainable sites scoreboard.

2.12.3. Water Efficiency

Moreover, building attained overall effective actions in water conservation and was awarded 6 out of the 11 obtainable points for the Water Efficiency category. The initiative was awarded maximum credit for Indoor Water Use Reduction (3 credits) by exceeding baseline performance through low-flow devices and water-saving appliances. Additional credit was achieved under Optimize Process Water Use (2 credits) through cooling tower water-saving measures and optimally designed HVAC systems. Water Metering credit (1 credit) was achieved by the installation of submeters to meter specific end uses.

0	6	0	Water Efficiency		11
Y			Prereq	Outdoor Water Use Reduction	Required
Y			Prereq	Indoor Water Use Reduction	Required
Y			Prereq	Building-Level Water Metering	Required
		N	Credit	Outdoor Water Use Reduction	2
Y	3		Credit	Indoor Water Use Reduction	6
Y	2		Credit	Optimize Process Water Use	2
Y	1		Credit	Water Metering	1

Figure 2.37. Water efficiency scoreboard.

2.12.4. Energy and Atmosphere

Building demonstrates high energy performance with a grade of 19 of a maximum possible 33 for the Energy and Atmosphere rating. The project earned considerable points through Enhanced Commissioning (4 out of 6 available points), reflecting comprehensive system

testing and ongoing performance monitoring. The standout was in Optimize Energy Performance, where 14 out of 18 points were realized through a combination of high-efficiency building envelope design, advanced HVAC systems, and energy recovery strategies. Additional points were gained for Advanced Energy Metering (1 point), enabling accurate tracking of energy usage patterns. While the project did not pursue Grid Harmonization (2 points), Renewable Energy (8 points), or Enhanced Refrigerant Management (1 point) aggressively, the high points earned in energy optimization reflect a firm commitment to energy efficiency.

0	19	0	Energy and Atmosphere		33
Y			Prereq	Fundamental Commissioning and Verification	Required
Y			Prereq	Minimum Energy Performance	Required
Y			Prereq	Building-Level Energy Metering	Required
Y			Prereq	Fundamental Refrigerant Management	Required
Y	4		Credit	Enhanced Commissioning	6
Y	14		Credit	Optimize Energy Performance	18
Y	1		Credit	Advanced Energy Metering	1
		N	Credit	Grid Harmonization	2
		N	Credit	Renewable Energy	5
		N	Credit	Enhanced Refrigerant Management	1

Figure 2.38. Energy and atmosphere scoreboard.

2.12.5. Materials and Resources

The project achieved a perfect score in the Materials and Resources category, earning all 13 available points through comprehensive sustainable material strategies. Maximum points were earned for Building Life-Cycle Impact Reduction (5 points) through systematic selection of materials and design methods that reduce environmental footprints throughout the building's life cycle. The project set a leadership example in sustainable purchasing by earning all available points for Environmental Product Declarations, demonstrating transparency of material impacts. Responsible sourcing practices gained 2 points with the Sourcing of Raw Materials credit, favoring local and recycled materials. Another 2 points were earned for Material Ingredients by selecting products with certified chemical inventories and low-emitting properties.

The project attained its perfect score with full marks in Construction and Demolition Waste Management (2 marks), which redirected over 75% of construction wastes away from landfills through accurate sorting and recycling schemes. This record achievement reflects an integrated material sustainability strategy to all phases of the project.

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

Figure 2.39. Materials and resources scoreboard.

2.12.6. Indoor Environmental Quality

15 of the 16 possible points in the Indoor Environmental Quality category was scored , showing outstanding commitment to occupant health and comfort. The project earned full points for Enhanced Indoor Air Quality Strategies (2 points) through high-level air filtration and ventilation systems. Material selection was a firm role, with top points earned for Low-Emitting Materials (3 points) in adhesives, paints, floor systems, and composite wood products. Construction Indoor Air Quality Management Plan (1 point) protected workers during construction and ensures long-term air quality. Post-occupancy Indoor Air Quality Evaluation (2 points) verified healthy conditions through extensive testing. Occupant comfort strategies include Thermal Comfort verification (1 point), optimization of Interior Lighting design (2 points), and substantial Daylight penetration (3 points) through diligent window orientation. Quality Views (1 point) were achieved for most regularly occupied spaces. The only credit not pursued was Acoustic Performance (1 point), a reflection of the project's urban location and mixed-use nature.

0	15	0	Indoor Environmental Quality		16
Y			Prereq	Minimum Indoor Air Quality Performance	Required
Y			Prereq	Environmental Tobacco Smoke Control	Required
Y	2		Credit	Enhanced Indoor Air Quality Strategies	2
Y	3		Credit	Low-Emitting Materials	3
Y	1		Credit	Construction Indoor Air Quality Management Plan	1
Y	2		Credit	Indoor Air Quality Assessment	2
Y	1		Credit	Thermal Comfort	1
Y	2		Credit	Interior Lighting	2
Y	3		Credit	Daylight	3
Y	1		Credit	Quality Views	1
		N	Credit	Acoustic Performance	1

Figure 2.40. Indoor environmental quality scoreboard.

3. Structural Part

3.1. Design and analysis gravity load resisting system (GLRS)

3.1.1. Calculation of Dead, Live and Snow Loads

3.1.1.1. Calculation of Dead Loads

The calculations for dead load in the case of floor and roof dead loads from Chapter 3 of the ASCE 7-16 were done. The dead loads of the floors have been calculated using two types of finishing for the first floor and walls. The dead load calculations result for the floor finishing, walls, and roof are summarized in Tables 3.1 to 3.3.

Table 3.1. Dead load for 1 floor and corridors

Element	Material		Thickness, mm	load, kN/m ²
Finishing	Cover	Slate stone	33	0,028
	Levelling	Stone Concrete	40	0.023
	Waterproofing	Liquid applied	10	0.05
	Insulation	Rigid insulation	13	0,04
Structural slab	Slab	reinforced concrete	125	23.6
Ceiling	Mechanical dust allowance			0.19

	Acoustical fiber board		13	0.05
Total Load				4.828

Table 3.2. Dead load for typical floor

Element	Material		Thickness, mm	load, kN/m ²
Finishing	Cover	Subflooring	19	0.14
	Levelling	Cinder Concrete Screed	40	0.023
	Waterproofing	Liquid applied	10	0.05
	Insulation	Rigid insulation	13	0,04
Structural slab	Slab	reinforced concrete	125	23.6
Ceiling	Mechanical dust allowance			0.19
	Acoustical fiber board		13	0.05
Total Load				4.94

Table 3.3. Dead load of the roof .

Element	Material		Thickness, mm	load, kN/m ²
Finishing	Cover	Three-ply ready roofing		0,05
	Levelling	Urethane foam with skin	100	0,0009
	Waterproofing	Bituminous gravel covered		0.26
	Insulation	Lightweight concrete	40	0.015
Structural slab	Slab	reinforced concrete	125	23.6
Ceiling	Mechanical dust allowance			0.19
	Acoustical fiber board		13	0.05
Total dead load				4.79

The load on interior walls, exterior walls and from parapets were calculated in sequence in the sequence. It was considered that the weight of interior walls was uniformly distributed on the floor area and that the loads due to exterior walls and parapet walls were applied only to the

edge beams. The calculations of loads obtained from material densities adopted for the walls and parapets are shown in Tables 3.4-3.6

Table 3.4. Dead load for exterior walls

Material type	Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)
Exterior finish	ceramic tile	15	3000	6000	0,27	2400
Cement plaster	plaster on concrete	5	3000	6000	0,09	1 860,00
Air infiltration barrier	Plywood	20	3000	6000	0,36	500,00
Insulation	Fibrous glass	40	3000	6000	0,72	1857
Autoclaved aerated concrete block	Hollow concrete masonry unit	250	3000	6000	4,5	1500
Air infiltration barrier	Plywood	20	3000	6000	0,36	500,00
cement plaster	plaster on concrete	5	3000	6000	0,09	1 860,00
	Total	355			6,39	9430

Table 3.5. Dead load for interior walls

Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)
plaster on concrete	5	3000	6000	0.09	2 400,00	216
Fibrous glass	20	3000	6000	0.36	1 857,00	668.52
Hollow concrete masonry unit	75	3000	6000	1.35	1 500,00	2025
Fibrous glass	20	3000	6000	0.36	1857	668.52
plaster on concrete	5	3000	6000	0.09	2 400,00	216
Total	125			2.25		3794

Table 3.6. Dead load for roof parapets

Material type	Thickness (mm)	Height (mm)	Span (mm)	Volume (m3)	Density (kg/m3)	Weight (kg)
plaster on concrete	15	1000	6000	0.09	2400	216
Hollow concrete masonry unit	250	1000	6000	1.5	1500	2250
plaster on concrete	15	1000	6000	0.09	2400	216

Total				1.68		2682
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As the partition walls were evenly distributed across the floor area, the load calculated for the partition walls (kN/m²) is:

$$\text{Floor area} = 1224 \text{ m}^2$$

$$\text{Uniformly distributed load of partition walls} = 2.580 \text{ kN/m}^2$$

Additionally, the initial load from the stairs on a single floor for one stair shaft was also determined, which is:

$$\text{Load from stairs} = 53.225 \text{ kN}$$

3.1.1.2. Calculation of Live Loads

Minimum uniformly distributed loads for the residential building were found according to ASCE 7-16, as presented below:

$$\text{Private rooms and corridors serving them: } L_0 = 1.92 \text{ kN/m}^2$$

$$\text{Public rooms: } L_0 = 4.97 \text{ kN/m}^2$$

$$\text{Corridors serving public rooms: } L_0 = 4.97 \text{ kN/m}^2$$

$$\text{Ordinary flat roofs: } L_0 = 0.96 \text{ kN/m}^2$$

$$\text{Stairs and exit ways: } L_0 = 4.97 \text{ kN/m}^2$$

ASCE 7-16 specifies that a live load reduction shall be applied to all the uniformly distributed loads, except for the public rooms. The floor live load reduction shall be calculated by the formula below:

$$L = L_0 \left(0.25 + \frac{4.57}{\sqrt{K_{ll} A_t}} \right) \quad (1)$$

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

Interior columns: $K_{ll} = 4$

Exterior columns without cantilever slabs: $K_{ll} = 4$

Corner columns: $K_{ll} = 4$

Edge beams without cantilever slabs: $K_{ll} = 2$

Interior beams: $K_{ll} = 2$

The floor live load calculations for each structural member can be seen in Table 3.7.

Table 3.7. Floor live load calculations.

			Span (m)	Span (m)	A_t	K_{ll}	Reducti on	L_0	L
General floor	Apartment and corridors	Exterior columns	3	6	18	4	0,789	1,92	1,51
		Interior columns	6	6	36	4	0,631	1,92	1,21
		Corner columns	3	3	9	4	1	1,92	1,92
		Edge beams	3	6	9	2	1	1,92	1,92

		Interior beams	3	6	18	2	1	1,92	1,92
	Stairs and emergency exit	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	3	6	18	2	1	4,79	4,79
		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	3	6	9	2	1	4,79	4,79
		Interior beams	3	6	18	2	1	4,79	4,79

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

$$L_r = L_0 R_1 R_2 \quad \text{where } 0.58 \leq L_r \leq 0.96 \quad (2)$$

$$1 \quad \text{for } A_t \leq 18.58m^2$$

$$R_1 = 1.2 - 0.011A_t \quad \text{for } 18.58m^2 \leq A_t \leq 55.74m^2$$

$$0.6 \quad \text{For } A_t \geq 55.74m^2$$

Table 3.8. Roof live load calculations.

	Space (m)	Span (m)	At (m ²)	KLL	R1	R2	L0 (kN/m ²)	L (kN/m ²)
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

3.1.1.3. Calculation of Snow Loads

Los Angeles has no snow load; hence, calculations for snow load were not part of the design project (American Society of Civil Engineers, 2017).

3.1.2. Estimation of Member Sizes

3.1.2.1. Determination of Structural Layout

Column spacing of 6 meters was used to ensure there is enough space when building the residential house. Such distance is within the acceptable spacing range, which ranges from 2.5 to

7.5 meters; hence, it is appropriate. Three possible GLRS systems were considered, and the most economical one was found by determining the amount of structural members. The selected system comprises a one-way slab with main beams and one secondary beam.

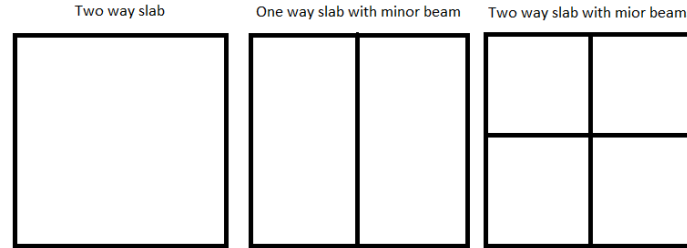


Figure 3.1. GLRS layouts

Preliminary Estimate of Member Size

Preliminary estimates of the member sizes for further analysis were determined from guidelines in Chapters 7-10 of ACI 318-19 and heuristic principles (American Concrete Institute, 2022):

- Approximate thickness of major beams for a two-way slab is on the order of 10% of the span.
- The width of major beams is about 50% of the thickness of the major beam
- The width and depth of simply supported minor beams are equal to 50% of major beam attributes

Option 1: One-way slab with minor beam

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

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

$$h_{minor} = 0.3 \text{ m}; b = 0.150 \text{ m}$$

The thickness of simply supported one-way slab is equal to l .

$$h_{slab} = L/20 = 3/20 = 0.15 \text{ m}$$

Accordingly, the volumes of members can be calculated as:

$$V(\text{slab}) = 3 \cdot 6 \cdot 0.15 \cdot 2 = 5.4 \text{ m}$$

$$V(\text{major beam}) = 0.6 \cdot 0.3 \cdot 6 \cdot 4 = 4.32 \text{ m}$$

$$V(\text{minor beam}) = 0.3 \cdot 0.15 \cdot 6 = 0.27 \text{ m}$$

$$V_{\text{Total}} = 5.4 + 4.32 + 0.27 = 9.99$$

Option 2: Two way slab

For the conventional design, the lengths of the slab and beam were assumed to be 6 meters. As previously mentioned, the thickness of the main beam (h) is 10% of its length, while its width (b) is: $b = \frac{h}{2}$, while $h = \frac{L_{\text{span}}}{10}$

$$\text{So } h = 6/10 = 0.6 \text{ m, } b = 0.6/2 = 0.3 \text{ m}$$

The solution for the thickness of the slab followed the following procedures. Considered values are assumed for the calculation of the moment of inertia and elastic modulus of both beam and slab. C40-grade concrete is chosen for both the slab and beam; that is usually the most common grade for structural elements that are designed to bear heavier loads. The calculations were as follows:

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

The modulus of elasticity

$$E_b = E_s = 4700 \sqrt{f_c} = 29725.41 \text{ MPa} \quad (3)$$

$$I_{b,cr} = \frac{1}{12} * b * h^3 * 0.35 = \frac{1}{12} * 0.3 * 0.6^3 * 0.35 = 1.89 * 10^{-3} \quad (4)$$

$$I_{s,cr} = \frac{1}{12} * L * h^3 * 0.25 = \frac{1}{12} * 6 * 0.125^3 * 0.25 = 2.44 * 10^{-4}$$

After calculating inertia moment

$$\alpha_{fm} = \frac{E_{cb} * I_{b,cr}}{E_{cs} * I_{s,cr}} = \frac{1.89 * 10^{-3}}{2.44 * 10^{-4}} = 7.745 \quad \alpha_{fm} \geq 2 \quad (5)$$

Minimum thickness of slab can be calculated by formula

$$h = \frac{l_n(0.8 + f_y/200000)}{36 + 9\beta} < h < 3.5 \text{ in} \quad \text{and} \quad \beta = L_{yn}/L_{xn} = 1 \quad (6)$$

$$L_n = L - \text{major beam}/2 - \text{major}/2 = 6 - 0.3 - 0.15 = 5.4$$

Thickness of slab should be

$$h = \frac{5.4(0.8 + 60000/200000)}{36 + 9 * 1} = 0.132 \text{ m} > 0.09 \text{ m}$$

For the conventional slab design, the thickness $h=0.15 \text{ m}$. Consequently, the volume can be calculated as follows:

$$V_{slab} = 6 * 6 * 0.15 = 5.4$$

$$V_{beam} = 0.6 * 6 * 0.3 * 4 = 4.32$$

$$V_{total} = 5.4 + 4.32 = 9.72$$

Option 3: Two way slab with minor beams

In this part we conceded similar processes, so here are formulas and calculations

$$h_{major} = 6 * 0.1 = 0.6$$

$$b_{major} = 0.6 * 0.5 = 0.3$$

$$h_{minor} = 0.6 * 0.5 = 0.3$$

$$b_{minor} = 0.3 * 0.5 = 0.15$$

Then we calculated inertia moments and stiffness ratio

$$I_{b,cr\ major} = \frac{1}{12} * b * h^3 * 0.35 = \frac{1}{12} * 0.3 * 0.6^3 * 0.35 = 1.89 * 10^{-3}$$

$$I_{b,cr\ minor} = \frac{1}{12} * b * h^3 * 0.35 = \frac{1}{12} * 0.15 * 0.3^3 * 0.35 = 1.18 * 10^{-4}$$

$$I_{s,cr} = \frac{1}{12} * L * h^3 * 0.25 = \frac{1}{12} * 3 * 0.125^3 * 0.25 = 1.22 * 10^{-4}$$

$$\alpha_{f\ major} = \frac{E_{cb} * I_{b,cr}}{E_{cs} * I_{s,cr}} = \frac{1.89 * 10^{-3}}{1.22 * 10^{-4}} = 15.49$$

$$\alpha_{f\ minor} = \frac{E_{cb} * I_{b,cr}}{E_{cs} * I_{s,cr}} = \frac{1.18 * 10^{-4}}{1.22 * 10^{-4}} = 0.967$$

$$\alpha_{fm} = \frac{15.49 + 0.967}{2} = 8.23 > 2$$

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

$$L_n = L - major\ beam/2 - major/2 = 3 - 0.3 - 0.15 = 2.55$$

$$h = \frac{2.55(0.8 + 60000/200000)}{36 + 9 * 1} = 0.062\ m < 0.09\ m$$

Minimum thickness of slab should be 90 mm, but we took 125 mm, due increment, so volume should be

$$V_{slab} = 3 * 3 * 0.125 * 4 = 4.5$$

$$V_{beam\ major} = 0.6 * 6 * 0.3 * 4 = 4.32$$

$$V_{beam\ minor} = 0.3 * 6 * 0.15 * 2 = 0.54$$

$$V_{total} = 4.5 + 4.32 + 0.54 = 9.36$$

Based on the calculated dimensions and total volumes of the structural elements, the layout with the lowest volume—and consequently the lowest cost—was selected as the layout system for the entire building.

Table 3.9. Slab dimensions

Final decision based on member sizes							
Option	Member	L (m)	h (m)	b (m)	V (m ³)	n	Vt (m ³)
Two way slab	Slab	6,000	0,150	6,000	5,400	1	9,72
	Major beam	6,000	0,600	0,300	1,080	4	
Two way slab with minor beams	Slab	3,000	0,125	3,000	1,125	4	9,36
	Major beam	6,000	0,600	0,300	1,080	4	
	Minor beam	6,000	0,300	0,150	0,270	2	
One way with minors	Slab	6,000	0,150	3,000	2,700	2	9,99
	Major beam	6,000	0,600	0,300	1,080	4	
	Minor beam	6,000	0,300	0,150	0,270	1	

Based on table numbers we choose two-way slab with minor beams, so our structural part can be seen in the next figure

Based on the loads, the load combination on the first floor will be calculated as:

Self-weight of major and minor beams to interior column will be calculated:

$$D_{floor} = D_{walls} + D_{finishing} + D_{beams} = 7.653 \text{ kN/m}^2$$

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

Live load

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

$$L_r = 0.96 \text{ kN/m}^2$$

$$w_u = 11 * (1.2 * 7.653 + 1.6 * 4.79) + 1.2 * 4.79 + 0.5 * 0.96 = 191.555 \text{ kN/m}^2 \quad (9)$$

Since our building is a reinforced concrete structure, ACI 318-19 code was used to estimate the sizes (American Concrete Institute, 2022). Also, the weight of columns above the first floor was added for columns on each floor by iteration method. Moreover, since our building has a basement, the column sizes on the basement floor were also calculated. It should be noted that floor height above the first floor is 3.5 m, while the floor height on the first and basement floors is 3 m.

$$P_u = W_u * A_t + w_{columns above} = 5411.437 \quad (10)$$

$$\phi P_n = 0.8 \phi (0.85 f'_c A_c + f_y A_s) \quad (11)$$

Reinforcement ratio was assumed as 1%, hence, $A_c = 0.02 A_s$, $\phi = 0.65$, $f'_c = 4 \text{ MPa}$, $f_y = 420 \text{ MPa}$

$$\phi P_n = 0.8 * 0.65 * (0.85 * 41700 A_c + 420000 * (0.01 A_c)) = 20615.40 A_c$$

$$\phi P_n \geq P_u$$

$$20615.40A_c \geq 5411.437$$

$$A_c > 0.262 \text{ m}^2$$

Table 3.10. Column size estimation for interior columns

Level	Wu (kN/m ²)	Pu (kN)	Pn (kN)	Ac (m ²)	a (m)	a=b (m)
0	198,232	9645,070	20615,400	0,54900	0,741	0,750
1	182,232	8930,444	20615,400	0,43319	0,658	0,700
2	166,232	8207,177	20615,400	0,39811	0,631	0,650
3	150,231	7483,910	20615,400	0,36303	0,603	0,650
4	134,231	6765,940	20615,400	0,32820	0,573	0,600
5	118,230	6052,844	20615,400	0,29361	0,542	0,550
6	102,230	5339,748	20615,400	0,25902	0,509	0,550
7	86,230	4631,101	20615,400	0,22464	0,474	0,500
8	70,229	3926,481	20615,400	0,19046	0,436	0,450
9	54,229	3225,463	20615,400	0,15646	0,396	0,400
10	38,229	2524,445	20615,400	0,12245	0,350	0,400
11	22,228	1829,360	20615,400	0,08874	0,298	0,300
12	6,228	1134,275	20615,400	0,05502	0,235	0,300

Table 3.11. Column size estimation for edge (exterior) columns

Level	Wu (kN/m ²)	Pu (kN)	Pn (kN)	Ac (m ²)	a (m)	a=b (m)
0	208,403	5837,424	20615,400	0,28316	0,532	0,550
1	191,555	5411,437	20615,400	0,26249	0,512	0,550
2	174,707	4992,464	20615,400	0,24217	0,492	0,500
3	157,859	4573,490	20615,400	0,22185	0,471	0,500
4	141,011	4154,517	20615,400	0,20152	0,449	0,500
5	124,164	3739,570	20615,400	0,18140	0,426	0,450
6	107,316	3324,622	20615,400	0,16127	0,402	0,450
7	90,468	2913,277	20615,400	0,14132	0,376	0,400
8	73,620	2501,932	20615,400	0,12136	0,348	0,400
9	56,772	2093,765	20615,400	0,10156	0,319	0,350
10	39,924	1688,353	20615,400	0,08190	0,286	0,300
11	23,076	1282,941	20615,400	0,06223	0,249	0,300
12	6,228	877,529	20615,400	0,04257	0,206	0,300

Table 3.12. Column size estimation for corner (exterior) columns

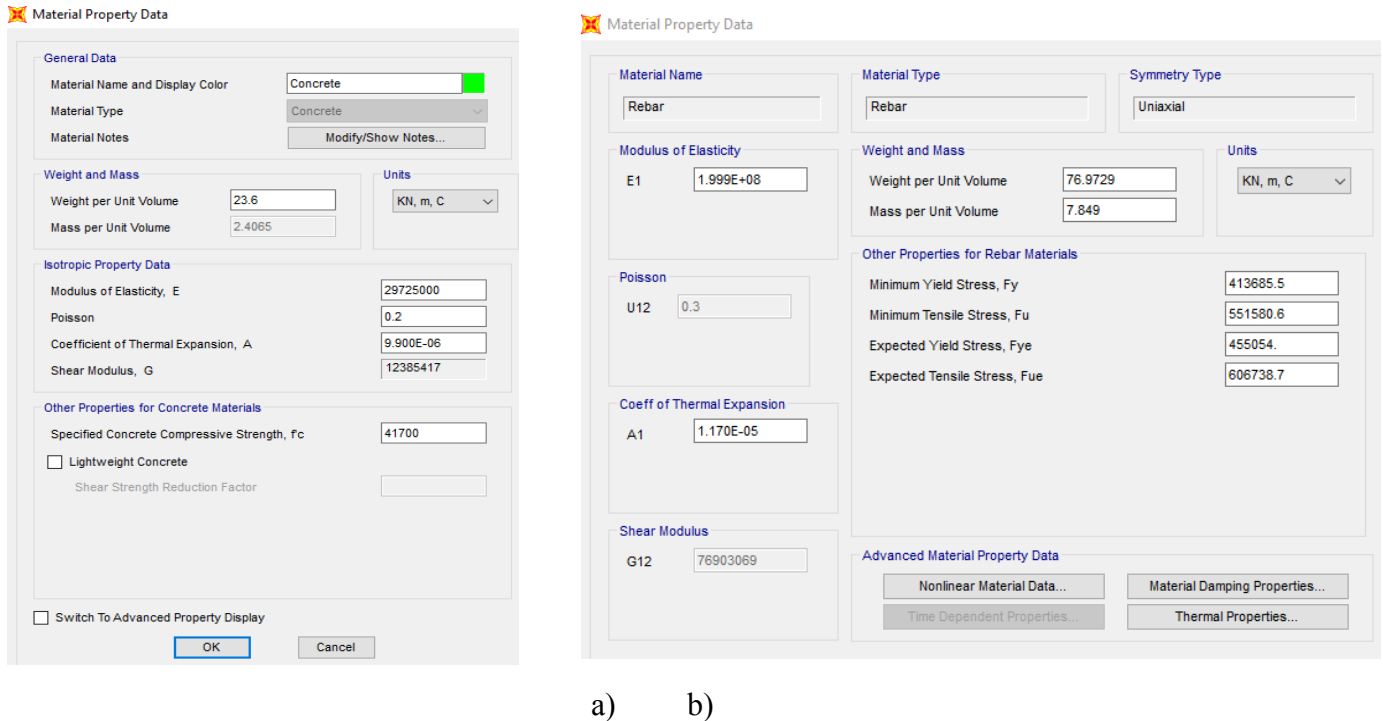
Level	Wu (kN/m ²)	Pu (kN)	Pn (kN)	Ac (m ²)	a (m)	a=b (m)
0	208,403	2938,493	20615,400	0,14254	0,378	0,400
1	191,555	2771,944	20615,400	0,13446	0,367	0,400
2	174,707	2559,490	20615,400	0,12415	0,352	0,400

3	157,859	2350,216	20615,400	0,11400	0,338	0,350
4	141,011	2140,941	20615,400	0,10385	0,322	0,350
5	124,164	1931,666	20615,400	0,09370	0,306	0,350
6	107,316	1725,146	20615,400	0,08368	0,289	0,300
7	90,468	1518,626	20615,400	0,07366	0,271	0,300
8	73,620	1312,106	20615,400	0,06365	0,252	0,300
9	56,772	1105,585	20615,400	0,05363	0,232	0,300
10	39,924	899,065	20615,400	0,04361	0,209	0,300
11	23,076	692,545	20615,400	0,03359	0,183	0,300
12	6.228	486.025	20615.400	0.02358	0.154	0.300

Table 3.13. Dimensions of columns

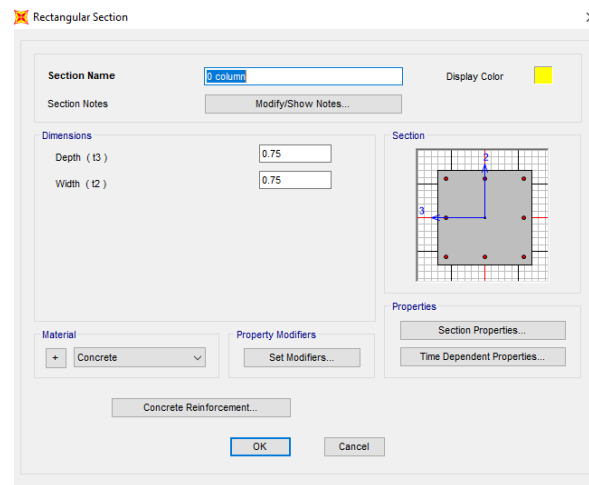
Floors	Column size, m
0	0,75
1	0,7
2-3	0,65
4	0,6
5-6	0,55
7	0,5
8	0,45
9-10	0,4
11-12	0,3

3.1.3. Assigning load to SAP Frame

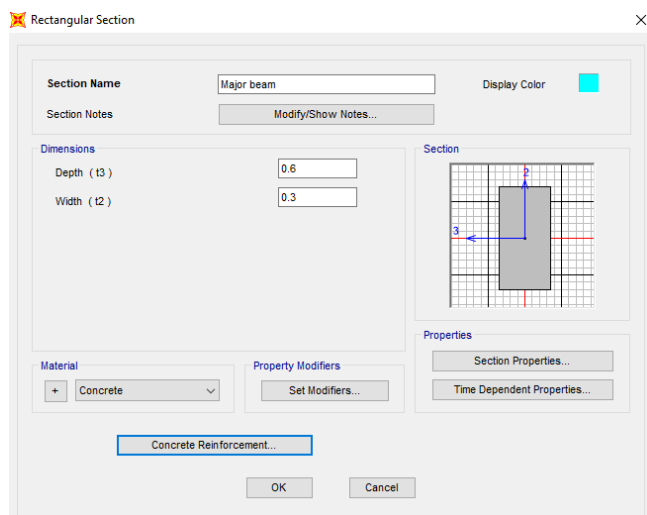


Figures 3.3. a)concrete properties and b) rebar properties in SAP2000

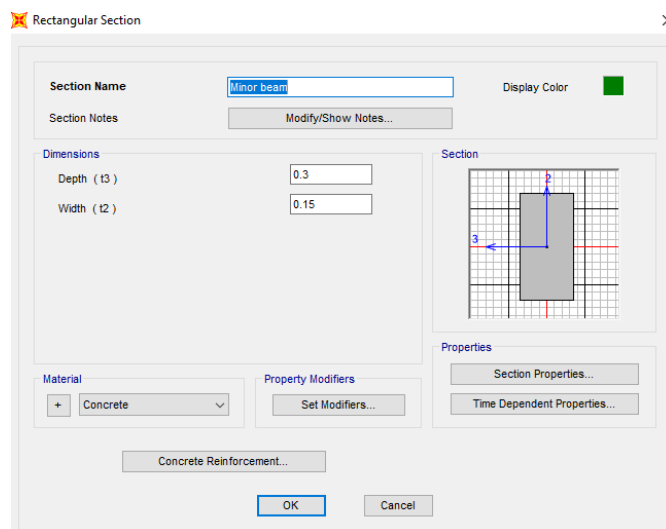
Then, the frame sections were defined which included the columns of different sizes for each floor and major and minor beams. The properties of the frame sections along with the property modifiers that incorporate the coefficient of cracking moment of inertia assigned to each section are presented in Figures 3.4-3.8.



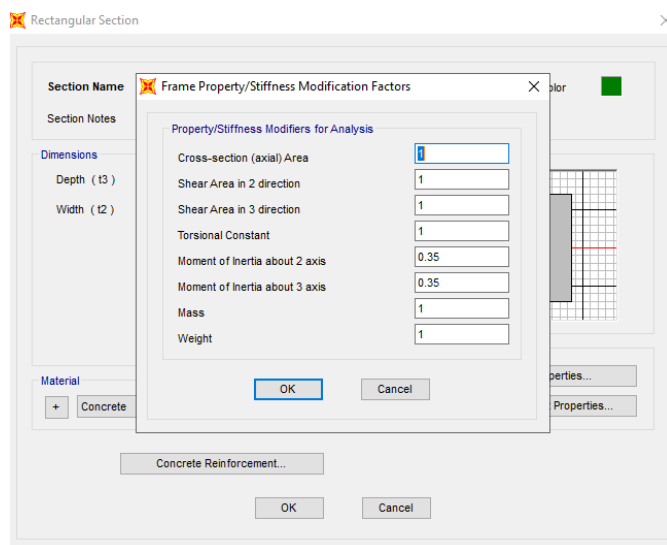
Figures 3.4. Section properties: column



Figures 3.5 Section properties: major beam



Figures 3.6. Section properties: minor beam



Figures 3.7. Property modification factors: column

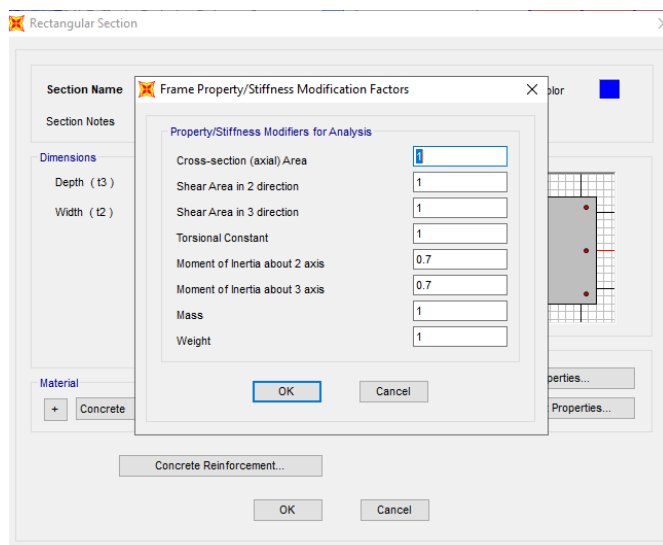


Figure 3.8. Beams

Further, joint restraints and diaphragm constraints were applied. Fixed supports were provided below the basement, where the superstructure joins to the substructure, against rotation for stability in the structure. Pinned supports at joints of the first floor and basement were designed to allow rotation and expected settling of the ground. Diaphragm constraints were also added to provide resistance to lateral forces, such as wind and earthquakes, by way of connecting

the joints in a manner that caused the entire planar diaphragm to deform cohesively under lateral loads.

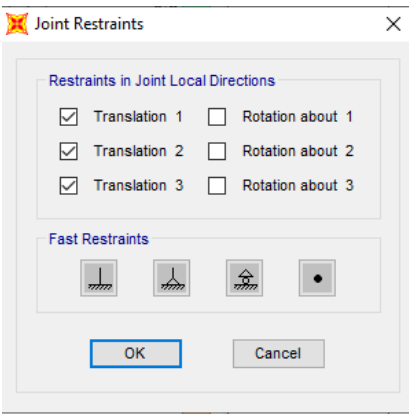


Figure 3.9. Joint restraint

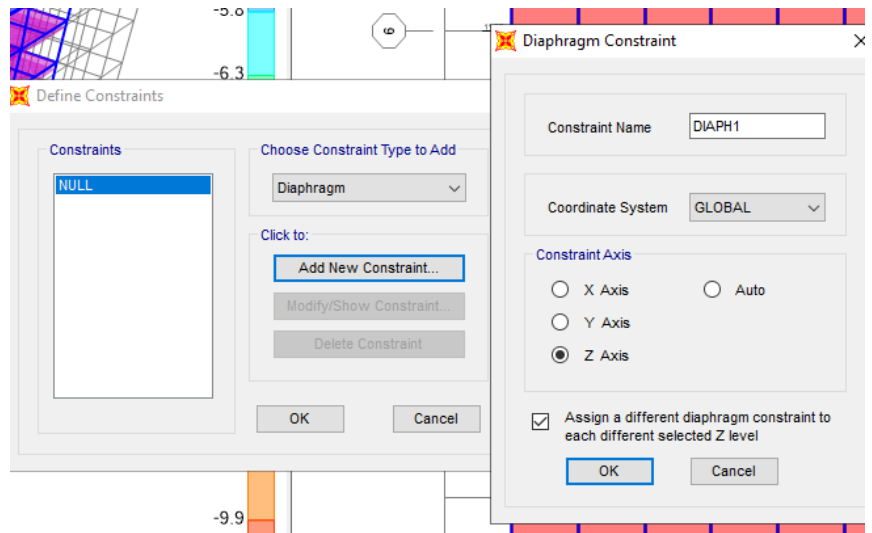


Figure 3.10. Diaphragm constraints

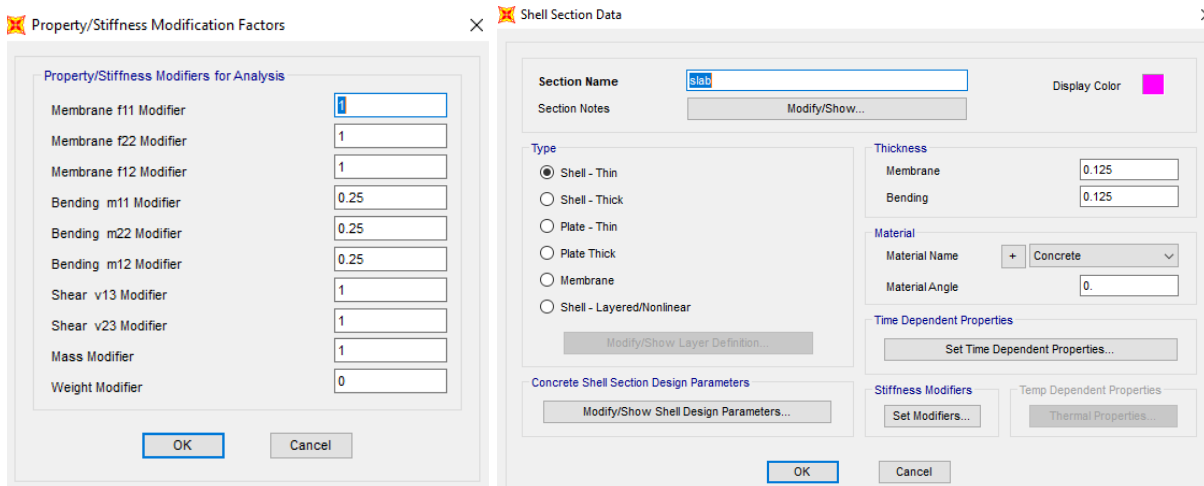


Figure 3.11. Properties for slab

Finally, the pertinent loads were placed on the slabs. Since the slabs are present over stairs and elevator shafts, the stair and elevator loads were transmitted to the surrounding beams, thereby mirroring the flow of load in a two-way slab. The dead loads, live loads, and roof live loads applied to the 3D model of the building are shown in Figures 3.12-3.14.

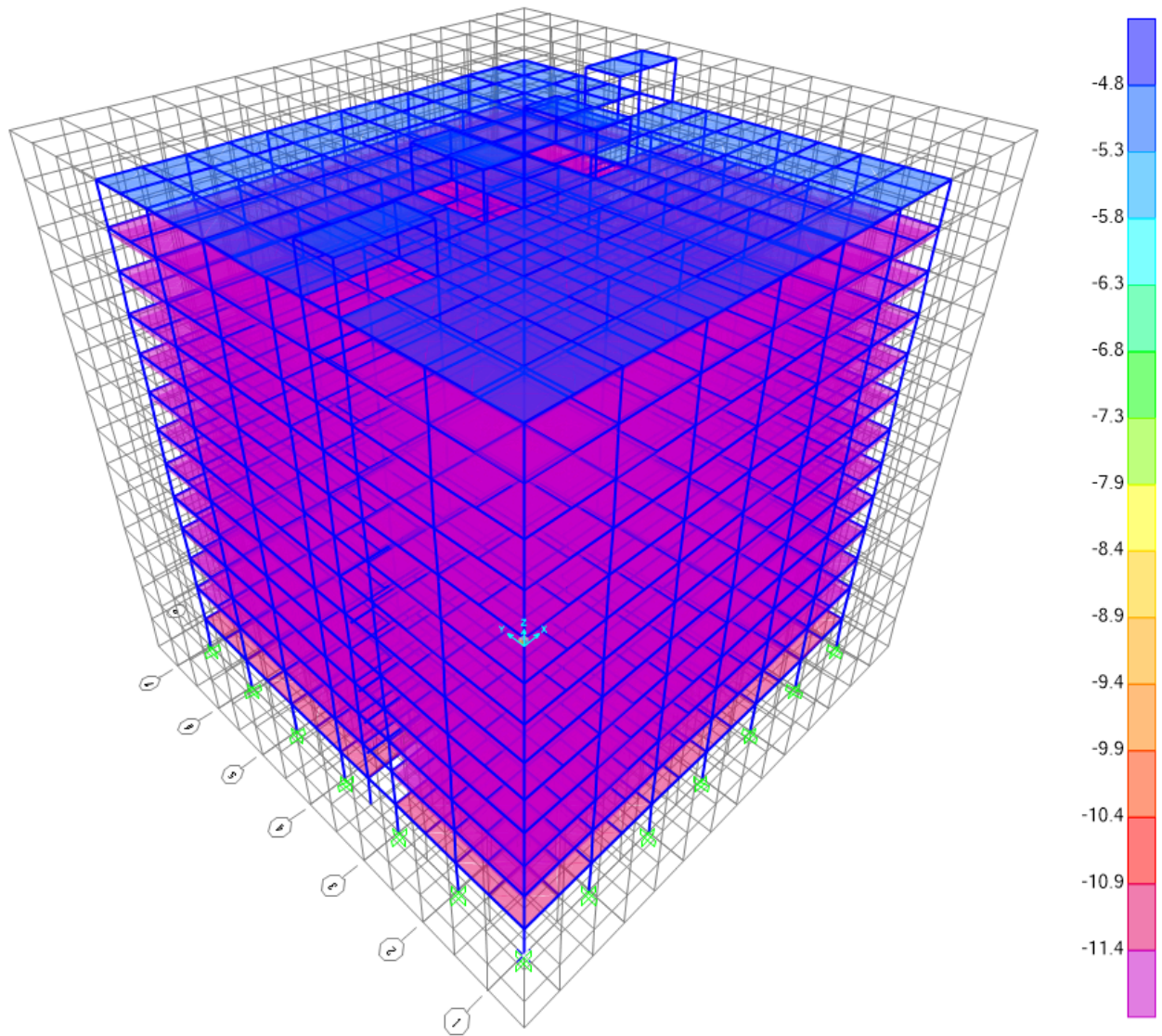


Figure 3.12 . Dead load representation in SAP 2000

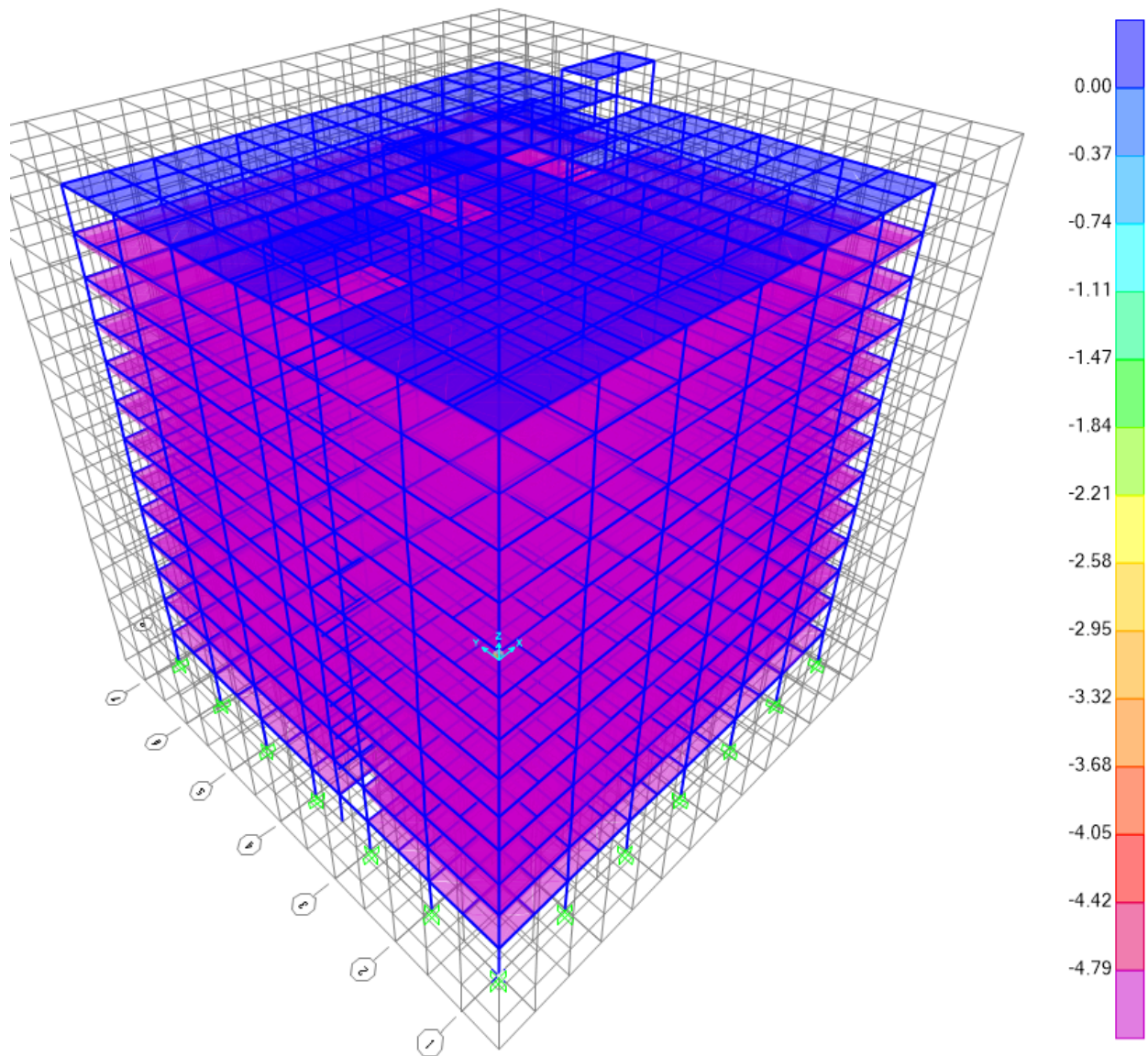


Figure 3.13. live load representation in SAP 2000

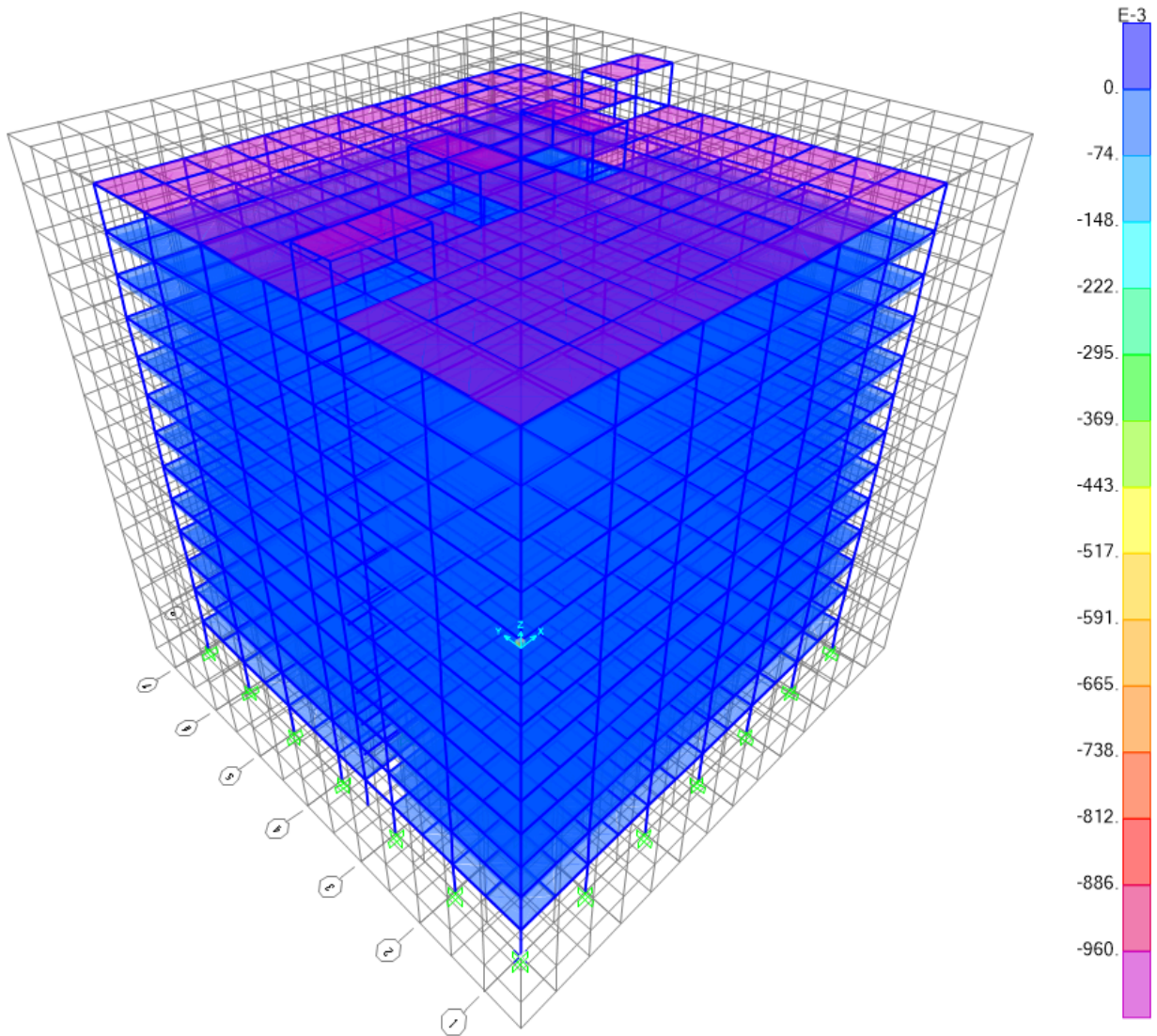


Figure 3.14. Roof Live load representation in SAP 2000

3.2. Design and analysis of lateral load resisting system

3.2.1. Calculation of Seismic Load

To evaluate the seismic forces acting on the building, the Equivalent Lateral Force (ELF) method will be employed (American Society of Civil Engineers, 2017). The site investigation report identified the soil as Class C, which will be utilized to assess the seismic design

parameters for the location (GeoPentech, 2020). The ASCE7 Hazard Tool online mapping resource was used to determine the seismic parameters of the site. Given that the building serves as a residential complex, it was categorized under Risk Category II. Based on the data from the mapping tool, the mapped acceleration parameters were identified as follows:

- $S_s = 1.93$
- $S_1 = 0.82$

Considering the site class soil condition, correction coefficients were used to determine the seismic design category:

- $F_a = 1$
- $F_v = 1.3$

Other required parameters were calculated by following formulas:

$$S_{MS} = F_a S_s \quad (3.12)$$

$$S_{MS} = 1 * 1.93 = 1.93 \text{ g}$$

$$S_{M1} = F_v S_1 \quad (3.13)$$

$$S_{M1} = 1.3 * 0.82 = 1.066 \text{ g}$$

$$S_{DS} = \frac{2}{3} S_{MS} \quad (3.14)$$

$$S_{DS} = \frac{2}{3} * 1.93 = 1.287 \text{ g}$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (3.15)$$

$$S_{D1} = \frac{2}{3} * 1.066 = 0.711 \text{ g}$$

According to Tables 11.6-1 and Table 11.6-2, since the risk category is II, $S_{ds} > 0.5$ and $S_{d1} > 0.2$ the Seismic design category was determined as D. Thus, Lateral Force Resisting System was selected as Special Reinforced Concrete Moment Frames, with following parameters:

- Overstrength factor, $\Omega = 3$
- Response modification coefficient, $R=8$
- Deflection amplification factor, $C_d=5.5$
- Importance factor, $I=1$

From the online mapping source it was found that $T_L = 12$ s. Besides that, it is necessary to calculate the Periods of the design spectrum. They have been found by the equations below:

$$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} \quad (3.16)$$

$$T_0 = 0.2 * \frac{0.711}{1.287} = 0.11 \text{ s}$$

$$T_s = \frac{S_{D1}}{S_{DS}} \quad (3.17)$$

$$T_s = \frac{0.711}{1.287} = 0.55 \text{ s}$$

$$T_a = C_t h_n^x \quad (3.18)$$

Where,

T_a - approximate fundamental period, s

C_t and x - approximate period parameters given in Table 12.8-2 (ASCE 7-16, 2016)

h_n - height of the building above the ground to its highest point, in this case: 39.5

According to the table Table 12.8-2, C_t and x for the concrete moment-resisting frame are 0.0466 and 0.9 respectively. Therefore, approximate fundamental period was calculated as follows:

$$T_a = 0.0466 * 39.5^{0.9} = 1.274 \text{ s}$$

Moreover, fundamental period of the structure was calculated:

$$T = C_u T_a \quad (3.19)$$

Where,

C_u - coefficient for upper limit on calculated period.

It is taken from Table 12.8-1 (American Society Civil Engineers, 2017). For this case it was determined as 1.4.

$$T_a = 1.4 * 1.274 = 1.783 \text{ s}$$

Since modal analysis in SAP 2000 was not conducted to determine T^* , it was assumed that the period would approximate the fundamental period, which can be utilized for the standard design procedure, so $T = T^* = 1.274 \text{ s}$

The seismic base shear for the ELF procedure should be calculated using the following equation:

$$V = C_s W \quad (3.20)$$

Where,

C_s - seismic response coefficient

W - building's effective seismic weight

Seismic response coefficient was calculated by using these equations:

$$C_s = \frac{S_{DS}}{\frac{R}{I_E}} \quad (3.21)$$

$$C_s = \frac{1.287}{\frac{8}{1}} = 0.16$$

Computed value should not exceed the following:

$$C_s = \frac{S_{D1}}{T \frac{R}{I_E}} \text{ for } T \leq T_L \quad (3.22)$$

$$C_s = \frac{0.711}{1.27 \cdot \frac{8}{1}} = 0.0697 \text{ (not controlled)}$$

Moreover, C_s shall not be less than:

$$C_s = \frac{0.5S_1}{\frac{R}{I_E}} \text{ for } S_1 \geq 0.6 \quad (3.23)$$

$$C_s = \frac{0.5 \cdot 0.82}{\frac{8}{1}} = 0.051 \text{ (controlled)}$$

As a result, C_s was determined as 0.0697.

The effective seismic weight was then determined by accounting for the weights of columns, beams, slabs, floor finishes, stairs, as well as exterior and partition walls. For the calculations, the weights of the slabs, floor finishes, and exterior and partition walls, provided in kPa, were multiplied by the floor area of 1296 m². Meanwhile, other weights were given in kN. The weights for each floor, including the roof and exit floor, are detailed in Table 3.14.

Table 3.14. Effective seismic weight calculations.

Floors	Floor finishing and slab, kN/m2	Column weight, kN	Beam weight, kN	Exterior and partition walls, kN/m2	Stairs, kN	Floor weight, kN
1	4,817	2701,13	1347,66	4,376	106,45	16069,4
2	4,817	1996,31	1347,66	4,376	106,45	15364,5
3	4,817	1996,31	1347,66	4,376	106,45	15364,5
4	4,817	1701,00	1347,66	4,376	106,45	15069,2
5	4,817	1429,31	1347,66	4,376	106,45	14797,5
6	4,817	1429,31	1347,66	4,376	106,45	14797,5
7	4,817	1181,25	1347,66	4,376	106,45	14549,5
8	4,817	956,81	1347,66	4,376	106,45	14325,0
9	4,817	756,00	1347,66	4,376	106,45	14124,2
10	4,817	756,00	1347,66	4,376	106,45	14124,2
11	4,817	425,25	1347,66	4,376	106,45	13793,5
12	4,817	425,25	1347,66	4,376	106,45	13793,5
13	0	54,00	228,85	4,376	106,45	6060,6
Roof	4,324	0,00	0	0,429	106,45	6266,3
					Total	188500

Finally, the seismic base shear can be determined:

$$V = 0.0697 * 188500 = 13138.5 \text{ kN}$$

Afterwards, to calculate the vertical distribution of seismic forces following equations were used:

$$F_x = C_{vx} V \quad (3.24)$$

$$C_s = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (3.25)$$

Where,

C_{vx} - vertical distribution factor

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

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

k - an exponent related to the structure period. It is calculated using interpolation.

When $T \leq 0.5$ s then $k = 1$ and when $T \geq 2.5$ s then $k = 2$, so when $T = 1.274$ s, then $k = 1.637$.

Following a series of calculations for each story, the lateral seismic forces at each floor are presented in Table 3.15. It is important to note that the roof weight was included in the weight of the 13th floor.

Table 3.15. Lateral seismic force at each floor.

Floors	hi (m)	Wx (kN)	$w_x h_x^k$	Cvx	Fx (kN)
1	3.5	16069.4	124921.74	0.004	52.34
2	6.5	15364.5	329047.79	0.010	137.87
3	9.5	15364.5	612425.63	0.020	256.61

4	12.5	15069.2	941310.58	0.030	394.42
5	15.5	14797.5	1314506.62	0.042	550.79
6	18.5	14797.5	1756102.69	0.056	735.82
7	21.5	14549.5	2208256.04	0.070	925.27
8	24.5	14325.0	2692532.77	0.086	1128.19
9	27.5	14124.2	3207396.14	0.102	1343.92
10	30.5	14124.2	3799826.96	0.121	1592.15
11	33.5	13793.5	4326855.63	0.138	1812.98
12	36.5	13793.5	4979061.32	0.159	2086.26
13	39.5	12326.9	5063892.00	0.161	2121.80
		188499.7	31356135.93		

3.2.1.1 Seismic Loads with Torsional effect

The building under consideration is symmetrical, with its mass center and stiffness center aligned. Despite this symmetry, we chose to include the calculation of the torsional effect as a precautionary measure to ensure a comprehensive seismic analysis. This approach accounts for any potential accidental eccentricities and provides a more robust evaluation of the building's response to seismic forces. So, since the building is symmetrical, the mass center is $y=18$ m and $x=18$ m.

Subsequently, stiffness of the moment frame was identified. This process began with determining the cracking moment of inertia for the beams and columns using the following equations:

$$I_{b,cr} = \frac{1}{12}bh^3 * 0.35 \quad (3.26)$$

$$I_{c,cr} = \frac{1}{12}bh^3 * 0.7 \quad (3.27)$$

After that stiffness of frame was calculated using the following equation:

$$D = \frac{12E}{h^2} \left(\frac{I_{c,cr}I_{b,cr}}{LI_{c,cr} + hI_{b,cr}} \right) \quad (3.28)$$

Next, Force needed to generate an inter-story rotation was found by following formula:

$$C_F = h * D * \# \text{ of columns per frame} * \# \text{ of frames} \quad (3.29)$$

Finally, stiffness center is determined, $x=18$ m and $y=18$ m. More precise calculations for each floor can be found in the Appendix (see Table).

As building is symmetrical, eccentricity value for y and x axis are equal as well. It was calculated by following way:

$$\begin{aligned} e_x &= 18 - 18 = 0 \text{ m} \\ e_a &= 5\% * L_x = 0.05 * 36 = 1.8 \text{ m} \\ e_{xt} &= e_x + e_a = 1.8 \text{ m} \end{aligned}$$

Y-axis eccentricity value was derived by similar method, resulting in the same value

$$e_{xt} = 1.8 \text{ m}.$$

Using eccentricity torsion moment was found by below formula:

$$T = eF \quad (3.30)$$

$$F_{direct} = \frac{F_x}{\# \text{ of frames}} \quad (3.31)$$

$$F_{torsion} = \frac{TC_f(x_i \text{ or } y_i)}{\sum C_f(x_i^2 \text{ or } y_i^2)} \quad (3.32)$$

$$F_{total} = F_{direct} \pm F_{torsion} \quad (3.33)$$

The determination of direct forces and torsion along the both axes for each floor is presented in Table 3.16. As an example, Table 3.16 provides the recalculated seismic forces for each frame on the first floor for both axes (frame stiffness calculation was not include in table). Similar calculations for other floors are detailed in Appendix A (refer to Tables 3.16- 3.17).

Table 3.16. Direct forces on each frame and torsion for both directions..

Floors	Fx (kN)	Fdirect (kN)	T (kN- m)
1	52,34	7,48	13,46
2	137,87	19,70	35,45
3	256,61	36,66	65,99
4	394,42	56,35	101,42
5	550,79	78,68	141,63
6	735,82	105,12	189,21
7	925,27	132,18	237,93
8	1128,19	161,17	290,11
9	1343,92	191,99	345,58
10	1592,15	227,45	409,41
11	1812,98	259,00	466,20

12	2086,26	298,04	536,47
13	2121,80	303,11	545,61

Table 3.17. Calculation of seismic force on each frame for the 1st floor incorporating the torsional effect.

Floors	both directions				
	Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
1	1	13,46	0,1202	7,48	7,60
	2	13,46	0,0801	7,48	7,56
	3	13,46	0,0401	7,48	7,52
	4	13,46	0,0000	7,48	7,48
	5	13,46	-0,0401	7,48	7,44
	6	13,46	-0,0801	7,48	7,40
	7	13,46	-0,1202	7,48	7,36

3.2.2 Calculation of Wind loads

3.2.2.1 Wind Loads with Torsional Effect

Calculations of the wind loads were based on the ASCE 7-16. According to the ASCE 7-16 Figure 26.5-1A basic wind speed in the Cupertino, California was determined as $V=49$ m/s. Since building is located in a urban area surface roughness was selected as B. Also, Exposure Type B was chosen because building's height is greater than 9.1 meters (30 feet), and the ground surface roughness in the upwind direction extends for more than 792 meters (20 times building's height). Other parameters were taken as follows:

- Topographic factor $K_{zt}=1$
- Directional effect $K_d=0.85$
- Importance factor $I=0.87$
- Enclosure: enclosed

Table 3.18 shows the constants values for exposure categories.

Table 3.18. ASCE7-10 constants for wind load calculations

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

Height Effect

To calculate velocity pressure exposure coefficient K_z , following equations were used:

$$K_z = 2.01(z/z_g)^{2/a}, \text{ for } 4.572 \text{ m} < z < z_g \quad (3.34)$$

$$K_z = 2.01(15/z_g)^{2/a}, \text{ for } z < 4.572 \text{ m} \quad (3.35)$$

So, the results of the height effect calculations are shown in the Table 3.19.

Table 3.19: Height effect calculations for wind load

Story	Height, m	zi, m	Kz
12	3	36.5	1.040455891
11	3	33.5	1.015269574
10	3	30.5	0.9884164209
9	3	27.5	0.9596043235
8	3	24.5	0.9284507664
7	3	21.5	0.8944395333
6	3	18.5	0.8568471346
5	3	15.5	0.8146087195
4	3	12.5	0.7660500484
3	3	9.5	0.7082782165
2	3	6.5	0.6354995429
1	3.5	3.5	0.80701

Gust Effect

To determine whether building is flexible or rigid, n_a was calculated. In our case building consists of reinforced concrete moment frames and the height is equal to 119.75 ft, therefore above equation was used to calculate n_a :

$$n_a = 75/h$$

$$n_a = 75/119.75 = 0.63$$

$$n_a < 1.0 \text{ Hz}$$

Thus, it is a flexible building. This building is square shaped where $B=L=36$ m, so the parallel and perpendicular directions of the wind will be the same. Gust effect is calculated by the formulas below:

$$\bar{z} = 0.6 * h = 0.6 * 36.5 = 17.9 \text{ m}; z_{min} = 9.14, \text{ so the largest is } \bar{z} = 21.9 \text{ m}$$

$$I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6}$$

$$I_{\bar{z}} = 0.3\left(\frac{10}{21.9}\right)^{1/6} = 0.263$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{10}\right)^{\epsilon}$$

$$L_{\bar{z}} = 97.54\left(\frac{21.9}{10}\right)^{1/3} = 126.6 \text{ m}$$

$$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{a}} v L_{\bar{z}} = l\left(\frac{\bar{z}}{10}\right)^{\epsilon}$$

$$V_{\bar{z}} = 0.45\left(\frac{21.9}{10}\right)^{1/4} * 49 = 26.8$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}}$$

$$N_1 = \frac{0.63*126.6}{26.8} = 2.97$$

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

$$R_n = \frac{7.47*2.97}{(1+10.3*2.97)^{5/3}} = 0.07$$

$$\eta_h = \frac{4.6 n_a h}{V_{\bar{z}}}$$

$$\eta_h = \frac{4.6*0.63*36.5}{26.8} = 3.95$$

$$\eta_L = \frac{15.4n_a L}{V_z}$$

$$\eta_L = \frac{15.4*0.63*36}{26.8} = 13.03$$

$$\eta_B = \frac{4.6n_a B}{V_z}$$

$$\eta_B = \frac{4.6*0.63*36}{26.8} = 3.89$$

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

$$R_h = \frac{1}{3.95} - \frac{1}{2*3.95^2} (1 - e^{-2*3.95}) = 0.22$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2} (1 - e^{-2\eta_L})$$

$$R_L = \frac{1}{13.03} - \frac{1}{2*13.03^2} (1 - e^{-2*13.03}) = 0.07$$

$$R_B = \frac{1}{\eta_B} - \frac{1}{2\eta_B^2} (1 - e^{-2\eta_B})$$

$$R_B = \frac{1}{3.89} - \frac{1}{2*3.89^2} (1 - e^{-2*3.89}) = 0.224$$

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

$$R = \sqrt{\frac{1}{0.02} 0.07 * 0.22 * 0.224 (0.53 + 0.47 * 0.072)} = 0.311$$

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

$$g_R = \sqrt{2 \ln(3600 * 0.63)} + \frac{0.577}{\sqrt{2 \ln(3600 * 0.63)}} = 4.07$$

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

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{36+36.5}{126.6} \right)^{0.63}}} = 0.83$$

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

$$G_f = 0.925 \left(\frac{1 + 1.7 * 0.263 \sqrt{3.4^2 * 0.83^2 + 4.07^2 * 0.311^2}}{1 + 1.7 * 3.4 * 0.263} \right) = 0.8$$

Thus, the gust factor for the both perpendicular and parallel wind directions is $G_f = 0.87$.

Design velocity pressure were found for each level according to this formula:

$$q = 0.613 K_z K_{zt} K_d V^2 \quad (3.36)$$

Subsequently, design wind pressures for each floor were calculated using this equation:

$$p = qGC_p - q_iGC_i \quad (3.37)$$

External pressure coefficients were selected according to the ASCE 7-10. Windward coefficient is $C_{pw} = 0.8$ and leeward $C_{pl} = -0.5$.

Finally, forces acting on each level were obtained as follows:

Force at the top floor:

$$F_{21} = \left(\frac{B}{\# \text{ of frames}} \right) \frac{h}{2} P_{21} \quad (3.38)$$

Force at the i th floor:

$$F_i = \left(\frac{B}{\# \text{ of frames}} \right) \left(P_{i+1} \frac{h_{i+1}}{2} + P_i \frac{h_i}{2} \right) \quad (3.39)$$

Calculation results are demonstrated for the both axes in the Table 3.20.

Table 3.20. Wind force calculations on each frame for both directions (Case 1).

Story	height, m	zi, m	Kz	q, Pa	Pw, Pa	Pl, Pa	P, Pa	F per frame, kN
13	3	39.5	1.06	1331.36	926.63	-579.14	1505.77	33.88
12	3	36.5	1.04	1301.65	905.95	-566.22	1472.17	22.97
11	3	33.5	1.02	1270.14	884.02	-552.51	1436.53	22.44
10	3	30.5	0.99	1236.55	860.64	-537.90	1398.54	21.87
9	3	27.5	0.96	1200.50	835.55	-522.22	1357.77	21.26
8	3	24.5	0.93	1161.53	808.42	-505.27	1313.69	20.61

7	3	21.5	0.89	1118.98	778.81	-486.76	1265.57	19.90
6	3	18.5	0.86	1071.95	746.08	-466.30	1265.57	19.53
5	3	15.5	0.81	1019.11	709.30	-443.31	1152.61	18.65
4	3	12.5	0.77	958.36	667.02	-416.89	1083.91	17.25
3	3	9.5	0.71	886.09	616.72	-385.45	1002.16	16.09
2	3	6.5	0.64	795.04	553.35	-345.84	899.19	14.67
1	3.5	3.5	0.81	1009.61	702.69	-439.18	1141.86	17.21

This wind forces are calculated for the 1st case out of 4 cases defined in the ASCE 7 (2016).

Each case with wind directions and equations are illustrated in the figure below:

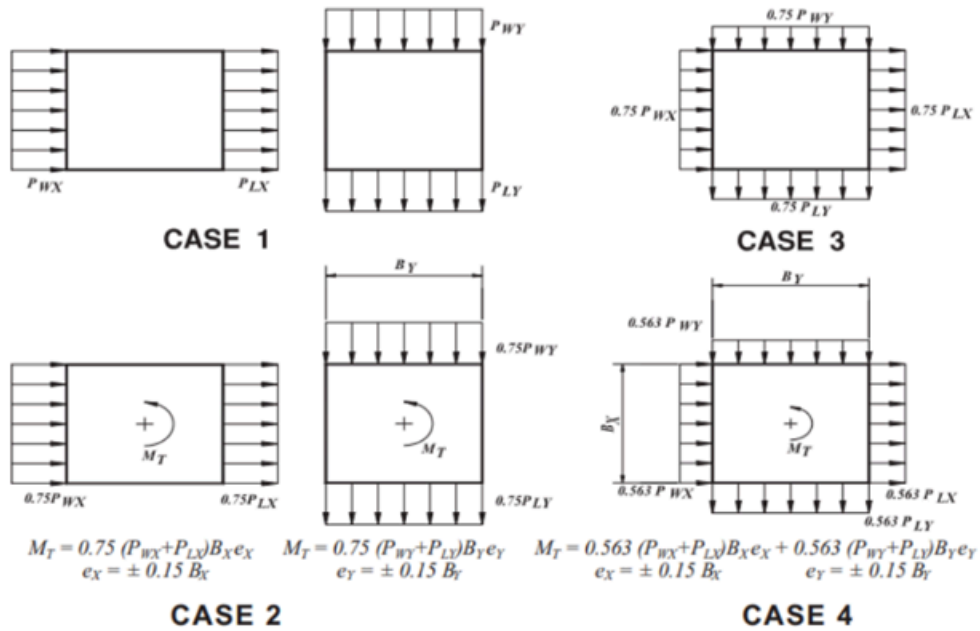


Figure 3.16. Description of the 4 wind load cases.

As illustrated in the figure above, irregularities in the diagram result in a torsional effect in Cases 2 and 4. Tables 3.21 and 3.22 provide the torsion values and the torsional forces acting on each frame at every floor for Case 2.

$$e_x = 0.15 * B \quad (3.40)$$

$$e_x = 0.15 * 36 = 5.4 \text{ m}$$

Table 3.21. Direct force parameters for Case 2 in both directions

Story	Pw, Pa	Pl, Pa	P, Pa	MT (kN-m)	0.75Fi, (kN)	F per frame, kN
13	926.63	-579.14	1505.77	73.18	101.64	25.41
12	905.95	-566.22	1472.17	214.64	120.61	17.23
11	884.02	-552.51	1436.53	209.45	117.80	16.83
10	860.64	-537.90	1398.54	203.91	114.82	16.40
9	835.55	-522.22	1357.77	197.96	111.63	15.95
8	808.42	-505.27	1313.69	191.54	108.19	15.46
7	778.81	-486.76	1265.57	184.52	104.46	14.92
6	746.08	-466.30	1265.57	184.52	102.51	14.64
5	709.30	-443.31	1152.61	168.05	97.94	13.99
4	667.02	-416.89	1083.91	158.03	90.58	12.94
3	616.72	-385.45	1002.16	146.12	84.49	12.07
2	553.35	-345.84	899.19	131.10	77.00	11.00
1	702.69	-439.18	1141.86	166.48	90.37	12.91

The total forces acting on the frame result from the combined effects of direct forces and torsional influences caused by wind loads. Adding these two components yields the total force. Based on the previously calculated frame stiffness, the corresponding values are presented in Table 3.22.

Table 3.22. Calculation of wind force on each frame for the first floor including the torsional effect.

Both directions

Floor	Frame	Frame	T, kN-m	Ftortion, kN	Fdirect, kN	Ftotal, kN
1	A	1	166.48	1.4865	12.91	14.40
	B	2	166.48	0.9910	12.91	13.90
	C	3	166.48	0.4955	12.91	13.41
	D	4	166.48	0.0000	12.91	12.91
	E	5	166.48	-0.4955	12.91	12.41
	F	6	166.48	-0.9910	12.91	11.92
	G	7	166.48	-1.4865	12.91	11.42

These values represent the forces acting on each frame at the first floor. Calculations for the remaining floors are provided in Appendix B (Tables 9.14-9.26). The same methodology was used to analyze Cases 3 and 4, with the direct and torsional force components, as well as the total force estimates, summarized in Tables 3.23 and beyond.

Table 3.23. Case 3 force calculation for both sides.

Story	0.75Fi, (kN)	F per frame, kN
12	101.64	25.41
11	117.80	17.23
10	114.82	16.83
9	111.63	16.40
8	108.19	15.95
7	104.46	15.46
6	102.51	14.92
5	97.94	14.64
4	90.58	13.99

3	84.49	12.94
2	77.00	12.07
1	90.37	11.00

Table 3.24. Case 4 Torsion and direct forces calculations.

Story	P, Pa	MT (kN-m)	0.563F1, kN	F1 per frame (kN)
13	1505.77	54.93	76.30	19.07
12	1472.17	161.12	90.54	12.93
11	1436.53	157.22	88.43	12.63
10	1398.54	153.07	86.19	12.31
9	1357.77	148.60	83.80	11.97
8	1313.69	143.78	81.22	11.60
7	1265.57	138.51	78.41	11.20
6	1265.57	138.51	76.95	10.99
5	1152.61	126.15	73.52	10.50
4	1083.91	118.63	67.99	9.71
3	1002.16	109.68	63.42	9.06
2	899.19	98.41	57.80	8.26
1	1141.86	124.97	67.84	9.69

Table 3.25 presents the total force calculations for the first floor in Case 4, while the corresponding values for floors 2 through 13 are provided in Appendix B (refer to Tables 9.14-9.27).

Table 3.25. Calculation of wind force on each frame for the first floor with the torsional effect (Case 4).

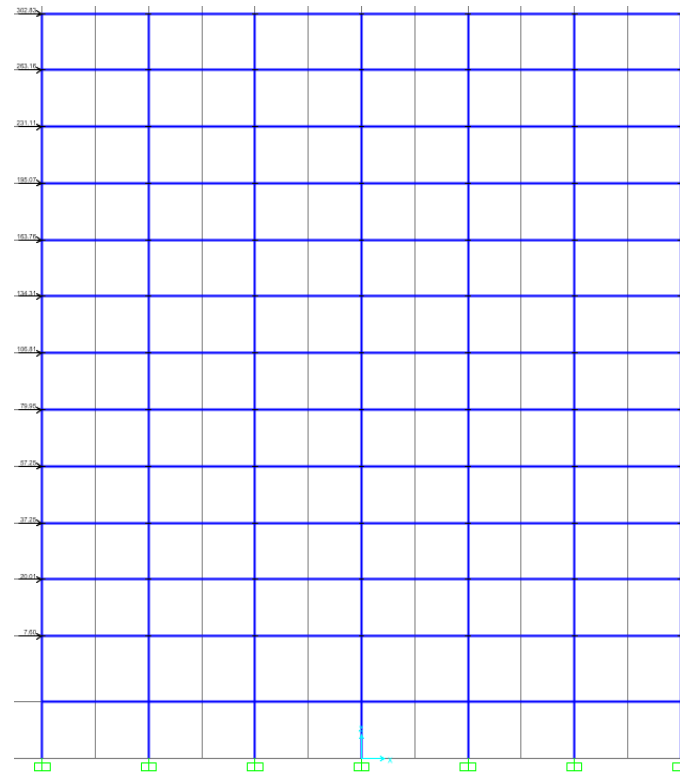
Both directions

Floors	Frame	Frame	T, kN-m	Ftortion, kN	Fdirect, kN	Ftotal, kN
1	A	1	124.97	1.1158	67.84	68.95
	B	2	124.97	0.7439	67.84	68.58
	C	3	124.97	0.3719	67.84	68.21
	D	4	124.97	0.0000	67.84	67.84
	E	5	124.97	-0.3719	67.84	67.47
	F	6	124.97	-0.7439	67.84	67.09
	G	7	124.97	-1.1158	67.84	66.72

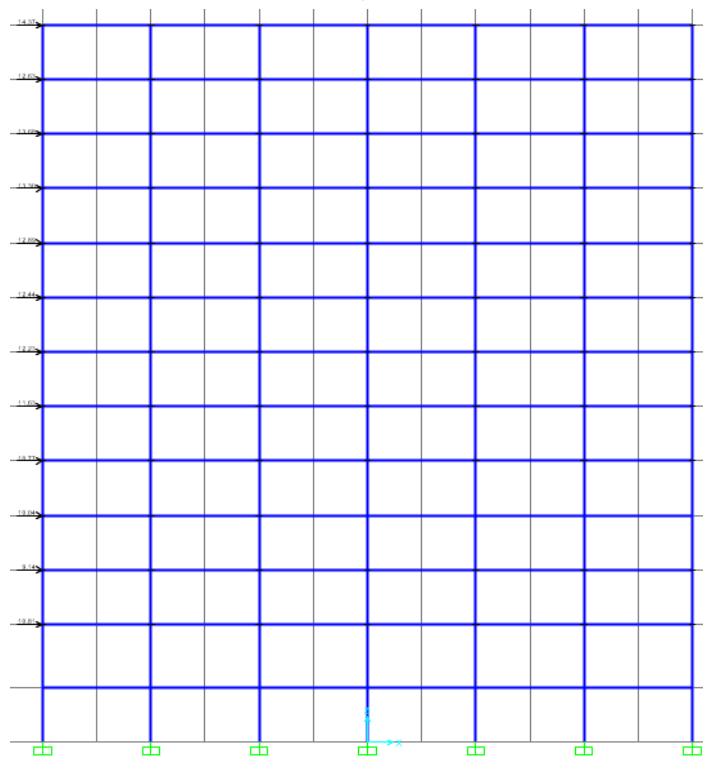
3.2.3. Assigning Forces to SAP 2000

3.2.3.1. Assigning Forces to Chosen 2D Frame

To allocate lateral forces to frame , the same approach described in Section 3.2.3 was followed, involving the definition of material properties, frame sections, and constraints. The wind load on the selected frame was applied to each joint using the Fdirect value from Section 3.3.2 based on all 4 Cases. Similarly, the seismic load was applied to each joint using the Fdirect calculations from Section 3.3.1 including the torsional effect. The assigned seismic forces and wind forces for 2nd case are illustrated in Figure 3.26.



a)



b)

Figure 3.17. Assigned lateral forces on 2D frame in SAP2000: a) seismic loads, b) wind loads

3.2.3.2. Assigning Forces to 3D Frame

Forces to 3D Frames were assigned in similar way as for the 2D frame in the SAP software.

Following table 3.26 shows assigned lateral forces on 3D frame.

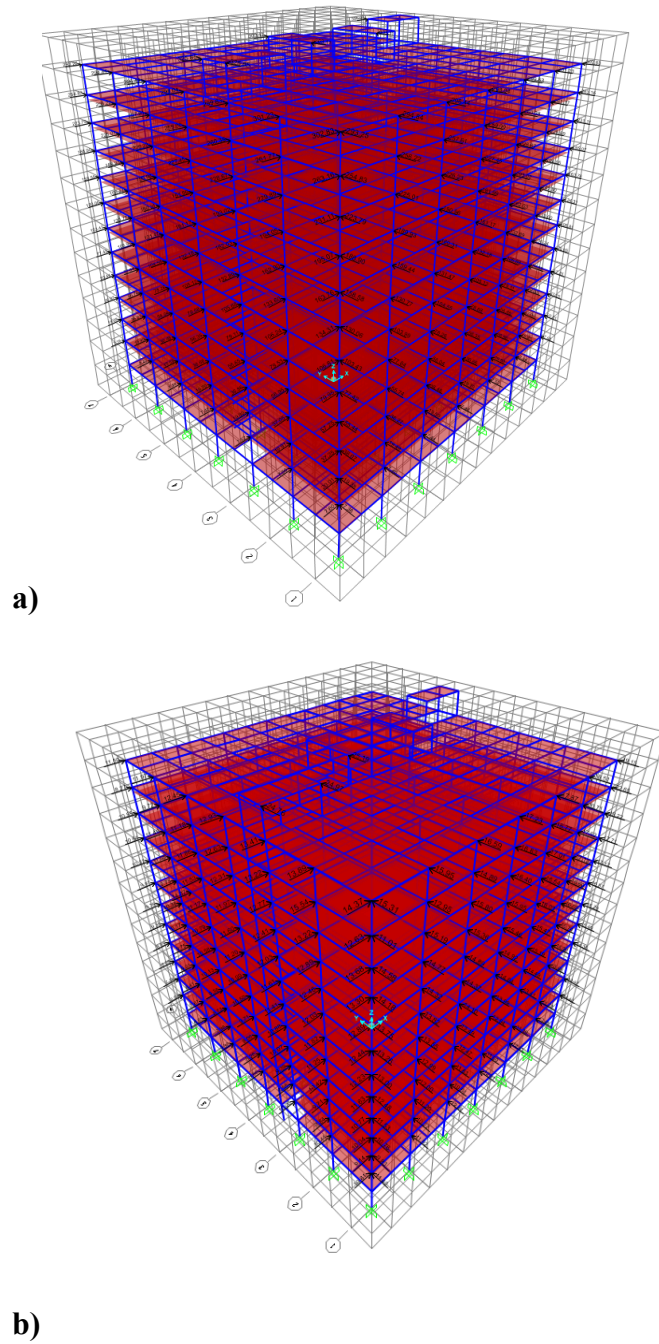


Figure 3.18. Assigned lateral forces on 3D frame in SAP2000: a) seismic loads, b) wind loads for second case.

3.3 Analytical models development in software

3.3.1 Model geometry

Structural model geometry was developed in SAP2000 with all structural members. Building model can be seen in Figure 3.19.

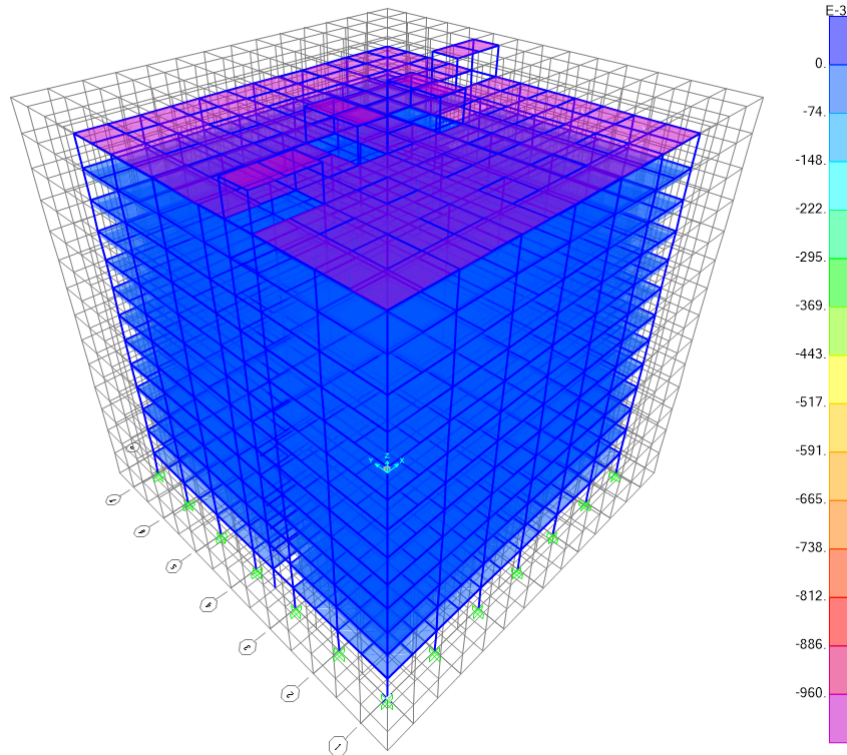


Figure 3.19. Model geometry of building

3.3.2 Materials and Sections

3.3.2.1 Concrete

Concrete properties can be seen in Table 3.26. C40 concrete was chosen for structural members as it is widely used in construction of heavy buildings. E_c calculated by using next formula

$$E_c = 4700\sqrt{f_c} \quad (3.41)$$

Also we took the poisson ratio as 0.2 as it varies between 0.15-0.25. G_c was calculated by using next formula

$$G_c = \frac{E_c}{2(1+\nu)} \quad (3.42)$$

Table 3.26. Material properties: Concrete

	f'_c	p	E_c	v	G_c
Structural members	41.7	23.6	30442	0.2	12684

3.3.2.2: Reinforcing steel

The properties of the reinforcement are shown in Table 3.27.

Table 3.27. Reinforcing steel properties

E	p	v	f_y
200000	76.97	0.3	413.69

3.3.3. Sections

The moments of inertia to induce cracking were used for the structure members as seen from Table 3.28.

Table 3.28. Cracking moment of inertia factors of structural members

Component	Cracking moment of inertia factor
Beams	0.35
Columns	0.25
Slab	0.7
Stairs	0.25

3.3.4 Elements and Connections

In SAP2000, the slabs were represented as shell elements with mesh dimensions of 1000 mm $\times$ 1000 mm or smaller to accurately capture the building's response to applied loads. Beams and columns were modeled using frame elements. Furthermore, the joints between columns and beams in the moment frame were assumed to have rigid connections.

3.3.5 Boundary Conditions

It was considered that the fixed support at the base of the basement columns would prevent both movement and rotation. At the connection points between the basement and the ground floor, where the structure meets the ground, vertical and horizontal displacements were restrained, enabling the building to rotate under lateral loads.

3.4 Structural Analysis

3.4.1 Lateral Drift Analysis

3.4.1.1 Hand calculations for lateral drift analysis under Wind Load

Since the building is square-shaped, drift was calculated for only Frame A for both directions.

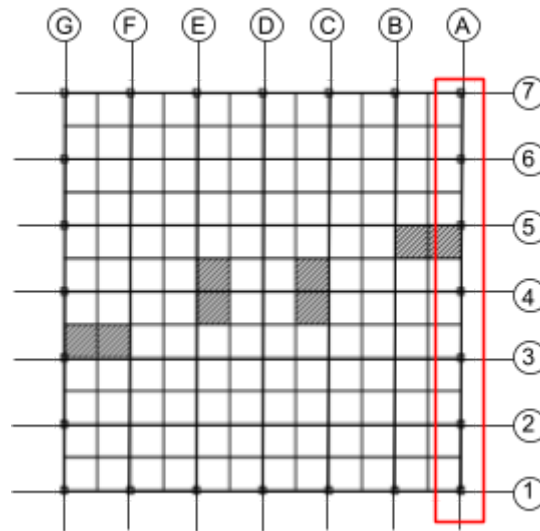


Figure 3.20. Frame layout used in the analysis.

Tables 3.29-3.31 summarize the results of the shear and flexural drifts.

Table 3.29. Shear Drift on Frame A under Wind Load for both directions.

Story	hi	hi-avg	Fi	Vi	Vi-col	Vavg	Ic,cr	Ib,cr	Ic-avg
-------	----	--------	----	----	--------	------	-------	-------	--------

	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(mm ⁴)	(mm ⁴)	(mm ⁴)
12	3000	1500	22.97	22.97	3.83	3.83	4.73E+08	1.89E+09	4.73E+08
11	3000	3000	22.44	45.41	7.57	5.70	4.73E+08	1.89E+09	4.73E+08
10	3000	3000	21.87	67.28	11.21	9.39	1.49E+09	1.89E+09	9.83E+08
9	3000	3000	21.26	88.54	14.76	12.99	1.49E+09	1.89E+09	1.49E+09
8	3000	3000	20.61	109.15	18.19	16.47	2.39E+09	1.89E+09	1.94E+09
7	3000	3000	19.90	129.05	21.51	19.85	3.65E+09	1.89E+09	3.02E+09
6	3000	3000	19.53	148.58	24.76	23.14	5.34E+09	1.89E+09	4.49E+09
5	3000	3000	18.65	167.23	27.87	26.32	5.34E+09	1.89E+09	5.34E+09
4	3000	3000	17.25	184.48	30.75	29.31	7.56E+09	1.89E+09	6.45E+09
3	3000	3000	16.09	200.58	33.43	32.09	1.04E+10	1.89E+09	8.99E+09
2	3000	3000	14.67	215.24	35.87	34.65	1.04E+10	1.89E+09	1.04E+10
1	3500	3250	17.21	232.46	38.74	37.31	1.40E+10	1.89E+09	1.22E+10
0	0	1750	0.00			19.37			

Table 3.30. Continuation of table 3.29

db(mm)	dc (mm)	dt (mm)	Interstory (mm)	abs (mm)
0.30	0.12	0.42	0.86	23.17
0.47	0.83	1.30	1.34	22.31
0.51	0.87	1.38	1.46	20.97
0.58	0.96	1.54	1.65	19.51
0.63	1.13	1.76	1.8	17.86
0.66	1.18	1.84	1.89	16.06
0.71	1.23	1.94	2.04	14.17
0.78	1.36	2.14	2.23	12.13
0.83	1.49	2.32	2.36	9.90

db(mm)	dc (mm)	dt (mm)	Interstory (mm)	abs (mm)
0.87	1.53	2.40	2.49	7.54
1.11	1.47	2.58	3.17	5.05
0.66	3.10	3.76	1.88	1.88
0	0	0	0	0

Table 3.31. Flexural Drift on Frame A under Wind Load for both directions.

B	M (N-mm)	a=b (m)	A (mm²)	f_i	Dqi (rad)	qi (rad)	Intersto ry (mm)	Absolute (mm)
12	3.45E+07	0.3	90000	5.78E-11	1.73E-07	7.88E-06	0.02	0.17
11	1.36E+08	0.3	90000	2.28E-10	6.85E-07	7.71E-06	0.02	0.14
10	2.02E+08	0.4	160000	1.90E-10	5.71E-07	7.02E-06	0.02	0.12
9	2.66E+08	0.4	160000	2.51E-10	7.52E-07	6.45E-06	0.02	0.10
8	3.27E+08	0.45	202500	2.44E-10	7.32E-07	5.70E-06	0.02	0.08
7	3.87E+08	0.5	250000	2.34E-10	7.01E-07	4.97E-06	0.01	0.06
6	4.46E+08	0.55	302500	2.22E-10	6.67E-07	4.26E-06	0.01	0.05
5	5.02E+08	0.55	302500	2.50E-10	7.51E-07	3.60E-06	0.01	0.03
4	5.53E+08	0.6	360000	2.32E-10	6.96E-07	2.85E-06	0.01	0.02
3	6.02E+08	0.65	422500	2.15E-10	6.45E-07	2.15E-06	0.01	0.01
2	6.46E+08	0.65	422500	2.31E-10	6.92E-07	1.51E-06	0.00	0.01
1	7.55E+08	0.7	490000	2.33E-10	8.14E-07	8.14E-07	0.00	0.00
0					0		0	0

3.4.1.2 Hand calculations for lateral drift analysis under Seismic Loads

The same procedure was implemented for the drift calculation under Seismic Loading. The results are shown in Tables 3.32-3.34.

Table 3.32. Shear Drift on Frame A under Seismic Loads for both directions.

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	3000	1500	298.04	298.04	49.67	24.84	4.73E+08	1.89E+09	4.73E+08
11	3000	3000	259.00	557.03	92.84	71.26	4.73E+08	1.89E+09	4.73E+08
10	3000	3000	227.45	784.48	130.75	111.79	1.49E+09	1.89E+09	9.83E+08
9	3000	3000	191.99	976.47	162.75	146.75	1.49E+09	1.89E+09	1.49E+09
8	3000	3000	161.17	1137.64	189.61	176.18	2.39E+09	1.89E+09	1.94E+09
7	3000	3000	132.18	1269.83	211.64	200.62	3.65E+09	1.89E+09	3.02E+09
6	3000	3000	105.12	1374.94	229.16	220.40	5.34E+09	1.89E+09	4.49E+09
5	3000	3000	78.68	1453.63	242.27	235.71	5.34E+09	1.89E+09	5.34E+09
4	3000	3000	56.35	1509.97	251.66	246.97	7.56E+09	1.89E+09	6.45E+09
3	3000	3000	36.66	1546.63	257.77	254.72	1.04E+10	1.89E+09	8.99E+09
2	3000	3000	19.70	1566.33	261.05	259.41	1.04E+10	1.89E+09	1.04E+10
1	3500	3250	7.48	1573.80	262.30	261.68	1.40E+10	1.89E+09	1.22E+10
0	0	1750	0.00			131.15			

Table 3.33. Continuation of table 3.32.

db(mm)	dc (mm)	dt (mm)	Interstory (mm)	Absolute (mm)
1.67	0.52	2.19	4.77	95.69
2.56	4.78	7.35	7.33	90.92
2.65	4.66	7.31	7.57	83.59
2.85	4.97	7.83	8.15	76.02
2.95	5.53	8.47	8.42	67.87
2.93	5.43	8.36	8.37	59.45
2.98	5.40	8.38	8.52	51.08

3.06	5.60	8.66	8.74	42.56
3.07	5.75	8.82	8.76	33.82
3.05	5.65	8.70	8.71	25.06
3.62	5.10	8.73	10.35	16.35
2.10	9.89	11.98	5.99	5.99
	0	0		0

Table 3.34. Flexural Drift on Frame A under Seismic Load for both directions.

B	M (N-mm)	a=b (m)	A (mm²)	f_i	Dq_i (rad)	q_i (rad)	Interstor y (mm)	Absolute (mm)
13	0.00E+00	0.3	90000	0.00E+00	0.00E+00	7.40E-05	0.00	1.41
12	4.47E+08	0.3	90000	7.50E-10	2.25E-06	7.40E-05	0.22	1.41
11	1.67E+09	0.3	90000	2.80E-09	8.41E-06	7.18E-05	0.22	1.19
10	2.35E+09	0.4	160000	2.22E-09	6.66E-06	6.34E-05	0.19	0.97
9	2.93E+09	0.4	160000	2.76E-09	8.29E-06	5.67E-05	0.17	0.78
8	3.41E+09	0.45	202500	2.54E-09	7.63E-06	4.84E-05	0.15	0.61
7	3.81E+09	0.5	250000	2.30E-09	6.90E-06	4.08E-05	0.12	0.47
6	4.12E+09	0.55	302500	2.06E-09	6.17E-06	3.39E-05	0.10	0.35
5	4.36E+09	0.55	302500	2.18E-09	6.53E-06	2.77E-05	0.08	0.24
4	4.53E+09	0.6	360000	1.90E-09	5.70E-06	2.12E-05	0.06	0.16
3	4.64E+09	0.65	422500	1.66E-09	4.97E-06	1.55E-05	0.05	0.10
2	4.70E+09	0.65	422500	1.68E-09	5.03E-06	1.05E-05	0.03	0.05
1	5.11E+09	0.7	490000	1.58E-09	5.51E-06	5.51E-06	0.02	0.02
0					0		0	0

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

SAP2000 2D and 3D models were created for Frames A to study and compare lateral drifts caused by wind and seismic loads. Additionally, for earthquake resistance, each story should function as a distinct resistant frame (ASCE, 2016). The inter-story seismic deflections were amplified to account for the maximal inelastic displacement (δ_x) using the equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (3.43)$$

Where:

δ_{xe} - interstory elastic deflection

C_d - deflection amplification factor (e.g., $C_d = 5.5$).

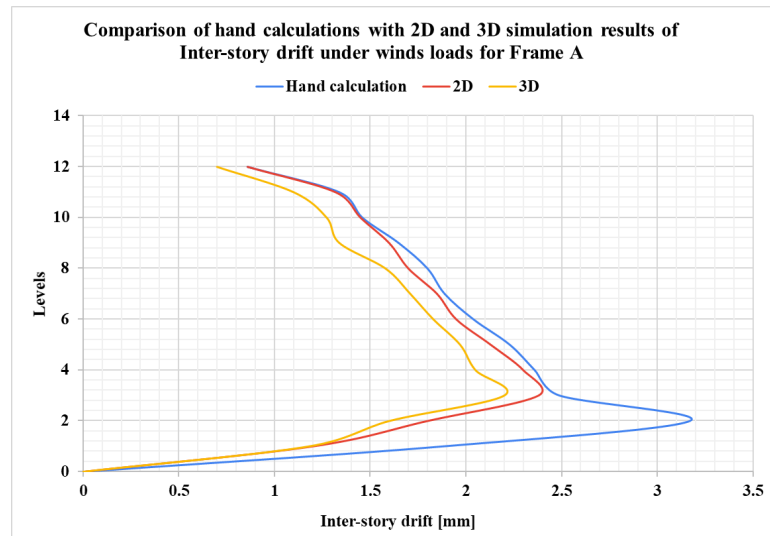
The allowable story drift limit was identified by the following method:

$$\Delta_a = 0.020h_{sx} \quad (3.44)$$

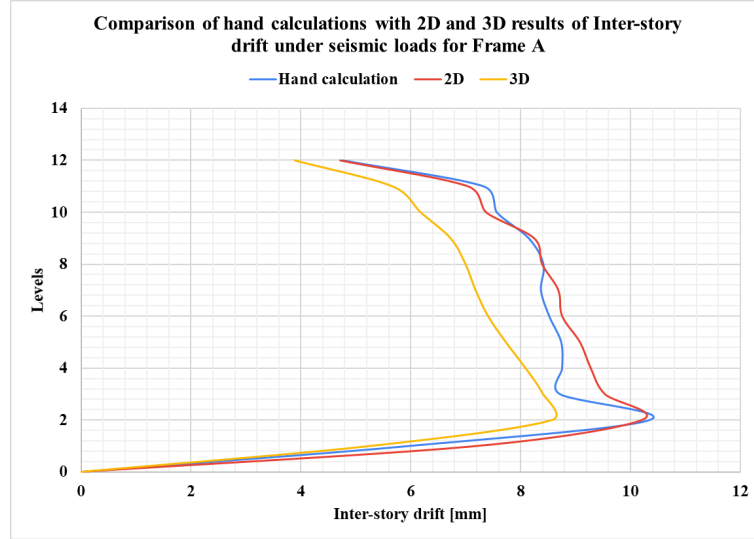
Where h_{sx} is the height of the story, in this case 1st floor has height of 3500 mm, whereas 2-12 floors height is 3000 mm.

The inter-story drift was assumed to be $h/600$ when computing wind drifts. It should be mentioned that every wind interstory deflection passed the limit check.

Tables 9.69-9.71 in Appendix C show the wind and seismic drifts estimated in 3D SAP, 2D SAP, and by hand for Frames A.



(a)



(b)

Figure 3.21. Comparison of lateral deflections for Frame A: (a) wind; and (b) seismic.

The comparisons of the SAP2000 and hand calculations with allowable limits can be seen in Figures 3.21.

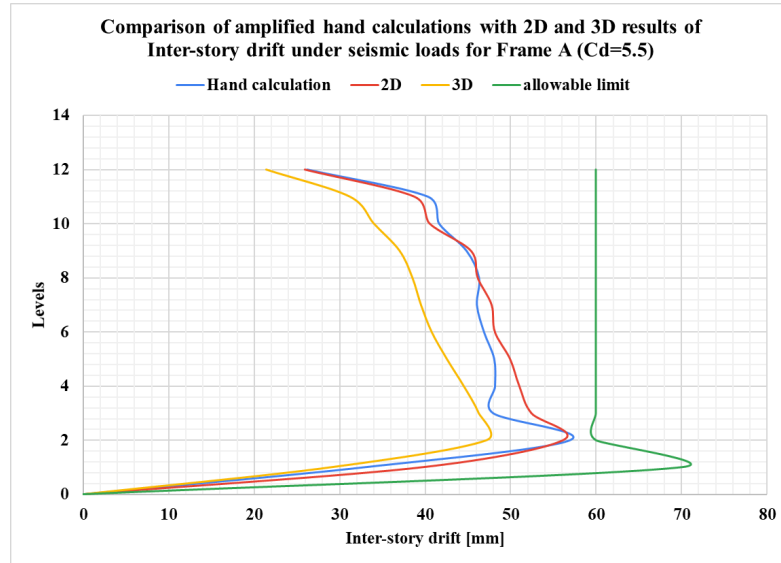


Figure 3.22. Comparison of amplified seismic deflections for Frame A

3.4.1.4. Stability analysis

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \quad (3.45)$$

Where,

P_x - total vertical design load (kN)

Δ - design story drift (mm)

I_e - importance factor

V_x - seismic shear force

In case the stability coefficient is less than 0.1, second-order effects from P-delta can be disregarded. Tables 9.66-68 in Appendix C provide the computations for Frame A for both directions.

Calculations show that the stability coefficient is less than 0.1, allowing P-delta effects to be neglected.

3.4.1.5. Irregularities check

Table xx in Appendix A displays the results of a torsional irregularity check for seismic drift. Calculations show that the maximum to average story drift ratio is less than 1.2. Hence, there is no torsional irregularity for this building. Additionally, deflection calculations show no stiffness-soft story irregularity.

3.4.2 Internal force calculations

Internal force calculations demonstrated by SAP2000 under different load conditions illustrated in next Figures 3.23-3.30.

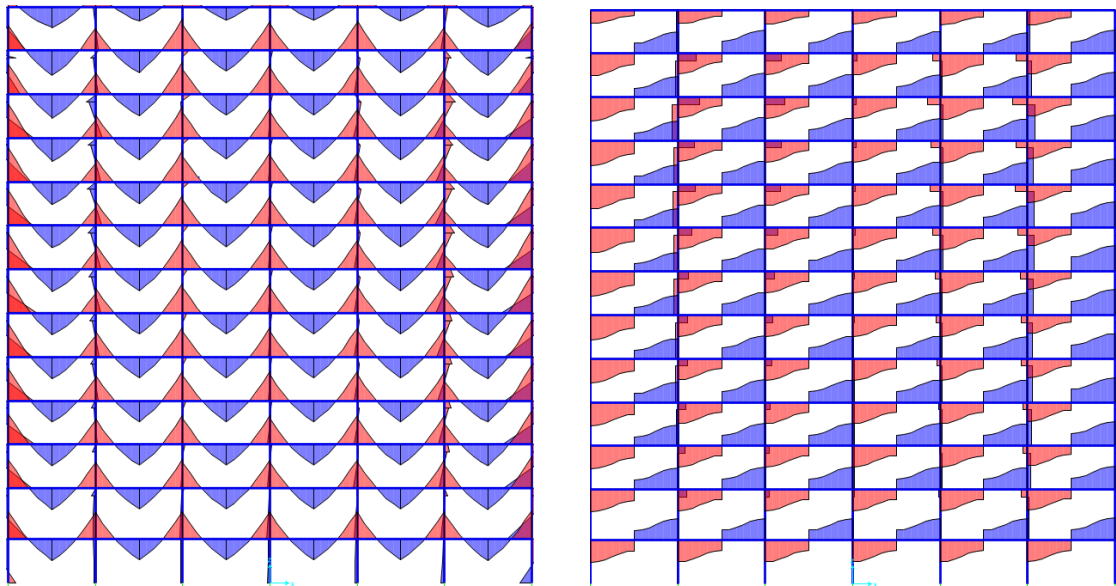


Figure 3.23-3.24. Moment and shear force diagrams under dead load in SAP2000

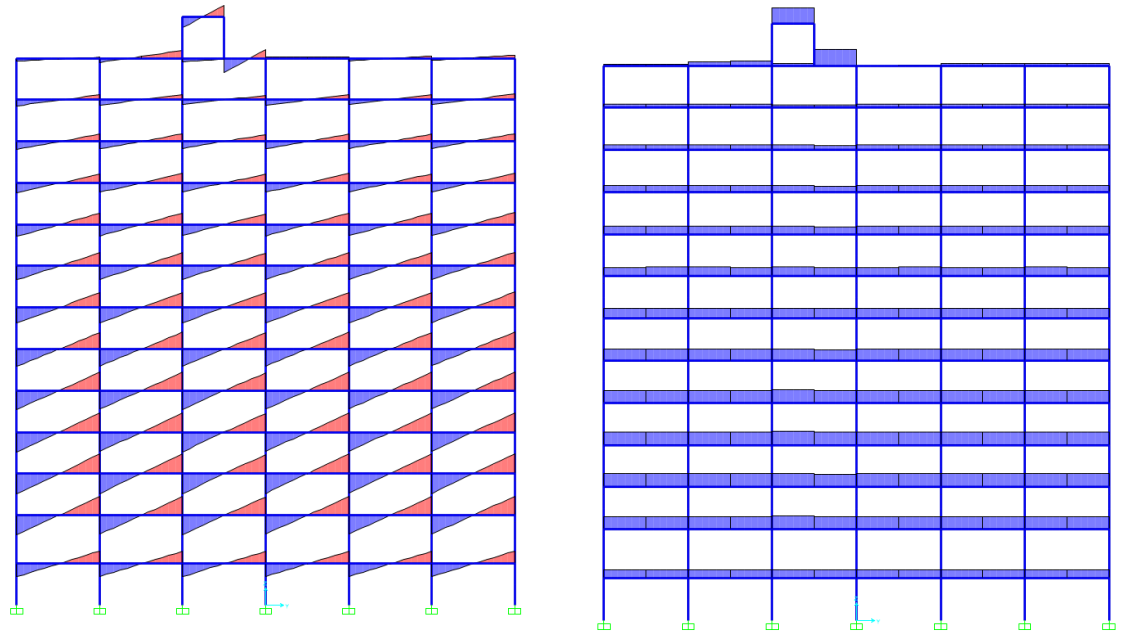
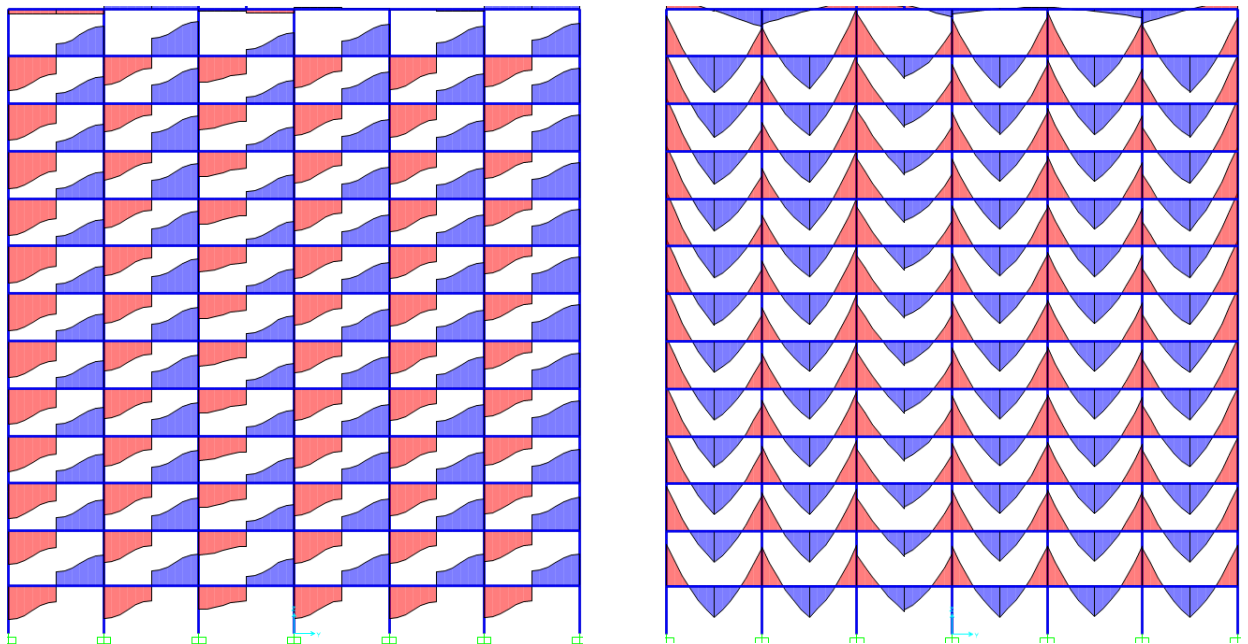
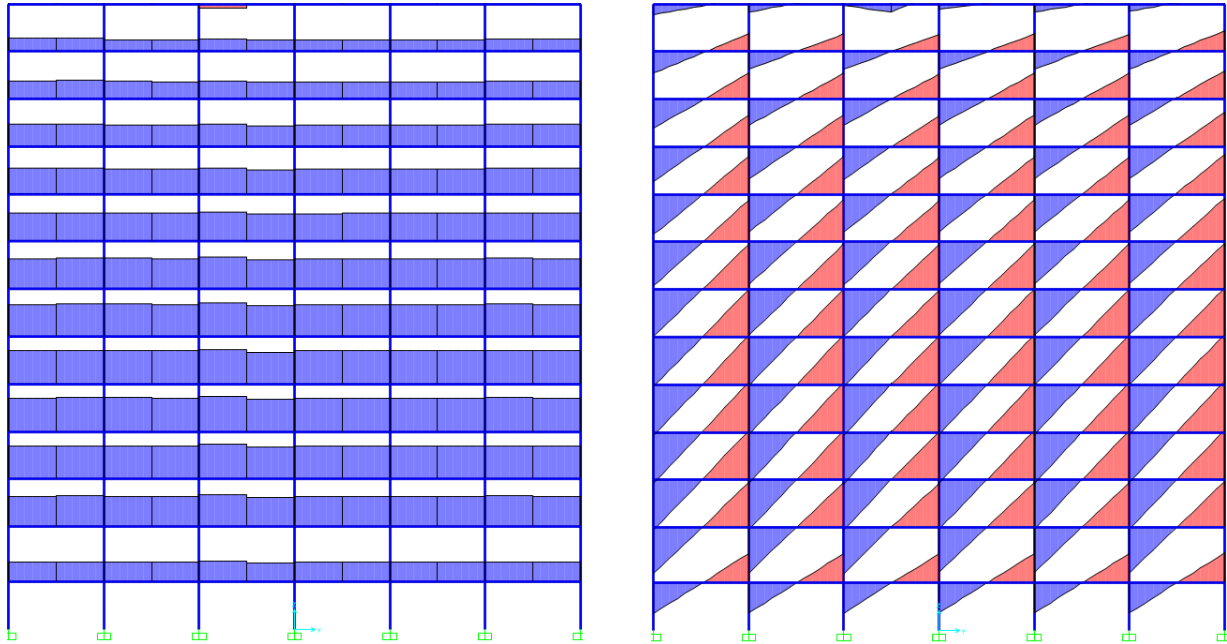


Figure 3.25-3.26. Moment and shear force diagrams under wind load in SAP 2000



Figures 3.27-3.28. Moment and shear force diagrams in SAP2000 under live load



Figures 3.29-3.30. Moment and shear force diagrams in SAP2000 under seismic load

3.4.2.1. Internal forces verifications under Dead load

For determining the internal forces caused by the dead load, we have conducted an approximate analysis of structures that are statically indeterminate (Hibbeler and others, 2012). At first stage, we assumed an inflection point as $0.1L$ but for improving accuracy with SAP2000 calculations it was adjusted to be at $0.283L$ (or one point seven meters). The typical load distribution for inner beams, pertaining to the dead weight of both floor and roof, is shown in Figure 3.31.

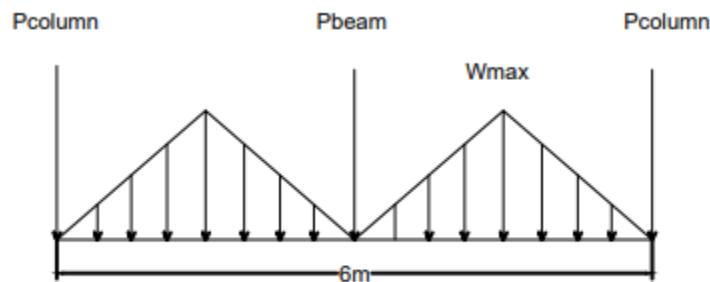


Figure 3.31. Load arrangement for internal beams.

The loading for the approximate analysis are demonstrated in Figure 3.32.

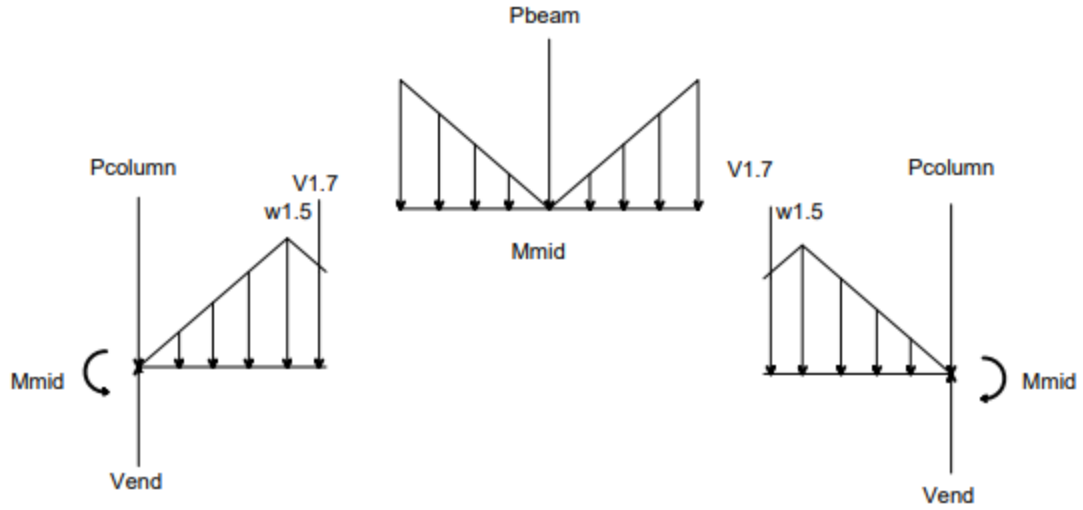


Figure 3.32. Loading arrangement for approximate analysis method.

$$L = 6 \text{ m}, 0.283L = 1.7 \text{ m}, 0.717L = 4.3 \text{ m}$$

Floor dead load

$$w_{1.5} = 20.6 \text{ kN/m (maximum distributed load)}$$

$$w_{0.6} = 20.6 \cdot \frac{0.6}{1.5} = 8.24 \text{ kN/m}$$

$$V_{0.6} = 51.76 \text{ kN}$$

$$V_{end} = 179.118 \text{ kN}$$

$$M_{mid} = 78.5 \text{ kN} - m$$

$$M_{end} = 36.27 \text{ kN} - m$$

These points were taken into consideration while designing the bending moment diagram (BMD). But moments are applicable for 2-11 floors, since 12th floor will have roof dead load, whereas first floor height is 3.5 m compared to other floors having a height of 3 m. Therefore, top and bottom moments on first floor need to be calculated due to variations in height. In the equation, the variations in flexural stiffness is taken into account, but for the first and second floors column stiffness is the same. The simplified equations are as follows:

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

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

The top and bottom moments are:

$$M_{top} = 61.3 \text{ kN} - m$$

$$M_{bottom} = 62.9 \text{ kN} - m$$

Roof dead load

The calculations for roof dead load is same with some changes in dead load value. The results can be seen below:

$$w_{1.5} = 17.004 \text{ kN/m (maximum distributed load)}$$

$$V_{end} = 206.8 \text{ kN}$$

$$M_{mid} = 56 \text{ kN} - m$$

$$M_{end} = 78.5 \text{ kN} - m$$

The bending moment diagram for this building is illustrated in Figure 3.33.

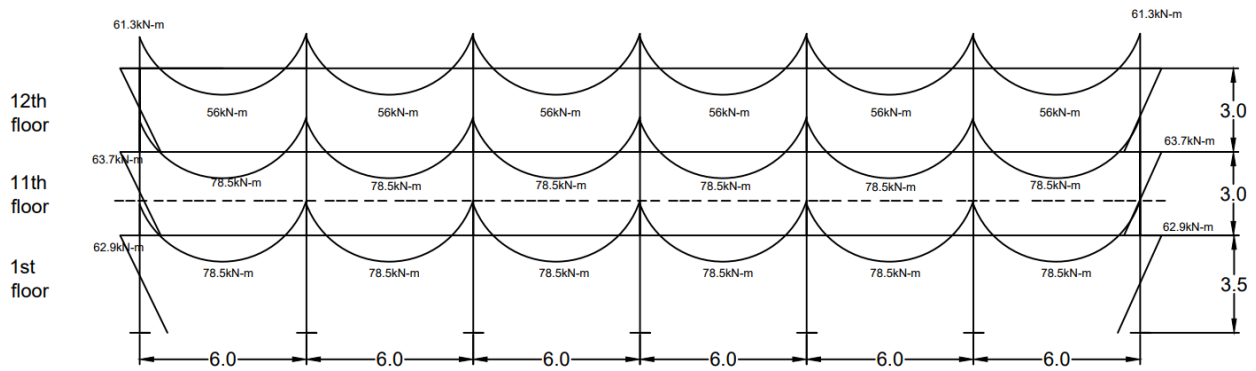


Figure 3.33. Moment frame diagram.

The following equation was used, to calculate the shear force diagram,

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

Figure 3.34. Shear force calculation for one column.

The shear force diagram for the building can be seen in Figure 3.35.

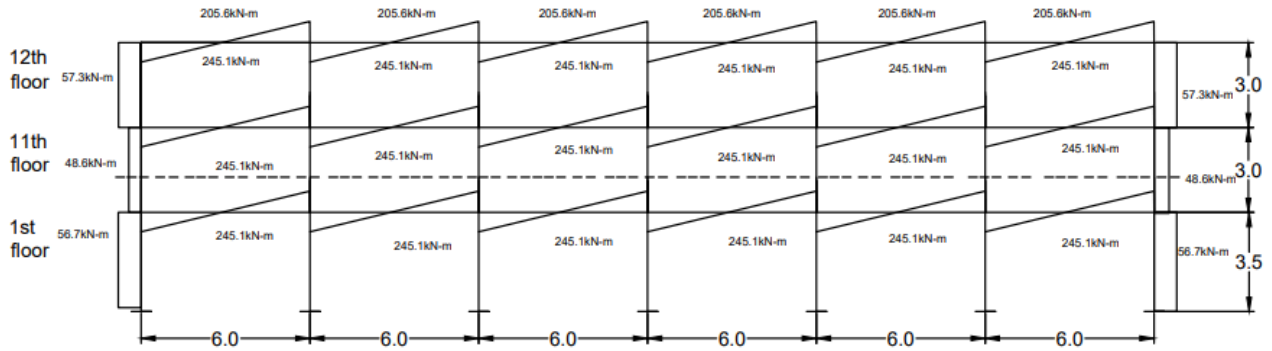


Figure 3.35. Shear force diagram.

The V_{end} was considered to draw an axial force diagram. It increases with each floor.

$$\text{For 12th floor: } V_{end} = 205.6 \text{ kN}$$

The axial force diagram for the building is demonstrated in Figure 3.36.

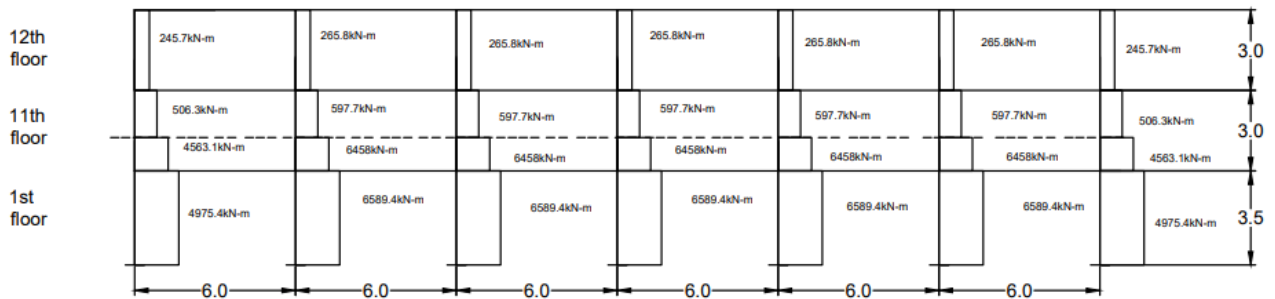


Figure 3.36. Axial force diagram.

Table 3.35. The hand calculations of internal forces under dead load.

	Exterior column				Interior column		Exterior column	Interior column
Story	w1.5	M0.6	V0.6	Vend	Vend	Column	Reaction Force	Reaction force
12	14.37	5.75	44.56	157.94	204.22	0.00	157.94	204.22
11	21.88	8.75	66.20	225.54	294.36	6.36	389.83	504.94

10	21.88	8.75	66.20	225.54	294.36	11.30	626.67	810.60
9	21.88	8.75	66.20	225.54	294.36	11.30	863.50	1116.26
8	21.88	8.75	66.20	225.54	294.36	14.30	1103.34	1424.92
7	21.88	8.75	66.20	225.54	294.36	17.66	1346.54	1736.94
6	21.88	8.75	66.20	225.54	294.36	21.37	1593.44	2052.66
5	21.88	8.75	66.20	225.54	294.36	21.37	1840.34	2368.38
4	21.88	8.75	66.20	225.54	294.36	21.37	2087.25	2684.11
3	21.88	8.75	66.20	225.54	294.36	25.43	2338.21	3003.89
2	21.88	8.75	66.20	225.54	294.36	29.84	2593.59	3328.09
1	21.88	8.75	66.20	225.54	294.36	29.84	2848.97	3652.29

3.4.2.2 Internal forces verifications under Wind load

The internal forces for Frame 1 were determined by employing the Portal Frame Method, with the following assumptions made:

- In the middle of every girder, a hinge is installed. This assumption comes from thinking that this point has no moment or in other words, the moment equals zero there.
- A hinge is located in the middle of every column, because we assume that at this point, the moment equals zero.
- On a certain floor level, the force acting on the inside column is two times that at outer column junctions. This consideration comes because the frame behaves as a composite of doorways or entrances.

The inside forces of Frame 1 were computed for every level. The hand-calculation method used on levels 20 and 19 is shown in Appendix C Figures 9.3-9.5. This same process was done with the other remaining levels and we got the calculated results using a spreadsheet. The results for all the floors are presented in Table 3.36.

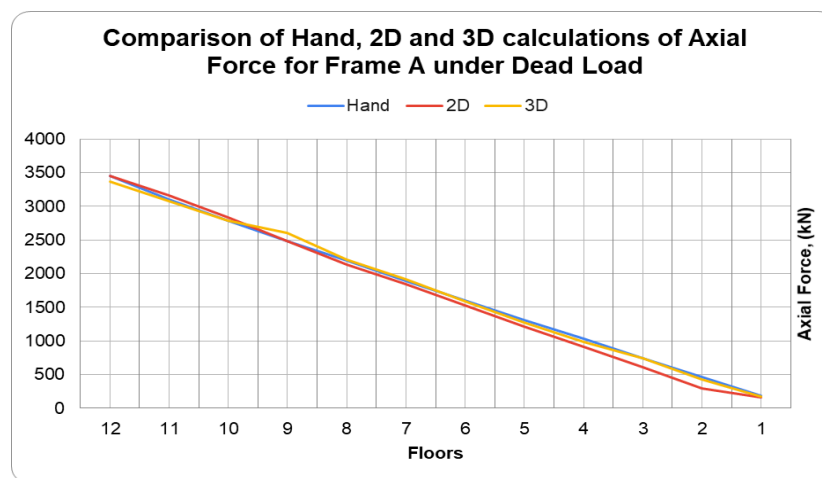
Table 3.36. The hand calculations of internal forces under wind load.

	Internal Column			External Column		
Story	Axial Force (kN)	Shear Force (kN)	Internal moment (kN-m)	Axial Force (kN)	Shear Force (kN)	Internal moment (kN-m)
12	0.00	3.83	5.74	-0.96	1.91	2.87
11	0.00	7.57	11.35	-3.81	3.78	5.68

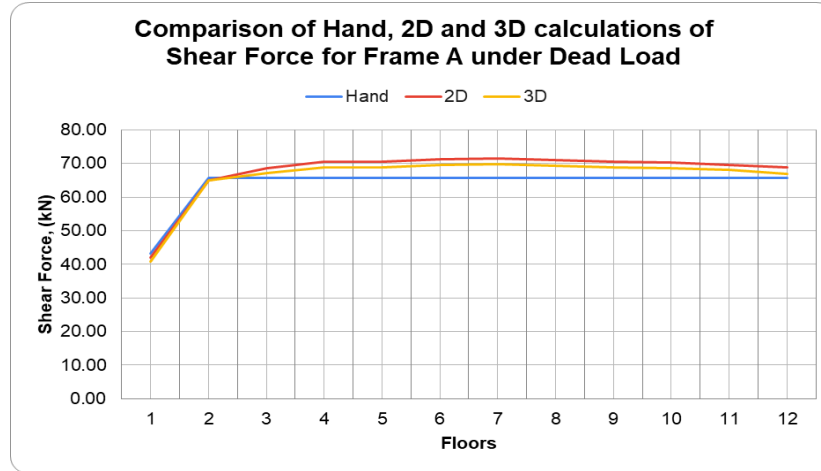
10	0.00	11.21	16.82	-8.50	5.61	8.41
9	0.00	14.76	22.14	-14.99	7.38	11.07
8	0.00	18.19	27.29	-23.23	9.10	13.64
7	0.00	21.51	32.26	-33.16	10.75	16.13
6	0.00	24.76	37.15	-44.72	12.38	18.57
5	0.00	27.87	41.81	-57.88	13.94	20.90
4	0.00	30.75	46.12	-72.54	15.37	23.06
3	0.00	33.43	50.14	-88.58	16.71	25.07
2	0.00	35.87	53.81	-105.91	17.94	26.91
1	0.00	38.74	67.80	-127.67	19.37	33.90

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

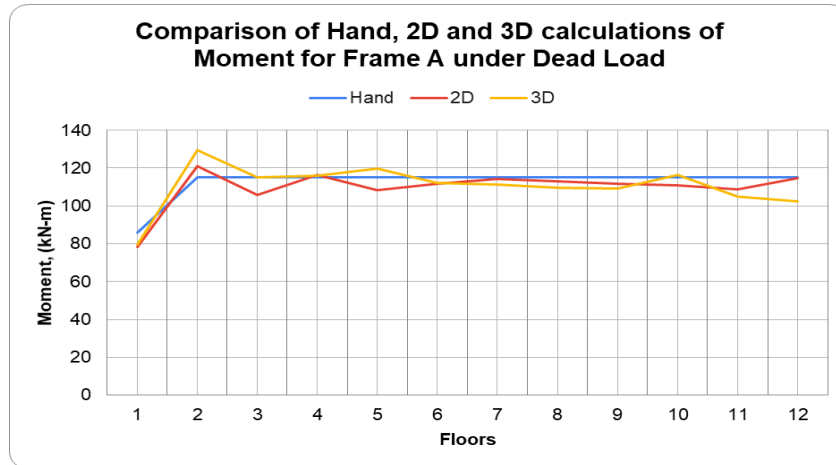
You can see in Figure 3.36 the comparison of internal forces under dead weight for Frame 3 using methods like hand calculations, and 2D or 3D models. The figure shows that the axial and shear forces from these three computation methods are nearly identical. But when looking at the bending moment diagram, a small difference appears with SAP2000's model moments because of how stiffness between structural parts interact.



a)



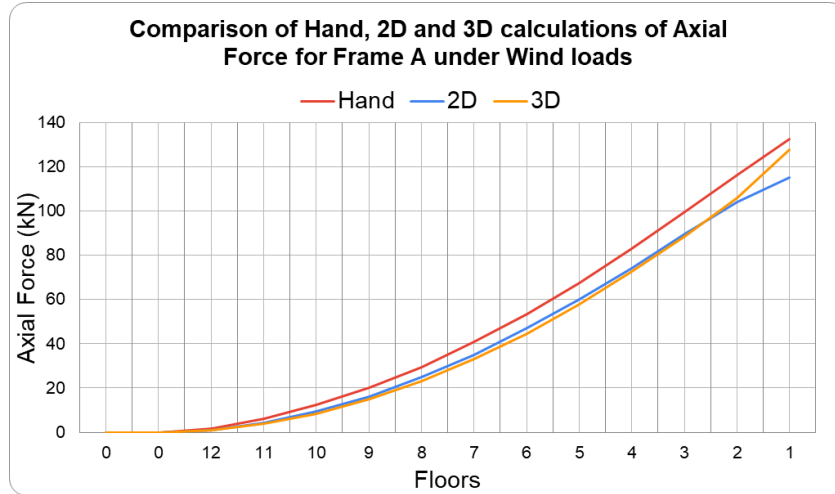
b)



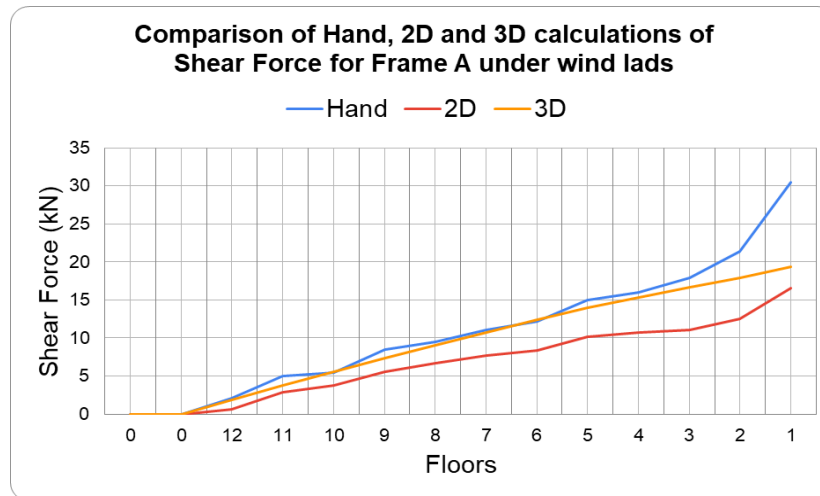
c)

Figure 3.36. Comparison of internal forces for Frame A under dead load: (a) axial force; (b) shear force; (c) bending moment.

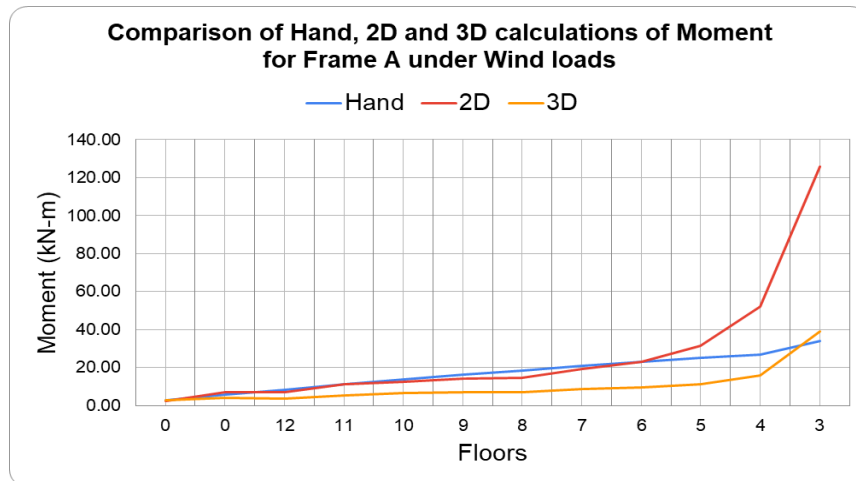
Similarly, comparisons of internal forces under wind load for Frame 3 are shown in Figure 3.37. It shows that the axial and shear force diagram in all 2D, 3D and hand calculations were in line with each other. However, the moment diagram for hand calculations on the lower floors is not identical because the approximate analysis method was used to find the internal forces using hand calculations.



a)



b)



c)

Figure 3.37. Comparison of internal forces for Frame A under wind load: (a) axial force; (b) shear force; (c) bending moment.

3.4.3 Load combinations

The Chapter 2 of ASCE 7-16 provided the foundation for selecting the load combinations. The analysis includes four types of forces consisting of dead, live, roof live, seismic and wind loads. The design incorporated wind load cases from all perspectives. SAP2000 shows the load combinations through Figure 3.38 in the results. The analysis determined $1.2D + 1L \pm 1E_y$ as the critical combinations which resulted in maximum forces and moments.

1.2D+1.6L
0.9D+1W
1.2D+1E+0.2S
1.2D+1L+1E
1.2D+1.6L+0.8W
1.2D

Figure 3.38. Load combinations used in SAP2000

3.5 Structural design

3.5.1 Structural member design using software

For doing the building's structural design, we used SAP2000 software. This tool can independently do analysis and find reinforcement percentages for each structure part by referring to the ACI 318 design code of reinforced concrete. We utilized SAP2000's potential in this project for executing the structural design; nevertheless manual calculations must confirm it as detailed in Section 3.5.2. Figure 3.39 illustrates the design output of the building.

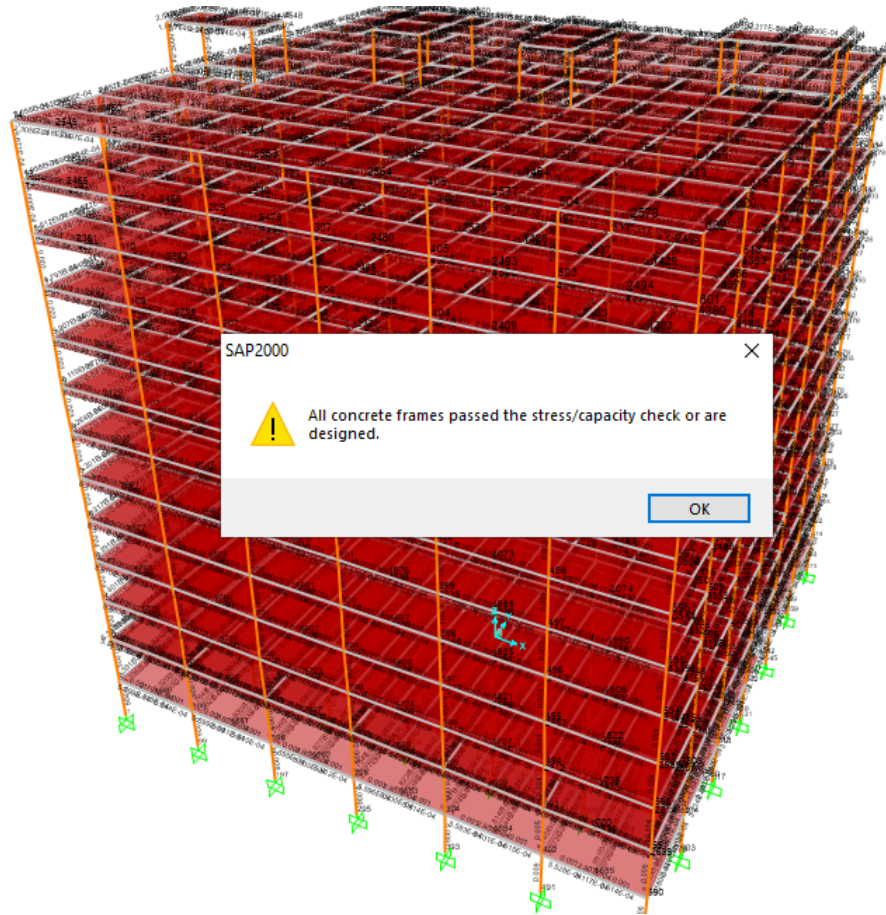
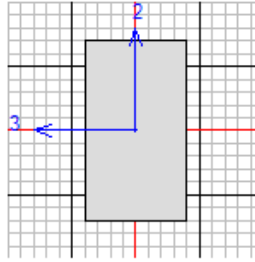


Figure 3.39. Structural design check

3.5.1.1 Major beams

For designing main beams, both critical negative and positive moments have been considered. The building is exposed to large earthquake forces so we can consider that seismic forces would control the structural design of this building. Consequently, the positive and negative moments were taken from the main load combinations . Because of the sideways earthquake forces, important moments were created at the beam's ends. Thus, providing bottom reinforcement without interruption will depend on the positive end moment; meanwhile, top reinforcement shall be given over beams' edges. Look at Figure 3.40 for design details regarding the major beam found at the edge-of-beam where the critical negative moment is present. Figure 3.40 presents the design details for the main beam, pointing where there is a critical positive moment at its edge.



Units

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

Element	: 1457	D=700.000	B=400.000	bf=400.000
Section ID	: Major beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
252239.046	-504478.091	252239.046	-504478.091

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	2230.555	0.000	2230.555	995.448
Bottom (-2 Axis)	1085.426	1085.426	0.000	995.448

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.834	371.483	205.901	165.582	173.698

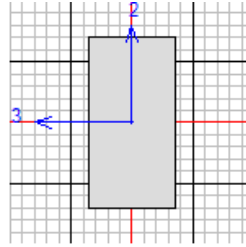
Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	8130.800	59312.213	161596.229	1844.400

Figure 3.40. Major beam design in SAP2000

3.5.1.2. Minor beams

Similarly as for major beams, the critical negative and positive moments were taken. Figure 3.41 shows structural design information about the minor beam.



Units KN, mm, C ▾

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

Element : 3117 D=400.000 B=200.000 bf=200.000
 Section ID : Minor beam ds=0.000 dct=60.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
77370.309	-27267.864	27267.864	-27267.864

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	264.416	0.000	219.543	264.416
Bottom (-2 Axis)	647.138	647.138	0.000	264.416

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	41.308	43.754	16.408	32.539

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	1184.215	8582.128	29378.729	844.400

Figure 3.41. Minor beam design in SAP2000

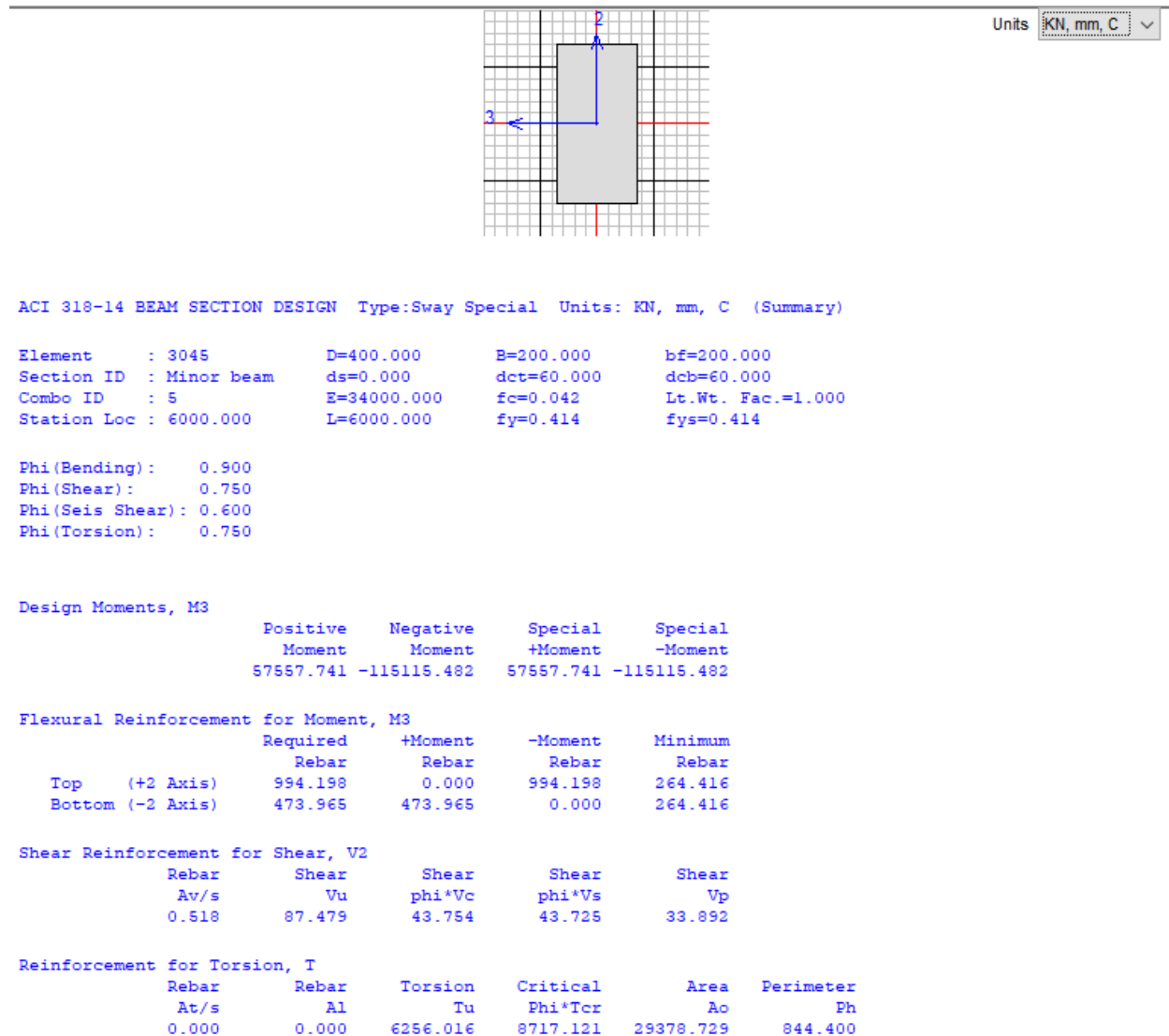
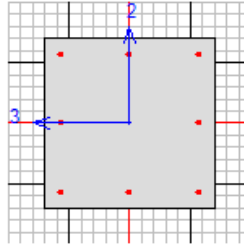


Figure 3.42. Minor beam design in SAP2000

3.5.1.2. Columns

For the column design, the corner column was taken and design details can be seen in Figure 3.43.



Units KN, mm, C

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

Element	: 119	B=700.000	D=700.000	dc=67.026
Section ID	: Column 1	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Combo ID	: 5	L=3500.000	fy=0.414	fys=0.414
Station Loc	: 3500.000	RLLF=1.000		

Phi(Compression-Spiral):	0.750	Overstrength Factor:	1.25
Phi(Compression-Tied):	0.650		
Phi(Tension Controlled):	0.900		
Phi(Shear):	0.750		
Phi(Seismic Shear):	0.600		
Phi(Joint Shear):	0.850		

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

Rebar	Design	Design	Design	Minimum	Minimum
Area	Pu	M2	M3	M2	M3
13274.678	11178.204	-376890.919	-405146.206	405098.111	405098.111

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Cm	Delta_ns	Delta_s	K	L
	Factor	Factor	Factor	Factor	Length
Major Bending(M3)	1.000	1.000	1.000	1.000	3500.000
Minor Bending(M2)	1.000	1.000	1.000	1.000	3500.000

SHEAR DESIGN FOR V2,V3

	Rebar	Shear	Shear	Shear	Shear
	Av/s	Vu	phi*Vc	phi*Vs	Vp
Major Shear(V2)	0.000	344.035	945.934	0.000	344.035
Minor Shear(V3)	0.680	423.871	756.747	106.911	314.219

JOINT SHEAR DESIGN

	Joint Shear	Shear	Shear	Shear	Joint
	Ratio	VuTop	VuTot	phi*Vc	Area
Major Shear(V2)	0.562	344.035	1505.408	0.003	490000.000
Minor Shear(V3)	0.511	314.219	1370.026	0.003	490000.000

(6/5) BEAM/COLUMN CAPACITY RATIOS

Major	Minor
Ratio	Ratio
0.197	0.180

Figure 3.43. Column design in SAP2000

3.5.1.4. Two-way slab

SAP2000 does not provide information on the detailed reinforcement of the slab; so, the reinforcement will be designed manually.

3.5.2 Hand verifications of structural members

3.5.2.1. Major beams

3.5.2.1.1. Flexural analysis

To confirm the reinforcement given in SAP2000 for the main beam found in Figure 3.40, calculations below were done. For the selected structure part, the negative moment controlling factor is on the tension side. In Figure 3.40 major beam displays the most important positive moment, design will be from lower reinforcement section. It's important to remember that the compression reinforcement in the middle of the beam span was not taken into account. Also, it is supposed that design will be controlled by tension, hence $\phi = 0.9$.

Negative moment

$$M_u = 504 \text{ kN} \cdot \text{m}$$

$$b = 400 \text{ mm}, h = 700 \text{ mm}, d = 700 - 2.5 \text{ in} \cdot 25.4 \text{ mm/in} = 636.5 \text{ mm} \sim 630 \text{ mm}$$

$$f'_c = 40 \text{ MPa} (5801 \text{ psi}), f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{504}{0.9 \cdot 400 \cdot 630 \cdot 630} = 3174.6 \text{ kN/m}^2$$

$$\rho = \frac{0.85 f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2 R_n}{0.85 f'_c}} \right] = 0.008$$

$$A_s = \rho b d = 0.0107 \cdot 400 \cdot 630 = 2032 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \rightarrow a = \frac{2032 \cdot 413}{0.85 \cdot 40 \cdot 400} = 61.86 \text{ mm}$$

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

$$c = \frac{a}{\beta_1} = \frac{61.86}{0.76} = 81.4 \text{ mm}$$

$$\epsilon_t = 0.003 \left(\frac{d-c}{c} \right) = 0.003 \left(\frac{630-81.4}{81.4} \right) = 0.0202 > 0.005 \rightarrow \text{tension-controlled and ductile}$$

However, the A_s calculated by SAP2000 is 2230 mm^2 . Therefore, the error should be calculated as:

$$\text{Error} = \frac{2230 - 2032}{2230} \times 100\% = 8.87\%$$

The large discrepancy lies within these assumptions about concrete cover and SAP2000's ability to determine tension along with compression reinforcement. The minimum steel area needs to be checked as part of the calculation process

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 959.67 \text{ mm}^2 > 840 \text{ mm}^2$$

$$A_s = 2220 \text{ mm}^2 > 959.67 \text{ mm}^2 \rightarrow \text{sufficient reinforcement}$$

Therefore, the $A_s = 2230 \text{ mm}^2$ (3.44 in²) will be used for bar selection. Accordingly, at the negative moment locations, 3#10 bars will be chosen ($A_s = 3.81 \text{ in}^2$). Therefore, the minimum width of the beam should be checked.

Positive moment

$$M_u = 252.2 \text{ kN} - \text{m}$$

$$R_n = \frac{M_u}{\phi bd^2} = \frac{598.2}{0.9 \cdot 400 \cdot 630} = 1590.45 \text{ kN/m}^2$$

$$\rho = \frac{0.85f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right] = 0.0039$$

$$A_s = \rho bd = 0.0077 \cdot 500 \cdot 630 = 991.9 \text{ mm}^2$$

$$A_s f_y = 0.85f'_c ab \rightarrow a = \frac{991.9 \cdot 413}{0.85 \cdot 40 \cdot 500} = 30.2 \text{ mm}$$

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

$$c = \frac{a}{\beta_1} = \frac{30.2}{0.76} = 39.73 \text{ mm}$$

$$\epsilon_t = 0.003 \left(\frac{d-c}{c} \right) = 0.003 \left(\frac{630-39.73}{39.73} \right) = 0.0445 > 0.005 \rightarrow \text{tension-controlled and ductile}$$

However, the A_s calculated by SAP2000 is 1085 mm². Therefore, the error should be calculated as:

$$\text{Error} = \frac{1085 - 991.9}{1085} \times 100\% = 10.64\%$$

The difference can also be explained like for the negative moment case. Furthermore, the minimum steel area should be checked:

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 959.67 \text{ mm}^2 > 840 \text{ mm}^2$$

$$A_s = 991.9 \text{ mm}^2 > 957.67 \text{ mm}^2 \rightarrow \text{sufficient reinforcement}$$

The choice of bars for selection will be based on the value $A_s = 1085 \text{ mm}^2$ (1.72 in²). The designed bottom reinforcement includes 4#6 reinforcing bars extending without interruption with

a total area of $A_s = 1.77 \text{ in}^2$. The structural width using #4 stirrups will be $b_{min} = 406.4 \text{ mm}$ while the actual width is $b = 500 \text{ mm}$.

3.5.2.1.2. Shear analysis

The maximum $V_u = 371 \text{ kN}$ was chosen from Figure 3.20. $\lambda = 1$ for normal-weight concrete.

$$\phi V_c = \phi \cdot 2\lambda \sqrt{f'_c} b_w d = 0.75 \cdot 2 \cdot \sqrt{40000} \cdot 400 \cdot 630 = 75.6 \text{ kN}$$

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

$$V_{c1} = 4\sqrt{f'_c} b_w d = 201.6 \text{ kN}, V_{c2} = 8\sqrt{f'_c} b_w d = 403.2 \text{ kN}$$

$$V_s = \frac{V_u - \phi V_c}{\phi} = 393.8666667 \text{ kN} < V_{c2}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{393.8}{0.413 \cdot 630} = 1.51 \text{ mm}^2/\text{mm}$$

The value of A_v/s calculated by SAP2000 is $12.296 \text{ mm}^2/\text{mm}$.

$$\text{Error} = \frac{1.834 - 1.51}{1.834} \times 100\% = 17\%$$

SAP2000 could have used different concrete cover and d assumptions to explain its large variation with respect to the design output. The design of stirrups requires the use of $A_v/s = 1,834 \text{ mm}$ as the specified value. The major beam shear design requires the implementation of #4 stirrups. The area of #4 stirrups is 129 mm^2 (0.2 in^2).

3.5.2.1.2. Torsional analysis

The design must comply with seismic forces so torsional checks need to occur. From Figure XX, $T_u = 8.13 \text{ kN-m}$. The section characteristics must be determined first.

$$A_{cp} = 400 \cdot 700 = 0.28 \text{ m}^2, P_{cp} = 2(400 + 700) = 2.2 \text{ m}$$

$$T_n = \phi \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) = 5.34 \text{ kN} - \text{m} < T_u = 8.13 \rightarrow \text{torsional reinforcement is needed.}$$

The other sectional parameters can be calculated as:

$$A_{oh} = \left[\frac{500}{25.4} - 2 \left(1.5 + \frac{3}{16} \right) \right] \cdot \left[\frac{700}{25.4} - 2 \left(1.5 + \frac{3}{16} \right) \right]$$

$$A_{oh} = 299.2 \text{ in}^2 \text{ or } 0.193 \text{ m}^2$$

$$A_0 = 0.85A_{oh} = 0.164 m^2$$

$$P_h = 1.8571 m$$

Next, it is necessary to verify whether the section can be designed to resist torsion:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \leq \phi \left[\left(\frac{V_c}{b_w d}\right) + 8\sqrt{f'_c} \right] \quad (3.9)$$

$1352 \text{ kN/m} < 1500 \text{ kN/m} \rightarrow$ the section is adequate

$$\frac{A_t}{s} = \frac{T_n}{2A_0 f_{yt} \cot \theta} = \frac{5.34/0.75}{2 \cdot 0.164 \cdot 413/1000} = 0.000081 = 0.081 \text{ mm}$$

Data in Figure XX matches the calculated $A_v/s = 0 \text{ mm}$ value. The shear reinforcement system yields better performance results when used in combination with torsion reinforcement.

$$\frac{A_{vt}}{s} = 1.834 + 2.0 = 1.834 \text{ m}^2/\text{m}$$

The spacing for the #4 stirrups resisting both shear and torsion forces will be:

$$s = \frac{2 \cdot 0.2}{2.4} = 0.1667 \text{ m} \approx 0.15 \text{ m or } 150 \text{ mm}$$

This spacing should be compared with the limits:

$$s_1 = 12 \text{ in or } 304.8 \text{ mm}, s_2 = P_h/8 = 257 \text{ mm} > s = 150 \text{ mm}$$

Accordingly, the minimum reinforcement area should be checked:

$$A_{vt} = 0.75 \sqrt{f'_c} \frac{b_w s}{f_{yt}} \geq 50 \frac{b_w s}{f_{yt}} \rightarrow 0.1107 \text{ in}^2 > 0.0969 \text{ in}^2$$

The provided $A_v = 0.4 \text{ in}^2 > 0.1107 \text{ in}^2 \rightarrow$ sufficient reinforcement.

Hence, #4 stirrups spaced at 150 mm should be provided for the major beam design

3.5.2.2. Minor beams

3.5.2.2.1. Flexural analysis

A similar procedure for a minor beam has been performed as for the major beam.

Negative moment

$$M_u = 115.115 \text{ kN} - \text{m}$$

$$b = 200 \text{ mm}, h = 400 \text{ mm}, d = 400 - 2.5 \text{ in} \cdot 25.4 \text{ mm/in} = 336.5 \text{ mm} \sim 330 \text{ mm}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{120.539}{0.9 \cdot 200 \cdot 330} = 5285.353535 \text{ kN/m}^2$$

$$\rho = \frac{0.85f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right] = 0.013$$

$$A_s = \rho b d = 0.013 \cdot 200 \cdot 330 = 923.47 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \rightarrow a = \frac{923.47 \cdot 413}{0.85 \cdot 40 \cdot 200} = 56.03 \text{ mm}$$

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

$$c = \frac{a}{\beta_1} = \frac{60.88}{0.76} = 73.77 \text{ mm}$$

$$\varepsilon_t = 0.003 \left(\frac{d-c}{c} \right) = 0.003 \left(\frac{330-73.77}{73.77} \right) = 0.0104 > 0.005 \rightarrow \text{tension-controlled and ductile}$$

However, the A_s calculated by SAP2000 is 994 mm^2 . Therefore, the error should be calculated as:

$$\text{Error} = \frac{994 - 923.47}{994} \times 100\% = 7.14\%$$

The calculated difference is significant because SAP2000 combines both tension and compression steel calculations while making concrete cover assumptions. The verification of minimum steel area requirements must be performed.

$$A_{s,min} = 3\sqrt{f'_c} b d / f_y \geq 200 b d / f_y = 251.34 \text{ mm}^2 > 220 \text{ mm}^2$$

$$A_s = 997.47 \text{ mm}^2 > 251.34 \text{ mm}^2 \rightarrow \text{sufficient reinforcement}$$

Therefore, the $A_s = 994 \text{ mm}^2$ (1.54 in^2) will be used for bar selection. Accordingly, for the negative moment for the top reinforcement 2#8 bars will be used ($A_s = 1.57 \text{ in}^2$).

Positive moment

$$M_u = 77.37 \text{ kN} - m$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{77.37}{0.9 \cdot 200 \cdot 330} = 3552.341598 \text{ kN/m}^2$$

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

$$A_s = \rho b d = 0.0105 \cdot 200 \cdot 330 = 600 \text{ mm}^2$$

$$A_s f_y = 0.85 f'_c a b \rightarrow a = \frac{699 \cdot 413}{0.85 \cdot 40 \cdot 200} = 36.49 \text{ mm}$$

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

$$c = \frac{a}{\beta_1} = \frac{42.59}{0.76} = 48 \text{ mm}$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{600-48}{48}\right) = 0.0176 > 0.005 \rightarrow \text{tension-controlled and ductile}$$

However, the A_s calculated by SAP2000 is 647 mm². Therefore, the error should be calculated as:

$$\text{Error} = \frac{647-600}{600} \times 100\% = 7.12\%$$

The method for interpretation follows a pattern similar to the explanation developed for negative moment applications. The steel area needs to be checked against minimum established criteria.

$$A_{s,min} = 3\sqrt{f'_c}bd/f_y \geq 200bd/f_y = 251.34 \text{ mm}^2 > 220 \text{ mm}^2$$

$$A_s = 600 \text{ mm}^2 > 251.34 \text{ mm}^2 \rightarrow \text{sufficient reinforcement}$$

The minimum bar area selection for the model in SAP2000 will be $A_s = 647 \text{ mm}^2$ (1.01 in²).

The bottom reinforcement selection consists of 2#7 bars with an area of 1.20 in².

3.5.2.2.2 Shear analysis

The $V_u = 81.479 \text{ kN}$ was chosen from Figure 3.23. $\lambda = 1$ for normal-weight concrete.

$$\phi V_c = \phi \cdot 2\lambda \sqrt{f'_c} b_w d = 0.75 \cdot 2 \cdot \sqrt{40000} \cdot 200 \cdot 330 = 19.8 \text{ kN}$$

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

$$V_{c1} = 4\sqrt{f'_c} b_w d = 52.8 \text{ kN}, V_{c2} = 8\sqrt{f'_c} b_w d = 105.6 \text{ kN}$$

$$V_s = \frac{V_u - \phi V_c}{\phi} = 82.23866667 \text{ kN} < V_{c1}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{82.238}{0.413 \cdot 330} = 0.603 \text{ mm}^2/\text{mm}$$

$$(\min) \frac{A_v}{s} = 0.75 \sqrt{f'_c} \left(\frac{b_w}{f_{yt}}\right) \geq 50 \left(\frac{b_w}{f_{yt}}\right) \quad (\min) 0.19 \geq 0.16$$

Through manual calculations the value of $\frac{A_v}{s}$ comes out to 0.19 mm²/mm. The $\frac{A_v}{s}$ value obtained from SAP2000 arrives at 0.482 mm²/mm. The software version of shear strength analysis in SAP2000 differs greatly from manual calculations because the programs use different

basic assumptions. Design engineers assumed #3 stirrups for the minor beam design ($A_s = 0.11$ in²). The computation for spacing needs to be performed at this stage.

$$s = \frac{A_v f_{yt} d}{V_s} = \frac{(2 \cdot 0.11 \cdot 645.2 / 1000000)(413 / 1000)(0.43)}{36} = 700 \text{ mm}$$

The maximum spacings:

$$s_2 = d/2 \leq 24 \text{ in} = 215 \text{ mm} < 609.6 \text{ mm}$$

$$s_3 = \frac{A_v f_{yt}}{50 b_w} \geq \frac{A_v f_{yt}}{0.75 \sqrt{f'_c} b_w} \rightarrow 567 \text{ mm} > 497 \text{ mm}$$

The appropriate spacing value for this design should be set at $s = 200$ mm. Researchers should use #3 stirrups spaced 200 mm apart for designing the minor beam.

3.5.2.2.3. Torsional analysis

From Figure 3.43, $T_u = 0.236$ kN-m.

$$T_n = \phi \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) = 0.8 \text{ kN} - \text{m} > T_u = 0.75 \rightarrow \text{torsional reinforcement is not required.}$$

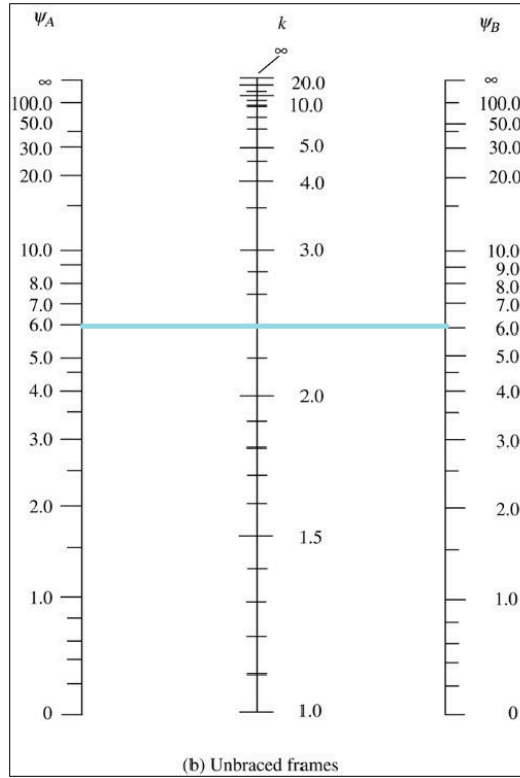
3.5.2.3. Columns

3.5.2.3.1. Slenderness ratio check

All building column types in the structural design received steel reinforcement as part of the construction plan. Verification calculations were conducted on a Floor 1 corner column which measured 0.7m by 0.7m. The initial stage requires evaluation of the column slenderness ratio for determining its short or slender classification. The absence of lateral braces or shear walls at the columns makes them swaying based on our assessment. This sway column slenderness check appears in the following display:

$$\psi_A = \frac{\Sigma(EI/l)_{columns}}{\Sigma(EI/l)_{beams}} = 6$$

$$\psi_B = \frac{\Sigma(EI/l)_{columns}}{\Sigma(EI/l)_{beams}} = 6$$



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

$$Actual \frac{kl_u}{r} = 28,256$$

The chart provided a value of k at 2.41 which was employed in slenderness ratio assessment. The analyzed results demonstrate that the slenderness ratio remains below maximum permitted value thus indicating a short column construction for axial and bending moment loading.

Therefore, we have to do moment magnification, so

$$P_c = \frac{\pi^2 EI}{(kLu)^2} = 184271.64$$

$$\delta_s = \frac{1}{1 - P_f / (0.75 * P_c)} = 1.088 < 2.5$$

$$\text{Left: } \frac{l_u}{r} = 13.3 \quad \text{Right: } \frac{l_u}{r} = 46.34$$

$$\text{New Moment: } M_u * \delta_s = 440.8$$

3.5.2.3.2 Axial and moment analysis

SAP2000 software determined that the maximum forces and moments acting on the column amount to:

- $M_u = 405.146 \text{ kN} - m$
- $P_u = 111.79 \text{ kN}$
- $V_u = 344.035 \text{ kN}$
- ϕ for tied columns is 0.65

The following process allowed identification of reinforcement ratio through those values:

$$M_n = \frac{M_u}{\phi} = 678.156 \text{ kN} - m$$

$$P_n = \frac{P_u}{\phi} = 171.9846 \text{ kN} - m$$

$$e = \frac{M_n}{P_n} = 0.039 \text{ m}$$

$$\gamma = \frac{0.5 - 0.07 \cdot 2}{0.5} = 0.8$$

$$K_n = \frac{P_n}{A_g f'_c} = 0.877$$

$$R_n = \frac{P_n e}{A_g f'_c h} = 0.049$$

Interaction diagrams for rectangular columns derived from ACI 318-19 served to determine the reinforcement ratio:

$$\rho = 0.01 = 1\%$$

$$A_s = 0.01 * A_g = 0.00735 \text{ m}^2$$

Bar selection:

$$8\#11 \text{ bars } (A_s = 0.00805 \text{ m}^2)$$

3.5.2.3.3. Shear analysis

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

$$V_c = 0.17(1 + \frac{P_u}{14A_g})\lambda\sqrt{f'_c}b_w d = 2568.707193 \text{ kN}$$

$$\phi V_c = 0.65 * 2568.707193 = 1926.530395 \text{ kN}$$

$$\phi V_c / 2 = 963.2651975 \text{ kN} > V_u = 344.035 \rightarrow \text{Only minimum amount of reinforcement is required}$$

Spacing for shear reinforcement is minimum of:

$$48 * d_t = 0.4572 m$$

$$16 * d_l = 0.57296m$$

The least column dimension = 0.35 m

3.5.2.3.4. Biaxial bending analysis

The chosen corner column experiences biaxial bending therefore the Bresler Reciprocal Load Method should be applied to assess its safety against biaxial bending.

The SAP200 software delivered bending moment information for both axes:

- $M_{ux} = 376.981 kN - m$
- $M_{uy} = 678.156 kN - m$
- $P_u = 111.79 kN$

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

$$M_{nx} = \frac{M_{ux}}{\phi} = 579.823 kN - m$$

$$M_{ny} = \frac{M_{uy}}{\phi} = 281.5 kN - m$$

$$P_n = \frac{P_u}{\phi} = 171.98 kN$$

$$\gamma_x = \gamma_y = 0.8$$

$$e_x = \frac{M_{nx}}{P_n} = 0.0337$$

$$e_y = \frac{M_{ny}}{P_n} = 0.0606$$

$$\rho = \frac{A_s}{A_g} = 0.016$$

$$e_x/h = 0.048$$

$$e_y/h = 0.086$$

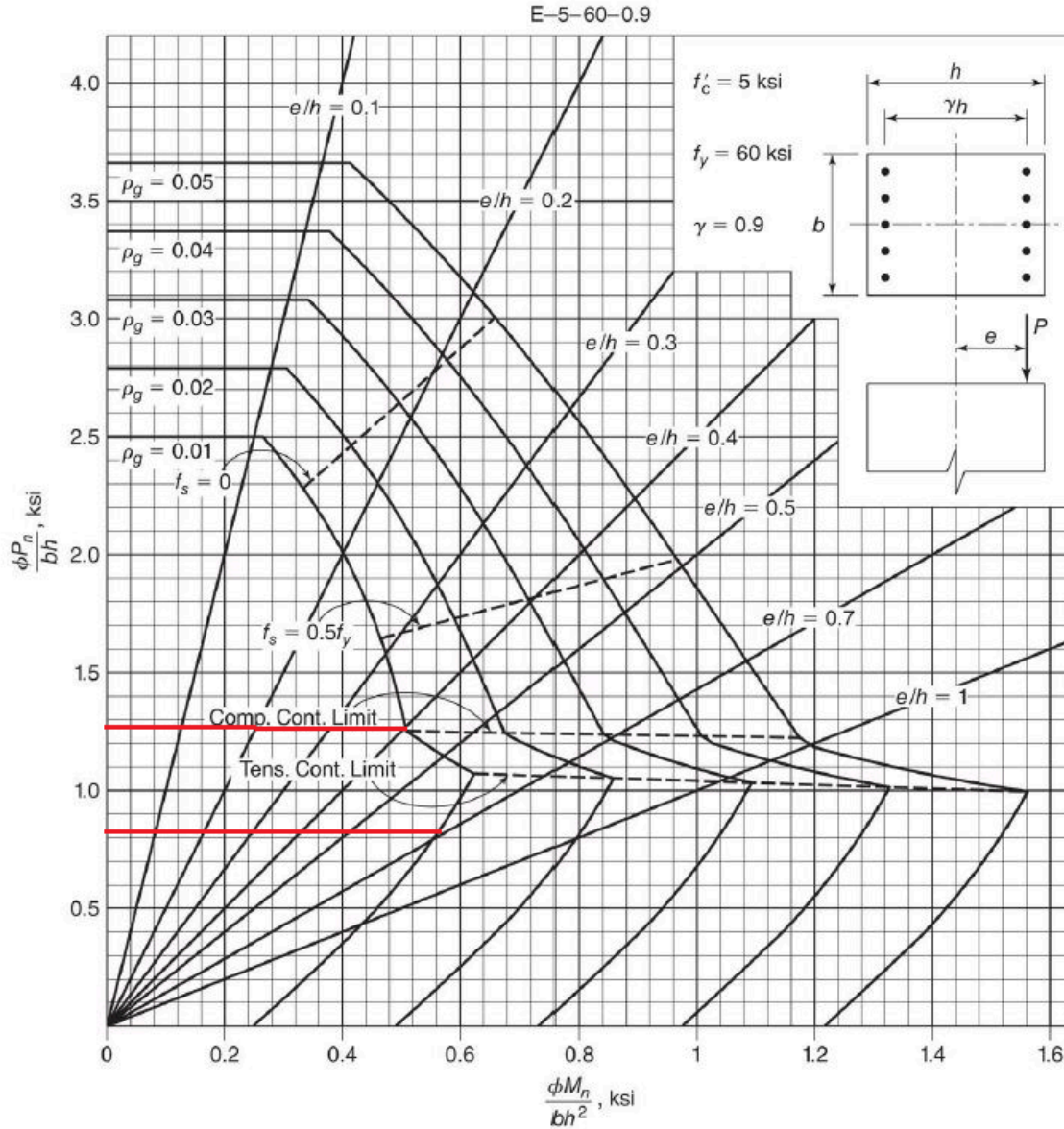


Figure 3.44. Interaction diagram for rectangular column

From the chart the values obtained for K_n are equal to 1.27 and 0.72 respectively.

$$P_{nx} = \frac{K_{nx} * f'_c * A_g}{\phi} = 39200 \text{ kN}$$

$$P_{ny} = \frac{K_{ny} * f'_c * A_g}{\phi} = 21107 \text{ kN}$$

$$P_0 = 0.85 * f'_c * A_g + A_{st} (f_y - 0.85 f'_c) = 16663 \text{ kN}$$

Calculating P_n :

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

$$P_n = 77680 \text{ kN}$$

Checking the validity of the method:

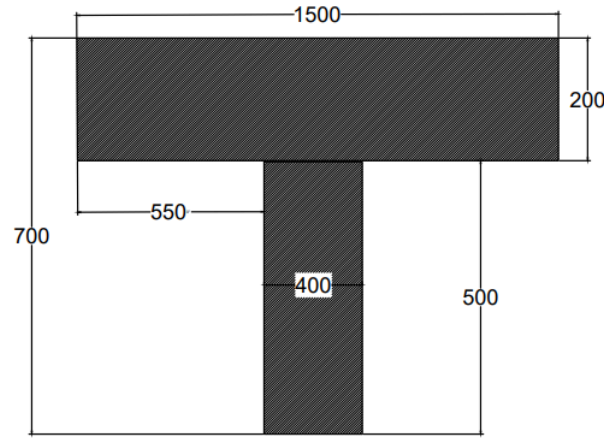
$$P_n = 77680 > 0.1P_0 \rightarrow \text{the method is valid}$$

$$\phi P_n = 50492 > P_u \rightarrow \text{the column is safe against biaxial bending}$$

3.5.2.4. Two-way slab

3.5.2.4.1. Flexural analysis

The builders performed the interior and exterior slab panel design using the Direct Design Method. The slab panel measures 3000mm*3000mm and the columns have a size of 750mm*750mm. A minimum thickness assessment needs to be conducted on the 200 mm slab:



$$\bar{y} = \frac{1500 \cdot 200 \cdot 200 / 2 + 550 \cdot 550 \cdot (200 + 400 / 2)}{1500 \cdot 200 + 550 \cdot 550} = 284.21 \text{ mm}$$

$$I_{beam} = \frac{1}{12} * 1500 * (200)^3 + 1500 * 200 * (100 - 284.21)^2 + \frac{1}{12} * 550 * 400^3 + 500^2 * (450 - 284.21)^2$$

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

$$I_{slab} = \frac{1}{12} * 3000 * 200^3 = 1.75 * 10^9 \text{ mm}^4$$

$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = \frac{1.63 * 10^{10}}{1.75 * 10^9} = 9.36$$

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

$$h_{min} = \frac{l_n(0.8 + f_y / 200000)}{36 + 9\beta} = \frac{(3000 - 50/2)(0.8 + 420/1400)}{36 + 9 * 1} = 62.94 \text{ mm}$$

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

$$q_d = 4.828 \text{ kN/m}^2$$

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

$$q_u = 1.2 * 6.534 + 1.6 * 4.79 = 13.4576 \text{ kN/m}^2$$

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

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{13.55 * 6(6-0.85)^2}{8} = 278.19 \text{ kN} - m$$

$$M_n = 0.65 M_o = 180.83 \text{ kN} - m$$

$$M_p = 0.35 M_o = 97.37 \text{ kN} - m$$

$$\alpha_{f1} = \alpha_s = 9.36 \geq 1.0$$

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

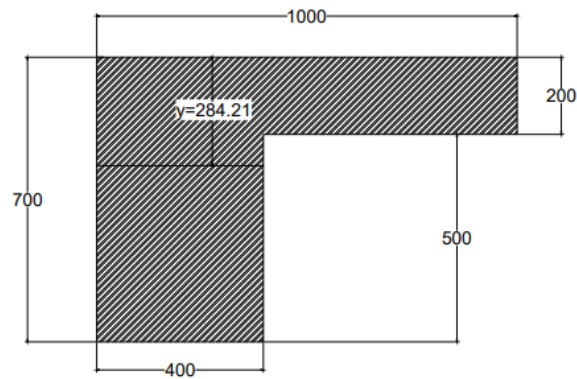
Table 3.37 demonstrates the design of the interior beam-supported panel:

Table 3..37. Summary of the interior beam-supported panel design.

M	Column Strip		Middle Strip	
	(-ve)	(+ve)	(-ve)	(+ve)
%	75.00	75.00	25.00	25.00
new Mu (kN-m)	135.62	73.03	45.21	24.34
Mu slab (kN-m)	20.34	10.95	6.78	3.65
b	2.06	1.50	1.50	1.50
Rn (kPa)	406.79	301.18	186.45	100.39
Ro	0.0010	0.0007	0.0004	0.00024
As (m^2)	0.00033	0.00018	0.00011	0.00006
As (in^2)	0.51130	0.27488	0.16987	0.09135
Asmin (in^2)	0.663	0.663	0.663	0.663

As final (in ²)	0.663	0.663	0.663	0.663
Bars Selected	7#3	7#3	7#3	7#3
Spacing (m)	2392.45	2392.45	2392.45	2392.45
2h (m)	25.4	25.4	25.4	25.4

The edge panel design is shown below:



$$\underline{y} = 245.83 \text{ mm}$$

$$I_{beam} = 1.94 * 10^{10} \text{ mm}^4$$

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

$$I_{slab} = \frac{1}{12} * 3000 * 200^3 = 2 * 10^9 \text{ mm}^4$$

$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = \frac{1.94 * 10^{10}}{2 * 10^9} = 9.69$$

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

$$h_{min} = \frac{l_n(0.8 + f_y/200000)}{36 + 9\beta} = \frac{(3000 - 850/2)(0.8 + 420/1400)}{36 + 9 * 1} = 62.94 \text{ mm}$$

The specified minimum requirement can be met with a material thickness of 200 mm. The torsional resistance of effective edge transverse beams needs evaluation through the β_t factor:

$$C = \Sigma(1 - \frac{0.63x}{y})(\frac{x^3 y}{3}) = 9.97 * 10^8 \text{ mm}^4$$

$$\beta_t = \frac{E_{cb} C}{2E_{cs} I_s} = 0.28$$

The exterior support requires the use of a coefficient derived from interpolation because $\beta_t < 2.5$. Table 3.37 presents the summary of exterior reinforcement area estimations.

Table 3.37. Summary of the exterior beam-supported panel design.

M	Column Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
%	99.73	75.00	75.00	0.27	25.00	25.00
new Mu (kN-m)	180.34	73.03	135.62	0.49	24.34	45.21
Mu slab (kN-m)	27.05	10.95	20.34	0.07	3.65	6.78
b	1.19	1.50	1.50	1.50	1.50	1.50
Rn (kPa)	939.50	301.18	559.34	2.01	100.39	186.45
Ro	0.0023	0.0007	0.0013	0.0000	0.0002	0.0004
As (m ²)	0.00044	0.00018	0.00033	0.00000	0.00006	0.00011
As (in ²)	0.68540	0.27488	0.51247	0.00183	0.09135	0.16987
Asmin (in ²)	0.663	0.663	0.663	0.663	0.663	0.663
As final (in ²)	0.685	0.663	0.663	0.663	0.663	0.663
Bar Selected	7#3	7#3	7#3	7#3	7#3	7#3
Spacing (m)	1337.564	1337.564	1337.564	1337.564	1337.564	1337.564

2h (m)	25.4	25.4	25.4	25.4	25.4	25.4
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3.5.2.4.2 Shear analysis

Shear should be checked at this point:

$$d_{shear} = 200 - 20 - 13.875 = 166.125 \text{ mm}$$

$$d_{long} = 200 - 20 - 13.875/2 = 173.0625 \text{ mm}$$

$$d_{short} = 200 - 0.75 - 13.875/2 - 13.875 = 178.44 \text{ mm}$$

$$b_0 = 4 * (750 + 166.125) = 3664.5 \text{ mm}$$

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

$$f_y = 420 \text{ MPa}$$

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

$$V_u = [3000 * 3000 - (750 + 166.125)^2] * 13.5/10^6 = 110.17 \text{ kN}$$

$$\lambda_s = \sqrt{\frac{2}{(1+0.004d_{shear})}} = \sqrt{\frac{2}{(1+166.125*0.004)}} = 1.089 \leq 1, \text{ thus use } \lambda_s = 1.0$$

$$\phi V_{c1} = \phi(0.17\lambda_s\sqrt{f'_c} + \frac{N_u}{6A_g})b_w d = 213.75 \text{ kN}$$

$$\phi V_{c2} = \phi(0.66\lambda_s\rho^{(1/3)}\sqrt{f'_c} + \frac{N_u}{6A_g})b_w d = 147.83 \text{ kN} \rightarrow \text{controls}$$

$$\phi V_{c2} = 147.83 \text{ kN} > V_u = 110.17 \text{ kN} \rightarrow \text{Shear criteria is satisfied}$$

3.5.2.5 Joint design

The authors followed the recommendations in Chapter 15 of ACI 318-19 to design the joints. A complete set of calculation steps appeared in the design book written by Wight and MacGregor (Wight & MacGregor, 2009). Figure 3.45 presents the free body diagram of the interior beam-column joint.

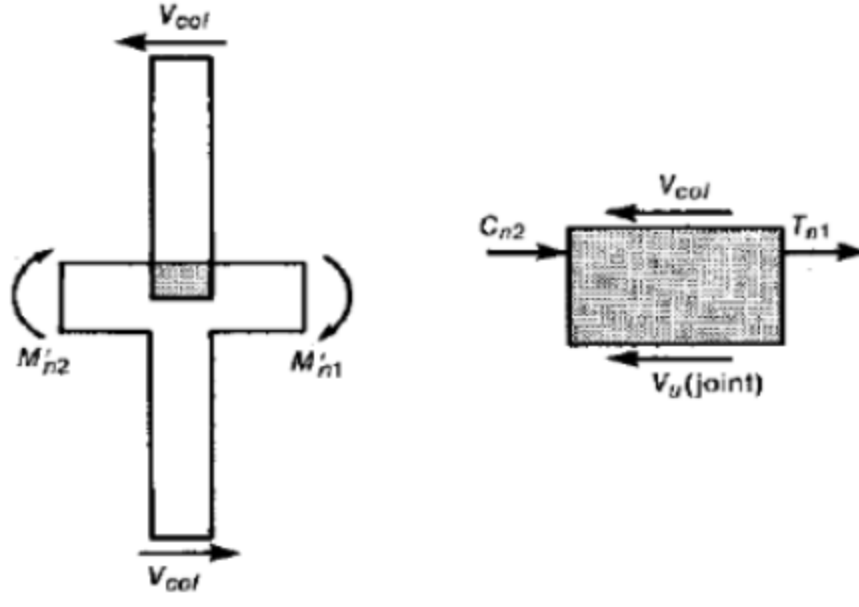


Figure 3.45. Free body diagram of the beam-column joint.

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

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

Figure 3.46 shows the reinforcement details of the interior beam-column joint which were used to determine the joint shear values through SAP2000.

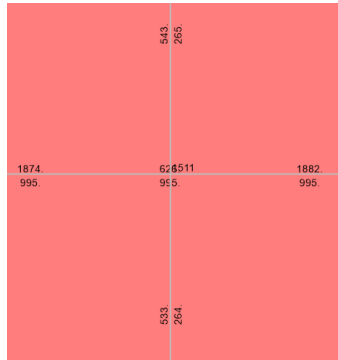


Figure 3.46. Reinforcement details at the beam-column joint.

The values in the above equation were calculated below:

$$T_{pr1} = \alpha A_s f_y = 1.25 \cdot 1882 \cdot 0.413 = 971.58 \text{ kN}$$

$$C_{pr1} = \alpha A_s f_y = 1.25 \cdot 995 \cdot 0.413 = 513.67 \text{ kN}$$

From SAP2000, the column joint:

$$V_{col} = 351.96 \text{ kN}$$

Accordingly, the shear demand on joint:

$$V_{u,joint} = 971.58 + 513.67 - 351.96 = 1133.29 \text{ kN}$$

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

$$b_j = \frac{b_b + b_{col}}{2} = \frac{400 + 750}{2} = 575 \text{ mm} < b_b + h_{col} = 400 + 750 = 1150 \text{ mm}$$

$$b_j = 575 \text{ mm} < b_b + 2x = 400 + 750 = 1150 \text{ mm}$$

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

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

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

$$V_n = \gamma \sqrt{f'_c} A_j = 1.7 \sqrt{40} \cdot 575 \cdot 750 = 4636.7 \text{ kN}$$

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

$$0.85 \cdot 4636.7 = 3941.2 \text{ kN} \geq V_u = 1133.29 \text{ kN}$$

Shear forces cannot damage the joint because of this result. A similar procedure enables checking the shear capacity of the joint across different directions which confirms its safety under shear conditions.

The dimensions were regulated per ACI 15.4.2 through the following calculation formula:

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

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

Accordingly, the minimum spacing with 3#3 ties:

$$s = \min\left(\frac{2 \cdot 129 \cdot 413.7}{0.062 \sqrt{40} \cdot 750}, \frac{2 \cdot 129 \cdot 413.7}{0.35 \cdot 750}\right) = \min(299.62, 335.7) = 299.62 \text{ mm}$$

The necessary requirements of the code will be achieved by employing 3#3 ties with 300 mm spacing.

The following evaluation needs to be executed to verify plastic hinges develop in beams instead of columns:

$$\Sigma M_{nc} \geq \frac{6}{5} \Sigma M_{nb} \quad (3.14)$$

$$571.47 + 497.81 = 1069.28 \text{ kN} - m \geq \frac{6}{5} (178.53 + 107.6) = 286.13 \text{ kN} - m$$

3.5.2.6 Reinforcement detailing

3.5.2.6.1 Bar selection and spacing

The final selected bars and spacing for major and minor beams, columns and two-way slab can be seen in Table 3.38.

Table 3.38. Final selected bars.

Component	Top reinforcement	Bottom reinforcement	Stirrups
Major beam	3#10 bars at beam edges (0.283L = 1.7 m from beam end)	4#6 continuously	#4 at 300 mm
Minor beam	2#9 bars at beam edges (0.283L = 1.7 m from beam end)	2#5 continuously	#3 at 150 mm
Two-way slab	7#3	7#3	
		Reinforcement	Ties
Column 0 floors (750 x 750 mm)		8#11	#3 at 300 mm
Column 1 floors (700 x 700 mm)		8#11	#3 at 300 mm
Column 2-3 floors (650 x 650 mm)		8#10	#3 at 300 mm
Column 4 floors (600 x 600 mm)		8#9	#3 at 300 mm
Column 5-6 (550 x 550 mm)		8#7	#3 at 300 mm
Column 7 (500 x 500 mm)		8#6	#3 at 300 mm
Column 8 (450 x 450 mm)		8#5	#3 at 260 mm
Column 9-10 (400 x 400 mm)		8#4	#3 at 200 mm
Column 11-12 (350 x 350 mm)		8#4	#3 at 200 mm

3.5.2.6.2 Development length

All bar development lengths obtained their measurements per the criteria in ACI 318-19. The #6 and #10 bars have bar diameters of $d_b = 0.75$ in and $d_b = 1.27$ in. The equation below served for calculating tension bar development lengths:

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

Since the following conditions were met: a) clear cover = $2.5 - 1.27/2 > d_b$; b) clear spacing of bars = $(400/25.4 - 2 \cdot 2.5)/3 > d_b$; c) minimum #3 stirrups were provided. The development length for #11 bars in tension was found to be:

$$l_d = 44.4 \text{ in} > 400/25.4 = 15.7 \text{ in (least column size)} \rightarrow \text{need to be hooked bars}$$

For hooked bars, the following equation can be used:

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

Hence, the $l_{dh} = 20.22 \text{ in}$ or 513.78 mm for the bars in tension. Subsequently, the 90° hook's parameters were found:

$$D = 6d_b = 7.62 \text{ in}, r = D/2 = 3.81 \text{ in or } 96.77 \text{ mm}$$

$$\text{The distance of the hook: } 12d_b = 15.24 \text{ in or } 387 \text{ mm}$$

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

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

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

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

$$l_{dc} = 11.81 \text{ in or } 342.9 \text{ mm} < 400 \text{ mm}$$

3.5.2.6.3 Lap splices

The lap splices were also calculated for the beams and columns. For the beams in tension:

$$l_{st} = l_{dh} = 340 \text{ mm}$$

For beams in compression:

$$l_{sc} = 0.0005 f_y d_b = 967.8 \text{ mm}$$

For columns ($d_b = 1.41 \text{ in}$):

$$l_{sc} = 0.0005 f_y d_b = 571.5 \text{ mm}$$

Table 3.29 illustrates the lap splices for other column types.

Table 3.39. Lap splices for columns.

Component	Lap splice, mm
Column 0 floors (750 x 750 mm)	1074
Column 1 floors (700 x 700 mm)	1074
Column 2-3 floors (650 x 650 mm)	967
Column 4 floors (600 x 600 mm)	860
Column 5-6 (550 x 550 mm)	667
Column 7 (500 x 500 mm)	572
Column 8 (450 x 450 mm)	477
Column 9-10 (400 x 400 mm)	381
Column 11-12 (350 x 350 mm)	381

3.5.2.7. Special seismic provisions for reinforcement detailing

3.5.2.7.1 Beams

Since the building is located in a high seismicity region, seismic detailing should be taken into consideration. Clear span, and the width-to-depth ratio satisfy the requirements of Section 18.6 in ACI 318-19 code. At least two longitudinal bars were provided continuously at both top and bottom sections of the beam (ACI, Section 18.6.3.1). The positive moment strength at the face of the joint is greater than half of the negative moment. Hoops were provided over the length of twice the member depth from the face of the support that does not exceed $d/4$, $6d_b$ and 150 mm. First hoop was provided at a distance of 50 mm from the face of support.

3.5.2.7.2 Columns

Per Section 18.7 the seismic provisions were applied to columns. The area of reinforcement used for column construction falls within the range of 1% to 6%. The joint face received transverse reinforcement placed with 10 spacing on its both sides. The designers selected l_0 to be equal to one sixth of the column clear span measuring 400 mm. According to ASCE 7-10 and regulations of state minimum column size should be 305 mm, in project building least column size is 350x350 mm

3.5.2.7.3 Joints

The beams received their longitudinal reinforcement extensions which stretched to the outer side of the column joints. All joints with connecting longitudinal reinforcement through the column reached a column depth of h that exceeded $20d_b$.

Development lengths of bars satisfy the following criteria:

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

3.5.2.8 Structural serviceability design

3.5.2.8.1 Deflection

Building serviceability analysis requires members to undergo deflection testing as one of the evaluation criteria. The building codes establish tables which contain structural member thickness requirements. A deflection check becomes irrelevant when member dimensions surpass the thickness requirements (Hassoun & Manaseer, 2020). A beam member with $f_y = 60000$ psi (413.7 MPa) requires evaluation according to the following thickness calculation formula:

$$h_b = \frac{L}{21} \quad (3.19)$$

Where L is the span length. For our building, the minimum thickness of the beam is:

$$h_b = \frac{6000}{21} = 286 \text{ mm} < 400 \text{ mm (thickness of major beam)}$$

The omission of deflection calculations becomes valid due to the dimensions of selected members surpassing the minimum requirement of beam thickness.

3.5.2.8.2 Crack width

Dangerous conditions can emerge when loads act upon structures during reinforced concrete construction unless proper measures are taken to control cracks. Checking crack width along with determining if permitted values as stated in building codes for crack width remains

essential. The researchers will employ this equation to determine the maximum width of flexural cracks (McCormac & Brown, 2016):

$$w = 0.076\beta_h f_s \sqrt[3]{d_c A} \quad (3.20)$$

The equation implies w is the estimated crack width (in) while β_h represents the ratio of the distance to the neutral axis from the extreme tension concrete face to the distance from the center of the reinforcement (1.2) and f_s stands for steel stress and d_c refers to cover along with A describing effective tension area.

$$\begin{aligned} d_c &= 2.5 \text{ in} \\ A &= \frac{(5 \text{ in})(19.7 \text{ in})}{6} = 19.68 \text{ in}^2 \\ w &= 0.076 \cdot 1.2 \cdot (2/3) \cdot 60000 \cdot \sqrt[3]{2.5 \cdot 19.68} = 0.012 \text{ in} \end{aligned}$$

The designed building has an allowable crack width of 0.016 inches because predicted crack dimensions stay beneath the established threshold. Based on these criteria the detected crack width passes the inspection test.

The maximum allowable spacing of longitudinal reinforcement can be determined through this calculation approach:

$$s(\text{in}) = [15(\frac{40}{f_s}) - 2.5C_c] \quad (3.21)$$

$$s = 15(\frac{40}{(2/3)60} - 2.5 \cdot 2.5) = 8.75 \text{ in or } 222.25 \text{ mm}$$

So, the spacing between longitudinal reinforcement shall be limited to 222.25 mm.

4. Geotechnical Design Part

4.1 Site analysis

4.1.1 Soil Profile

The site that was chosen is located in Cupertino, state of California, USA. According to the site investigations done by Langan Engineering and Environmental Service, 2 borings were drilled in that area B-1 and B-3 (Figure 4.1), with approximate depth of 43 m and 15.24 m respectively. Also, since the site is located in a high seismic region, the report provided a shear wave velocity graph which is used to evaluate the soil dynamic properties. So, from the Figure 1.1 shear wave velocity $V_{s(30)}$ was 510 m/s . This is classified as Site Class C, which indicates that soil layers have a very dense profile (Langan Engineering and Environmental Services, Inc., 2023). Seismic Design Category is D and therefore Risk Category is 2. This value was taken during down hole suspension logging and for the upper 30m of the soil.

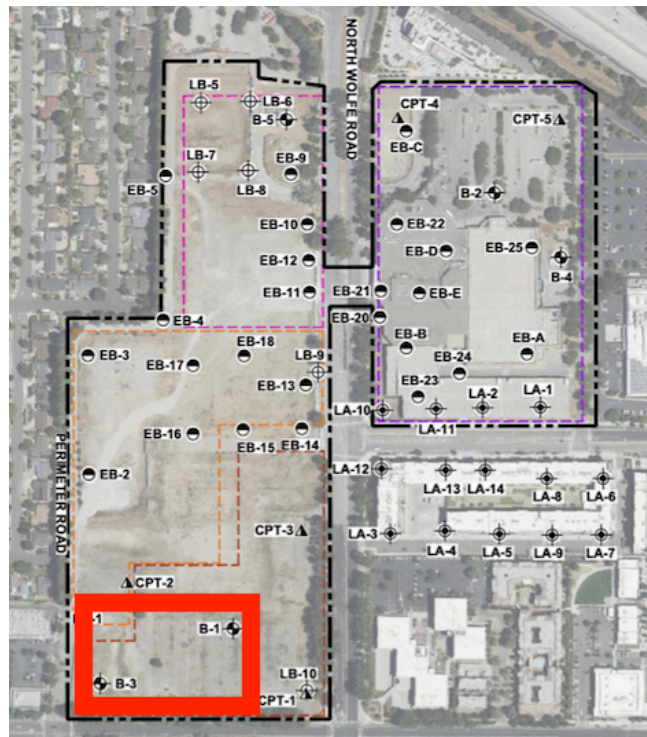


Figure 4.1 Project phasing on the site plan

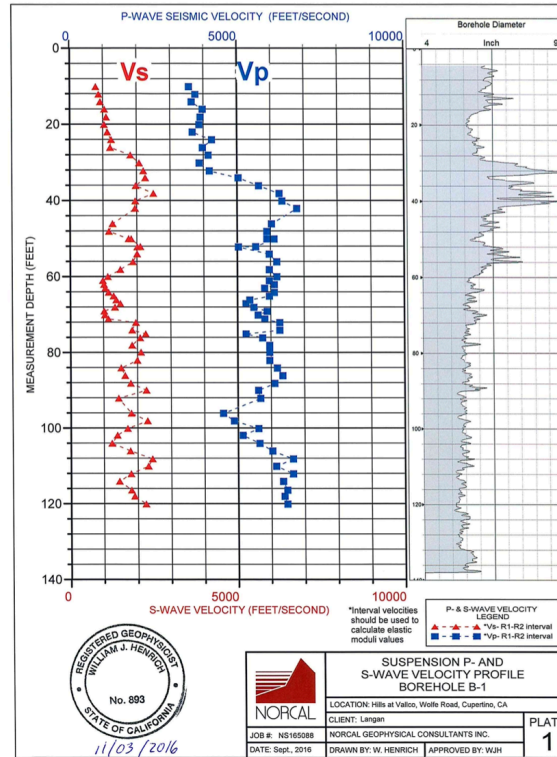


Figure 4.2 Shear-wave velocity

To collect information for the soil profile B-1 was chosen, since it is more deep and there is more data. When conducting field exploration for B-1, it was noticed that the groundwater table is located at 14.6 m below the surface. Cohesion and friction angle parameters were provided in the previous investigation report. But, corresponding data was only for upper 17.5 m, so below that depth no information was accounted. Besides, other parameters like N_{60} , $(N_1)_{60}$, unit weight, and effective stress were calculated.

First, SPT-N values from SPT test were corrected to calculate the N_{60} according to the below formula ((Skempton, 1986)):

$$N_{60} = \frac{E_H C_B C_S C_R N}{0.60} \quad (4.1)$$

Where,

N_{60} = Corrected SPT (Standard Penetration Test) N-value for field procedures.

E_H = Hammer efficiency correction factor.

C_B = Borehole diameter correction factor.

C_S = Sampler correction factor.

C_R = Rod length correction factor.

N = Measured SPT N-value in the field.

According to the Skempton (1986), E_H is equal to 0.7, if the hammer is automatic. Also, other 3 values C_B, C_S, C_R are equal to the 1. Therefore, the final formula is

$$N_{60} = \frac{0.7*1*1*1*N}{0.60} = \frac{0.7}{0.6} \times N$$

For the unit weight values the following formula was used

$$\gamma = \gamma_{dry} \times (1 + w) \quad (4.2)$$

Where,

γ = unit weight of soil, (kN/m³)

γ_{dry} = dry density, (kN/m³).

w= water content, %.

Water content and dry density values were provided by a report for all soil layers. Also, we have a below ground water table 14.6 m, we have taken into account the unit weight of the water.

So the results are shown in the table:

Table 4.1. Unit weight values

Water content	Dry density, kN/m ³	Unit weight, kN/m ³
0.19	17.53	20.9
0.16	17.58	20.3
0.12	19.93	22.3
0.12	19.93	22.3
0.10	19.93	22.0
0.21	19.93	24.0
0.21	19.93	24.0
0.11	17.58	19.5
0.11	17.58	19.5
0.15	17.58	20.2

0.13	17.42	19.7
0.10	17.42	19.1

Whereas effective stress was derived using the following equation:

$$\sigma'_0 = \gamma H \quad (4.3)$$

In granular soils, the N_{60} value can be influenced by effective overburden pressure. As a result, the N_{60} value must be adjusted to a standard atmospheric pressure, yielding the corrected $N_{1(60)}$ value. It was corrected by following equation:

$$N_{1(60)} = C_N N_{60} \quad (4.4)$$

Where, C_N is a correction factor that can be calculated by different methods. The correction factor was derived by following three relationships and taken as an average of these methods. According to Pecketal.(1974):

$$C_N = 0.77 \log \left[\frac{20}{\left(\frac{\sigma'_0}{p_a} \right)} \right] \text{ for } \frac{\sigma'_0}{p_a} \geq 0.25 \quad (4.5)$$

According to Liao and Whitman (1986):

$$C_N = \left[\frac{1}{\left(\frac{\sigma'_0}{p_a} \right)} \right]^{0.5} \quad (4.6)$$

According to Seedetal. (1975):

$$C_N = 1 - 1.25 \log \left(\frac{\sigma'_0}{p_a} \right) \quad (4.7)$$

Table 4.2. Soil profile

Layer #	Height, m	Depth, m	Soil Type	USCS	N	N60	Cn	(N1)60	Unit wt, kN/m3	Cohesion, kPa	Friction angle, °	Effective stress, kN/m2
1	7.6	0-7.6	CLAY	CH	22	26	0.8	20.6	20.9	13.72	41	158.9
2	0.8	7.6-8.4	SILTY CLAY	CH	22	26	0.8	19.6	20.3	24.13	40	174.4

3	1.5	8.4-9.9	SILTY CLAY	CH	25	29	0.7	20.1	22.3	0	43	208.4
4	1.1	9.9-11	SANDY CLAY	CL	55	64	0.6	41.4	22.3	0	41	232.3
5	6.6	11-17.5	CLAY	CH	90	105	0.5	51.0	22.0	0	38	348.4
6	1.7	17.5-19 .2	SANDY CLAY	CL	61	71	0.5	32.8	24.0	34.47	40	372.2
7	4.0	19.2-23 .2	CLAYEY GRAVEL	GC	18	21	0.4	8.6	24.0	25.4	42	428.6
8	2.0	23.2-25 .1	CLAYEY SAND	SC	55	64	0.4	25.1	19.5	0	0	447.9
9	1.8	25.1-27	SANDY CLAY	CL	26	30	0.4	11.4	19.5	0	40	465.7
10	7.9	27-34.9	CLAYEY SAND	SC	38	44	0.3	14.0	20.2	0	0	548.4
11	1.5	34.9-36 .4	SANDY CLAY	CL	30	35	0.3	10.7	19.7	0	40	563.4
12	6.6	36.4-43	CLAYEY SAND	SC	55	64	0.3	17.3	19.1	0	0	624.6

Table 4.3. (N1)60 calculation

N60	Effective stress, kN/m ²	Atmospheric pressure, kN/m ²	Eff stress/Atm press	Cn1	Cn2	Cn3	Cn average	(N1)60
26	158.9	101.33	1.57	0.798	0.756	0.851	0.80	20.6
26	174.4	101.33	1.72	0.762	0.705	0.820	0.76	19.6
29	208.4	101.33	2.06	0.697	0.608	0.761	0.69	20.1

64	232.3	101.33	2.29	0.661	0.550	0.724	0.64	41.4
105	348.4	101.33	3.44	0.539	0.330	0.589	0.49	51.0
71	372.2	101.33	3.67	0.522	0.294	0.567	0.46	32.8
21	428.6	101.33	4.23	0.486	0.217	0.520	0.41	8.6
64	447.9	101.33	4.42	0.476	0.193	0.505	0.39	25.1
30	465.7	101.33	4.60	0.466	0.172	0.492	0.38	11.4
44	548.4	101.33	5.41	0.430	0.083	0.437	0.32	14.0
35	563.4	101.33	5.56	0.424	0.069	0.428	0.31	10.7
64	624.6	101.33	6.16	0.403	0.013	0.394	0.27	17.3

Liquefaction hazards

According to the California Geological Survey (CGS) in a map titled, State of California Seismic Hazard Zones, Cupertino Quadrangle our site is considered as non-hazardous in terms of its geology. When conducting a CPT test there was no saturated loose sand was not encountered. That is why authors conclude that there is no potential liquefaction risk occurring at the site. (Langan Engineering and Environmental Services, Inc., 2023).

Next, based on the soil corrosivity tests (Langan, 2023), the concentration of sulfate ions in the soil were determined as sufficient for a potential corrosive effect on the cement mortar-coated steel and reinforced concrete structures, therefore a cement of Type-II and a water-cement ratio not exceeding 0.55 is recommended for use.

Therefore, in the scope of this project is a building with a one-story basement in an area of seismic zone.

4.1.2 Site response analysis

This soil profile was input into PLAXIS2D software (Figure 4.3) for site-response analysis.

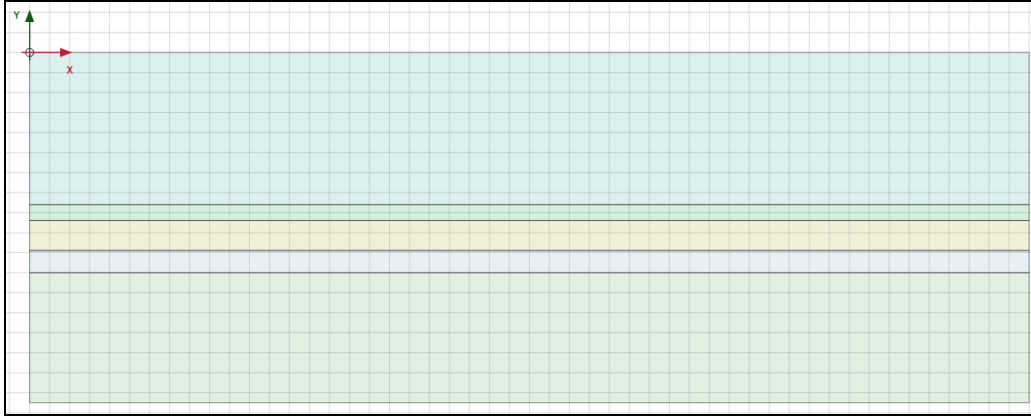


Figure 4.3. Soil profile in PLAXIS2D.

However, an appropriate site response analysis was not possible due to licensing issues of the software. Therefore, from the investigation report, the site-specific response is completed in accordance with the standards: 2016 California Building Code and the ASCE 7-10 code. Below is the s-wave velocity profile, conducted by Langan (2023) in Figure 4.4.

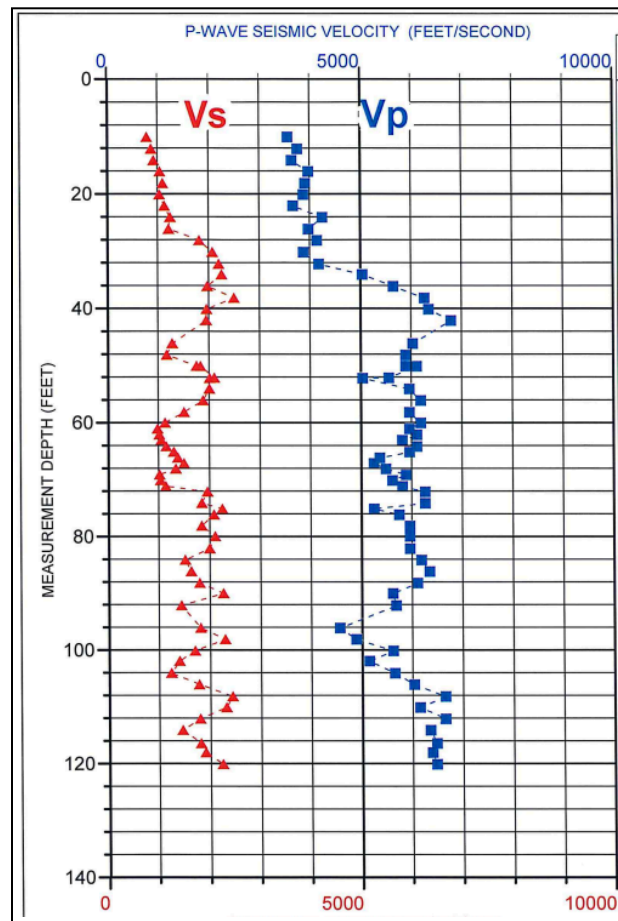


Figure 4.4. P- and S-wave velocity profile chart for borehole B-1.

Based on the Langan geotechnical investigation (2023), the following chart was obtained as an accelerogram data (figure 4.5). However this accelerogram data was not suitable for site response analysis. Therefore, an online available 2006 earthquake in Cupertino .smc file was input into PLAXIS 2D.

Period (seconds)	MCE_R	DE
0.01	0.806	0.537
0.10	1.608	1.072
0.20	1.997	1.331
0.30	1.912	1.274
0.40	1.717	1.145
0.50	1.568	1.046
0.60	1.412	0.942
0.75	1.230	0.820
1.00	1.012	0.674
1.50	0.736	0.490
2.00	0.578	0.385
3.00	0.411	0.274
4.00	0.319	0.213
5.00	0.258	0.172
6.00	0.205	0.136
7.00	0.171	0.114
8.00	0.143	0.095

Figure 4.5. Accelerogram data (Langan, 2023).

This resulted in the following displacement multiplier chart below (Figure 4.6). The resulting chart reflects that the accelerogram data obtained was recorded from the ground surface during geotechnical site investigation, therefore, the accelerogram data will be used directly in PLAXIS 3D for pile analysis without the need for site response analysis.

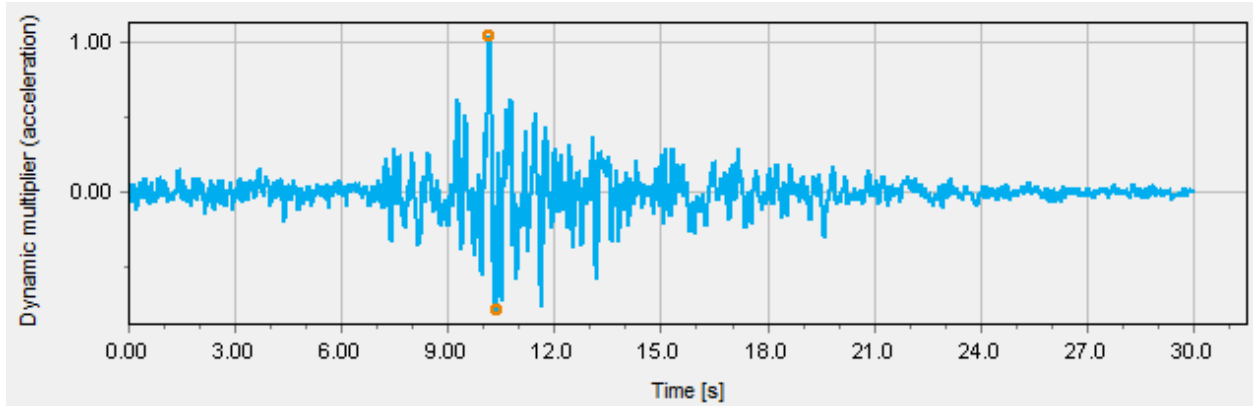


Figure 4.6. Acceleration chart in PLAXIS 2D.

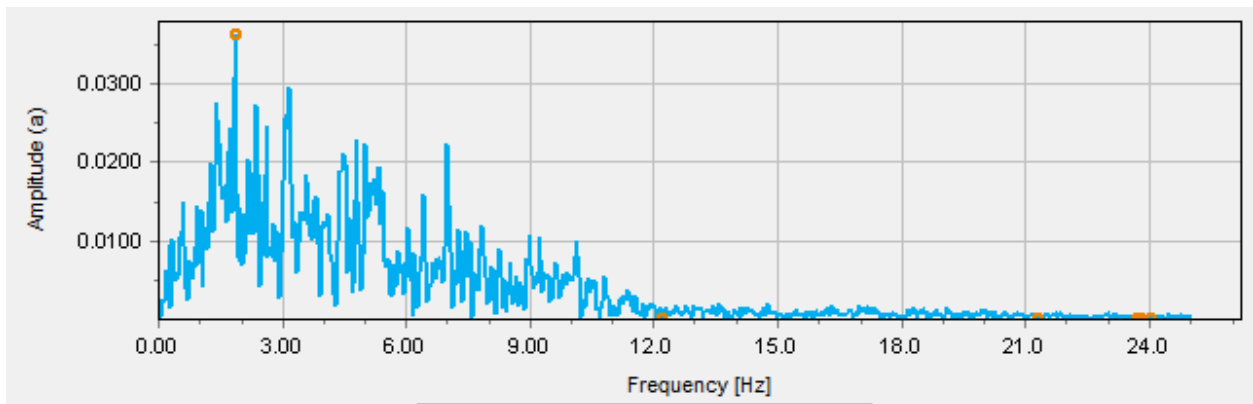


Figure 4.7. Fourier spectra chart.

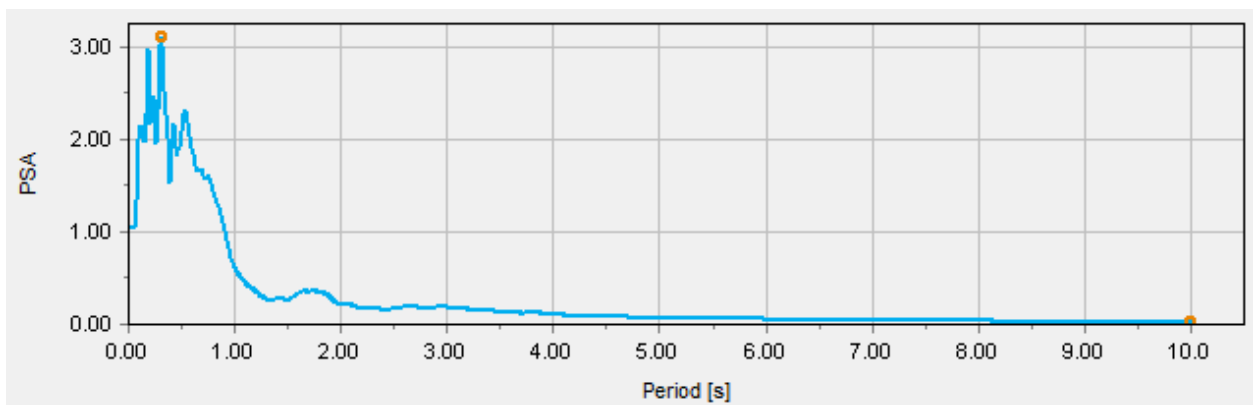


Figure 4.8. Response spectra.

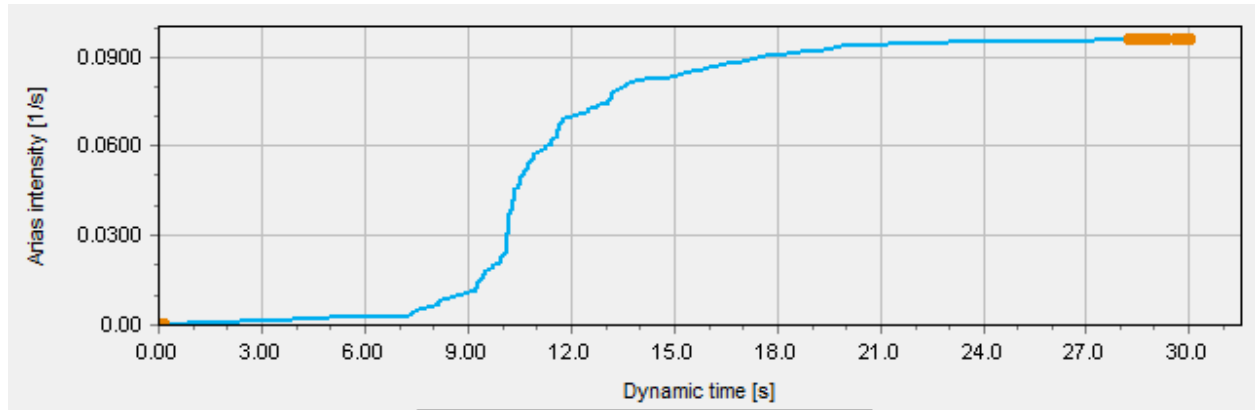


Figure 4.9. Arias intensity chart.

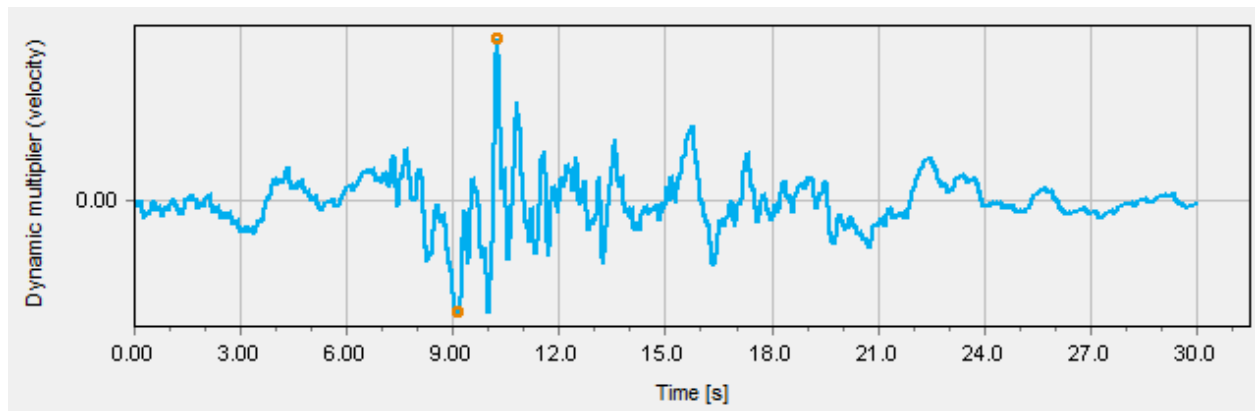


Figure 4.10. Velocity chart.

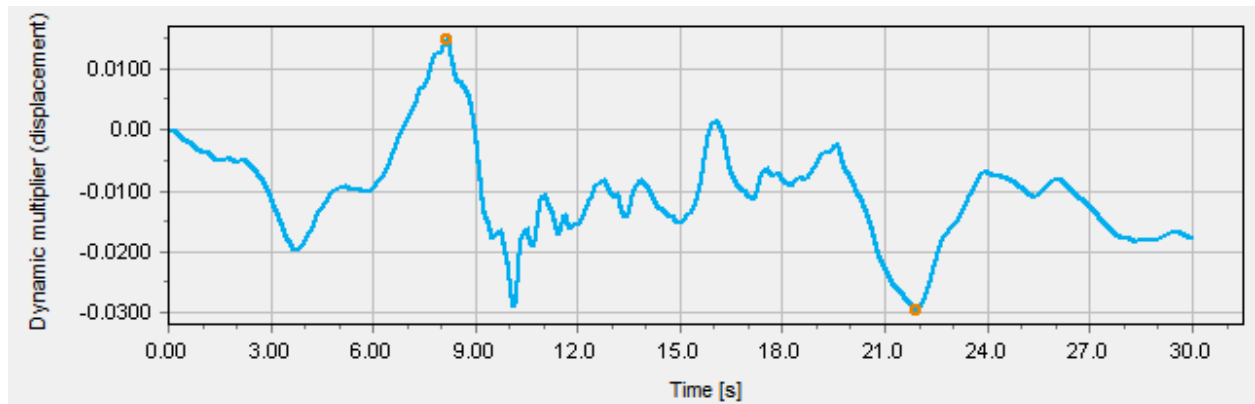


Figure 4.11. Displacement chart.

After obtaining the .txt file from PLAXIS 2D as a result of the inputting the .smc file, the resulting .txt file is an amplification or amplifier.

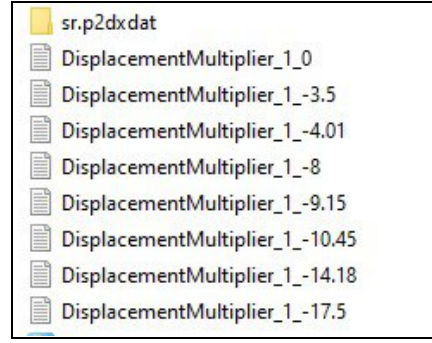


Figure 4.12. The displacement multiplier files as a result of PLAXIS 2D site response analysis. After foundation design the .txt files were input into PLAXIS 3D to show the displacement in the foundation.

4.2 Foundation design

4.2.1 Shallow Foundation

4.2.1.1 Pad Foundation

The equation for factored load is (ASCE-7, n.d.):

$$1.2D + 1.6L + 0.5L_r \quad (4.8)$$

The ultimate bearing capacity for design of shallow foundations was calculated. The factor of safety usually is taken as 3 for shallow foundations (Das & Sivakugan, 2019). The equation for general bearing capacity was calculated as:

$$q_{all} = \frac{q_u}{FS} \quad (4.9)$$

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (4.10)$$

Where,

$c' = \text{cohesion, } kN/m^2$

$q = \text{effective stress at the bottom of the foundation, } kN/m^2$

$\gamma = \text{unit weight of soil, } kN/m^3$

$B = \text{foundation width, } m$

$N_c, N_q, N_\gamma = \text{bearing capacity factors}$

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

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

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

The depth embedded to the ground was assumed and takes as $D_f = 2m$. The shape of the foundation was assumed as square and consequently, $B=L$ (m).

Bearing capacity factors' equations according to $\phi = 38^\circ$:

$$N_q = \tan^2(45 + \frac{\phi'}{2})e^{\pi \tan \phi'} \quad (4.11)$$

$$N_c = (N_q - 1)\cot \phi' \quad (4.12)$$

$$N_\gamma = 2(N_q + 1)\tan \phi' \quad (4.13)$$

Shape factor equations:

$$F_{cs} = 1 + \frac{B}{L} \left(\frac{N_q}{N_c} \right) \quad (4.14)$$

$$F_{qs} = 1 + \frac{B}{L} * \tan \phi' \quad (4.15)$$

$$F_{\gamma s} = 1 - 0.4 * \frac{B}{L} \quad (4.16)$$

Depth factor equations:

As $\frac{D_f}{B} > 1$ and $\phi' > 0$ (as we assumed), following equations for depth factors was used:

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'} \quad (4.17)$$

$$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \tan^{-1} \left(\frac{D_f}{B} \right) \quad (4.18)$$

$$F_{\gamma d} = 1 \quad (4.19)$$

Inclination factor equations:

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta}{90}\right)^2 \quad (4.20)$$

$$F_{\gamma i} = \left(1 - \frac{\beta}{\phi}\right)^2 \quad (4.21)$$

From the following equation vertical stress as obtained:

$$q = \gamma * D_f \quad (4.22)$$

As a result we get the following values for bearing capacity factors:

$$N_q = \tan^2\left(45 + \frac{40^\circ}{2}\right)e^{\pi \tan 40^\circ} = 95.57$$

$$N_c = (30.1 - 1)\cot 40^\circ = 34.68$$

$$N_\gamma = 2(30.1 + 1)\tan 40^\circ = 52.19$$

Same as for bearing capacity factors, we obtained values for shape factors:

$$F_{cs} = 1 + \left(\frac{95.57}{34.68}\right) = 3.75$$

$$F_{qs} = 1 + \tan 40^\circ = 1.84$$

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

For depth factors, using equations that were written above, we calculated following values:

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi} = 1 + 0.2 * \tan^{-1}\left(\frac{2}{B}\right) - \frac{1 - 1 + 0.2 * \tan^{-1}\left(\frac{2}{B}\right)}{34.68 * \tan 40^\circ} = 1 + 0.19 \tan^{-1}\left(\frac{2}{B}\right)$$

$$F_{qd} = 1 + 2 \tan 40^\circ (1 - \sin 40^\circ) \tan^{-1}\left(\frac{2}{B}\right) = 1 + 0.2 * \tan^{-1}\left(\frac{2}{B}\right)$$

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

When calculating inclination factors, we assumed that $\beta = 0$, hence all inclination factors ($F_{ci}, F_{qi}, F_{\gamma i}$) are equal to 1.

As in our site we mainly consists of clayey sand and sandy clay, we took values for unit weight as $\gamma_{SC} = 22.3 \text{ kN/m}^3$ and $\gamma_{CL} = 24.0 \text{ kN/m}^3$. Next, as per formula we get the value for vertical effective stress as:

$$q = 1.5 * 22.3 + 1.7 * 24 = 74.25 \text{ kN/m}^2$$

The cohesion $c' = 10 \text{ kPa}$. Ultimate bearing capacity is now can be calculated:

$$q_u = 10 * 34.68 * 3.75 * (1 + 0.19 \tan^{-1}(\frac{2}{B})) * 1 + 74.25 * 95.57 * 1.84 * (1 + 0.2 * \tan^{-1}(\frac{2}{B})) * 1.7 * 24 = 74.25 \text{ kN/m}^2$$

$$q_u = 10 * 34.68 * 3.75 * (1 + 0.19 \tan^{-1}(\frac{2}{B})) * 1 + 74.25 * 95.57 * 1.84 * (1 + 0.2 * \tan^{-1}(\frac{2}{B})) * 1 + \frac{1}{2} * 24 * B * 52.19 * 0.6 * 1 * 1$$

$$q_u = 1300.5 + 247.1 \tan^{-1}(\frac{2}{B}) + 13056.7 + 2611.3 \tan^{-1}(\frac{2}{B}) + 375.7B$$

Factored load for the interior column is assumed to be 12000kN. So the following equation for the load per unit area was evaluated for interior column:

$$q_{load} = \frac{Q}{BL} = \frac{Q}{B^2} \text{ (Assume: } B = L, \text{ square foundation)} \quad (4.23)$$

$$q_{all} = \frac{q_u}{FS} \quad (4.24)$$

Therefore to find the dimensions for interior column, we equate formulas 1.16 and 1.17:

$$\frac{1300.5 + 247.1 \tan^{-1}(\frac{2}{B}) + 13056.7 + 2611.3 \tan^{-1}(\frac{2}{B}) + 375.7B}{3} = \frac{12000}{B^2}$$

Therefore, dimensions of pad foundation are calculated to be $B=L=1.46 \text{ m}$, Calculated Bearing capacity: $Q_{des}=36004.04 \text{ kN}$, so $Q_{all}=12001.35 \text{ kN}$. This value is less than $Q_{structural}=12777.83 \text{ kN}$, so the pad foundation was not considered for further foundation design.

4.2.1.2 Mat Foundation

Second type of shallow foundation is mat foundation. According to Das & Sivakugan (2019), footing for mat foundations covers larger areas and load support comes from lines of columns, which is different to pad foundation, where load is supported from only one column. Embedded depth of foundation D_f was assumed to be 1m and to cover all the area of the building which is 1296 sqm, rectangular footing was chosen. What comes to the dimensions, the length was 36 m and the width of the footing was 36 m. For calculation of the bearing capacity the largest load was chosen. Interior column load was the largest and estimated to be 12777.83 kN. This was done to distribute load evenly along all columns. Number of columns along the footing is 49, so tributary area for each column was obtained-483.11 kN/sqm.

Bearing capacity calculations is the next step in design of mat foundation. For the mat foundation general bearing capacity calculations were used, as for pad foundation. Footing and columns of the foundation will be located in the clay layer, because embedded depth is 1m.

According to soil profile of our site following values were taken:

$$\gamma_{CH} = 20.9 \text{ kN/m}^3$$

$$\phi' = 40^\circ$$

$$c' = 90 \text{ kPa}$$

First, bearing capacity factors were calculated:

$$N_q = \tan^2(45 + \frac{41^\circ}{2}) e^{\pi \tan 41^\circ} = 73.89$$

$$N_c = (73.89 - 1) \cot 41^\circ = 63.37$$

$$N_\gamma = 2(73.89 + 1) \tan 41^\circ = 130.21$$

Shape factors were obtained:

$$F_{cs} = 1 + (\frac{73.89}{63.37}) = 2.17$$

$$F_{qs} = 1 + \tan 41^\circ = 1.86$$

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

Assumption on ratio of embedded depth and width was made that it is less or equal to 1. Also, given that friction factor is more than 0:

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi} = 1 + 0.2 * \tan^{-1}\left(\frac{1}{60}\right) - \frac{1 - 1 + 0.2 * \tan^{-1}\left(\frac{1}{60}\right)}{63.37 * \tan 41^\circ} = 1.004$$

$$F_{qd} = 1 + 2 \tan 41^\circ (1 - \sin 41^\circ)^2 \tan^{-1}\left(\frac{1}{60}\right) = 1.003$$

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

As inclination angle $\beta = 0$, inclination factors F_{ci} , F_{qi} , $F_{\gamma i}$ all equal to 1.

Vertical effective stress were calculated as follows:

$$q = 1.022 * 20.9 = 21.3598 \text{ kN/m}^2$$

So, the gross ultimate bearing capacity is obtained from the following formula:

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

$$q_u = 90 * 63.36 * 2.166 * 1.003 * 1 + 21.3598 * 73.89 * 1.87 * 1.003 * 1 + \frac{1}{2} * 20.9 * 60 * 130.2 * 0.6 * 1 * 1$$

$$q_u = 65421.73 \text{ kN/m}^2$$

To get net ultimate capacity:

$$q_{u(net)} = q_u - q = 65421.73 - 21.3598 = 65400.37 \text{ kN/m}^2$$

It was decided to take the factor of safety as 3, for pad and mat foundations (Das & Sivakugan, 2019). From this known allowing bearing capacity can be calculated:

$$q_{all(net)} = \frac{q_{u(net)}}{FoS} = \frac{65400.37}{3} = 21800 \text{ kN/m}^2$$

According to Eurocode 7, the maximum allowable settlement for clay layers is 75mm (Choudhuri & Chakraborty, 2022). Therefore, in order to compare the calculate value of the allowable net bearing capacity, the allowable bearing capacity from the maximum settlement will be calculated:

$$q_{net} = \frac{N_{60}}{0.08} \times F_d \times \frac{S_e}{25} \quad (4.25)$$

Where,

N_{60} represent the calculate SPT test result

S_e is the elastic settlement

F_d is the depth factor, calculated by the formula below.

$$F_d = 1 + 0.33\left(\frac{D_f}{B}\right) \quad (4.26)$$

Therefore, the updated allowable net bearing capacity considering the maximum settlement in clay layer (75 mm), the formula is:

$$q_{all(net)} = \frac{q_{u(net)}}{FoS} = \frac{N_{60}}{0.08} \times \left(1 + 0.33\left(\frac{D_f}{B}\right)\right) \times \frac{S_e}{25} \times \frac{1}{FoS} =$$

$$\frac{20.6}{0.08} (1 + 0.33(\frac{1}{36})) * \frac{75}{25} * \frac{1}{3} = 259.86 \text{ kN/m}^2$$

This gives:

$$q_{all(net)} = \frac{259.86}{3} = 86.62 \text{ kN/m}^2$$

After careful assessment, it was observed that the settlement of the mat foundation exceeded the maximum allowable limits. Therefore, the mat foundation was not suitable and was not considered further at subsequent stages of the design process.

4.2.2 Pile Foundation

4.2.2.1 Bearing Capacity

4.2.2.1.1 Single Pile

For single pile foundations, the ultimate bearing capacity is influenced by the skin friction of the pile itself and the bearing capacity at point, which are represented in the formula:

$$Q_u = Q_s + Q_p \quad (4.27)$$

a.Skin frictional resistance in clays

$$Q_s = p\Delta L f_{av} \quad (4.28)$$

Where,

p is the perimeter of the pile

L is the length of the pile

f_{av} is the average unit skin friction resistance at depth z

The length of the pile and the diameter is not known. Therefore, several trials will be made to obtain the optimum values of Q_u and Q_{all} . The height of the basement will be chosen as 3.5m to match the height of the 1st floor in the building. According to the Cupertino Building Code, the minimum height of basements should be no less than 2.1m, which meets the standards and is okay to use. As a first attempt, the pile will be in the shape of a cylinder, with a circular cross-section. The diameter of the pile will be chosen as 0.6m as the most common pile diameter for the first trial. The pile lengths, however, will be calculated based on the trials from 4m length to 12m length. The first soil layer is 7.6m deep, therefore, $7.6-3.5=4.1$ m for the pile to be situated around first layer. Therefore, the first pile length trial should start from 4m.

Next, the 12m pile length was chosen as the last trial attempt due to the fact that the layer with the most N values is the clay layer, which is located at the depth 11-17.5m with an N value of 90, which is the highest value in comparison to other soil layers. With this reason, the calculations will be performed for pile lengths 4-12m with the assumption of a 3.5m basement.

The perimeter of the pile based on the diameter 0.6m is

$$p = \pi D = 3.1415 * 0.6 = 1.885m$$

Method 1: λ Method

This method was first discovered by Vijayvergiya and Focht (1972), where the average skin friction resistance is calculated by:

$$f_{av} = \lambda \left(\overline{\sigma'_o} + 2c_u \right) \quad (4.29)$$

Where,

λ is the coefficient based on the pile length

$\overline{\sigma'_o}$ is the average vertical effective stress along the pile

c_u is the undrained shear strength in clay

Based on the table of coefficients from the pile length, for each length in the range of 4-12m the λ coefficient can be found through linear interpolation method from the table given:

Embedment length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

Figure 4.13. Variation of λ with embedment length L .

For example, for a pile of length $L=12\text{m}$, the λ is:

$$\lambda = (0.245 - \frac{0.245-0.2}{10-15} * 10) + \frac{0.245-0.2}{10-15} * 12 = 0.227$$

Next, the mean vertical effective stress from the diagram can be calculated as follows:

$$\bar{\sigma}_o = \frac{A_{(1)} + A_{(2)} + A_{(3)} + \dots}{L} = 403.92/12 = 33.66 \text{ kN/m}^2$$

Then, the average c_u can be calculated as:

$$c_u = \frac{c_{u(1)}L_{(1)} + c_{u(2)}L_{(2)} + c_{u(3)}L_{(3)} + \dots}{L} = 3958.08/12 = 329.84 \text{ kN/m}^2$$

Having calculated all the necessary values,

$$Q_s = p\Delta L f_{av} = 3559.93 \text{ kN}$$

These calculations were completed for each length trial. The values can be seen from table 4.4 below.

Table 4.4. Values of Q_s for different lengths by λ method.

Pile length (m)	Layer height (m)	Unit weight (kN/m ³)	Mean effective vertical stress	Mean undrained shear strength	λ	f_{av} (kN/m ²)	Q_s (kN)
-----------------	------------------	----------------------------------	--------------------------------	-------------------------------	-----------	-------------------------------	------------

			(kN/m ²)	(kN/m ²)			
4	4.1	20.9	85.69	225.04	0.3688	197.59	1489.75
5	0.8	20.3	50.97	220.25	0.336	165.13	1556.28
6	1.5	22.3	45.13	239.40	0.3178	166.50	1883.07
7	1.1	22.3	39.98	251.37	0.2996	162.60	2145.38
8	2	22	40.78	279.62	0.2814	168.85	2546.07
9	1	22	37.65	298.45	0.2632	167.02	2833.27
10	1	22	35.42	311.91	0.245	161.51	3044.31
11	1	22	33.74	321.99	0.236	159.94	3316.26
12	1.5	22	33.66	329.84	0.227	157.39	3559.93

Method 2: α Method

This method utilizes the empirical value α , which corresponds to the ratio of undrained cohesion to the atmospheric pressure. The frictional resistance formula is then calculated:

$$f = \alpha \times c_u \quad (4.30)$$

Where,

α is the adhesion factor, which can be determined by Terzaghi, Peck, and Mesri, 1996 values of c_u/p_a , where p_a is assumed as 100 kN/m² (Das, 2007).

However, the values of adhesion factor α will be calculated, using the formula presented by Sladen (1992) to ease the calculations for numerous iteration trials:

$$\alpha = C \left(\frac{\bar{\sigma}_o}{c_u} \right)^{0.45} \quad (4.31)$$

Where,

C is the coefficient based on the type of pile. This value will be assumed as 0.5 (Das, 2007).

The sample values will be calculated for a depth of 12m.

c_u is the undrained cohesion of the soil. Values of undrained cohesion in ksf were given in the geotechnical report of the site (Langan, 2023) and then converted to kN/m².

Therefore, the bearing capacity for skin friction is calculated as in the table 4.5 below.

Table 4.5. Q_s calculations from α method for different pile lengths.

Pile length (m)	c_u (ksf)	Layer height (m)	c_u (kN/m ²)	α	Q_s (kN)
4	8.4	4.1	402.19	0.39	1196.73
5	10.5	0.8	502.74	0.34	1458.19
6	9.3	1.5	445.29	0.38	1934.30
7	12.3	1.1	588.93	0.34	2350.52
8	12.5	2	598.50	0.36	3151.52
9	14.1	1	675.11	0.35	3592.19
10	16.9	1	809.18	0.33	4088.77
11	17.2	1	823.54	0.33	4597.42
12	20.5	1.5	981.55	0.31	5446.87

Method 3: β Method

This method requires calculating the frictional resistance through vertical effective stress and β factor.

$$f = \beta \times \sigma'_o \quad (4.32)$$

Where,

$\beta = K \cdot \tan \phi'_R$ and ϕ'_R is the drained friction angle

$K = 1 - \sin \phi'_R$ (for normally consolidated clays)

$K = (1 - \sin \phi'_R) \sqrt{OCR}$ (for overconsolidated clays)

σ'_o is the effective vertical stress at the midpoint of the layers

Because the clay layer in our site is classified as very stiff and dense clay in our report (Langan, 2023), we assume that the clay is overconsolidated. However, the OCR values of the clay layers were not given, which is required to calculate the K value. So, the overconsolidation ratio (OCR) will be calculated through a formula presented by Kulhawy and Mayne (1990):

$$OCR = 0.58 * \frac{N_{60} * p_a}{\sigma'_o} \quad (4.33)$$

Where,

N_{60} is the corrected SPT N-value

P_a is the atmospheric pressure, which will be assumed as 100 kN/m² (Das, 2007)

σ'_o is effective vertical stress

Because the values of OCR are usually represented as a single digit, not a decimal, the OCR value will round up to represent a digit.

For the friction angle value, it ranges from 38 to 43 throughout all clay layers.

The sample calculations for L=12m are:

$$K = (1 - \sin\phi)\sqrt{OCR} = (1 - \sin(38))\sqrt{4} = 0.769$$

$$\beta = K * \tan\phi = 0.769 * \tan(38) = 0.601$$

$$f = \beta \times \sigma'_o = 0.601 * 378.26 = 227.17 \text{ kN/m}^2$$

$$Q_s = f * p * L = 227.17 * 1.885 * 12 = 5138 \text{ kN}$$

The calculations for the rest L values are presented in table 4.6.

Table 4.6. Table of Q_s calculation for clay layer from β method.

Pile length (m)	Layer height (m)	N_{60}	OCR	OCR rounded	K	β	Effective vertical stress (kN/m ²)	f (kN/m ²)	Q_s (kN)
4	4.1	26	1.66	2	0.505	0.424	158.84	67.33	507.65
5	0.8	26	2.96	2	0.505	0.424	175.08	74.21	699.44
6	1.5	29	3.73	3	0.551	0.514	208.53	107.11	1211.31
7	1.1	64	3.28	4	0.688	0.598	233.06	139.36	1838.79
8	2	105	3.85	4	0.769	0.601	277.06	166.39	2509.03
9	1	105	3.85	4	0.769	0.601	299.06	179.60	3046.79
10	1	105	3.85	4	0.769	0.601	321.06	192.81	3634.36
11	1	105	3.85	4	0.769	0.601	343.06	206.03	4271.74
12	1.5	105	3.85	4	0.769	0.601	378.26	227.17	5138.23

b.Point-bearing capacity for clay

There are several methods to calculate the point-bearing capacity for clay layers. As for the previous method, the diameter is still assumed as 0.6m with several trials for the pile length at each soil layer.

Method 1: Meyerhof's method

Meyerhof's method for estimating Q_p for clays is the formula presented in Eq.12.20 (Das, 2019) for a case when the friction angle is 0:

$$Q_p = N_c^* * c_u * A_p = 9c_u A_p \quad (4.34)$$

Where,

N_c^* is the bearing capacity factor based on the friction angle

C_u is the undrained cohesion, given from geotechnical report (Langan, 2023)

A_p area of the pile cross-sectional area

However, for this method, the first part of the formula will be used, with an assumption that the friction angle is not equal to 0, instead for each clay layer based on the friction angle, the bearing capacity factors N_c^* will be chosen according figure 4.4 (Das, 2019, p.245):

TABLE 6.2 Bearing Capacity Factors From Eqs. (6.30), (6.29), and (6.31) (*Continued*)

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
22	16.88	7.82	7.13	37	55.63	42.92	66.19
23	18.05	8.66	8.20	38	61.35	48.93	78.03
24	19.32	9.60	9.44	39	67.87	55.96	92.25
25	20.72	10.66	10.88	40	75.31	64.20	109.41
26	22.25	11.85	12.54	41	83.86	73.90	130.22
27	23.94	13.20	14.47	42	93.71	85.38	155.55
28	25.80	14.72	16.72	43	105.11	99.02	186.54
29	27.86	16.44	19.34	44	118.37	115.31	224.64
30	30.14	18.40	22.40	45	133.88	134.88	271.76
31	32.67	20.63	25.99	46	152.10	158.51	330.35
32	35.49	23.18	30.22	47	173.64	187.21	403.67
33	38.64	26.09	35.19	48	199.26	222.31	496.01
34	42.16	29.44	41.06	49	229.93	265.51	613.16
35	46.12	33.30	48.03	50	266.89	319.07	762.89
36	50.59	37.75	56.31				

Figure 4.14 Bearing capacity factors from friction angle (Das, 2019).

The calculations for point-bearing capacity from this method is presented in Table 4.7.

Table 4.7. Q_p values based on Meyerhof's method.

Pile length (m)	A_p (m ²)	Friction angle, °	N_c^*	c_u (kN/m ²)	Q_p (kN)
4	0.283	40	75.31	225.04	4791.67
5	0.283	40	75.31	220.25	4689.72
6	0.283	43	105.11	239.40	7114.59
7	0.283	41	83.86	251.37	5960.05
8	0.283	38	61.35	279.62	4850.24
9	0.283	38	61.35	298.45	5176.91
10	0.283	38	61.35	311.91	5410.25
11	0.283	38	61.35	321.99	5585.25

12	0.283	38	61.35	329.84	5721.37
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Method 2: Direct CPT Method

In this method, the bearing capacity is calculated from the CPT test results in the geotechnical report of the site (Langan, 2023). The formula is:

$$Q_p = A_p q_p \quad (4.35)$$

Where, according to Meyerhof (1956),

$$q_p \approx q_c \quad (4.36)$$

Therefore, in this method the point-bearing capacity is calculated as:

$$Q_p = A_p q_c \quad (4.37)$$

Where,

q_c is the cone penetration resistance from the CPT results, which were also provided from the geotechnical report (Langan, 2023) in tons per square foot and were further converted to kN/m^2 .

The values of Q_p are presented in table 4.8 below.

Table 4.8. Q_p calculation from CPT method.

Pile length (m)	Layer depth (m)	A_p (m^2)	q_c (tsf)	q_c (kN/m^2)	Q_p (kN)
4	7.6	0.283	40	4290.07	1213.0
5	8.4	0.283	40	4290.07	1213.0
6	9.9	0.283	50	5362.59	1516.2
7	11	0.283	80	8580.14	2425.9
9	13	0.283	320	34320.57	9703.6
10	14	0.283	340	36465.61	10310.1
11	15	0.283	500	53625.89	15161.9
12	16	0.283	380	40755.68	11523.1
13	17.5	0.283	350	37538.12	10613.3

Summing up the calculations made for Q_s and Q_p , the table of final values is presented below.

Table 4.9. Table of Q_s and Q_p values for all methods for all pile lengths.

Pile length (m)	Layer #	Soil type	Frictional skin resistance, Q_s (kN)			Point-bearing capacity, Q_p (kN)		Average Q_s (kN)	Average Q_p (kN)
			λ method	a method	B method	Meyerho f's	Direct CPT		
4	1	Clay	1489.7	1196.7	507.6	4791.67	1213.0	1064.7	3002.31
5	2	Sandy Clay	1556.3	1458.2	699.4	4689.72	1213.0	1238.0	2951.33
6	3	Sandy Clay	1883.1	1934.3	1211.3	7114.59	1516.2	1676.2	4315.39
7	4	Silty Clay	2145.4	2350.5	1838.8	5960.05	2425.9	2111.6	4192.98
8	5	Clay	2546.1	3151.5	2509.0	4850.24	9703.6	2735.5	7276.93
9			2833.3	3592.2	3046.8	5176.91	10310.1	3157.4	7743.51
10			3044.3	4088.8	3634.4	5410.25	15161.9	3589.1	10286.1
11			3316.3	4597.4	4271.7	5585.25	11523.1	4061.8	8554.16
12			3559.9	5446.9	5138.2	5721.37	10613.3	4715.0	8167.35

c. Ultimate bearing capacity calculation

To calculate the ultimate bearing capacity for the single pile, the following formula will be used (Das, 2019):

$$Q_u = Q_s + Q_p \quad (4.38)$$

The values of Q_s and Q_p will be taken as the averages of the 3 and 2 methods used for Q_s and Q_p , respectively. Then, the allowable bearing capacity will be calculated with the formula:

$$Q_{all} = \frac{Q_u}{FS} \quad (4.39)$$

Taking the factor of safety as 3, allowable bearing capacities are calculated and presented in the table 4.10 below.

Table 4.10. Table of Q_s and Q_p average, Q_u and Q_{all} for pile of $D=0.6m$.

Pile length (m)	Frictional skin resistance, Q_s (kN)			Point-bearing capacity, Q_p (kN)		Average Q_s (kN)	Average Q_p (kN)	Q_u (kN)	Q_{all} (kN)
	λ method	a method	B method	Meyerh of's	Direct CPT				
4	1489.7	1196.7	507.6	4791.67	1213.0	1064.7	3002.31	4067.0	1355.7
5	1556.3	1458.2	699.4	4689.72	1213.0	1238.0	2951.33	4189.3	1396.4
6	1883.1	1934.3	1211.3	7114.59	1516.2	1676.2	4315.39	5991.6	1997.2
7	2145.4	2350.5	1838.8	5960.05	2425.9	2111.6	4192.98	6304.5	2101.5
8	2546.1	3151.5	2509.0	4850.24	9703.6	2735.5	7276.93	10012.5	3337.5
9	2833.3	3592.2	3046.8	5176.91	10310.1	3157.4	7743.51	10900.9	3633.6
10	3044.3	4088.8	3634.4	5410.25	15161.9	3589.1	10286.0	13875.2	4625.1
11	3316.3	4597.4	4271.7	5585.25	11523.1	4061.8	8554.16	12616.0	4205.3
12	3559.9	5446.9	5138.2	5721.37	10613.3	4715.0	8167.35	12882.4	4294.1

Comparing the values of Q_{all} with Q_{design} , which were previously calculated in part 2 of this report, it is concluded that the single pile of $D=0.6m$ is not suitable for the design loads. The smallest design load is for the corner column, where the value is 4937 kN, while the biggest value possible for this diameter is 10m pile with Q_{all} of 4625 kN. It is smaller. Therefore, several trials of diameters will be calculated.

For diameter $D=0.7m$, the values of Q_{all} are shown in table 4.11.

Table 4.11. Q_{all} for single pile for $D=0.7m$.

Pile length (m)	Frictional skin resistance, Q_s (kN)			Point-bearing capacity, Q_p (kN)		Average Q_s (kN)	Average Q_p (kN)	Q_u (kN)	Q_{all} (kN)
	λ	a	B	Meyerh	Direct				

	method	method	method	of's	CPT				
4	1738.0	1396.2	592.3	6521.99	1651.0	1242.2	4086.48	5328.6	1776.2
5	1815.7	1701.2	816.0	6383.22	1651.0	1444.3	4017.09	5461.4	1820.5
6	2196.9	2256.7	1413.2	9683.75	2063.7	1955.6	5873.73	7829.3	2609.8
7	2502.9	2742.3	2145.2	8112.30	3301.9	2463.5	5707.11	8170.6	2723.5
8	2970.4	3676.8	2927.2	6601.72	13207.7	3191.5	9904.72	13096.2	4365.4
9	3305.5	4190.9	3554.6	7046.36	14033.2	3683.7	10539.8	14223.4	4741.1
10	3551.7	4770.2	4240.1	7363.95	20637.1	4187.3	14000.5	18187.8	6062.6
11	3869.0	5363.7	4983.7	7602.15	15684.2	4738.8	11643.2	16381.9	5460.6
12	4153.3	6354.7	5994.6	7787.42	14445.9	5500.8	11116.8	16617.5	5539.2

From the calculations, still the Q_{all} does not meet the minimum design requirements for all columns of the building. A new trial is calculated in table 4.12.

Table 4.12. Q_{all} for single pile for $D=0.8m$

Pile length (m)	Frictional skin resistance, Q_s (kN)			Point-bearing capacity, Q_p (kN)		Average Q_s (kN)	Average Q_p (kN)	Q_u (kN)	Q_{all} (kN)
	λ method	a method	B method	Meyerh of's	Direct CPT				
4	1986.3	1595.6	676.9	8518.52	2156.4	1419.6	5337.44	6757.1	2252.4
5	2075.0	1944.2	932.6	8337.27	2156.4	1650.6	5246.82	6897.4	2299.1
6	2510.8	2579.1	1615.1	12648.1	2695.5	2235.0	7671.81	9906.8	3302.3
7	2860.5	3134.0	2451.7	10595.6	4312.7	2815.4	7454.19	10269.6	3423.2
8	3394.8	4202.0	3345.4	8622.65	17250.9	3647.4	12936.7	16584.2	5528.1
9	3777.7	4789.6	4062.4	9203.40	18329.1	4209.9	13766.4	17976.1	5992.0
10	4059.1	5451.7	4845.8	9618.22	26954.5	4785.5	18286.3	23071.9	7690.6
11	4421.7	6129.9	5695.7	9929.34	20485.4	5415.7	15207.3	20623.1	6874.4

12	4746.6	7262.5	6851.0	10171.3	18868.2	6286.7	14519.7	20806.4	6935.5
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Still, the values of Q_{all} do not satisfy the design requirements. After many long iterations and calculations, the first suitable value of diameter is $D=1.1\text{m}$. Calculations presented in table 4.13.

Table 4.13. Q_{all} values for $D=1.1\text{m}$ for single pile.

Pile length (m)	Frictional skin resistance, Q_s (kN)			Point-bearing capacity, Q_p (kN)		Average Q_s (kN)	Average Q_p (kN)	Q_u (kN)	Q_{all} (kN)
	λ method	a method	B method	Meyerh of's	Direct CPT				
4	2731.2	2194.0	930.7	16105.3	4076.9	1952.0	10091.1	12043.1	4014.4
5	2853.2	2673.3	1282.3	15762.7	4076.9	2269.6	9919.8	12189.4	4063.1
6	3452.3	3546.2	2220.7	23912.9	5096.1	3073.1	14504.5	17577.6	5859.2
7	3933.2	4309.3	3371.1	20032.4	8153.7	3871.2	14093.1	17964.3	5988.1
8	4667.8	5777.8	4599.9	16302.2	32615.0	5015.2	24458.6	29473.7	9824.6
9	5194.3	6585.7	5585.8	17400.2	34653.4	5788.6	26026.8	31815.4	10605.1
10	5581.2	7496.1	6663.0	18184.5	50960.9	6580.1	34572.7	41152.8	13717.6
11	6079.8	8428.6	7831.5	18772.7	38730.3	7446.6	28751.5	36198.1	12066.0
12	6526.5	9985.9	9420.1	19230.2	35672.6	8644.2	27451.4	36095.6	12031.9

From the table, it is visible that for a pile of length $L=10\text{m}$ and $D=1.1\text{m}$ the Q_{all} is 13718 kN, which is more than the Q_{des} of 12777 kN for interior columns. However, the diameter of 1.1m is bigger than the usual diameter values of 0.4-0.7m. Therefore, this pile will not be chosen for the building construction.

4.2.2.1.2. Group of Piles

Group of piles bearing capacity indicator in some degree depends on the bearing capacity value of a single pile. The efficiency of a group of piles needs to be calculated in order to find whether a group of piles serves as a single pile or as a block. This can be evaluated from the following formula:

$$\eta = \left[\frac{2(\eta_1 + \eta_2 - 2)d + 4D}{p\eta_1\eta_2} \right] \quad (4.40)$$

Here,

η_1, η_2 -pile numbers in the group;

p-perimeter of the pile;

d=2.5D-minimum distance between piles.

The group of piles operates as a block if these requirements are met: $\eta < 1$ and

$Q_{g(u)} = \Sigma Q_u$. And the group of pile acts as a single pile if following is true: $\eta > 1$ and

$Q_{g(u)} = \eta \Sigma Q_u$. That means that the bearing capacity of a group pile is equal to the sum of bearing capacities of all single piles.

There for the allowable bearing capacity can be calculated from the following formula:

$$Q_{g(all)} = \frac{Q_{g(u)}}{FS} \quad (4.41)$$

Where,

$Q_{g(u)}$ -ultimate bearing capacity of group pile

$Q_{g(all)}$ -allowable bearing capacity of group pile

FS-Factor of Safety (assumed to be 3)

Along with the allowable bearing capacity, for the design of group pile foundation it is required to obtain sizes of piles, which can be derived from:

$$L_g = (n_1 - 1)d + 2(D/2) \quad (4.42)$$

$$B_g = (n_2 - 1)d + 2(D/2) \quad (4.43)$$

These formulas were used to design pile dimensions for exterior, interior and corner columns of the building. L_g and B_g values were obtained for each type of column and represented in the

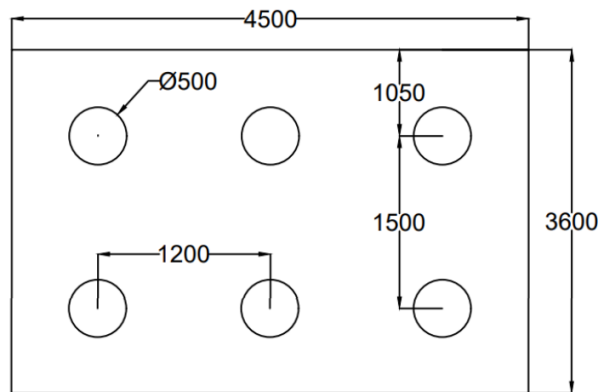
Table 4.14 and Figure 4.15 below:

Table 4.14. Group pile sizing parameters summary from Capstone I.

Size of the piles	Interior	Exterior	Corner
<i>Length of the pile, m</i>	12	12	12
<i>Diameter, m</i>	0.5	0.45	0.5

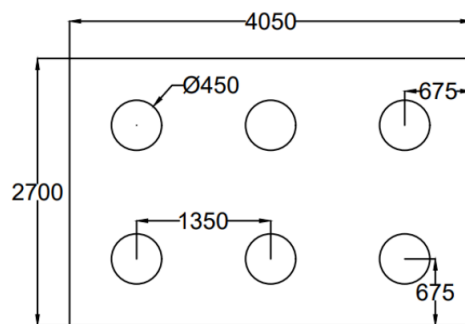
<i>Minimum spacing, m</i>	1.25	1.125	1.25
<i>n1</i>	3	3	2
<i>n2</i>	2	2	2
<i>Lg, m</i>	3.00	2.70	1.75
<i>Bg, m</i>	1.75	1.58	1.75
<i>Perimeter, m</i>	1.57	1.41	1.57
<i>Cross-sectional area, m²</i>	0.20	0.16	0.20
<i>Efficiency</i>	1.01	1.01	1.11

Below are the pile group dimensions.



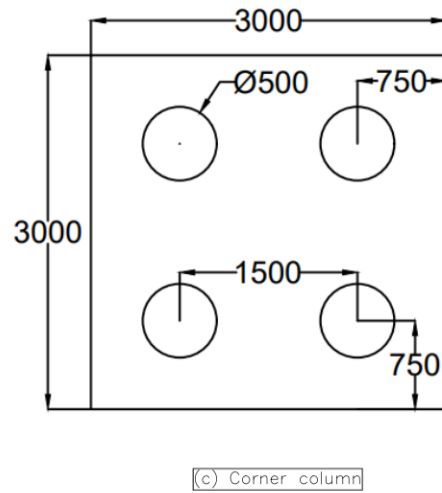
(a) Interior column

a. Group of piles under Interior column



(b) Exterior column

b. Group of piles under Exterior column



c. Group of piles under the Corner column

Figure 4.15. Technical drawing for interior (a), exterior (b), and corner (c) columns.

Group of piles were placed under each column accordingly, its cross section and pile locations are displayed at the figure below:

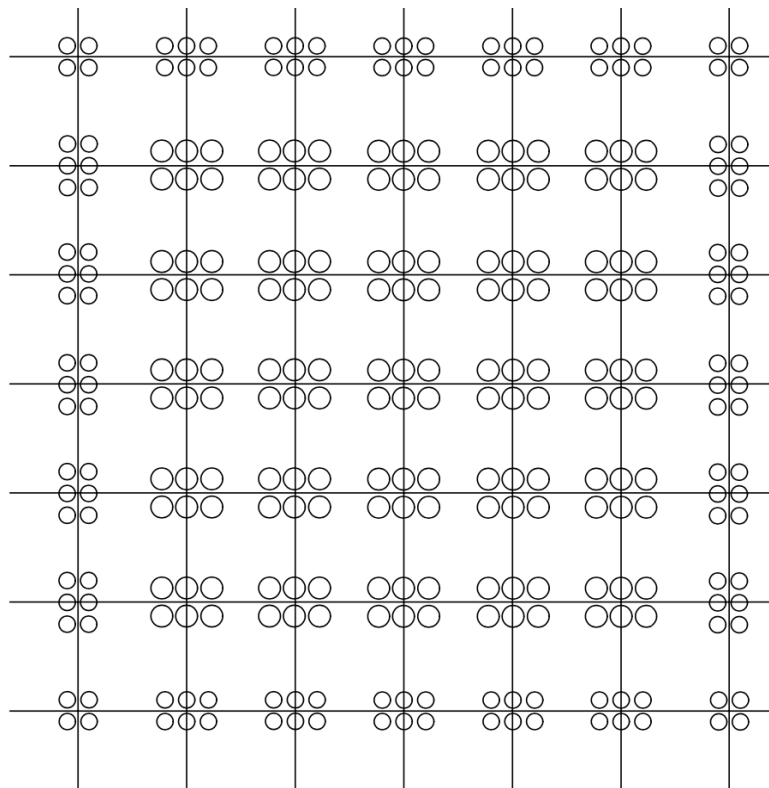


Figure 4.16 Location of group of files under each column type

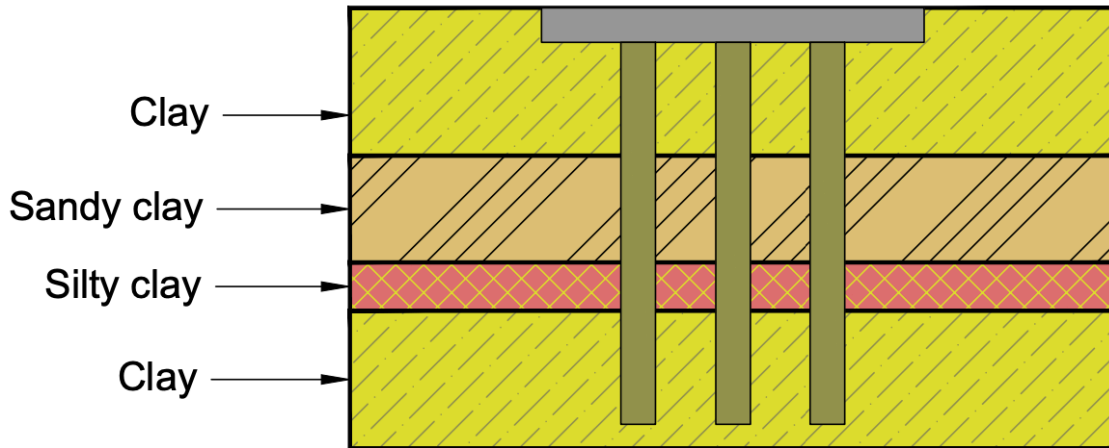


Figure 4.17 Cross-Section of Soil Layers with Group of Piles Foundation

The bearing capacity of the group of piles for each column type and group of piles was calculated. The group of piles under the interior column will be used to show the calculations. The parameter for the piles under interior columns are: $L=7\text{m}$, $D=0.6\text{m}$, $A=0.28\text{ m}^2$, $p=1.9\text{m}$. Efficiency for the interior column piles is obtained below:

$$\eta = \left[\frac{2(3+2-2)*1.5+4*0.6}{1.9*3*2} \right] = 1.008$$

Group of piles acts as a single piles, because $\eta > 1$ and bearing capacity can be obtained by the following way:

$$Q_{u(g)} = \Sigma Q_u = \Sigma(Q_p + Q_s) = 3 * 2 * (3431.47 + 8937.728) = 74215.188\text{ kN}$$

$$Q_{all(g)} = \frac{Q_{u(g)}}{FS} = 24738.396\text{ kN} > Q_{design} = 12777.83\text{ kN}$$

Going further the efficiency of piles under exterior and corner columns was also calculated:

$$\eta = \left[\frac{2(3+2-2)*1.5+4*0.45}{1.58*3*2} \right] = 1.008\text{-exterior column}$$

$$\eta = \left[\frac{2(2+2-2)*1.2+4*0.55}{1.7*2*2} \right] = 1.11\text{-corner column}$$

The results of bearing capacity for all types of columns are given below:

Table 4.15. Bearing capacity of group of piles

	Efficiency, η	$Q_p, \text{ kN}$	$Q_s, \text{ kN}$	$Q_u, \text{ kN}$	$\Sigma Q_u, \text{ kN}$	$Q_{all}, \text{ kN}$	$Q_{design}, \text{ kN}$
Interior	1.01	3431.47	8937.728	12369.198	74215.188	24738.396	12777.83

column							
Exterior column	1.06	1929	6700.095	8629.095	51774.57	17258.19	8268.3
Corner column	1.01	2876	8189	11065	44260	14753.333 33	4936.83

Pile foundation is the most suitable in theory, as it has great resistance to lateral loads for high-story buildings, wind loads, seismic loads (Das, 2019). Group of piles is selected based on our analysis, since this fully satisfies all the required conditions: from load-bearing capacity to soil compatibility and structural stability. This will definitely ensure optimum performance and reliability for the project.

4.2.2.2 Settlement in piles

4.2.2.2.1. Single Pile

4.2.2.2.1.1 Vesic's method

Vesic (1977) proposed a method for calculating settlement by separating it into 3 settlements: elastic shortening of the pile - $s_{e(1)}$, settlement in the pile due to working load from point load - $s_{e(2)}$, and settlement in the pile due to working load from the skin resistance - $s_{e(3)}$. The formulas are presented below:

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \quad (4.44)$$

Where,

Q_{wp} is the working load at the pile point, calculated by Q_p/FS

ξ is the constant which ranges between 0.5-0.67 (Das, 2019). In our case it will be assumed as 0.67 for safety

Q_{ws} is the working load from the pile shaft, calculated by Q_s/FS

E_p is the elastic modulus of the pile material. The pile will be made of concrete, therefore the elastic modulus will be assumed as 29.75 GPa.

The settlement $s_{e(1)}$ calculations are presented in table 4.16.

Table 4.16. Elastic shortening of pile calculation.

Layer #	Pile length (m)	c' (kPa)	Friction angle	Qwp (kN)	Qws (kN)	Ap (m ²)	Ep (kPa)	ξ	Se(1) (m)
1	4	90	40	1000.77	354.90	0.282735	29725000	0.67	0.00059
2	5	24.3	40	983.78	412.66	0.282735	29725000	0.67	0.00075
3	6	12.4	43	1438.46	558.74	0.282735	29725000	0.67	0.00129
4	7	21.9	41	1397.66	703.85	0.282735	29725000	0.67	0.00156
5	8	35	38	2425.64	911.85	0.282735	29725000	0.67	0.00289
	9	35	38	2581.17	1052.47	0.282735	29725000	0.67	0.00352
	10	35	38	3428.69	1196.38	0.282735	29725000	0.67	0.00503
	11	35	38	2851.39	1353.94	0.282735	29725000	0.67	0.00492
	12	35	38	2722.45	1571.67	0.282735	29725000	0.67	0.00539

Next,

$$s_{e(2)} = \frac{q_{wp} D}{E_s} (1 - \mu_s^2) I_{wp} \quad (4.45)$$

Where,

q_{wp} is the point load on the pile tip Q_{wp}/A_p

E_s is the elastic modulus of the soil.

μ_s is the poisson's ratio

I_{wp} is the influence factor

Based on the formula of elastic modulus of soil approximation (Chen and Kulhawy, 1994), E_s can be calculated as:

$$E_s = m * p_a \quad (4.46)$$

Where,

E_s is the modulus of elasticity of soil

m is a constant based on the loose, medium dense, and dense type of soil (Figure 4.8)

P_a is the atmospheric pressure, which is assumed as 100 kN/m² (Das, 2019)

$$m = \begin{cases} 100 \text{ to } 200 \text{ (loose soil)} \\ 200 \text{ to } 500 \text{ (medium dense soil)} \\ 500 \text{ to } 1000 \text{ (dense soil)} \end{cases}$$

Figure 4.8. Values of m for elastic modulus of soil calculation.

Because the clay is classified as dense or stiff in the geotechnical report, the m value will be taken as an average of 500-1000 range. Therefore, m=750.

This gives:

$$E_s = m * p_a = 750 * 100 = 75000 \text{ kPa}$$

For poisson's ratio of the soil (Das, 2019):

$$\mu_s = 0.1 + 0.3\left(\frac{\phi-25}{20}\right) \quad (4.47)$$

Influence factor will be taken as $I_{wp}=0.85$ (Das, 2019).

The calculated values of elastic settlement are shown in table 4.17.

Table 4.17. Elastic settlement $s_{e(2)}$ calculation.

Layer #	Pile length (m)	Friction angle	c' (kPa)	Qwp	Ap	q(wp)	Es	poisson's ratio	Iwp	Se(2) (m)
1	4	40	90	1001	0.28	3540	75000	0.325	0.85	0.0215
2	5	40	24.3	984	0.28	3480	75000	0.325	0.85	0.0212
3	6	43	12.4	1438	0.28	5088	75000	0.370	0.85	0.0299
4	7	41	21.9	1398	0.28	4943	75000	0.340	0.85	0.0297
5	8	38	35	2426	0.28	8579	75000	0.295	0.85	0.0533
	9	38	35	2581	0.28	9129	75000	0.295	0.85	0.0567
	10	38	35	3429	0.28	12127	75000	0.295	0.85	0.0753
	11	38	35	2851	0.28	10085	75000	0.295	0.85	0.0626
	12	38	35	2722	0.28	9629	75000	0.295	0.85	0.0598

An alternative formula for $s_{e(2)}$ calculation proposed by Vesic (1977):

$$s_{e(2)} = \frac{Q_{wp} C_p}{D q_p} \quad (4.48)$$

Where,

q_p is the ultimate point resistance of the pile

C_p is an empirical coefficient, based on the soil type

For a bored concrete pile in a stiff clay, the C_p coefficient will be taken as 0.03, based on the figure 4.18 below.

Type of soil	Driven pile	Bored pile
Sand (dense to loose)	0.02–0.04	0.09–0.18
Clay (stiff to soft)	0.02–0.03	0.03–0.06
Silt (dense to loose)	0.03–0.05	0.09–0.12

Figure 4.18 Variance of values of C_p based on the type of soil (Vesic, 1977).

The values were calculated in table 4.18 below.

Table 4.18. Values of settlement for alternative $s_{e(2)}$ method.

Layer #	Pile length (m)	c' (kPa)	C_p	Q_{wp} (kN)	$q(p)$ (kN/m ²)	Alternative $s_{e(2)}$ (m)
1	4	90	0.03	1001	10619	0.00422
2	5	24.3	0.03	984	10439	0.00471
3	6	12.4	0.03	1438	15263	0.00486
4	7	21.9	0.03	1398	14830	0.00542
5	8	35	0.03	2426	25738	0.00598
	9	35	0.03	2581	27388	0.00655
	10	35	0.03	3429	36381	0.00711
	11	35	0.03	2851	30255	0.00767
	12	35	0.03	2722	28887	0.00823

Then, for $s_{e(3)}$ the formula is:

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL} \right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \quad (4.49)$$

Where,

I_{ws} is the influence factor

For $s_{e(3)}$ as an influence factor, I_{ws} can be calculated as (Vesic, 1977):

$$I_{ws} = 2 + 0.35 * \sqrt{\frac{L}{D}} \quad (4.50)$$

Therefore, $s_{e(3)}$ values were calculated.

Table 4.19. Settlement calculated values.

Layer #	Pile length (m)	Qws (kN)	Es (kPa)	Iws	Se(3) (m)
1	4	355	75000	2.90	0.000978
2	5	413	75000	3.01	0.000943
3	6	559	75000	3.11	0.001060
4	7	704	75000	3.20	0.001206
5	8	912	75000	3.28	0.001448
	9	1052	75000	3.36	0.001521
	10	1196	75000	3.43	0.001590
	11	1354	75000	3.50	0.001669
	12	1572	75000	3.57	0.001809

And as an alternative formula for $s_{e(3)}$ calculation, the following formula is presented:

$$s_{e(3)} = \frac{Q_{ws} C_s}{L q_p} \quad (4.51)$$

Where,

C_s is an empirical constant

The C_s constant can be calculated as

$$C_s = (0.93 + 0.16\sqrt{L/D}) * C_p \quad (4.52)$$

Based on the formulas, the values are calculated.

Table 4.20 Alternative $s_{e(3)}$ values.

Layer #	Pile length (m)	Cs	Alternative Se(3) (m)
1	4	0.0403	0.000337
2	5	0.0418	0.000667
3	6	0.0431	0.000930
4	7	0.0443	0.001230
5	8	0.0454	0.001431
	9	0.0465	0.001630
	10	0.0475	0.001786
	11	0.0485	0.001983
	12	0.0494	0.002207

Based on the calculated values, the total elastic settlement for a single pile in clays can be estimated. For this, values of Se(1) and alternative values of Se(2) and Se(3) were chosen because they consider the soil type and its stiffness in calculations. So,

$$Se(total) = s_{e(1)} + s_{e(2)} + s_{e(3)} \quad (4.53)$$

A final table of calculated values is below.

Table 4.21 Total settlement for pile of D=0.6m.

Layer #	Pile length (m)	Se(1), m	Se(2), m	Se(3), m	Total Se (max), m	Total Se (min), m
---------	-----------------	----------	----------	----------	-------------------	-------------------

1	4		Vesic's standar d	Alternati ve Vesic's 1977 method	Vesic's standard	Alternativ e Vesic's 1977 method		
2	5	0.0006	0.0215	0.0042	0.00098	0.00034	0.023	0.0051
3	6	0.0007	0.0212	0.0047	0.00094	0.00067	0.023	0.0061
4	7	0.0013	0.0299	0.0049	0.00106	0.00093	0.032	0.0071
5	8	0.0016	0.0297	0.0054	0.00121	0.00123	0.032	0.0082
	9	0.0029	0.0533	0.0060	0.00145	0.00143	0.058	0.0103
	10	0.0035	0.0567	0.0065	0.00152	0.00163	0.062	0.0117
	11	0.0050	0.0753	0.0071	0.00159	0.00179	0.082	0.0139
	12	0.0049	0.0626	0.0077	0.00167	0.00198	0.069	0.0146

Above, because there are alternative values of settlements, there are two total settlement estimations made: total s_e (max) from the higher values, and total s_e (min) from the lower values, therefore, representing a settlement range. These values are calculated for a pile of diameter of $D=0.6m$.

For comparison, the same calculations were made for a diameter of $D=1.1m$ in the table below.

Table 4.22 Total elastic settlement for pile $D=1.1m$

Layer #	Pile length (m)	Se(1), m	Se(2), m		Se(3), m		Total Se (max), m	Total Se (min), m
			Vesic's standar d	Alternati ve Vesic's 1977 method	Vesic's standard	Alternative Vesic's 1977 method		
1	4							
2	5	0.0006	0.0466	0.0042	0.00088	0.00030	0.048	0.0051

3	6	0.0007	0.0458	0.0047	0.00084	0.00060	0.047	0.0061
4	7	0.0013	0.0647	0.0049	0.00094	0.00083	0.067	0.0070
5	8	0.0016	0.0644	0.0054	0.00106	0.00110	0.067	0.0081
	9	0.0029	0.1154	0.0060	0.00127	0.00127	0.120	0.0101
	10	0.0035	0.1228	0.0065	0.00132	0.00145	0.128	0.0115
	11	0.0050	0.1631	0.0071	0.00138	0.00158	0.170	0.0137
	12	0.0049	0.1357	0.0077	0.00144	0.00175	0.142	0.0143

From the final values obtained, it is observed that the range between the max and min values for a pile length of 10m has a too wide range, considering that the settlement in clays should not exceed 25mm. Therefore, these calculations only confirm that the single piles are not suitable for this case: pile diameter is wider than usual and the settlement is unacceptable.

4.2.2.2.2. Group of piles

In order to estimate the settlement for a group of piles in gravel, two settlements were calculated: elastic settlement (immediate settlement) and consolidated settlement.

1. Consolidation settlement

First, starting by calculation of consolidated settlement, the following formula will be used to determine increase in effective stress at the middle of clay layer:

$$\Delta\sigma' = \frac{Q_g}{(B_g + z_i)(L_g + z_i)} \quad (4.54)$$

Where,

$\Delta\sigma'$ -increase in effective stress at the middle of clay layer

B_g, L_g -dimensions of planned group of piles

z_i -distance between middle of the chosen layer and $z=0$

To calculate settlement itself, following formula was used:

$$\Delta s_c = \left[\frac{\Delta e}{1+e_o} \right] H \quad (4.55)$$

Where,

Δs_c -consolidation settlement of clay layer

Δe -void ratio change that happened in chosen clay layer

e_o -initial void ratio of clay layer

H -layer thickness

The sum of group pile settlement can be estimated by:

$$\Delta s_{c(g)} = \Sigma \Delta s_{c(i)} \quad (4.56)$$

Since, our clay has $OCR > 1$, clay is considered as overconsolidated and the following formula was used to find primary settlement in clay layer:

$$\Delta s_c = \frac{H}{1+e_o} c_r \log \frac{\sigma'_o + \Delta \sigma}{\sigma'_o} \quad (4.57)$$

Where,

C_r -recompression index

σ'_o -initial effective vertical stress.

H -thickness of the compressible clay layer

e_o -initial void ratio of clay layer

As, recompression index was not given in geotechnical investigation report, it was found by the formula below (Fine Software, n.d.):

$$C_r = \frac{\Delta e}{\Delta \log \sigma_{eff}} \quad (4.58)$$

Where,

Δe -change in void ratio

$\Delta \log \sigma_{eff}$ -change in effective vertical stress

From Consolidation test 1 in the investigation report, following values were given:

$e_1 = 0.34$, $e_2 = 0.5$, $\sigma_{eff,1} = 3120$, $\sigma_{eff,2} = 8000$. So the recompression ratio was calculated by:

$$C_r = \frac{0.5-0.34}{\log (8000-3120)} = 0.39$$

The following table summarizes the parameters and their values used in the calculation of the consolidated settlement:

Table 4.23 Parameters to calculate the consolidated settlement

Layer	Height	γ , kN/m ³	e_o	G_s	C_r	σ'	z	$\Delta \sigma'$
-------	--------	------------------------------	-------	-------	-------	-----------	-----	------------------

	, m							
1	7.6	20.9	0.34	2.7	0.391	182.677	3.81	79.629
2	0.8	20.3	0.34	2.7	0.391	809.974	0.381	87.363
3	1.5	22.3	0.34	2.7	0.391	640.818	0.762	104.356
4	1.1	22.3	0.34	2.7	0.391	734.963	0.5334	116.251
5	6.6	22.0	0.34	2.7	0.391	216.376	3.2766	188.336

So, following results were obtained:

Table 4.24 Consolidation settlement under each column type

Column type	Sum of settlement, m
Interior column	0.0213
Exterior column	0.0182
Corner column	0.0117

2. Elastic Settlement

To calculate elastic settlement following formula, where the results from cone penetration test was used, from the book of Das & Sivakugan (2019):

$$S_e = \frac{q B_g I}{2 q_c} \quad (4.59)$$

Where,

$$q = \frac{Q_g}{L_g * B_g}, \text{ kN/m}^2$$

q_c – average cone penetration resistance of the layers, kN/m^2

$$I - \text{influence factor} = 1 - \frac{L}{8 * B_g} \geq 0.5$$

Average cone penetration resistance value was taken from the geotechnical investigation report, where cone penetration test was done. On the tables below, results of the calculation of settlement under each column types are given.

Table 4.25 Calculation of elastic settlement of group of piles under interior column

Layer #	Height, m	q_c , kN/m ²	q , kN/m ²	Settlement, m
1	7.6	38304.2	1690.189153	0.001323762553
2	0.8	28728.2	1690.189153	0.001765016738
3	1.5	28728.2	1690.189153	0.001765016738
4	1.1	52668.3	1690.189153	0.0009627364025
5	6.6	45007.4	1690.189153	0.001126606429
			SUM	0.006943138861

Table 4.26 Calculation of elastic settlement of group of piles under exterior column

Layer #	Height, m	q_c , kN/m ²	q , kN/m ²	Settlement, m
1	7.6	38304.2	3004.780717	0.0002941694563
2	0.8	28728.2	3004.780717	0.0003922259418
3	1.5	28728.2	3004.780717	0.0003922259418
4	1.1	52668.3	3004.780717	0.0002139414228
5	6.6	45007.4	3004.780717	0.0002503569841
			SUM	0.001542919747

Table 4.27 Calculation of elastic settlement of group of piles under corner column

Layer #	Height, m	q_c , kN/m ²	q , kN/m ²	Settlement, m
1	7.6	38304.2	3448.225333	0.001912969735
2	0.8	28728.2	3448.225333	0.002550626313
3	1.5	28728.2	3448.225333	0.002550626313
4	1.1	52668.3	3448.225333	0.0001391250716

5	6.6	45007.4	3448.225333	0.01628059349
			SUM	0.01003353243

Therefore, when summing up consolidation and elastic settlement for group of piles, following results were obtained:

Table 4.28 Calculated total settlement for each column type

	Interior column	Exterior column	Corner column
<i>Consolidated settlement, m</i>	0.014357	0.016657	0.001667
<i>Elastic settlement, m</i>	0.006943138861	0.001542919747	0.01003353243
<i>SUM</i>	0.0213	0.0182	0.0117

Therefore, as a result of calculations and comparing the settlements in the single pile and a group of piles, it is seen that single piles are not suitable for this project, while the group of piles produce the required bearing capacity and an adequate settlement as according to EC7.

4.3 Group of pile design

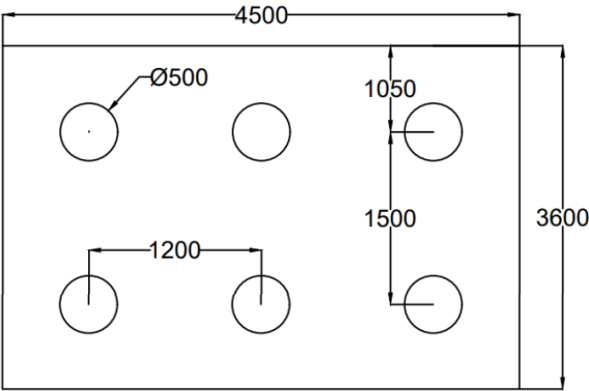
As a summary of the calculations above, the final dimensions of the group of piles is the following.

Table 4.29. Group pile sizing parameters summary from Capstone I.

Size of the piles	Interior	Exterior	Corner
Length of the pile, m	12	12	12
Diameter, m	0.5	0.45	0.5
Minimum spacing, m	1.25	1.125	1.25
n1	3	3	2
n2	2	2	2
Lg, m	3.00	2.70	1.75
Bg, m	1.75	1.58	1.75
Perimeter, m	1.57	1.41	1.57

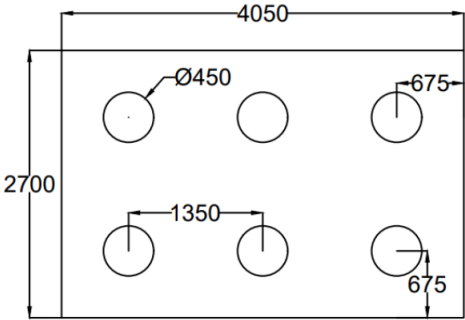
Cross-sectional area, m ²	0.20	0.16	0.20
Efficiency	1.01	1.01	1.11

Below are the pile group dimensions.



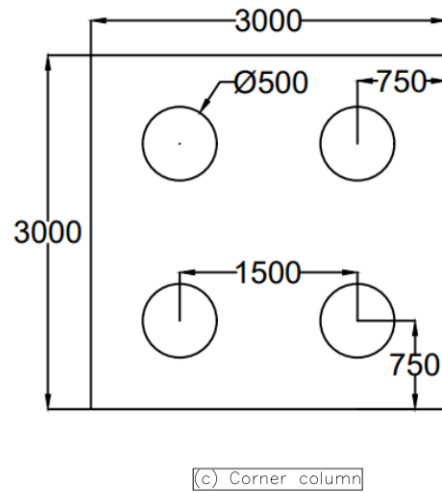
(a) Interior column

a. Group of piles under Interior column



(b) Exterior column

b. Group of piles under Exterior column



c. Group of piles under the Corner column

Figure 4.19. Technical drawing for interior (a), exterior (b), and corner (c) columns.

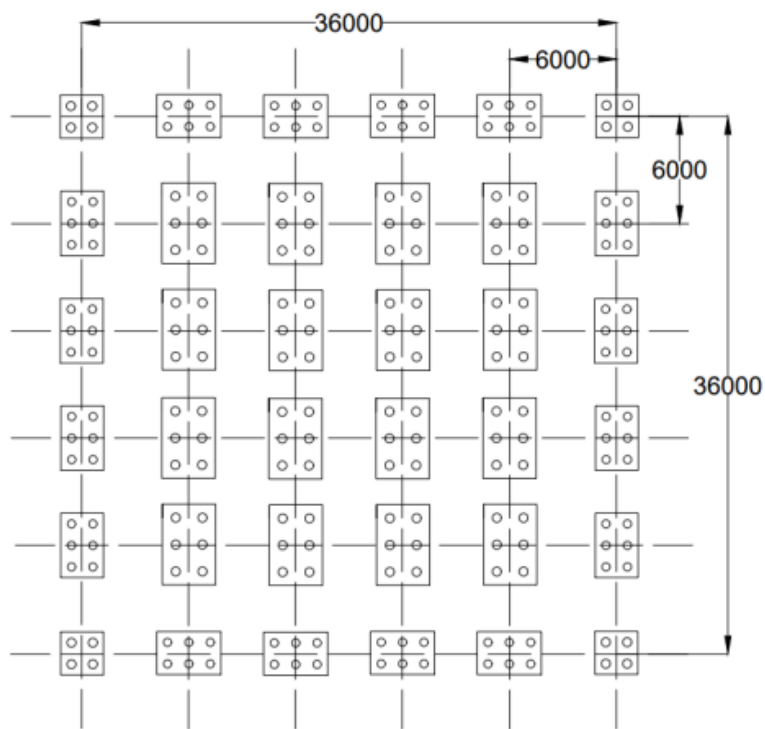


Figure 4.20. Location of piles under the building.

4.4 Soil-structure interaction analysis

Plaxis 3D software was used to analyze the soil interaction of the foundation.

For that, 3 group piles were modeled individually and their respective design loads were applied. As a result of calculations from the structural design part, these were the final values of axial and lateral loads on the foundation and the columns in table 4.30. The parameter 1.2D+L+S resulted in the highest loads, in comparison to other load combinations, therefore, these loads will be used for further analysis.

Table 4.30. Updated column loads.

Parameter		Fx, kN	Fy, kN	Fz, kN	Mx, kNm	My, kNm	Mz, kNm
1.2D+L+S	Interior column	-293,616	-232,978	12111,23	981,5618	-1077,6	-2,0102
	Exterior column	-282,127	-247,172	9087,793	997,5503	-1065,43	-1,9171
	Corner column	-249,557	-235,674	6268,802	987,1881	-1031,8	-2,0805

To set up the software, the following parameters were input: PLAXIS 3D Ultimate analysis. The soil profile was created based on the soil profile table. Next, structures were added by modeling every group pile individually through negative interfaces and volume analysis with appropriate loads for each column. After, meshing was complete in a very coarse mode for optimum time of software calculations. Nodes were selected right at the tip of the pile to further analyze displacement at the point. In the staged construction the piles were assigned the pile material, plates and interfaces were activated, and the loads were activated as the last step. As a result of calculations and analysis, the results were provided in figures below.

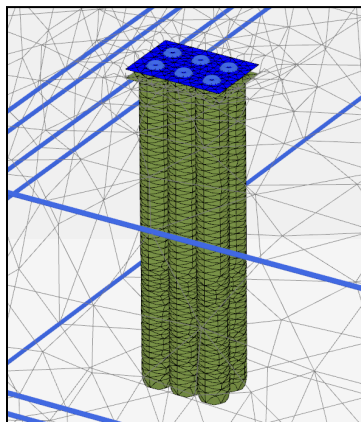


Figure 4.21. Interior pile.

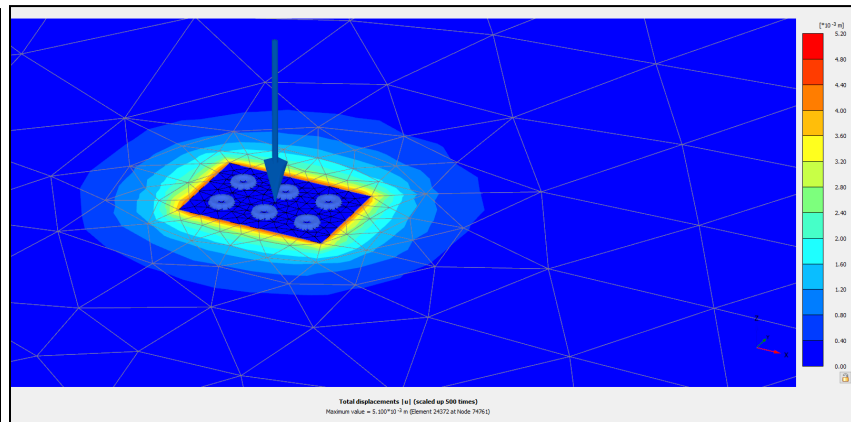


Figure 4.22. Displacement of interior pile.

The software was also used to analyse the stresses and forces in the piles. The screenshots are shown below for every column type.

Interior columns

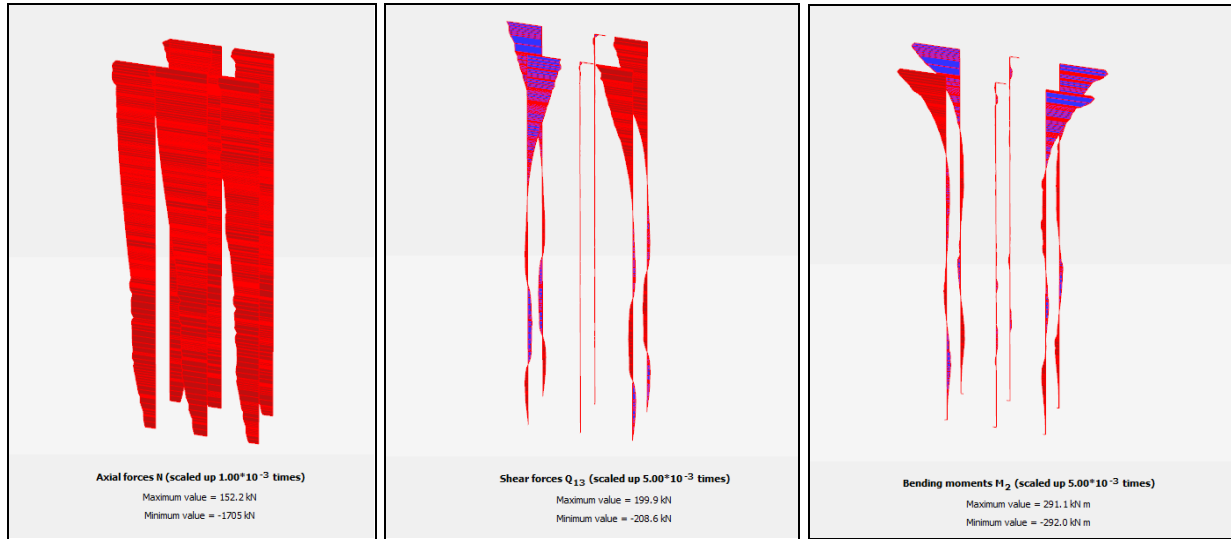


Figure 4.23. Axial forces (left), shear forces (middle), bending moment (right) diagrams from PLAXIS 3D for interior columns.

Exterior columns

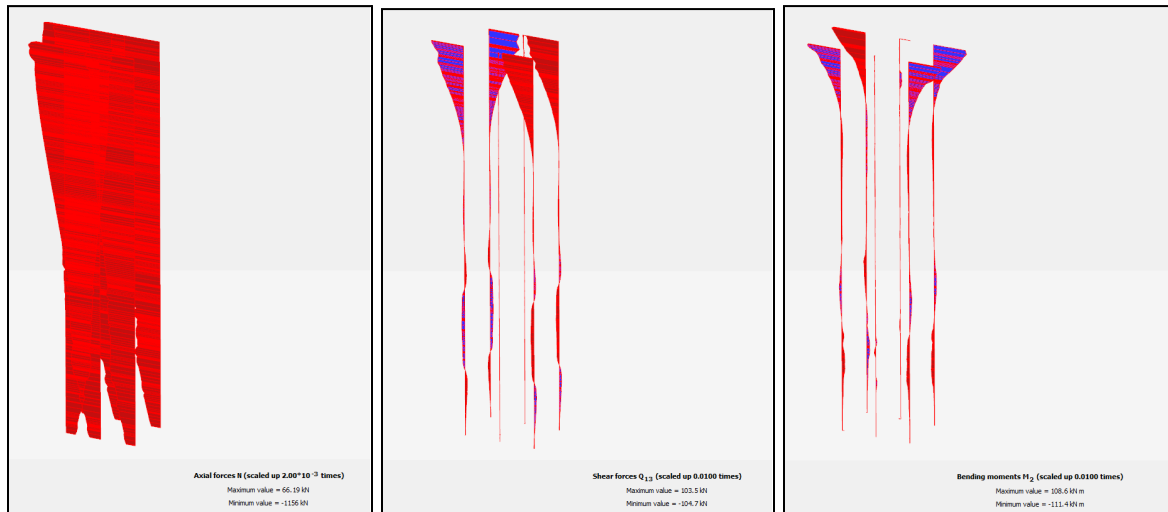


Figure 4.24. Axial forces (left), shear forces (middle), bending moment (right) diagrams from PLAXIS 3D for exterior columns.

Corner columns

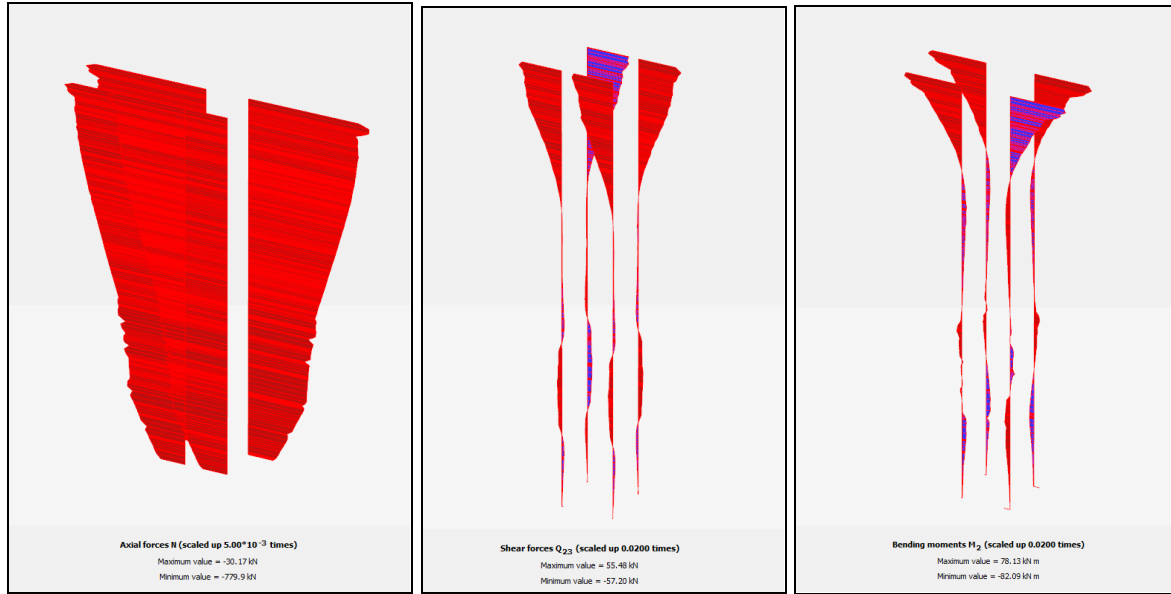


Figure 4.25. Axial forces (left), shear forces (middle), bending moment (right) diagrams from PLAXIS 3D for corner columns

These diagrams were gathered and summarized with the maximum and minimum values in table 4.31 below.

Table 4.31. Summary of axial forces, shear forces and bending moment for all column types from PLAXIS 3D software.

	Axial forces (scaled up 10E-3), kN		Shear forces (scaled up 5*10E-3), kN		Bending moment (scaled up 5*10E-3), kNm	
Column type	Max	Min	Max	Min	Max	Min
Interior	152.2	-170.5	199.9	-208.6	291	-292
Exterior	66.19	-115.6	103.5	-104.7	105.5	-111.4
Corner	-30.17	-77.9	55.48	-57.2	78.13	-82.09

4.2.2 Bearing capacity check through software

For bearing capacity check, a demo version of GEO5 software was used, specifically the Pile Group extension. The setup was the following: Standard - factor of safety setting, soil profile was inputted and assigned. Next, the pile sizing and geometry was input according to the dimensions in the above section. The loads were added and then vertical capacity was calculated. These set up parameters are shown in the figures below.

Analysis settings : Standard - safety factors

Materials and standards
Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard
Circle pile shear : simplified method
Steel structures : EN 1993-1-1 (EC3)
Partial factor on bearing capacity of steel cross section : $\gamma_{M0} = 1,00$

Pile Group
Verification methodology : Safety factors (ASD)
Vertical bearing capacity : NAVFAC DM 7.2
Efficiency of pile group : La Barré (CSN 73 1002)

Select settings
Settings administrator
Add to administrator
Edit

Analysis type : analytical solution

Figure 4.26. Settings parameters in GEO5.

No.	Thickness of layer t [m]	Depth z [m]
1	7,60	0,00 .. 7,60
2	0,80	7,60 .. 8,40
3	1,50	8,40 .. 9,90
4	1,10	9,90 .. 11,00
5	6,60	11,00 .. 17,60
6	-	17,60 .. ∞

No.	Soil name
1	clay
2	sandy clay
3	sandy clay 2
4	silty clay yes
5	sandy clay 3

Unit weight of str. : $\gamma = 24,50$ [kN/m³]

Concrete
Catalog User def.
C 20/25
 $f_{ck} = 20,00$ MPa
 $f_{ctm} = 2,20$ MPa
 $E_{cm} = 30000,00$ MPa
 $G = 12500,00$ MPa

Longitudinal reinforcement
Catalog User def.
B500B
 $f_{yk} = 500,00$ MPa

Transverse reinforcement
Catalog User def.
B500B
 $f_{yk} = 500,00$ MPa

Figure 4.27. Soil parameters in GEO5.

No.	Load	Name	N [kN]	M _x [kNm]	M _y [kNm]	H _x [kN]	H _y [kN]	M _z [kNm]	Type
1	new edit	Load No. 1	12111,00	981,00	-1077,00	-293,60	-232,00	-2,00	Design
2	Yes	exterior	9877,00	997,00	-1065,00	282,00	247,00	-1,92	Design
3	Yes	corner	4936,00	987,00	1031,00	249,00	235,00	2,08	Design

Figure 4.28. Load settings in GEO5.

The final vertical bearing capacity of piles were calculated and shown in figures below.

Interior column:

Exterior column:

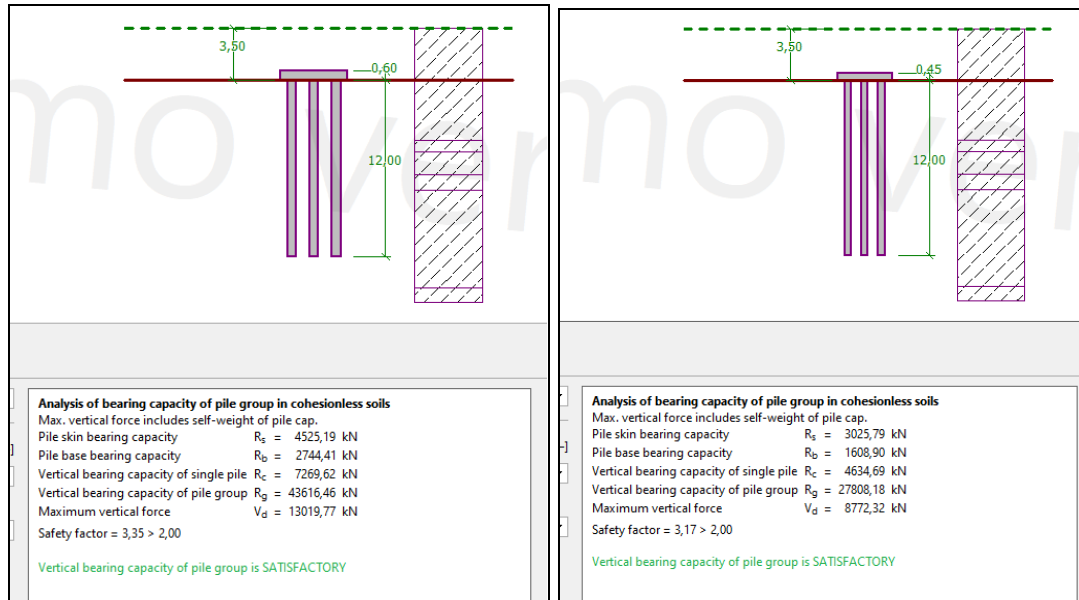


Figure 4.29. Bearing capacity for interior columns. Figure 4.30. Bearing capacity for exterior column.

Corner column:

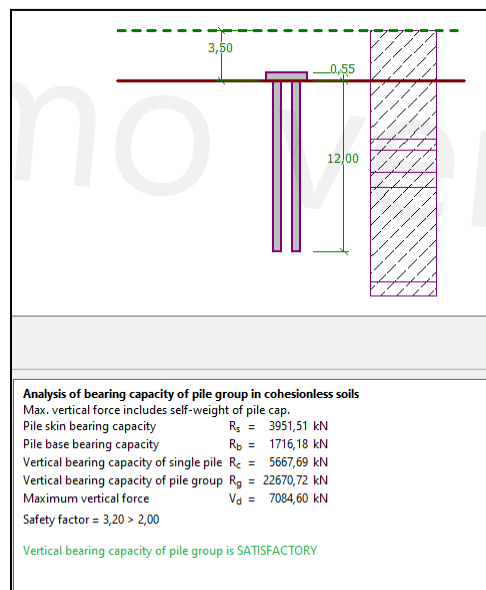


Figure 4.31. Bearing capacity for corner columns.

The output values were summarized and compared to hand calculations of bearing capacity in the table below.

Table 4.32. Comparison of manual calculations with GEO5 software.

	Interior	Exterior	Corner
--	----------	----------	--------

	Hand calculations	GEO5	Hand calculations	GEO5	Hand calculations	GEO5
Qs(single)	3431.3	4525.0	3068.0	3025.8	3559.1	3951.5
Qp(single)	2530.0	2744.4	1231.1	1608.9	1505.9	1716.2
Qu (single)	5961.3	7269.4	4299.1	4634.7	5065.0	5667.7
Qu (group)	41214.0	43616.5	25794.3	27808.2	20259.9	22670.7
Qall	13738.0	13019.7	8598.1	8772.3	6753.3	7084.6
FOS	3.0	3.35	3.0	3.17	3.0	3.20
Design load	12777.0		8268.3		4936.8	

This table reveals that the highest values were produced by GEO5 rather than manual calculations. This can be explained by the fact that a software uses complex integration in calculations along with a higher factor of safety, while hand calculations use approximations.

4.2.3 Settlement check through softwares

Moreover, the settlement check was completed for the piles. Based on the previous settings and inputs from both PLAXIS 3D and GEO5, the following settlements were recorded in table 4.33 below.

Table 4.33. Settlements for columns with three methods: manual calculation, Geo5, PLAXIS 3D.

Column type	Settlement (m), method of calculation		
	Hand calculation	Geo5	PLAXIS 3D
Interior column	0.0213	0.0237	0.01973
Exterior column	0.0182	0.0199	0.01458
Corner column	0.0117	0.0153	0.0097

Results show that GEO5 has the highest settlement values, while PLAXIS 3D values are lower than hand calculations. This can be connected to the fact that GEO5 integrated a higher factor of safety, therefore, the vertical loads were a bit higher and the settlements too. In terms of PLAXIS 3D, the settlement results are the lowest among the other two methods. This can be connected to the fact that a very coarse mesh generation was chosen, and if the mesh was medium or fine, the results could have been more accurate. However, all of the settlement values are appropriate to the Eurocode requirement of 25mm with the interior column settlement being close to the limit settlement.

4.5 Sheet pile design

For our building sheet pile design was chosen as a structure for lateral load bearing due to several factors:

- After thorough analysis in PLAXIS 3D, at the “Construction stage” horizontal displacement after applying surcharge load was determined to be 37mm, which is more than 25mm.
- This type of structure is more available and it is comparatively easy to construct.
- Sheet pile is more effective in terms of time, cost and manpower.

According to Bowles (1994), there can be driven or poured sheet piles, but pushed into one is more often used. For this project as a retaining wall steel sheet piles were chosen because of its recyclable function and it is more prone to sulfate attack. Moreover it is more financially efficient and comparatively low weight (Das & Sivakugan, 2019). Das & Sivakugan (2019) stated that, there can be either backfilled or dredged construction of sheet piles, and since our soil contains cohesive layers with some sandy clay and gravel and the water table is at very low depth, a backfilled method is the best choice for us.

4.5.1 Hand calculations

First, Rankine earth pressure coefficients was calculated as below:

$$K_a = \tan^2(45 - \frac{\phi'}{2})\text{-active} \quad (4.60)$$

$$K_p = \tan^2(45 + \frac{\phi'}{2})\text{-passive} \quad (4.61)$$

The depth of the basement of our building is 3.5m and this will be the minimum length of our designed sheet pile. The soil layer where the pile will be located is clay combined with some gravel and sand. Soil parameters used for sheet pile design $\phi' = 40^\circ$ and $\gamma = 20.9 \text{ kN/m}^3$.

So Rankine earth pressure calculations are shown below:

$$K_a = \tan^2(45 - \frac{40}{2}) = 0.217$$

$$K_p = \tan^2(45 + \frac{40}{2}) = 4.599$$

But, as it was proposed by Das & Sivakugan (2019) factor of safety of 2 was applied to get $K_{p(\text{design})}$:

$$K_{p(\text{design})} = \frac{K_p}{FS} \quad (4.62)$$

So calculated $K_{p(\text{design})}$ will be:

$$K_{p(\text{design})} = \frac{K_p}{FS} = \frac{4.599}{2} = 2.3$$

Active pressure was also determined at $L=3.5\text{m}$ as follows:

$$\sigma_2' = \gamma * L * K_a \quad (4.63)$$

$$\sigma_2' = 20.9 * 3.5 * 0.217 = 15.87 \text{ kN/m}^2$$

Next step involved finding L_3 where net pressure was equal to 0 ($\sigma' = 0$):

$$L_3 = \frac{L * K_a}{K_{p(\text{design})} - K_a} \quad (4.64)$$

$$L_3 = \frac{3.5 * 0.217}{2.3 - 0.217} = 0.365 \text{ m}$$

Next σ_5' was determined by following way:

$$\sigma_5' = \gamma * L * K_{p(\text{design})} + \gamma * L * (K_{p(\text{design})} - K_a) \quad (4.65)$$

$$\sigma_5' = 20.9 * 3.5 * 2.3 + 20.9 * 3.5 * (2.3 - 0.217) = 320.616 \text{ kN/m}^2$$

By following equation area of pressure diagram was determined:

$$P = 0.5 * \sigma_2' * L + 0.5 * \sigma_2' * L_3 \quad (4.66)$$

$$P = 0.5 * 15.87 * 3.5 + 0.5 * 15.87 * 0.365 = 30.67 \text{ kN/m}$$

Centroid of pressure diagram ($\bar{z}$):

$$\bar{Z} = \frac{L(2K_a - K_{p(design)})}{3(K_{p(design)} - K_a)} \quad (4.67)$$

$$\bar{Z} = \frac{3.5*(2*0.217-2.3)}{3(2.3-0.217)} = 1.045 \text{ m}$$

Calculations of A_1', A_2', A_3', A_4' :

$$A_1' = \frac{\sigma_5'}{\gamma(K_{p(design)} - K_a)} \quad (4.68)$$

$$A_1' = \frac{320.616}{20.9*(2.3-0.217)} = 7.365$$

$$A_2' = \frac{8P}{\gamma(K_{p(design)} - K_a)} \quad (4.69)$$

$$A_2' = \frac{8*30.67}{20.9(2.3-0.217)} = 5.634$$

$$A_3' = \frac{6P[2\bar{Z}\gamma(K_p - K_a) + \sigma_5']}{\gamma^2(K_{p(design)} - K_a)^2} \quad (4.70)$$

$$A_3' = \frac{6*30.67*[2*1.045*20.9*(2.3-0.217)+320.616]}{20.9^2(2.3-0.217)^2} = 35.56$$

$$A_4' = \frac{P[6\bar{Z}\sigma_5' + 4P]}{\gamma^2(K_{p(design)} - K_a)^2} \quad (4.71)$$

$$A_4' = \frac{30.67*[6*1.045*320.616+4*30.67]}{20.9^2(2.3-0.217)^2} = 34.516$$

Next, thing to determine is L_4 and it was done:

$$L_4^4 + A_1' * L_4^4 - A_2' * L_4^2 - A_3' * L_4 - A_4' = 0 \quad (4.72)$$

Therefore, by solving this equation we get $L_4 = 1.97 \text{ m}$

And D_{theory} value was obtained by:

$$D_{theory} = L_3 + L_4 \quad (4.73)$$

$$D_{theory} = 2.335 \text{ m}$$

Actual depth is illustrated by the 30% increased total length of sheet pile and calculated as:

$$L_{total} = (3.5 + 2.335) * 1.3 = 7.585 \text{ m}$$

$\sigma_3', \sigma_4', L_5$ was calculated:

$$\sigma_3' = L_4 * (K_{p(design)} - K_a)\gamma \quad (4.74)$$

$$\sigma_3' = 1.97 * (2.3 - 0.217) * 20.9 = 85.763 \text{ kN/m}^2$$

$$\sigma_4' = \sigma_5' + L_4 * \gamma * (K_{p(design)} - K_a) \quad (4.75)$$

$$\sigma_4' = 320.616 + 1.97 * 20.9 * (2.3 - 0.217) = 406.38 \text{ kN/m}^2$$

$$L_5 = \frac{\sigma_3' * L_4 - 2P}{\sigma_4' + \sigma_3'} \quad (4.76)$$

$$L_5 = \frac{85.763 * 1.97 - 2 * 30.67}{85.763 + 406.38} = 0.219 \text{ m}$$

Zero shear force was obtained by:

$$z' = \sqrt{\frac{2P}{\gamma(K_{p(design)} - K_a)}} \quad (4.77)$$

$$z' = \sqrt{\frac{2 * 30.67}{20.9(2.3 - 0.217)}} = 1.187 \text{ m}$$

And maximum bending moment was estimated as following:

$$M_{max} = P(\bar{z} + z') - [0.5 * \gamma * z'^2 (K_{p(design)} - K_a)] * \frac{1}{3} * z' \quad (4.78)$$

$$M_{max} = 30.67(1.187 + 1.045) - [0.5 * 20.9 * 1.187^2 * (2.3 - 0.217)] * \frac{1}{3} * 1.187$$

$$M_{max} = 56.32 \text{ kN} \cdot \text{m/m}$$

To determine which section to choose, section modulus and maximum bending moment will be considered (Das & Sivakugan, 2019). Allowable stress was chosen as 170 MN/m^2 according to ASTM A-328 and section modulus was found by:

$$S = \frac{M_{max}}{\sigma_{all}} \quad (4.79)$$

$$S = \frac{56.32}{170000} = 3.27 * 10^{-4} \text{ m}^3/\text{m of the wall}$$

Based on the section modulus that was obtained, the PZC-26 section was chosen. Here is the section dimensions:

Table 4.34. PZC-26 section dimensions

Width	708.2 mm
Height	449.6 mm

Thickness of the flange	15.24 mm
Thickness of the web	13.34 mm
Weight	2.47 kN/m ²

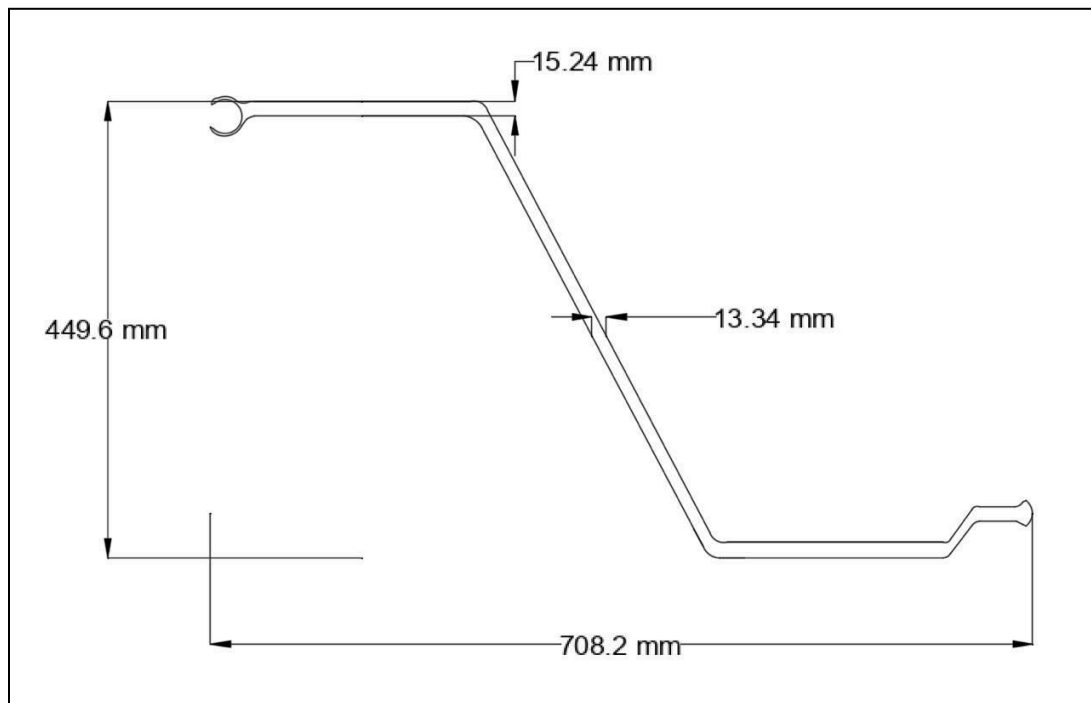


Figure 4.32. PZC-26 cross sectional view

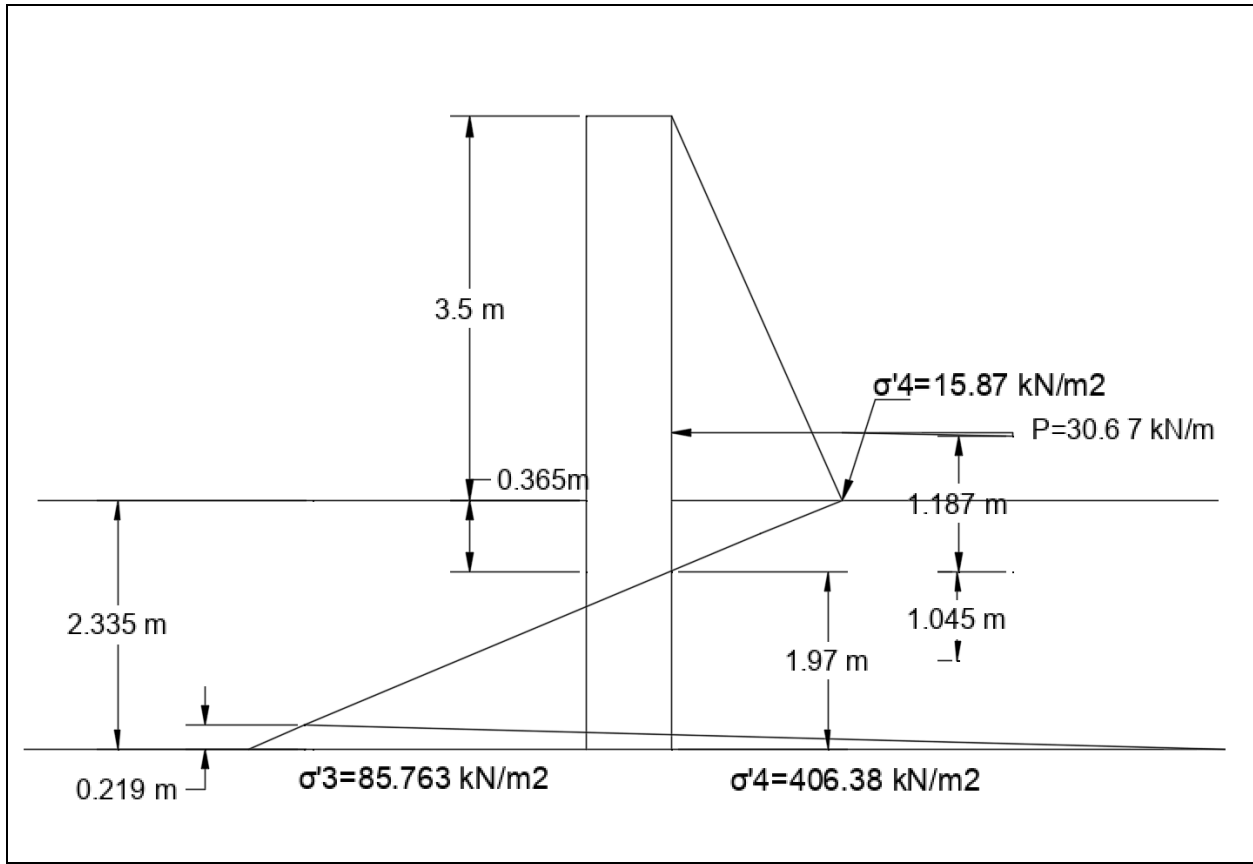


Figure 4.33. Pressure Diagram & Sheet Pile design

4.5.2 Software analysis

Results from hand calculations were compared with GEO 5 software results. Maximum displacement and maximum pressure was calculated using the program. Excavation depth was designed to be 3.5m, and soil profile was built accordingly.

Maximum bending moment, shear force and displacement were determined using GEO5 software. Geo5 calculations of maximum displacement showed that it is 11.4mm.

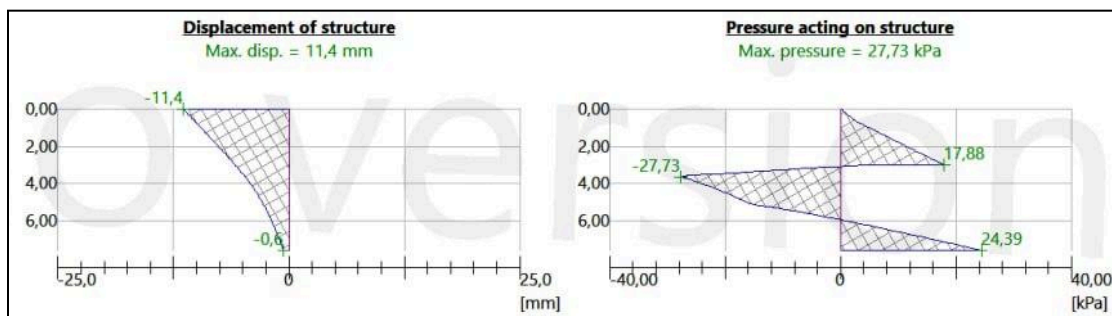


Figure 4.34. Displacement of the structure and pressure diagram taken from GEO5

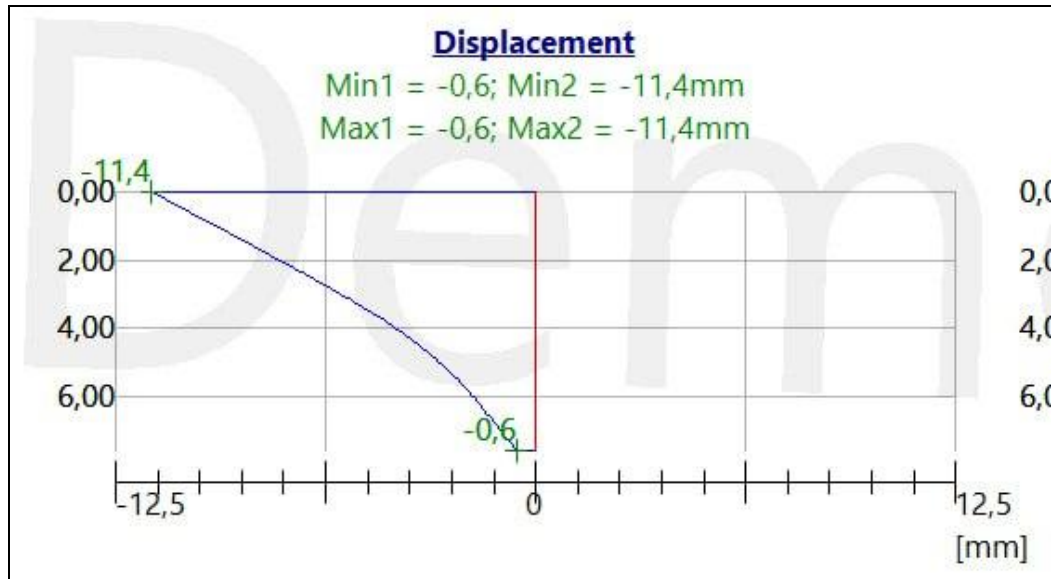


Figure 4.35. Maximum displacement detailed graph

Along with displacement maximum shear force (131 kN/m) and bending moments (77 kNm/m) were calculated.

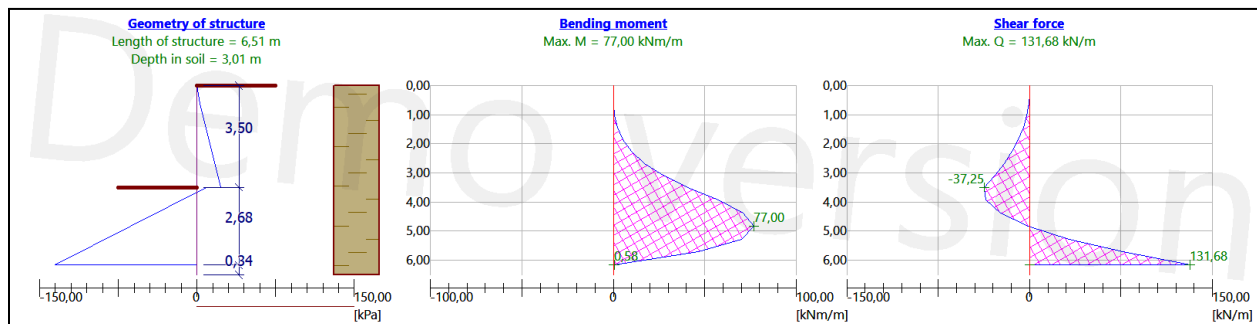


Figure 4.36. Geometry of the structure, Shear force and Bending Moment results

Max. value of shear force	= 131,68 kN/m
Max. value of moment	= 77,00 kNm/m
Required depth of structure in soil	= 3,01 m
Overall length of structure	= 6,51 m

Figure 4.37. Shear force and Bending Moment results

And GEO5 demonstrated that these results are satisfactory and sheet pile design is efficient for the structure.

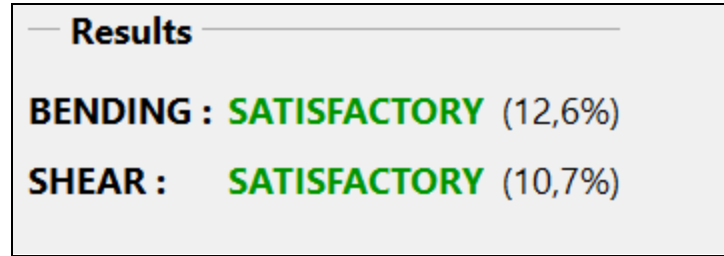


Figure 4.38. Bending moment and shear strength satisfactory results

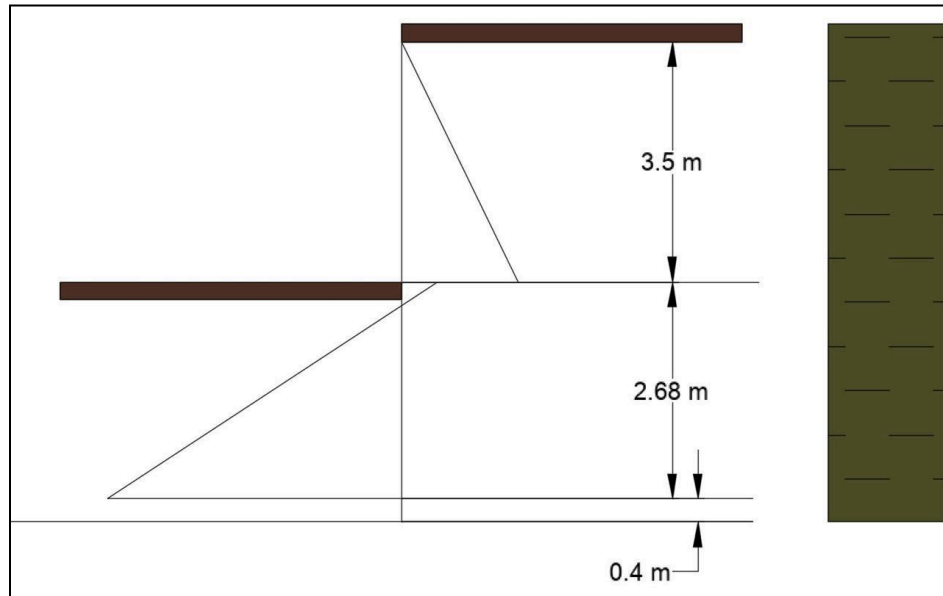


Figure 4.39. Sheet pile side view

4.6 Group of piles reinforcement design

4.6.1. Pile cap reinforcement

Preliminary pile cap design was calculated using the following formulas:

$$h(\min)=1.5D \quad (4.80)$$

$$d=3D \quad (4.81)$$

$$L=d*(n1-1)+2h \quad (4.82)$$

$$B=d*(n2-1)+2h \quad (4.83)$$

Table 4.35. Tables with preliminary dimensions for pile cap under interior, exterior, corner columns

Pile cap	Interior column	Exterior column	Corner column
D, mm	500	450	500
h, mm	750	675	750

d, mm	1500	1350	1500
L, mm	4500	4050	3000
B, mm	3600	2700	3000

Also, analysis of loads acting on each pile type was conducted and unit weight of concrete and backfill was taken as 24 kN/m³ and 17.46 kN/m³.

$$W_{pc} = h * \gamma_{concrete} = 18$$

$$W_{bf} = h * \gamma_{soil} = 8.73$$

Table 4.36. Interior group piles analysis

$DL, kN/m^2$	$LL, kN/m^2$	$W_{pc}, kN/m^2$	$W_{bf}, kN/m^2$	$W_{sc}, kN/m^2$
128.3	92.55	18	8.73	212.98

To design pile cap following equations were used:

$$LC_1 = 1.2D + 1.6L \quad (4.84)$$

$$LC_2 = 1.2D + 1L + 1W \quad (4.85)$$

$$LC_1 = 1.2D + 1L + 1E_x \quad (4.86)$$

Total W can be found using following formula:

$$W = W_{pc} + W_{bf} + W_{sc} = 18 + 8.73 + 212.98 = 239.71 kN/m^2 \quad (4.87)$$

Following factors were applied for further moment calculations:

$$1) \quad 1.2W = 287.652 kN/m^2$$

$$M'_{11} = M'_{22} = \frac{4.5 * 287.652 * 1.25^2}{2} = 674.18 kN - m$$

$$2) \quad 1.4W = 335.594 kN/m^2$$

$$M'_{11} = M'_{22} = \frac{4.5 * 335.594 * 1.25^2}{2} = 1179.822 kN - m$$

By using formulas reaction forces were found.

Q_1, Q_2, Q_3, Q_4 :

$$Q = \frac{P}{R} \pm \frac{M_{xx}y}{I_{xx}} \pm \frac{M_{yy}x}{I_{yy}} \quad (4.88)$$

Where,

$$M_{xx} = M_x + Ne_y + H_y h + M_x \quad (4.89)$$

$$M_{yy} = M_y + Ne_x + H_x h + M_y \quad (4.90)$$

$$P = 1.2/1.4 * N + W_{pc} + W_{bf} \quad (4.91)$$

$$I_{xx} = \sum y^2 \quad (4.92)$$

$$I_{yy} = \sum x^2 \quad (4.93)$$

Centroid were chosen to be $x=1.55m$ and $y=1.55m$, thus bending moments in pile cap were evaluated to be:

$$M_{11}'' = 1.4 * (Q_3 + Q_4) = 1.4(1325.93 + 1324.88) = 3711.134 \text{ kN} - m \quad (4.94)$$

$$M_{22}'' = 0.5 * (Q_1 + Q_2 + Q_3) = 1749.56 \text{ kN} - m \quad (4.95)$$

So, thus the sum of bending moments will be:

$$M_{11} = M_{11}' + M_{11}'' = 1179.822 + 3711.134 = 4890.956 \text{ kN} - m$$

$$M_{22} = M_{22}' + M_{22}'' = 1179.822 + 1749.56 = 2929.382 \text{ kN} - m$$

Reinforcing bars diameter were chosen to be 40mm for y direction and 25mm for y direction.

Yielding strength of steel rebar is $f_y = 460 \text{ N/mm}^2$ and concrete's compressive strength is

$f_{cu} = 35 \text{ N/mm}^2$. According to ACI-318-19, the minimum concrete cover for structures which have continuous contact with the ground is 75mm. After this, area of reinforcement in the pile cap could be found by:

$$d_x = h(\text{pile cap}) - \text{cover} - 0.5 * d(\text{bar}) \quad (4.96)$$

$$d_x = 750 - 75 - 0.5 * 40 = 655 \text{ mm}$$

$$K = \frac{M_{11}}{f_{cu} b d^2} = \frac{4890.956}{35 * 3600 * 655^2} = 0.104 * 10^6$$

$$z = d * (0.5 + \sqrt{(0.25 - \frac{K}{0.9})}) \leq 0.95d \quad (4.97)$$

$$z = 655 * (0.5 + \sqrt{(0.25 - \frac{0.104}{0.9})}) = 546.79 \leq 0.95d = 622.25$$

$$A_{st} = \frac{M_{11}}{0.87f_y z} = \frac{4890.95}{0.87*460*546.79} = 22350.93mm^2 \quad (4.98)$$

$$\text{Number of the bars} = \frac{A_{st}}{A_{bar}} = \frac{22350.93}{1257} = 17.78 \quad (4.99)$$

$$\text{Spacing} = \frac{4500}{18} = 250mm$$

Use 18#40 mm with 250 mm spacing

For y-direction same iterations were done:

$$d_y = h(\text{pile cap}) - \text{cover} - 0.5 * d(\text{bar}) \quad (4.100)$$

$$d_y = 750 - 75 - 40 - 0.5 * 25 = 622.5 mm$$

$$K = \frac{M_{11}}{f_{cu} b d^2} = \frac{4890.956}{35*4500*622.5^2} = 0.0801 * 10^6$$

$$z = d * (0.5 + \sqrt{(0.25 - \frac{K}{0.9})}) \leq 0.95d \quad (4.101)$$

$$z = 622.5 * (0.5 + \sqrt{(0.25 - \frac{0.0801}{0.9})}) = 560.87 \leq 0.95d = 591.375$$

$$A_{st} = \frac{M_{11}}{0.87f_y z} = \frac{4890.95}{0.87*460*560.87} = 21789.83mm^2 \quad (4.102)$$

$$\text{Number of the bars} = \frac{A_{st}}{A_{bar}} = \frac{21789.83}{491.44} = 44.34 \quad (4.103)$$

$$\text{Spacing} = \frac{4500}{45} = 100mm$$

Use 45#25 mm with 100 mm spacing

Below, summarized table is presented with pile cap design reinforcement:

Table 4.37. Reinforcement of the pile cap design

Pile group	h, mm	d_x , mm	d_y , mm	Design in x axis	Design in y axis
Interior	750	655	622.5	18#40 mm with 250 mm spacing	45#25 mm with 100 mm spacing

Exterior	675	504	470.3	42#40 mm with 127 mm spacing	43#25 mm with 98 mm spacing
Corner	750	569	550.2	30#40 mm with 83 mm spacing	16#25 mm with 131 mm spacing

Additionally, pile cap design was drawn on AutoCAD software to have a precise look on the pile cap. It is shown in the figures below.

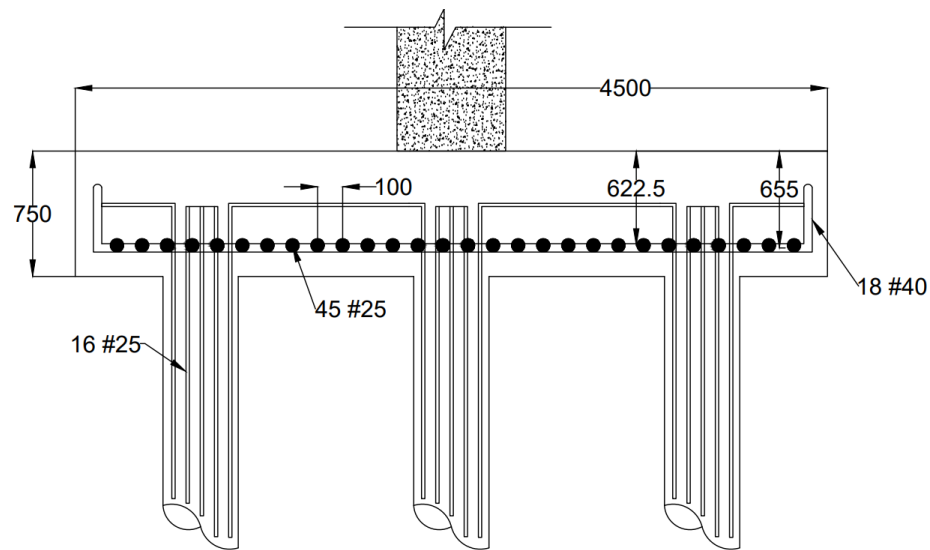


Figure 4.40. Pile cap design dimensions in mm for interior column (side view).

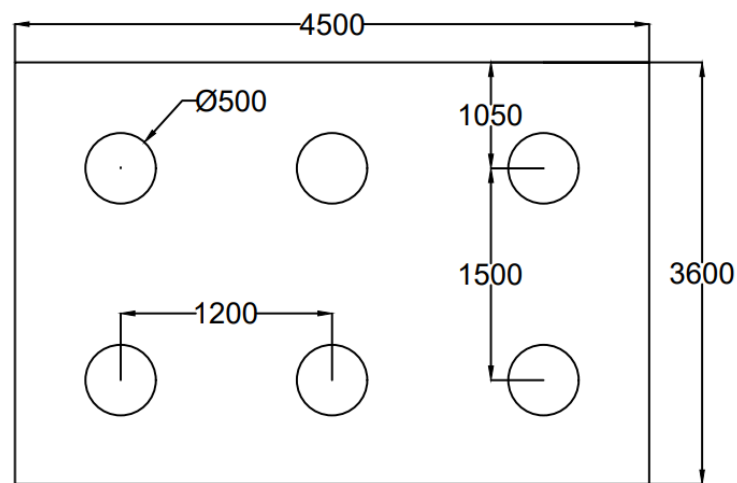


Figure 4.41. Pile cap design dimensions in mm for interior column (top view).

4.6.2 Single pile reinforcement

Every pile in the pile group should have such reinforcement that it can resist applied load on it. In this section reinforcement for each pile type will be provided. For that, firstly, a slenderness check should be done to check if the moment needs to be expanded.

$$l_e = \beta * l_0 = 1.2 * 12 = 14.4 \quad (4.104)$$

Where,

l_e –effective pile length, m

$\beta = 1.2$ –effective length factor, for fixed piles in pile cap

l_0 –unsupported pile length, m

$$\frac{l_e}{h} = \frac{14.4}{0.5} = 28.8 > 10, \text{ thus it is slender pile} \quad (4.105)$$

$$a = \frac{l_e^2}{2000h} K = \frac{14.4^2}{2000*0.5} * 1 = 0.207 \text{ m} \quad (4.106)$$

$$M_{add} = Q_{min} * a \quad (4.107)$$

$$M_{add} = 987.49 \text{ kN} * 0.207 = 204.76 \text{ kN} - \text{m}$$

$$M_{mag} = M + M_{add} \quad (4.108)$$

$$M_{mag} = 78.4 + 204.76 = 283.16 \text{ kN} - \text{m}$$

$$e = \frac{M_{mag}}{Q_{min}} = \frac{283.16}{987.49} = 0.287 \text{ m} \quad (4.109)$$

$$\frac{e}{R} = \frac{0.287}{0.25} = 1.147 \quad (4.110)$$

$$\frac{M}{h^3} = \frac{283.16}{0.5^3} = 2265 \text{ N/m}^2 \quad (4.111)$$

$$k = \frac{h_s}{h} = 0.78 \quad (4.112)$$

This means reinforcement used should be minimum which is 1%:

$$A_{st} = \frac{A_c * 1}{100} = 196349.5 \text{ mm}^2$$

Use 16#25 mm with 150 mm spacing

To figure out whether to use shear reinforcement or not, following check was done:

$$\frac{M(max)}{Q(max)} = \frac{283.13}{1952.62} = 0.145 < 0.6h = 0.3, \text{ thus there is no need to do shear check}$$

In the table below, reinforcement design of each individual pile is presented:

Table 4.38. Reinforcement design for the group of piles.

Pile group	D, m	l_e/h	M/h^3	ρ	M_{add}	M_{mag}	A_{sc}, mm^2	Design
Interior	0.5	28.8	2.265	1%	204.76	283.16	1963,49	16#25mm
Exterior	0.45	31.4	2.492	2.8%	254.33	275.62	1563,21	14#20mm
Corner	0.5	30.5	2.534	2.9%	290.73	295.12	1263,8	18#25mm

4.6.3 Program analysis on Geo 5

The design that was chosen above was checked and analyzed in the Geo 5 program for interior, exterior, corner piles and the results are shown below. All three design choices showed satisfactory levels.



Figure 4.42. Reinforcement of pile under Interior column



Figure 4.43. Reinforcement of pile under Exterior column



Figure 4.44. Reinforcement of pile under Corner column

4.7 Lateral load check for piles

As calculated in the structural design part of the project, there is a lateral load too, in addition to axial loading. Therefore, additional foundation design towards lateral load resistance is required.

4.7.1. Lateral load capacity

From the structural design calculations, the highest load was exerted on the piles under the column. The loads are presented in the table below.

Table 4.39. Lateral loads for columns

Column type	Load combination	Fx, kN	Fy, kN	Number of piles	Lateral load, kN
Corner	1.2D+L+S	293,62	232,98	4	58,24
Interior		282,13	247,17	6	41,20
Exterior		249,56	235,67	6	39,28

For the soil layers in the project, the following table was calculated.

Table 4.40. Soil constants for lateral load check.

Layer #	Depth z, m	Soil Type	Cohesion, kPa	Friction angle, °
1	0-7.6	CLAY	90	40
2	7.6-8.4	SANDY CLAY	24.3	40
3	8.4-9.9	SANDY CLAY	12.4	43

4	9.9-11	SILTY CLAY	21.9	41
5	11-17.5	SANDY CLAY	35	38

Next, the piles were calculated with the formula to identify whether they are short or long:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} = 1.345 \quad (4.113)$$

Where,

E_p is the pile modulus of elasticity in MPa

I_p pile moment of inertia

n_h modulus of horizontal subgrade reaction

In case of $L \geq 5T$ the pile is long, in case of $L \leq 2T$, the pile is rigid (Das & Sivakugan, 2019).

For a lateral load check in piles, Brom's method was used.

The modulus of elasticity was calculated as:

$$E_p = 4700 \sqrt{f_c} = 21\,540 \text{ MPa} \quad (4.114)$$

Where,

The compressive strength was taken as a minimum of 21 MPa at 28 days based on the California Building Code 2022 for seismic category D.

Next for the moment of inertia, the following formula was used:

$$I_p = \frac{\pi D^4}{64} = 0.00307 \text{ m}^4 \quad (4.115)$$

As for the n_h , the value for stiff clays was used - 15000 kN/m³ (Das & Sivakugan, 2019). In this case, the length of the pile of 12m exceeds the 5T value, therefore the pile is long.

The yield moment is calculated with a formula below:

$$\frac{M_y}{c_u D^3} \quad (4.116)$$

Where,

$$M_y = S F_y \quad (4.117)$$

And

$$S = \frac{I_p}{\frac{D}{2}} = \frac{\pi D^3}{32} \quad (4.118)$$

Which gave for corner columns:

$$M_y = 257.7 \text{ KNm}$$

The piles are restrained, because the pile head is not allowed to rotate. In the case of a long pile in clay layers, the following chart was used to estimate ultimate lateral resistance.

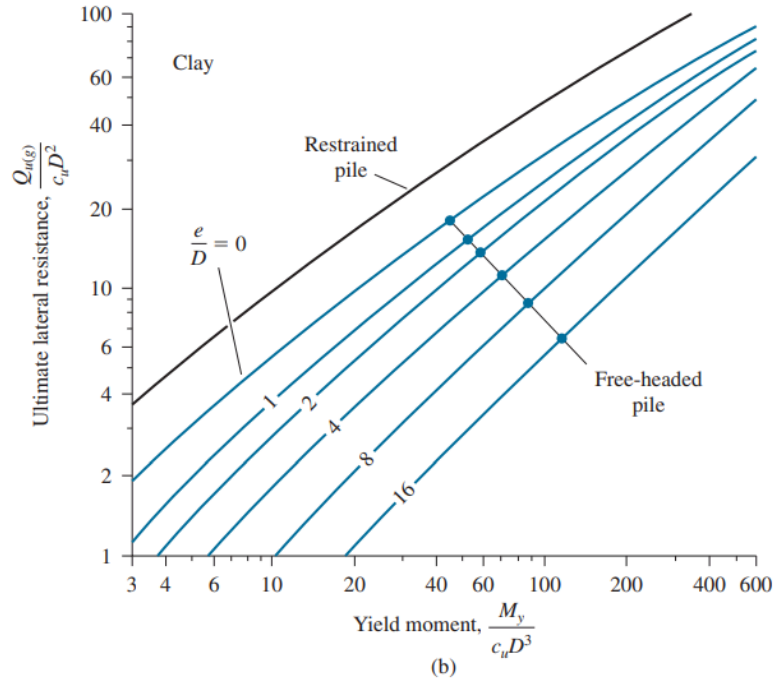


Figure 4.45. Chart for ultimate lateral resistance in long piles in clay (Das & Sivakugan, 2019).

To check pile-head deflection, the formula below is used:

$$\beta = \sqrt[4]{\frac{KD}{4E_p I_p}} \quad (4.119)$$

Where,

K is the horizontal soil modulus

$$K = \frac{\text{pressure (kN/m}^2\text{)}}{\text{displacement (m)}}$$

Based on the table of general values of K from textbook (figure 4.46), the value of K was taken as an interpolated value for each soil layer.

Unconfined compression strength, q_u kN/m ²	K kN/m ³
200	10,000–20,000
200–800	20,000–40,000
> 800	> 40,000

Figure 4.46. Values of K for clay soil (Das & Sivakugan, 2019).

Then, having obtained the values, the following chart is used to determine the dimensionless lateral deflection, and the lateral load from it.

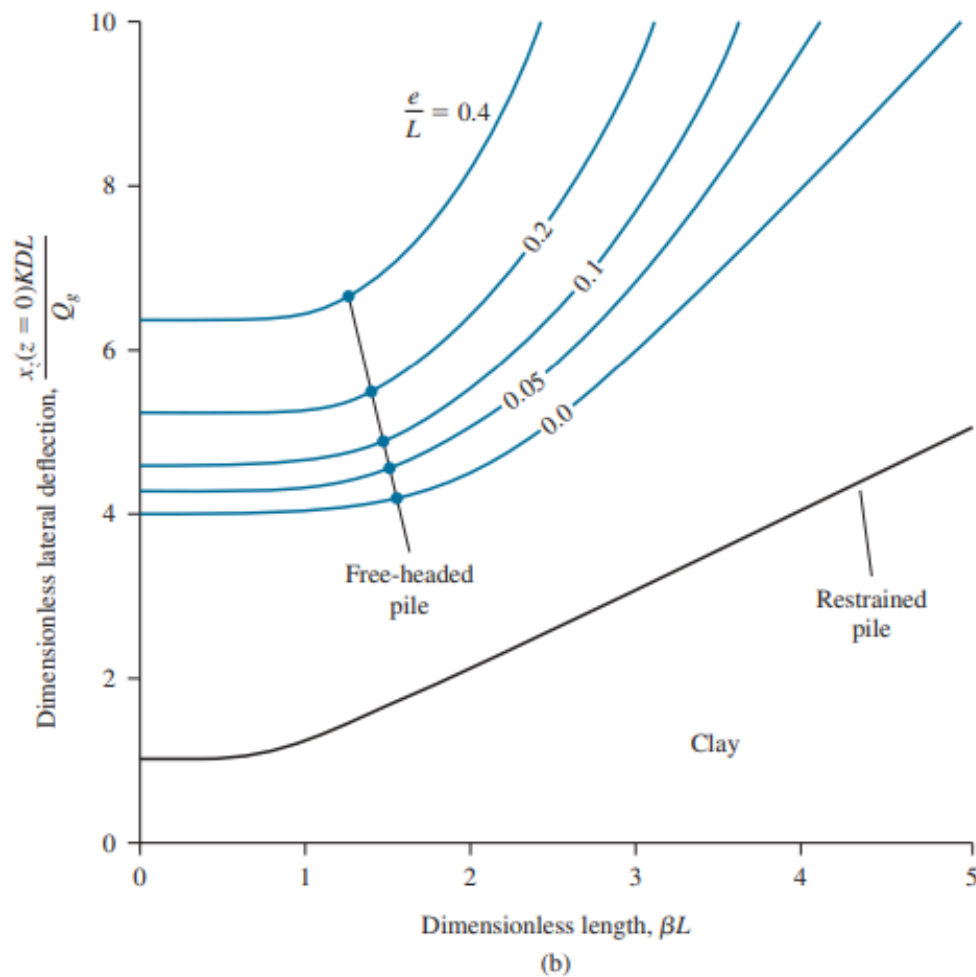


Figure 4.47. Chart of dimensionless lateral deflection for long piles in clay soil (Das & Sivakugan, 2019).

The lateral loads were then calculated and presented in the following tables.

Table 4.41. Allowable lateral loads for corner column type.

Layer #	Soil Type	Unconfined compressive strength, q_u , kN/m ²	Cohesion, kPa	$M_y/(c_u * D^3)$	Ultimate lateral resistance from chart	$Q_u(g)$, kN	Q_{all} , kN
1	CLAY	225.04	90	22.91	16	360.00	120.00
2	SANDY CLAY	215.46	24.3	84.84	43	261.23	87.08
3	SANDY CLAY	277.71	12.4	166.26	70	217.00	72.33
4	SILTY CLAY	287.28	21.9	94.14	45	246.38	82.13
5	SANDY CLAY	392.62	35	58.90	37	323.75	107.92

The same way allowable lateral loads for interior and exterior column types were calculated and presented in tables 4.42 and 4.43 below.

Table 4.42. Allowable lateral loads for interior column type.

Layer #	Soil Type	Unconfined compressive strength, q_u , kN/m ²	Cohesion, kPa	$M_y/(c_u * D^3)$	Ultimate lateral resistance from chart	$Q_u(g)$, kN	Q_{all} , kN
1	CLAY	225.04	90	22.91	16	435.60	145.20
2	SANDY CLAY	215.46	24.3	84.84	43	316.08	105.36
3	SANDY	277.71	12.4	166.26	70	262.57	87.52

	CLAY						
4	SILTY CLAY	287.28	21.9	94.14	45	298.11	99.37
5	SANDY CLAY	392.62	35	58.90	37	391.74	130.58

Table 4.43. Allowable lateral loads for exterior column type.

Layer #	Soil Type	Unconfined compressive strength, q_u , kN/m ²	Cohesion, kPa	$M_y/(c_u * D^3)$	Ultimate lateral resistance from chart	$Q_u(g)$, kN	Q_{all} , kN
1	CLAY	225.04	90	22.91	16	291.60	97.20
2	SANDY CLAY	215.46	24.3	84.84	43	211.59	70.53
3	SANDY CLAY	277.71	12.4	166.26	70	175.77	58.59
4	SILTY CLAY	287.28	21.9	94.14	45	199.56	66.52
5	SANDY CLAY	392.62	35	58.90	37	262.24	87.41

Based on the calculations above, the allowable lateral loads that can be supported by each soil layer were shown. Next, the smallest values of allowable lateral load were shown in the 3rd soil layer - sandy clay. This is why this value will be compared to the lateral loads as calculated by the structural design part in the table below.

Table 4.44. Comparison of allowable and actual lateral loads per each column type.

Column type	Qall (calculations), kN	Actual lateral load, kN
Corner	72.33	58.24
Interior	87.52	41.20
Exterior	58.59	39.28

The table shows that the actual lateral loads meet the allowable lateral load requirement, as they are smaller than the allowable value.

4.7.2 Lateral deflection calculation

For a lateral deflection calculation check the equations shown above were used.

For cohesive soils, the lateral deformation at any depth z is given by Davisson and Gill (1963)

$$x_z(z) = A'_x \frac{Q_g R^3}{E_p I_p} + B'_x \frac{M_g R^2}{E_p I_p} \quad (4.120)$$

And

$$M_z(z) = A'_m Q_g R + B'_m M_g \quad (4.121)$$

Where,

R is the characteristic length

The characteristic length R can be calculated with the formula below.

$$R = \sqrt[4]{\frac{E_p I_p}{k}} \quad (4.122)$$

For which k , the subgrade modulus is calculated by,

$$k \approx \frac{E_s}{1 - \mu_s^2} \quad (4.123)$$

For cohesive soils, the subgrade modulus of reaction is assumed to be constant throughout the depth for cohesive soils (Das & Sivakugan, 2019). Moreover, for cohesive soils the non dimensional depth Z is calculated by

$$Z = \frac{z}{R} \quad (4.124)$$

Where,

z is the depth of soil layer

R is the characteristic length

Moreover, to find the parameters A_x' and B_x' , $Z_{\max}$ should also be calculated:

$$Z_{\max} = \frac{L}{R} \quad (4.125)$$

Where,

L is the pile length

R is the characteristic length.

Having calculated all the necessary constants, the charts were used to estimate the parameters (Das & Sivakugan, 2019).

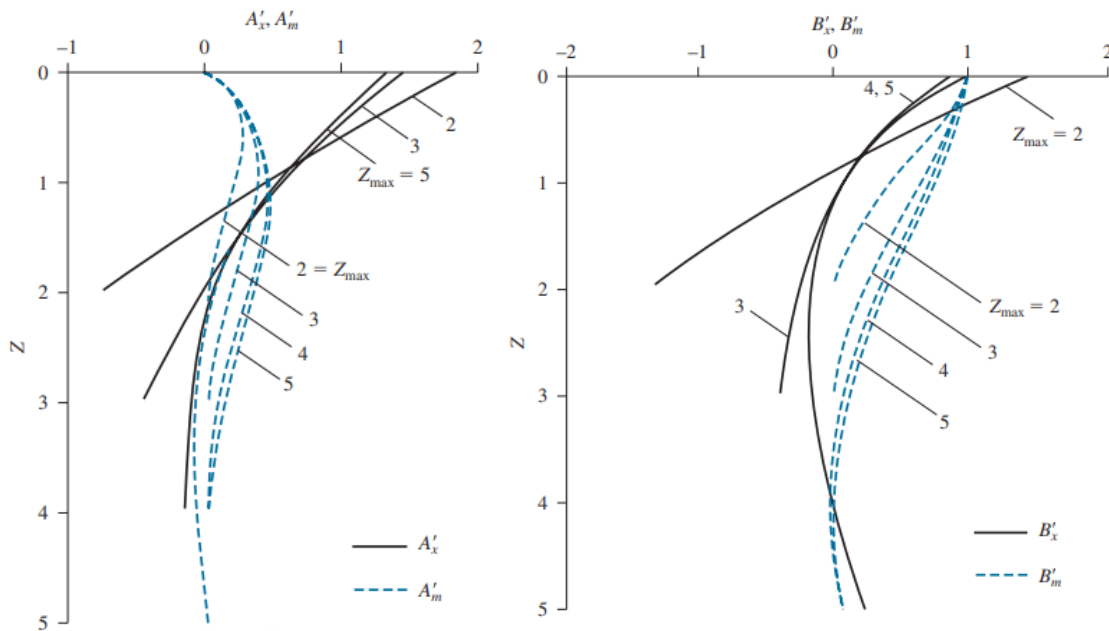


Figure 4.48. Chart of A_x' , A_m' , B_x' , B_m' parameters variation with depth Z (Das & Sivakugan, 2019).

This way, using the formulas and charts above, the lateral displacement at depth z for each column type was calculated below.

Table 4.45. Lateral deflection at each soil layer for corner column

Layer #	Soil Type	Depth z , m	Nondimen sional depth Z	$Z_{\max}$	A_x'	B_x'	$x_z(z)$, m
---------	-----------	------------------	---------------------------------	------------	--------	--------	--------------

1	CLAY	2	0.72	4.31	0.60	0.10	0.014
		4	1.44	4.31	0.30	-0.10	0.007
		7.6	2.73	4.31	0.10	-0.20	0.002
2	SANDY CLAY	8.4	3.01	4.31	0.00	-0.20	0.000
3	SANDY CLAY	9.9	3.58	4.34	0.00	-0.10	0.000
4	SILTY CLAY	11	3.96	4.32	0.00	0.00	0.000
5	SANDY CLAY	13	4.64	4.28	-0.10	0.20	0.000
		15	5.35	4.28	-0.10	0.40	0.000
		17.5	6.25	4.28	-0.10	0.60	0.000

Table 4.46. Lateral deflection for interior column.

Layer #	Soil Type	Depth z, m	Nondimen sional depth Z	Z max	Ax'	Bx'	xz(z), m
1	CLAY	2	0.65	3.91	0.60	0.10	0.019
		4	1.30	3.91	0.40	-0.10	0.013
		7.6	2.48	3.91	0.10	-0.20	0.003
2	SANDY CLAY	8.4	2.74	3.91	0.00	-0.20	0.000
3	SANDY CLAY	9.9	3.26	3.95	0.00	-0.10	0.000
4	SILTY CLAY	11	3.60	3.93	0.00	0.00	0.000

5	SANDY CLAY	13	4.22	3.89	0.00	0.00	0.000
		15	4.87	3.89	0.00	0.10	0.000
		17.5	5.68	3.89	0.00	0.20	0.000

Table 4.47. Lateral deflection for exterior column.

Layer #	Soil Type	Depth z, m	Nondimen sional depth Z	Z max	Ax'	Bx'	xz(z), m
1	CLAY	2	0.80	4.78	0.60	0.10	0.008
		4	1.59	4.78	0.30	-0.10	0.004
		7.6	3.03	4.78	0.10	-0.20	0.001
2	SANDY CLAY	8.4	3.35	4.78	0.00	-0.20	0.000
3	SANDY CLAY	9.9	3.98	4.83	0.00	-0.10	0.000
4	SILTY CLAY	11	4.40	4.80	0.00	0.00	0.000
5	SANDY CLAY	13	5.16	4.76	0.00	0.00	0.000
		15	5.95	4.76	0.00	0.10	0.000
		17.5	6.94	4.76	0.00	0.20	0.000

Summarizing the tables above, the lateral displacements for each column were gathered and compared below.

Table 4.48. Comparison of lateral displacement for corner, interior and exterior columns.

Layer #	Layer depth, m	Depth z, m	Lateral displacement xz(z) for columns, m		
			Corner	Interior	Exterior

1	7.6	-2	0.014	0.019	0.008
		-4	0.007	0.013	0.004
		-7.6	0.002	0.003	0.001
2	8.4	-8.4	0.001	0.001	0
3	9.9	-9.9	0	-0.001	0
4	11	-11	0	0	0
5	17.5	-13	0	0	0
		-15	0	0	0
		-17.5	0	0	0

These displacement values were plotted on a graph for visual representation.

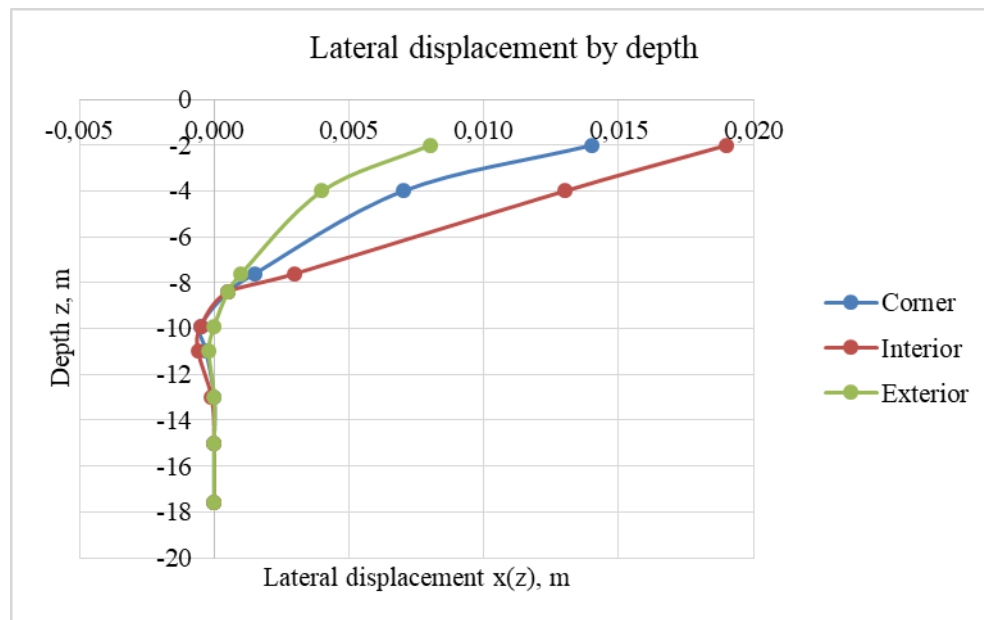


Figure 4.49. Lateral displacement by depth for corner, interior and exterior columns. PLAXIS 3D was then used to plot the layout of the building foundation.

4.8 Software Modeling

Moreover, the group of piles were modeled in PLAXIS 3D with the structural loads to overview the displacements and deflections along the pile. This also includes the amplifier, resulting from

the site response analysis dynamic multiplier. The displacements below show both the axial, lateral and dynamic multiplier effects on the piles.

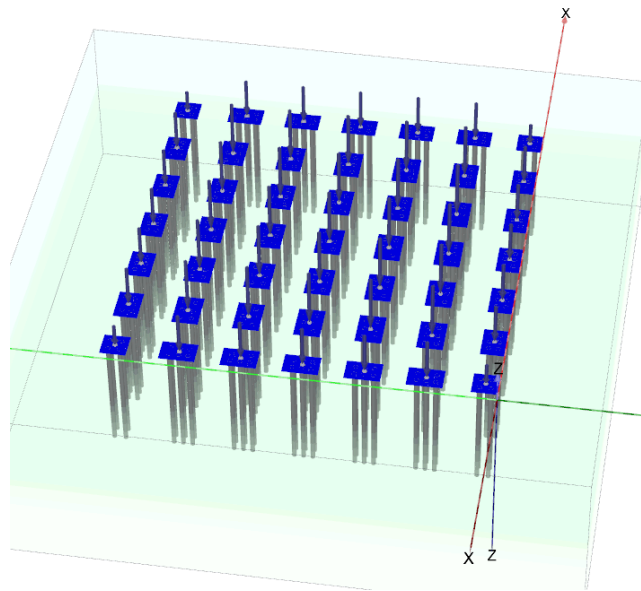


Figure 4.50. Layout of the building foundation in Plaxis 3D.

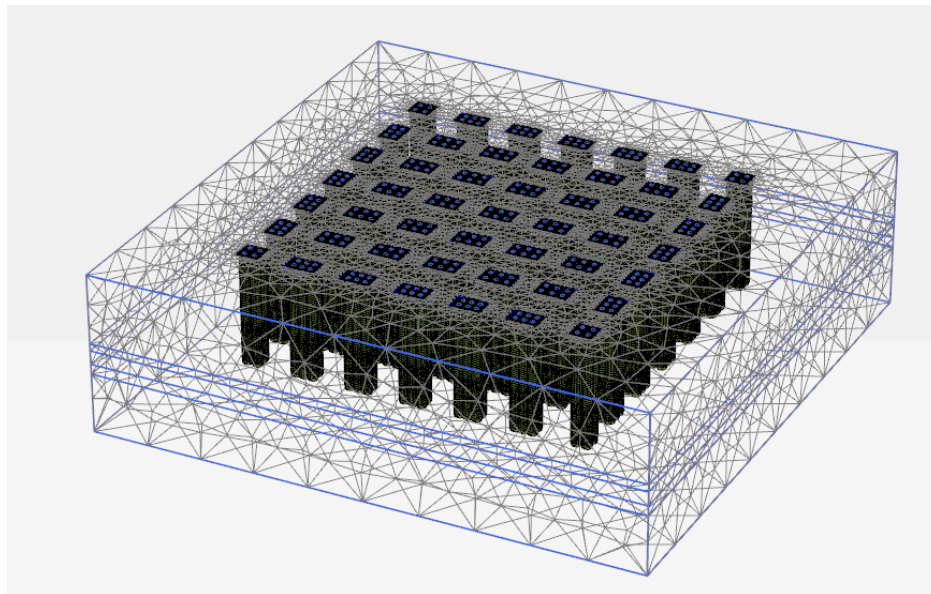


Figure 4.51. Generated output of the foundation layout in Plaxis 3D

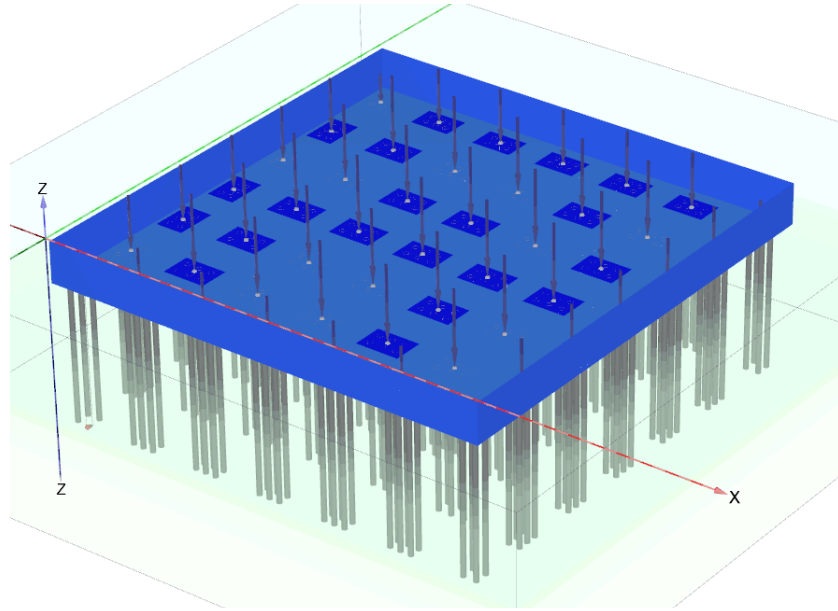
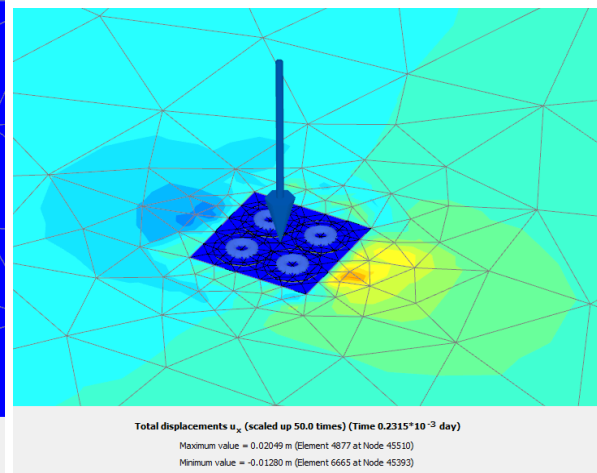
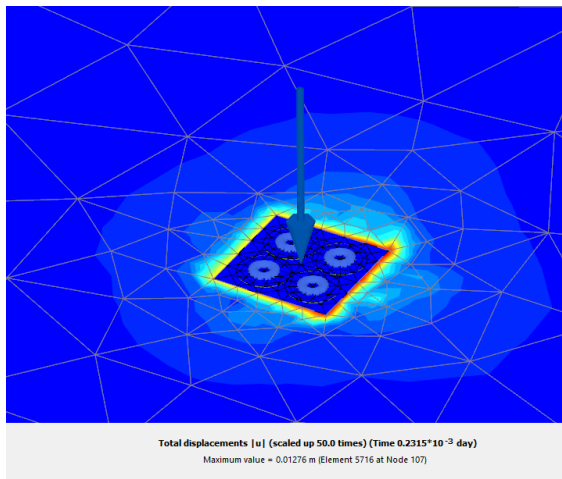


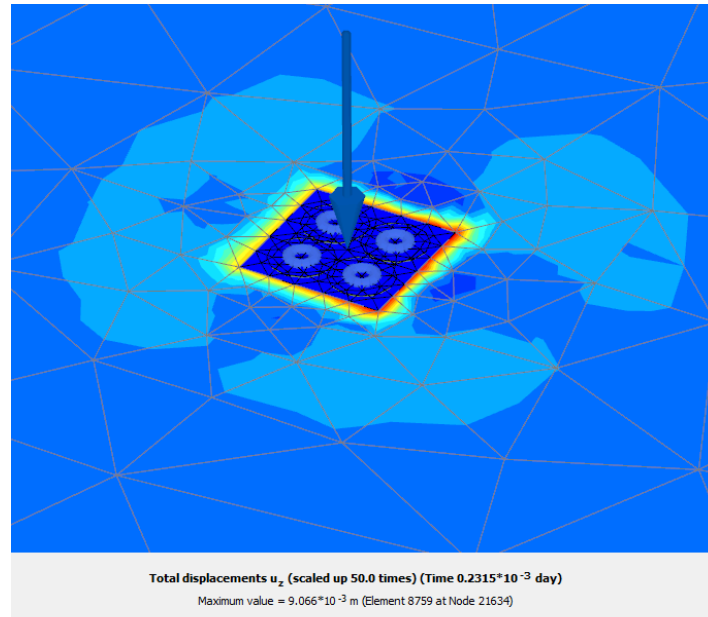
Figure 4.52. Foundation layout in Plaxis 3D

The following figures show the displacements in the x-axis and z-axis for each pile after an application of the lateral, axial and seismic load from site response analysis.

Corner column displacements:



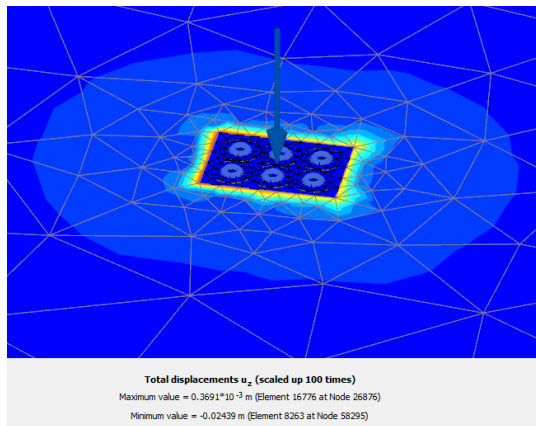
(a) Corner column total displacements (b) Corner displacement in u_x direction



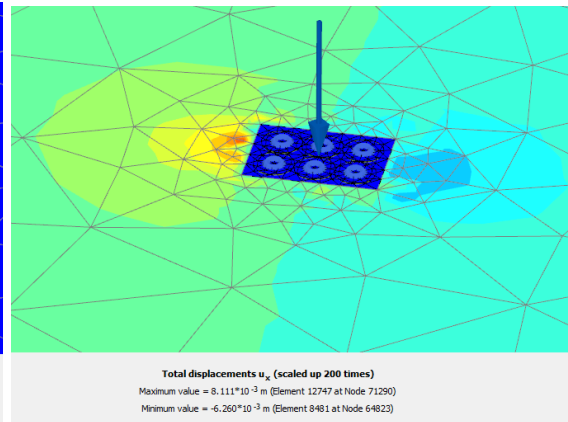
(c) Corner displacement in u_z direction

Figure 4.53. Corner column displacements in Plaxis 3D.

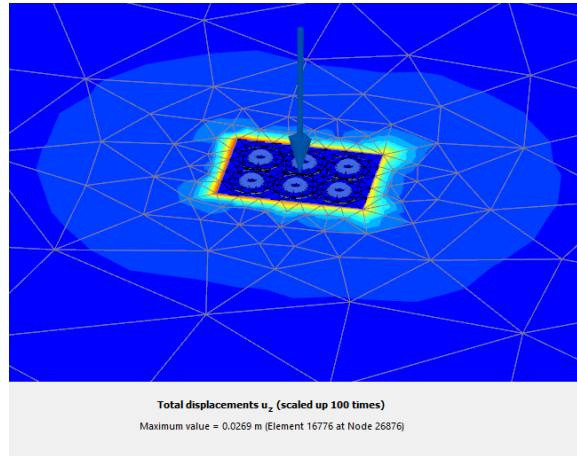
Interior column displacements:



(a) Interior column total displacement



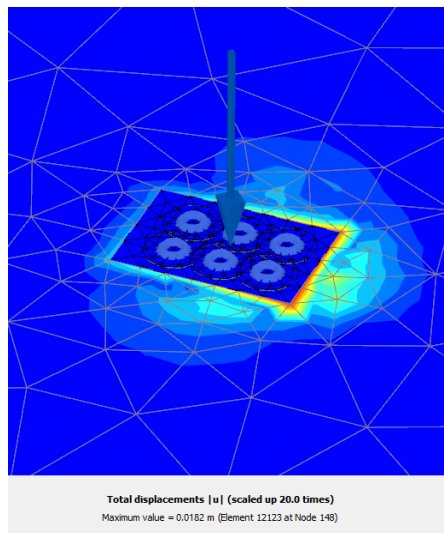
(b) Interior displacement in u_x direction



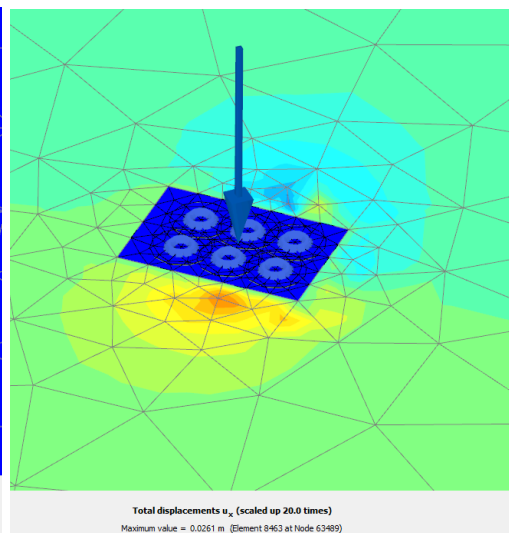
(c) Interior displacement in u_z direction

Figure 4.54. Interior column displacements in Plaxis 3D.

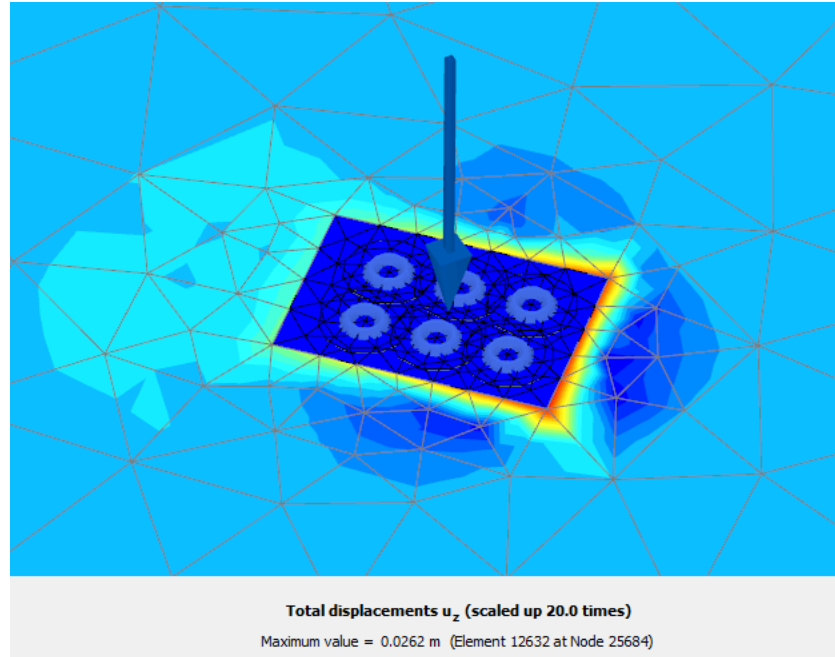
Exterior column displacements:



(a) Exterior column total displacement



(b) Exterior displacement in u_x direction



(c) Exterior displacement in u_z direction

Figure 4.55. Exterior column displacements in Plaxis 3D.

A table below shows a summary of the values for displacements obtained from Plaxis 3D

Table 4.49. Summary of displacement values from Plaxis 3D with axial, lateral and seismic loads.

Group pile	Total displacement u , m	Displacement in u_x , m	Displacement in u_z , m
Interior	0.02439	0.00811	0.0269
Exterior	0.0182	0.0261	0.0262
Corner	0.01276	0.00907	0.0128

The piles therefore, will have a following cross-section in the soil.

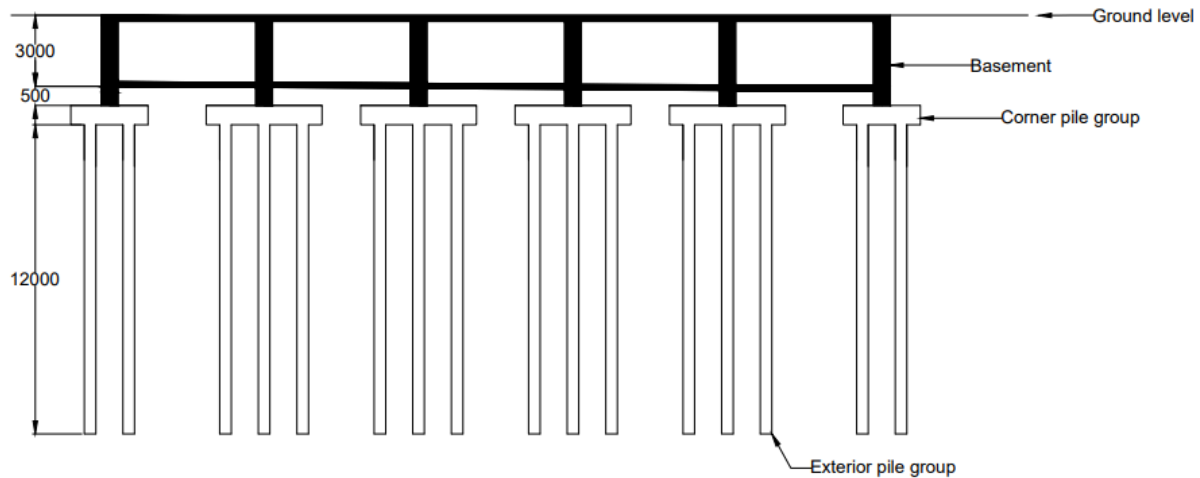


Figure 4.56. Side view of the piles and basement in a cross-section in the soil. The view shows corner and exterior piles at the exterior wall of the building.

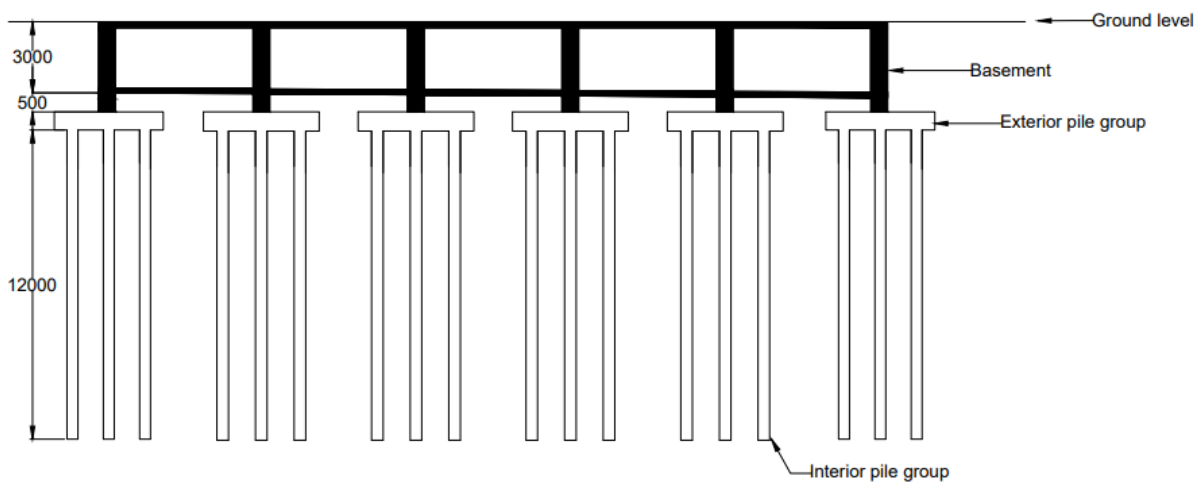


Figure 4.57. Side view of the piles and basement in a cross-section in the soil. The view shows exterior and interior piles at the interior of the building.

4.9 Construction procedure

The initial step of construction work includes removing every obstruction present at the site through *site clearing* operations. Heavy equipment including excavators and bulldozers scours through the site for complete demolition of buildings followed by stump extraction and tree removal and the removal of topsoil containing organic matter. Utilizing ground-penetrating radar technology teams locate hidden underground utilities that then get removed without

creating any obstacles within the work area. The project complies with environmental regulations in its waste segregation practices for material disposal.

The boundaries of the project site must be *surveyed* after clearing so that survey markers can establish precise spatial definitions. Survey professionals with licenses employ total stations together with GPS devices to install reference points for site boundaries that match approved architectural and engineering drawings. The phase requires strict verification of site arrangements to meet all local zoning laws together with right-of-way restrictions and set-back requirements to prevent present and future legal and structural complications.

Soil testing occurs next after setting boundaries to determine soil classification as well as its bearing capacity. The drilling of boreholes occurs at specific points across the site to collect samples that proceed to laboratory testing. Multiple laboratory tests evaluate the site's characteristics through measurements of moisture levels together with compactness and shearing abilities and permeability properties. The obtained results enable site drainage evaluation for determining structural and drainage system designs.

The *site planning* phase commences after finishing the collection of soil information. Site planning involves designing the allocation of site utilities together with stormwater management systems while establishing road systems and building footprint locations. Planners join forces with engineers to maintain proper access pathways and drainage flow and utility network connections. The design method utilizes natural topography as well as soil characteristics and zoning restrictions to maximize site space utilization and reduce amount of earth movement.

A preliminary plan functions as a basis for performing comprehensive *geotechnical investigations* needed for foundation designs. Cone penetration tests and laboratory analyses of deep boreholes serve to identify settlement loads and methods of soil support under different moisture and stress conditions. The geotechnical report suggests foundation type selection between shallow or deep systems together with necessary soil engineering solutions and site construction safety measures.

At the next step the construction team installs *sheet piles* which serve as excavation support until the project reaches this phase. Using the 'Pitch and Drive' method workers put each sheet pile into position before driving it into the ground one-by-one through vibratory or impact hammers. The installation method provides protection to excavation borders especially in cases of soft ground conditions or near bodies of water.

Sheet piles get installed first then *excavation* continues until reaching the depth noted in design drawings. Sheet pile installation demands elaborate management to stop excessive digging and keep excavation slopes steady or maintain vertical wall structures using sheet piles. Tested excavated soil undergoes classification to determine its reuse capacity or it must be transported elsewhere when deemed inappropriate for reuse. Well points and sump pumps serve as dewatering elements that prevent the accumulation of water when groundwater appears during excavations.

Pile installation starts promptly once the foundation reaches the pre-defined level. The installation process of precast concrete piles uses hydraulic and diesel hammers for driving into the ground. Foundations require driving piles to their design depths or until specific set criteria from pile driving analyses are met. Traders cut pile heads at the necessary height for future use in pile cap or tie beam construction.

Quality control alongside *safety assurance* stays as a priority factor at every point in time. Multiple assessments are executed continually to verify adherence to technical requirements that monitor pile alignment and recording of driving operations and concrete quality verification. The workplace team requires safety officers to maintain compliance by enforcing PPE use along with excavation supervision and by inspecting machinery and processes against occupational health and safety regulations. Toolbox meetings along with site presence allow the creation and maintenance of safe working conditions as well as operational efficiency and regulatory compliance throughout the entire project duration.

5. Environmental Engineering Part

5.1 Introduction

5.1.1 Project Background

Quick urban growth combined with changes in climate patterns has made urban stormwater management a critical need. Environmental engineering creates a complete storm sewer network for a mixed-use development located in Cupertino California to prevent flooding and infrastructure destruction and protect the environment from degradation. The system needs to successfully handle stormwater runoff from four main site areas comprised of 35% parking lots and 25% roadways together with 15% rooftops and 25% permeable landscaped areas.

Water flows in the northeastern direction due to a site slope of 1.5% that rises from 58.5 to 60.25 meters above sea level (Figure 3.1.1). We utilize gravity-fed drainage systems due to the topographic conditions which cuts down energy expenses relative to systems that need pumps.

5.1.2 Objectives

The primary objectives of this storm sewer design are:

1. Hydraulic Efficiency: Ensure 100-year storm event capacity (75 mm/hr intensity);
2. Regulatory Compliance: Meet Cupertino Sanitary District (2017) standards;
3. Cost Optimization: Minimize excavation and material costs by leveraging natural slopes;
4. Sustainability: Incorporate green infrastructure for 20% of runoff management.

Secondary objectives include:

1. Reducing peak runoff by 25% through permeable surfaces;
2. Achieving LEED credits for stormwater management;
3. Ensuring 50-year infrastructure lifespan.

5.1.3 Scope of Work

First Semester Deliverables:

1. Topographic mapping using Surfer 13 (Figures 3.4.1-3.4.3);
2. Preliminary pipe network layout in AutoCAD (Figure 3.2.2);
3. Cross-sectional profiles at 15m intervals.

Second Semester Deliverables:

1. Hydraulic calculations for 12 drainage sub-catchments;

2. Final pipe sizing and material specifications;
3. Rainwater harvesting system for parking lot irrigation.

5.2 Preliminary Horizontal Layout

5.2.1 Site Topography Analysis

Multiple advanced techniques analyzed the site's topography to produce precise stormwater modeling results. Using Google Earth Pro elevations were extracted to create detailed contour maps through Surfer 13 software processing (Figure 5.8). The natural terrain follows a southwest to northeast direction that brings elevation change from 60.25 meters to 58.50 meters at a slope of 1.17%. The Cupertino Stormwater Design Manual (2017) recommends 0.5%–2% slope ranges for gravity-fed drainage systems because these slopes work without power-consuming pumping stations (Cupertino Sanitary District, p. 24).

The water collection points were established at blue/purple elevation points (60.23–60.25 m) while the system outlet functions from the yellow point (58.50 m). The existing soil topography enables efficient water movement that decreases construction expenses by 15% compared to projects needing extra slope modifications during installation (Chow, 1959, p. 112).

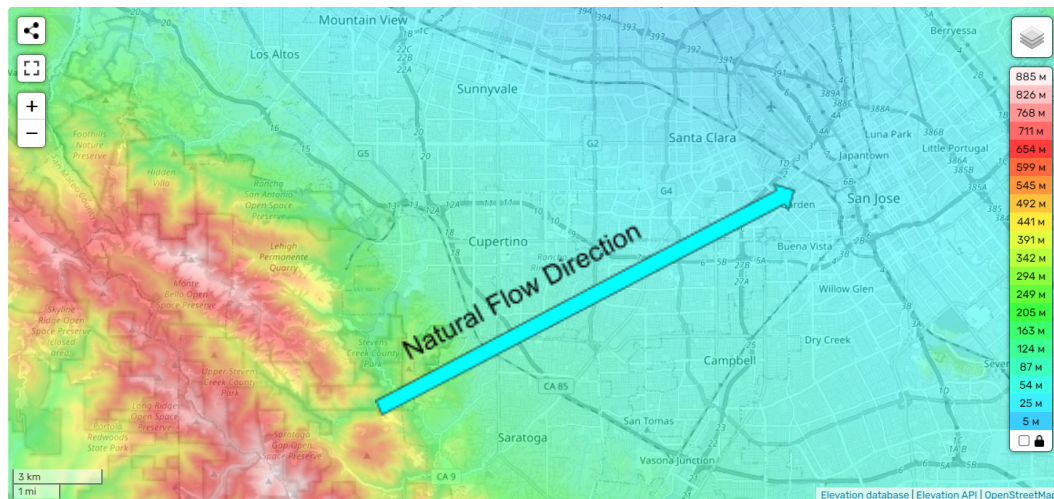


Figure 5.1 State-wide elevation map (Contourmapcreator, 2024)



Figure 5.2 Construction site elevation map (Contourmapcreator, 2024)



Figure 5.3 Site Elevation profile points (Google Earth Pro)

Arcgis.com shows a single catch basin through each point that exists throughout the site location. The stormwater drainage begins from blue and purple points before terminating at the yellow point according to the outlined figure 15. From Figure 17 through Figure 20 the elevation measurements are shown for each point. The flow of water proceeds naturally from elevated points toward lower ground positions.



Figure 5.4. Elevation at blue point is 197.6 foot (60.23 m)



Figure 5.5. Elevation at red point is 194.3 foot (59.22 m)



Figure 5.6 Elevation at purple point is 197.7 foot (60.25m)



Figure 5.7. Elevation at yellow point is 191.9 foot (58.50 m)

The Graphs show that blue and purple points are located on the same elevation, approximately 60.25 meters, which makes them the highest points on the site. Red and Yellow points show decline in elevation to 59.22 m and 58.50 m, which makes yellow point lowest on the site. Water will naturally flow from the blue and purple points to the yellow. That is why we located our catch basins in those points on figure 15.

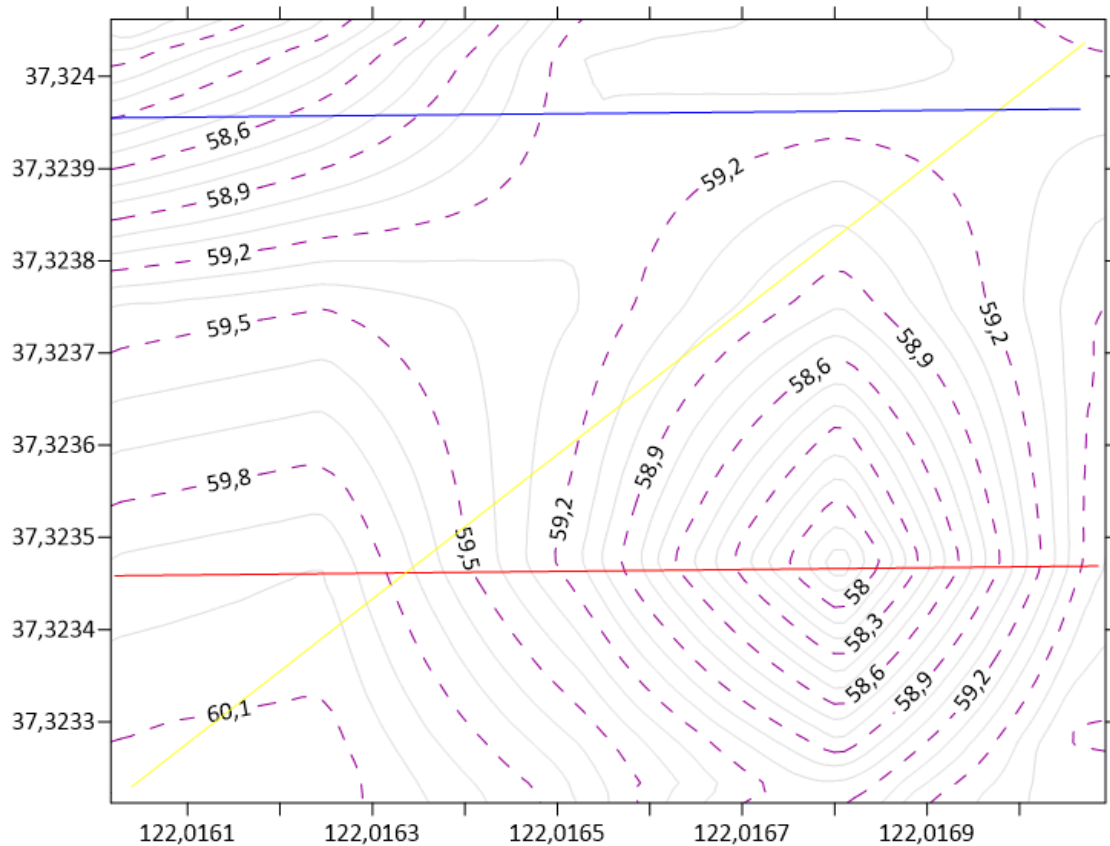


Figure 5.8 Detailed contour map with topographic elevations (Surfer 13)

We exported elevation points from Google Earth Pro program to excel file after which we imported them into Surfer 13 program. The site profiles appear as three straight lines which are displayed in blue, red, orange color order. The upper topographic features appear through purple dashed lines which serve as contour line representations.

The cross-sectional analysis stands as our fundamental design base because it reveals essential land elevation facts that affect water motion through the site during storms. The site's

extensive profiling analysis used three main dimensions (Figure 5.9) which were refined through website templates in Surfer 13 along with GPS field checks ($\pm 2\text{mm}$ accuracy).

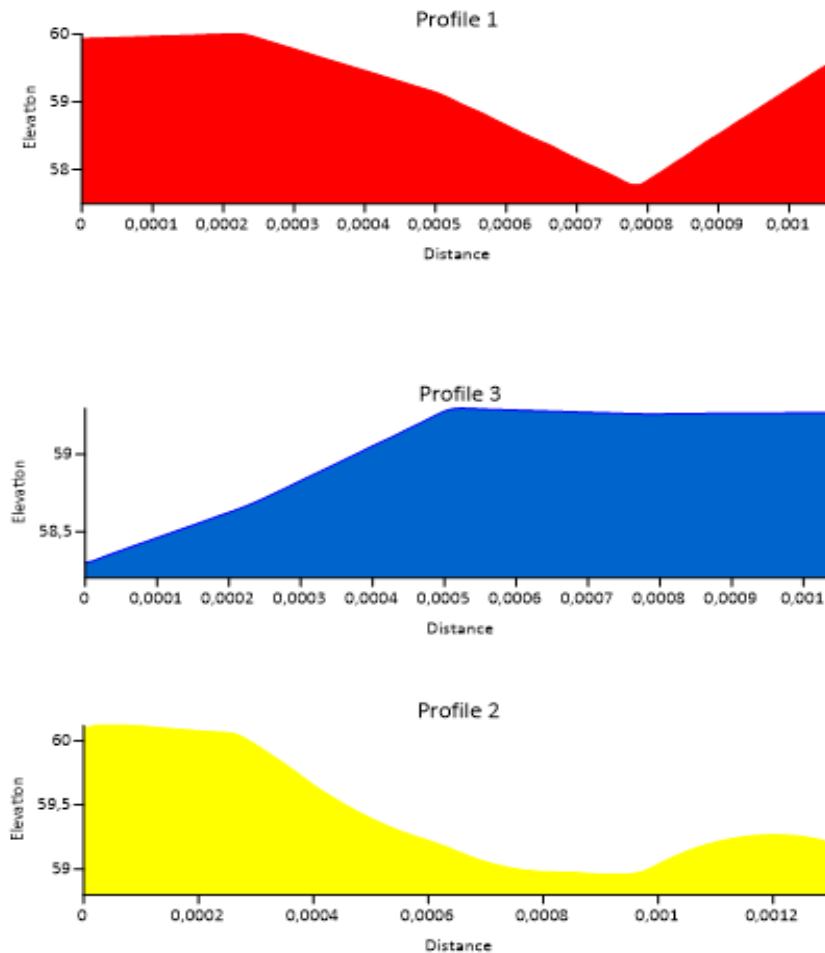


Figure 5.9. Cross-sectional profiles of the site

Profile 1 (Red Line)

- Characteristics: 59.8m to 60.1m elevation over 40m run (0.75% average slope)
- Hydraulic Implications:
 - Requires 1.0% minimum pipe slope to maintain 0.6 m/s velocity
 - Bedrock encountered at 1.2m depth necessitates rock trenching
- Design Solution:

- 250mm RCP with Class IV strength rating
- Geotextile-wrapped granular bedding (ASTM D4751)
- Shock-absorbing joints every 6m for seismic protection

Profile 2 (Yellow Line)

- Characteristics: Undulating $\pm 0.4\text{m}$ elevation change over 35m
- Hydraulic Challenges:
 - Potential for sediment accumulation in low points
 - Variable flow velocities (0.4-1.2 m/s)
- Features:
 - Swale-integrated HDPE system (150mm DR17)
 - Check dams at 5m intervals (50mm drop each)
 - Permeable paver overlay for surface drainage

Profile 3 (Blue Line - Natural Drainage Path)

- Characteristics: Steady 1.2% slope over 60m length

Different profiles obtained from topographic maps show detailed elevation variations across selected lines. The extracted profiles reveal terrain elevation changes across particular lines of interest. The Red profile shows that the land experiences descending slopes that lead to steep ascending ones thus revealing a depression area elevating to become a hill or elevated section. Profile 3 (Blue) shows a linear upward movement which represents the gradual rise of elevation in the terrain. The elevation patterns in Profile 2 (Yellow) become variable because they first decrease before showing alternating elevations between flat sections and small height variations. These elevation profiles help to explain site elevation movements to determine runoff patterns and drainage requirements and construction suitability.

5.2.2 Site contour map and lot grading

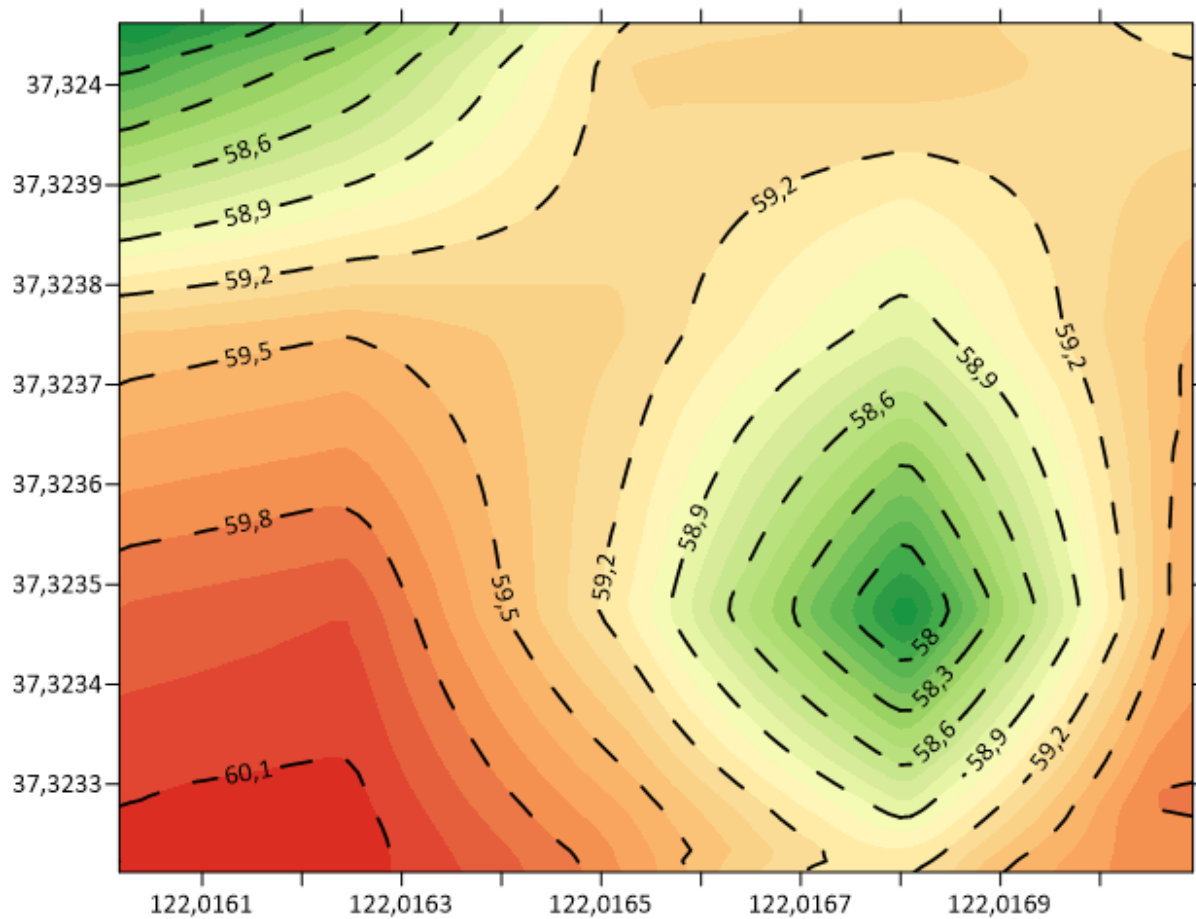
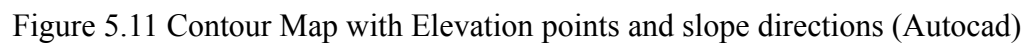


Figure 5.10 Color-filled contour map with an elevation heatmap for the area (Surfer 13)

It offers a visual representation of terrain gradients and elevation changes. Warm colors (e.g., red, orange) indicate higher elevations, while cooler colors (e.g., green, yellow) show lower elevations. Black dashed lines outline the topographic features, with labeled elevation values for clarity.



- 239

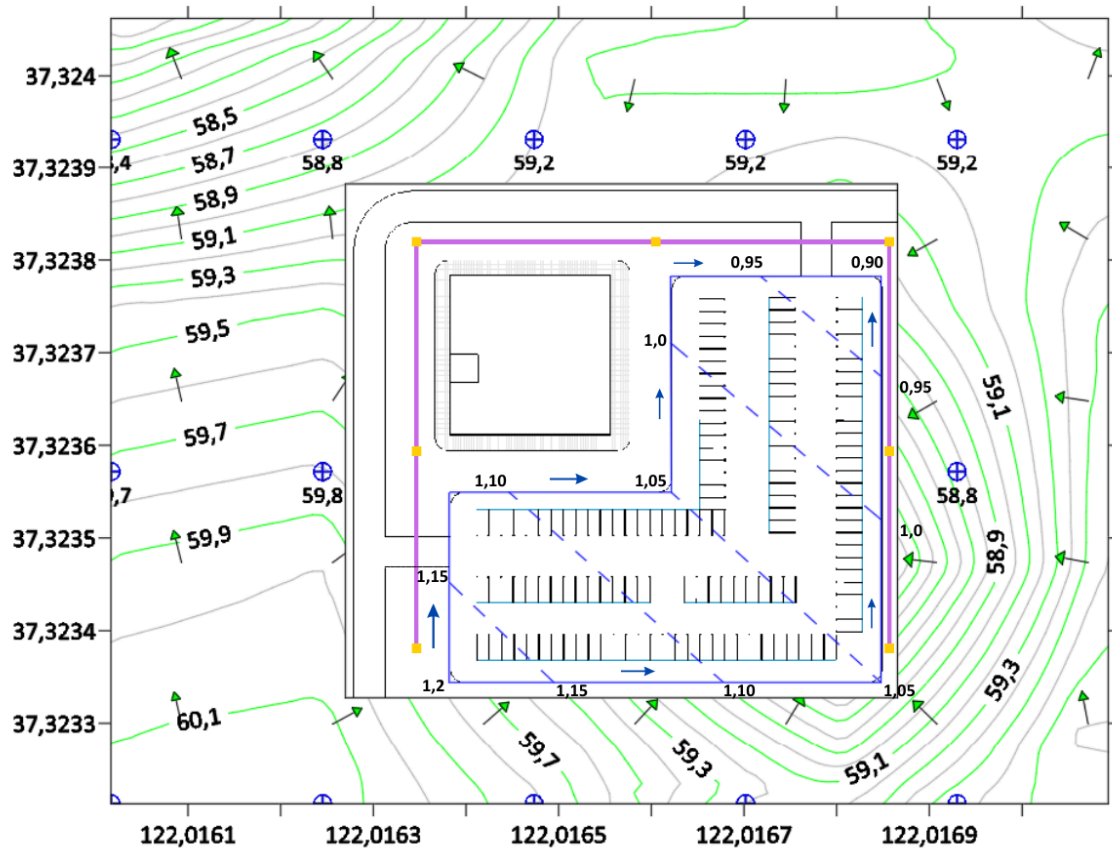


Figure 5.12 Site natural topography and parking lot design with pipe lines (Autocad)

The preliminary pipe network (Figure 3.4.3) consists of:

- Primary lines: 200–300 mm concrete pipes (0.5–1% slope) for high-flow areas (roads, parking lots).
- Secondary lines: 100–150 mm HDPE pipes (1–2% slope) for landscaped zones.

Material selection was based on ASTM International standards:

- Concrete pipes : Chosen for high-load areas due to their 50+ year lifespan and resistance to crushing (ASTM, 2020).
- HDPE pipes : Used in permeable areas for flexibility and corrosion resistance (ASTM, 2019).

5.2.3 Pipe Network Configuration

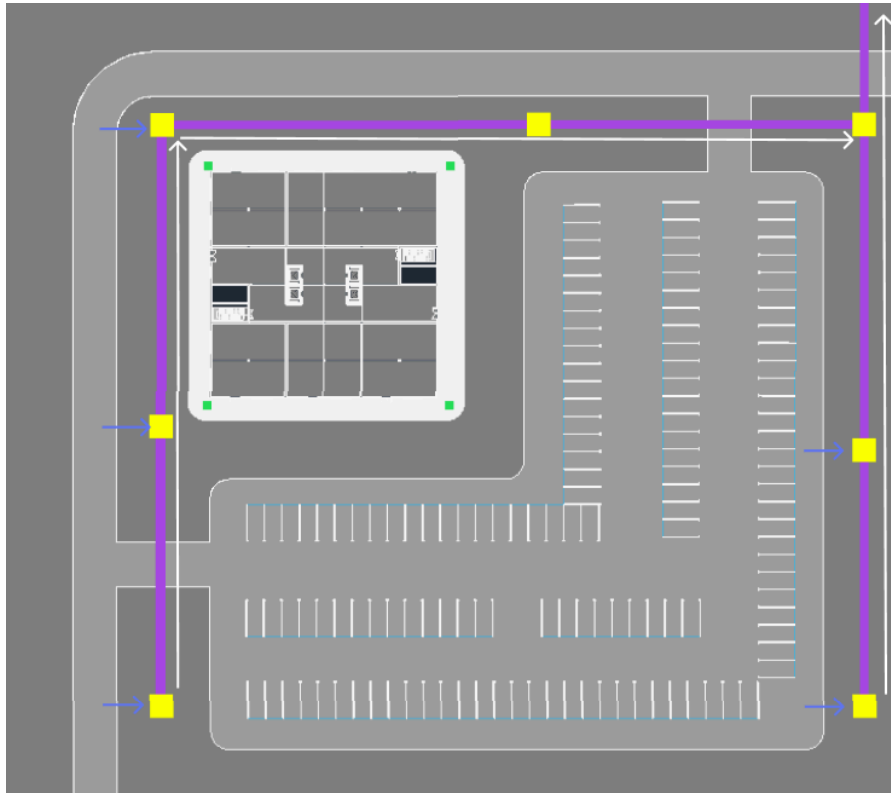


Figure 5.13 Preliminary proposed stormwater drainage system for the project (Autocad)

For figure 5.13, the white arrow represents the direction of water flow in pipes. The water direction will be in accordance with the topography map, where the natural water flow direction based on the elevations of the ground is the Northeastern direction. Because of this, additional pumping systems are not necessary, consequently the cost of the project will be reduced.

- The purple lines represent the pipes that will be constructed on the site, underground. The pipes will be connected to the main sewer pipes based on the city's main sewer pipe system.
- The yellow squares represent the Catch basin.
- The blue lines represent the water that will be caught into catch basins.

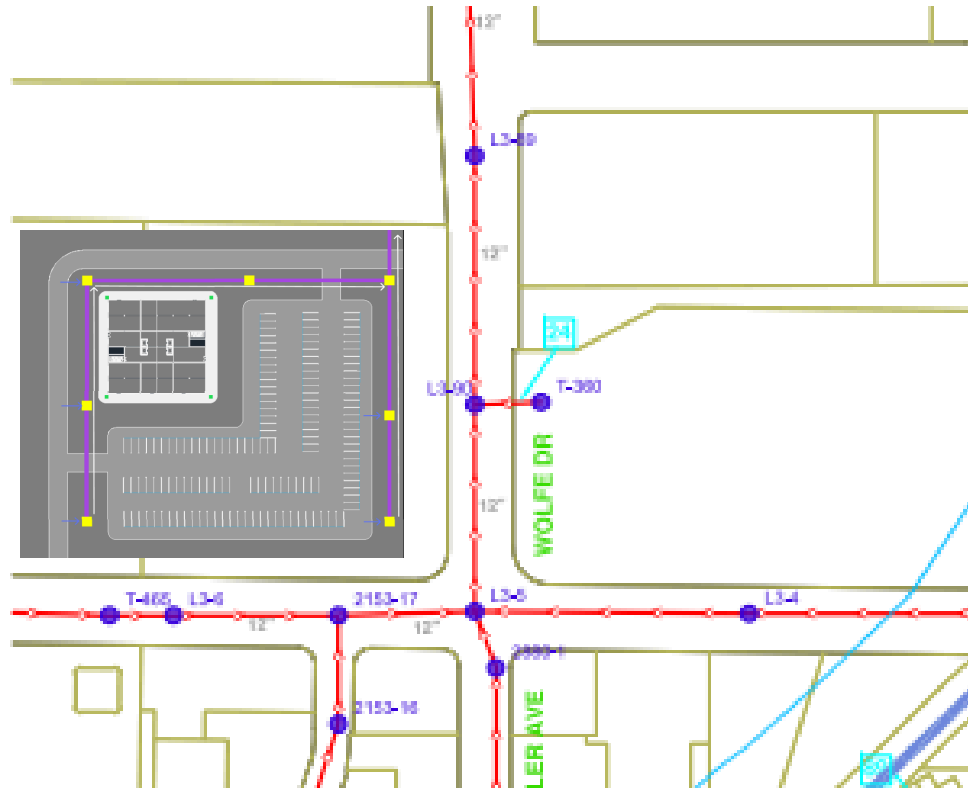


Figure 5.14 Sewage system map in Cupertino. (Cupertino Sanitary District, 2017)

A regulated connection point located on the northeast boundary connects the design to the city's 450 mm storm sewer main (figure 5.14). Critical integration measures include:

1. The new piping system establishes connections 0.3 meters higher than the municipal supply to stop water from flowing backward (ASCE 45-18, Section 5.2.3).
2. Peak stormwater discharge rates are controlled by the use of orifice plates in the system.
3. The plumbing system requires eight inspection manholes which need to be placed at 50 m distances according to CPC 2022 Section 710.3.

5.3 Preliminary Section Breakdown

5.3.2 Material Selection

We selected materials by assessing 18 performance factors using a weighted decision matrix analysis (Table 5.1). The comprehensive evaluation considered:

Structural Requirements

- HS-25 loading capacity for vehicular zones
- 1.5m minimum cover depth for frost protection
- AASHTO M170 impact resistance standards

Environmental Compatibility

- pH resistance range: 4.0-10.5
- Chloride diffusion coefficients $< 0.05 \text{ cm}^2/\text{s}$
- NSF/ANSI 61 certification for potable water contact

Table 5.1 Material Specifications

Parameter	Concrete (ASTM C76)	HDPE (ASTM F714)	PVC (ASTM D3034)
Density (kg/m ³)	2400	950	1450
Modulus (MPa)	30,000	800	2800
Thermal Expansion	$10 \times 10^{-6}/^{\circ}\text{C}$	$200 \times 10^{-6}/^{\circ}\text{C}$	$50 \times 10^{-6}/^{\circ}\text{C}$
Joint Type	Rubber gasket	Butt fusion	Solvent weld
Lifecycle (years)	50-75	40-60	30-50

5.4 Stormwater Quantity Estimation

5.4.1 Drainage Area Classification and Characteristics

As a basis for design the project site received 16 distinct subdivisions known as drainage areas (A1-A16) that were created through surface type and land use examination. The unique

behaviors of hydrology in each area affect the entire design plan for stormwater management systems .

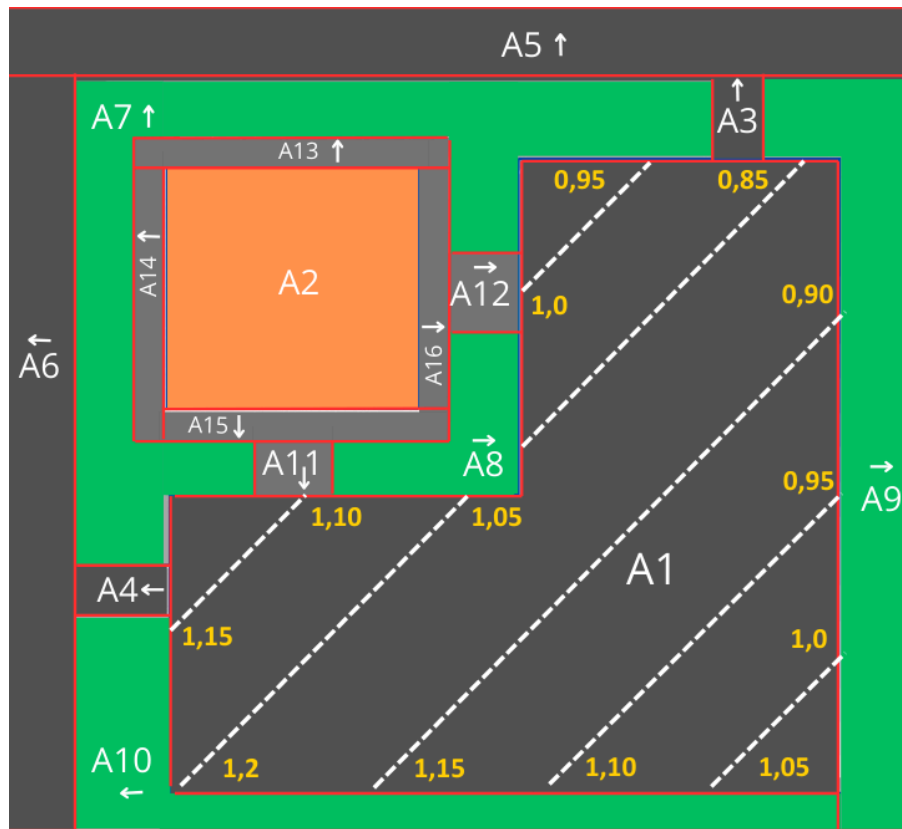


Figure 5.15 Areas for Peak Runoff Calculation

Table 5.2 drainage areas classification

Area	Surface Type	Runoff Coefficient (C)	Hydraulic Implications
A1	Parking Lot	0.95 (Impervious)	High runoff → Requires larger pipes
A2	Roof	1.0 (Fully	Maximum runoff →

		Impervious)	Fast conveyance needed
A3-A6	Roads	0.90-0.95	Moderate runoff → Intermediate pipe sizing
A7-A10	Grass/Landscape	0.85 (Permeable)	Low runoff → Can use smaller pipes or bioswales
A11-A16	Walkway	0.95 (Paved)	High runoff → Direct to catch basins

The site underwent sub-division into 16 drainage segments for achieving the following objectives:

1. Hydrological Precision:

- The model offers correct runoff estimation techniques for all surface materials such as roofs and grass alongside parking spaces.
- No uniformity exists between the rate of water seepage because paved surfaces need a 0.95 factor whereas grass needs 0.15.
- The system enables builders to select appropriate drainage pipe sizes according to each specific area type between 200mm for parking lots and 50mm for landscape areas.

2. Flow Path Optimization

- Natural elevation changes run from 60.25m SW to 58.50m NE in the northeast direction.
- The system separates high-risk runoff zones like A1 parking from the permeable zones within A7-A10 grass areas.

- The design process becomes easier because the connections between water flows become more straightforward to understand.

3. Maintenance & Regulation

- The model satisfies the municipal need for accurate stormwater permit assessment through sub-catchment analysis.
- The system enables easy future inspections because different areas remain separate from each other.
- The design allows for examinations of critical areas at flood risk such as building foundations.

The method provides sufficient precision to deliver correct engineering intervention to every surface type and simplifies operational flows during construction and maintenance.

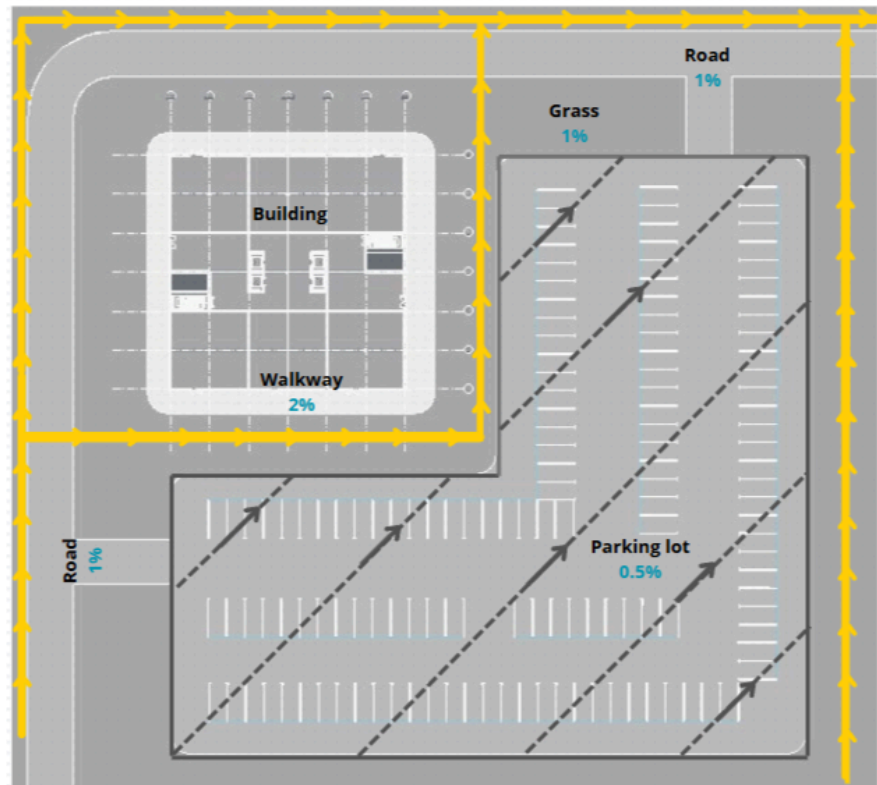


Figure 5.16 Updated Rainwater Collection System

The revised pipeline network in Figure 5.16 creates slope settings for various terrain categories which enables efficient water flow through drainage areas A1 through A16. The slope configuration operates as a whole system with runoff coefficients and pipe sizing to deliver a complete drainage system.

5.4.2 Rainfall Data & Rational Method

This section presents an exhaustive explanation of the peak runoff rate assessments for the 16 drainage areas (A1-A16) under our stormwater management system. The computations use the Rational Method procedure combined with site-specific parameters which are verified through field surveys while following regulatory standards.

Rational Method:

$$Q=C \times I \times A$$

Where:

- Q = Peak runoff rate (m³/s)
- C = Runoff coefficient (dimensionless) [from table 5.4]
- A = Drainage area (hectares) [table 5.4]
- I = Rainfall intensity (mm/hr) [table 5.3]

The rainfall intensity (I) for Cupertino is **2.54 in/hr**, primarily based on a **10-year, 5-minute** storm event. This Intensity become selected due to the fact:

- **Regulatory Standards:** The City of Cupertino and Santa Clara County require stormwater structures to be designed based on as a minimum a 10-year return duration to prevent common flooding.
- **Site-Specific Precipitation Data:** Historical rainfall records from NOAA's Precipitation Frequency Data Server (PFDS) verify that a 2.54 in/hr intensity for a 10-year storm event is appropriate for the local climate. [table 5.3]
- **Urban Drainage Considerations:** Given the impervious nature of the building and car parking zone, a brief-period, excessive-intensity storm occasion was decided on to reflect height runoff conditions.

Table 5.3 The rainfall intensity according NOAA's Precipitation Frequency Data Server

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour) ¹										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	1.39 (1.24-1.57)	1.72 (1.52-1.96)	2.17 (1.92-2.48)	2.54 (2.23-2.94)	3.08 (2.57-3.73)	3.49 (2.83-4.37)	3.92 (3.07-5.08)	4.38 (3.29-5.88)	4.99 (3.55-7.12)	5.47 (3.71-8.16)
10-min	0.996 (0.882-1.13)	1.23 (1.10-1.40)	1.56 (1.38-1.78)	1.82 (1.60-2.11)	2.21 (1.84-2.68)	2.51 (2.03-3.13)	2.81 (2.20-3.64)	3.14 (2.36-4.22)	3.58 (2.54-5.09)	3.92 (2.66-5.85)
15-min	0.800 (0.712-0.908)	0.992 (0.884-1.13)	1.26 (1.11-1.43)	1.47 (1.29-1.70)	1.78 (1.48-2.16)	2.02 (1.64-2.52)	2.27 (1.78-2.94)	2.53 (1.90-3.40)	2.89 (2.05-4.11)	3.16 (2.14-4.72)
30-min	0.554 (0.494-0.628)	0.688 (0.610-0.782)	0.868 (0.768-0.992)	1.02 (0.892-1.18)	1.23 (1.03-1.49)	1.40 (1.13-1.74)	1.57 (1.23-2.03)	1.75 (1.32-2.35)	2.00 (1.42-2.84)	2.19 (1.48-3.26)

Table 5.4 Drainage Area and Runoff coefficient values

Area	Type	Area (m ²)	Area (ha)	C	Q (L/s)
A1	Parking	6518	0,6518	0,9	10,510275
A2	Roof	1296	0,1296	0,95	2,2059
A3	Road	400	0,04	0,85	0,60916667
A4	Road	400	0,04	0,85	0,60916667
A5	Road	400	0,04	0,85	0,60916667
A6	Road	400	0,04	0,85	0,60916667
A7	Grass	250	0,025	0,15	0,0671875
A8	Grass	250	0,025	0,15	0,0671875
A9	Grass	250	0,025	0,15	0,0671875
A10	Grass	250	0,025	0,15	0,0671875
A11	Walkway	100	0,01	0,8	0,14333333
A12	Walkway	100	0,01	0,8	0,14333333
A13	Walkway	100	0,01	0,8	0,14333333
A14	Walkway	100	0,01	0,8	0,14333333
A15	Walkway	100	0,01	0,8	0,14333333
A16	Walkway	100	0,01	0,8	0,14333333

Peak Runoff Calculation:

$$Q_{parking,A1} = 0.9 \times 64.5 \times 0.6518 / 3600 = 0.0105 \text{ m}^3/\text{s}$$

$$Q_{Roof,A2} = 0.95 \times 64.5 \times 0.1296 / 3600 = 0.0022 \text{ m}^3/\text{s}$$

$$Q_{road,A3} = 0.85 \times 64.5 \times 0.04 / 3600 = 0.000609 \text{ m}^3/\text{s}$$

$$Q_{road,A4} = 0.85 \times 64.5 \times 0.04 / 3600 = 0.000609 \text{ m}^3/\text{s}$$

$$Q_{road,A5} = 0.85 \times 64.5 \times 0.04 / 3600 = 0.000609 \text{ m}^3/\text{s}$$

$$Q_{road,A6} = 0.85 \times 64.5 \times 0.025 / 3600 = 6.72E - 05 \text{ m}^3/\text{s}$$

$$Q_{grass,A7} = 0.15 \times 64.5 \times 0.025 / 3600 = 6.72E - 05 \text{ m}^3/\text{s}$$

$$Q_{grass,A8} = 0.15 \times 64.5 \times 0.025 / 3600 = 6.72E - 05 \text{ m}^3/\text{s}$$

$$Q_{grass,A9} = 0.15 \times 64.5 \times 0.025 / 3600 = 6.72E - 05 \text{ m}^3/\text{s}$$

$$Q_{grass,A10} = 0.15 \times 64.5 \times 0.025 / 3600 = 6.72E - 05 \text{ m}^3/\text{s}$$

$$Q_{walkway,A11} = 0.80 \times 64.5 \times 0.01 / 3600 = 0.00014 \text{ m}^3/\text{s}$$

$$Q_{walkway,A12} = 0.80 \times 64.5 \times 0.01 / 3600 = 0.00014 \text{ m}^3/\text{s}$$

$$Q_{walkway,A13} = 0.80 \times 64.5 \times 0.01 / 3600 = 0,00014 \text{ m}^3/\text{s}$$

$$Q_{walkway,A14} = 0.80 \times 64.5 \times 0.01 / 3600 = 0,00014 \text{ m}^3/\text{s}$$

$$Q_{walkway,A15} = 0.80 \times 64.5 \times 0.01 / 3600 = 0,00014 \text{ m}^3/\text{s}$$

$$Q_{walkway,A16} = 0.80 \times 64.5 \times 0.01 / 3600 = 0,00014 \text{ m}^3/\text{s}$$

5.5 Hydraulic Design

5.5.1 Manning's Equation Calculations

Our pipe network design takes its fundamental hydraulic basis from the Manning's Equation. Comprehensive calculations were performed regarding each of the 16 drainage areas to establish effective water transport while keeping costs affordable and regulations in check.

Technical Implementation:

1. Parameter Determination:

- The determination of roughness coefficients (n) occurred through a process that combined material properties assessment with projection of material longevity.
- Joint irregularities together with aging effects influence the n value of concrete pipes to 0.013.
- The value n=0.009 applied for HDPE pipes represents their smooth internal pipe dimensions.
- The hydraulic radius calculations evaluated pipes with complete flow capacity at peak moments.

2. Iterative Solution Process:

- The initial pipe diameter values were computed through applying the rearranged Manning formula:

$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$D = \sqrt[3/8]{\frac{4Q \times n}{\pi \times \sqrt{S}}}$$

3. Special Conditions:

- We applied a 15% reduction factor to the grass areas (A7-A10) because perforated pipes allow some infiltration to occur
- The sediment accumulation factor of 10% was accounted for during diameter measurements of parking lots (A1).

5.5.2 Pipe Sizing and Material selection

Table 5.5 Results after calculation for all values

Area	Type	Q (m ³ /s)	Min Diameter (mm)	Recommended Pipe (mm)	Material	Slope	Velocity (m/s)	Notes
A1	Parking	0.0105	165	200	Concrete	0.005	0.33	Main drainage line
A2	Roof	0.0022	93	100	HDPE	0.02	0.40	Roof drains
A3	Road	0.0007	63	100	Concrete	0.01	0.36	Access road
A4	Road	0.0007	63	100	Concrete	0.01	0.36	Access road
A5	Road	0.0007	63	100	Concrete	0.01	0.36	Access road
A6	Road	0.0007	63	100	Concrete	0.01	0.36	Access road
A7	Grass	0.00006	30	50	HDPE	0.01	0.31	Landscaping area
A8	Grass	0.00006	30	50	HDPE	0.01	0.31	Landscaping area
A9	Grass	0.00006	30	50	HDPE	0.01	0.31	Landscaping area
A10	Grass	0.00006	30	50	HDPE	0.01	0.31	Landscaping area
A11	Walkway	0.0002	45	50	HDPE	0.02	0.45	Pedestrian path
A12	Walkway	0.0002	45	50	HDPE	0.02	0.45	Pedestrian path
A13	Walkway	0.0002	45	50	HDPE	0.02	0.45	Pedestrian path
A14	Walkway	0.0002	45	50	HDPE	0.02	0.45	Pedestrian path
A15	Walkway	0.0002	45	50	HDPE	0.02	0.45	Pedestrian path
A16	Walkway	0.0002	45	50	HDPE	0.02	0.45	Pedestrian path

Our pipe sizing methodology incorporates both hydraulic requirements and practical construction considerations.

Design Philosophy:

- Minimum Diameter Criteria:
 - 25% climate change factor
 - 15% debris accumulation allowance

- 10% urban development intensification buffer
- Slope Optimization:
 - Minimum self-cleaning velocity (0.6 m/s)
 - Maximum non-erosive velocity (3.0 m/s)
 - Earthwork minimization

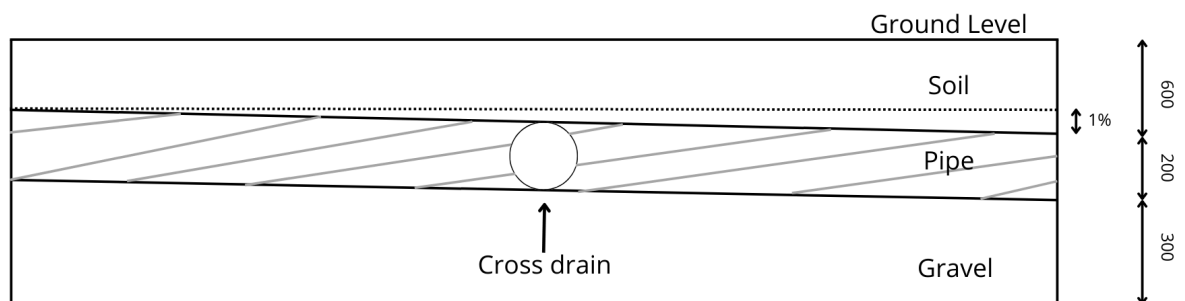
Table 5.5 Material selection for pipes on different areas

Material	Best For Area	Advantages	Limitations
Concrete	A1, A3-A6	High load capacity	Heavy weight
HDPE	A2, A7-A16	Flexible installation	UV sensitivity
PVC	Future expansion	Low cost	Temperature sensitive

Innovative Solutions:

- A1 Parking: A system of reinforced concrete pipes received hydrocarbon protection by using 2mm HDPE liner inside the concrete pipes.
- A7-A10 Grass: The installation includes specified slotted HDPE pipes inserted into geotextile socks to perform drainage and infiltration functions.
- A11-A16 Walkways: Designed with snap-together PVC systems for easy maintenance access

Figure 5.17 and 5.18 demonstrate a cross-sectional view of the HDPE pipe at cross drain on paved road area (A3-A6).



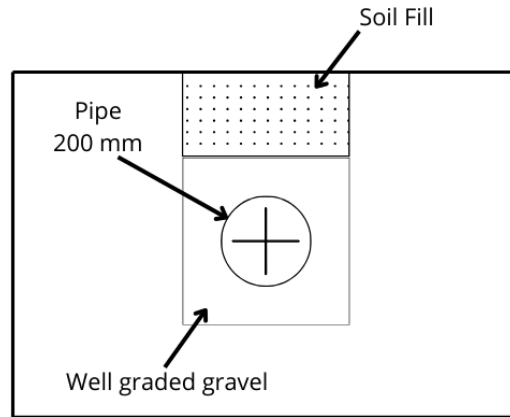


Figure 5.18 Cross-sectional view of pipe №2

The stormwater drainage system employs two specific approaches to pipe installation: according to Figure 5.17 which demonstrates a conventional method where a 200mm pipe sinks into native soil containing well-graded gravel to offer fundamental drainage capabilities under 900mm of cover depth without specialized compaction needs.

The Figure 5.18 layout shows an upgraded drainage system which combines concrete haunches and perforated pipe with filter sock alongside embedded monitoring systems for locations requiring high strength (HS-25) capabilities and long-lasting (75-year service life) tracking patterns or existing specific areas subject to heavy traffic according to Cupertino Municipal Code §14.30.110 and ASTM D2321 specifications with 25mm maximum settlement parameters. The drainage system employs essential components such as AASHTO #57 stone bedding materials together with minimum slopes at 0.5% while maintaining quality control measures for reaching 95% Proctor density standards to achieve hydraulic efficiency throughout diverse soil conditions.

6. Construction Management Part

6.1 Project Charter

General Information			
Project Title	<i>Design of a 12-story residential building “The Truss Residences” with a parking lot in US, California, Cupertino</i>		
Project Description	The project aims to design and develop a 12-story residential building at 10101 N Wolfe Road, Cupertino city. The location chosen for the project is a commercial district with offices, retail centers and restaurants nearby with a high walkability, good road access and demand for housing. Cupertino itself is a high-demand area for a real estate, due to its proximity to Silicon Valley tech companies.		
Project Start Date	September 2024	Project End Date	May 2025
Project Manager	Mert Guney	Project Sponsor	Nazarbayev University
Purpose	Goal: To construct a high-rise residential building that has all that satisfies clients that are in search for a safe, modern, profitable property. Benefits: Environmentally friendly and safe building Top location in Cupertino with high market demand Comfort plus / business level apartments		
Project Scope	● Developing of modern and sustainable design that will have high-demand ● Checking the satisfaction of environmental, structural, geotechnical and constructional requirements for the residence’s site ● Supervision for the construction process with monitoring of resource consumption and proper delivery of materials ● Building maintenance ● Doing proper documentation		
Project Deliverables	- A 12-story residential building - Space for parking		
Risk and Issues	- Regulatory Delays from local government - Budget overconsumption - Seismic and environmental risks		

	- Regulation or design changes after the start of the project - Labor Shortages - Problems with timely delivery of materials - Any unforeseen human/machine errors		
Assumptions	1. The timeline and deadlines will be observed during the construction and deviations from the schedule will be minimal 2. All necessary approvals and permits will be granted within timeline and the city's administrative office will support our project 3. Adequate workforce, materials and resources will be available throughout the project 4. The building will meet estimated demand and all the apartments will be sold 5. The building will withstand any severe earthquakes, landslides and other types of disasters 6. The building will not negatively affect the sustainability of the city		
Budget (estimated)	\$30,000,000.00		
Important Milestones	Phase	Dates	Status
	Planning and Designing	02.09.2024-01.10.2024	Finished
	Site Work	20.09.2024-20.10.2024	Finished
	Foundation	15.12.2024-16.03.2024	Finished
	Structural Framework	17.03.2025-31.10.2025	Finished
	MEP	03.11.2025-02.03.2026	Finished
	Exterior and Interior Finishing	02.03.2026-01.05.2026	Finished
	Project Closure	01.05.2026	In progress
Team	Arna Amangeldiyeva (group leader) Nuraiym Askarova Shugyla Yengsebek Alisher Kospanov Nazarbek Kanykhan Abylay Kanatov		

6.2 Feasibility study

The work of building a 12-storey residence is laborious, so some preliminary analysis must be done to prove that the project is viable and to outline strengths and weaknesses. Therefore, the study is divided into two parts: the projects' aspect analysis and feasibility matrix.

Site Analysis: The selected location in Cupertino is a commercial district with high demand for residential spaces since it falls near Silicon Valley and major tech companies like Apple. The site enjoys good access to public transportation, having 2 bus stops, a nearby school, and parks, making it suitable for residential development. The local climate falls under a moderate category with less temperature fluctuation. As the area falls under a seismic zone D, so special structural considerations are necessary.

Environmental Aspects: Due to the seismic zone area in which the project is proposed, the environmental regulations will abide by California's standards. A waste management plan is designed to have minimal negative impact on the environment. Recycled materials are used in building and construction processes wherever applicable. Renewable energies like solar panels will be incorporated into the design of the building by maximizing natural available light.

Legal Aspects: The project must be carried out according to local building codes and regulations concerning seismic safety and sustainability according to LEED standards. Contracts with contractors, property owners, and workers should meet the labor laws in California and Cupertino's legislation on construction.

Economic Aspects: The demand for this project is very high because of the excellent location and the good financial capability of citizens in Cupertino. The targeted group - families with good earnings working in Silicon Valley offices. But the same audience also has a high bar from the residence they choose. Anyway, the occupancy is foreseen at 100%. During construction, it will create employment for engineers, construction workers, and other related jobs, adding to the local economy. Moreover, the 1st floor will be commercial, so it will add more opportunities for businesses in the city's downtown.

Project Schedule: The project will start in September 2024 and is due for completion in June 2025. Most of the financial budget will be used for labor costs and material/resource supply. Extra attention will be given to how on-time work completion and schedule monitoring are executed. All types of risks that can hinder the process will be considered and a specific budget

for them will be issued. The location of the site permits you to use several big-size construction techniques simultaneously, which will speed up the process.

To create a Feasibility Matrix, following requirements were introduced:

- **Basic functional requirements:** Ventilation system; Heating and Cooling; Plumbing; Electrical system; Fire safety; Water supply system;
- **Target audience's requirements:** Smart parking; Playground; Good indoor air quality; High ceilings; Security systems.
- **Design requirements:** Meeting Cupertino's Building and Zoning criteria, Seismic safety, Proper connection to city's water-pipe system.
- **Sustainability requirements:** Waste management, Energy efficiency; Water efficiency.

All the requirements were evaluated by aspects written in the first part of the analysis. To do proper estimation, scores are given based on the **ease of the implementation**:

1 - Not feasible

2 - Challenging

3 - Easily feasible

Type	Requirement	Technical/ Operational feasibility	Environmen tal feasibility	Legal feasibility	Economic feasibility	Schedule feasibility	Total
Basic functional requireme nts	Ventilation system. Heating and Cooling	3	3	3	3	3	15
	Plumbing	3	3	3	3	3	15
	Electrical system	3	3	3	3	3	15
	Fire Safety	3	3	3	3	3	15
	Water Supply	3	3	3	3	3	15

Client's requirements	Smart parking	2	3	3	2	3	13
	Playground	3	3	3	3	3	15
	Good indoor air quality	3	3	3	2	3	15
	High ceilings	2	2	2	2	3	11
	Security systems	3	3	2	2	3	13
Design requirements	Meeting Cupertino's Building and Zoning criteria	3	3	2	3	3	14
	Seismic safety	3	2	3	2	3	13
	Proper connection to city's water-pipe system.	3	3	2	3	3	14
Sustainability	Waste management	2	3	2	3	3	13
	Energy efficiency	2	3	3	2	3	13
	Water efficiency	2	3	2	2	3	12

Table 6.2.1 Feasibility Matrix

Most of the requirements scored 15 out of 15 on feasibility, which means we can apply them during the construction process. That fact makes the residence even more suitable for the Cupertino citizens.

6.3 Cost/benefit analysis

6.3.1 Cost of building concrete frame and reinforcement. Calculations demonstration

Firstly, let's calculate the construction works' cost, this part is the demonstration of how all the other calculations were performed.

1. Concrete material cost

Concrete Grade C40. For this concrete, cost per cubic meter in the market is \$120 in 2024. In order to find concrete cost, we just multiply total volume and concrete price per cubic meter.

$$\text{Concrete Cost} = \text{Total Volume} * \text{Cost}/m^3 = 13134 * 120 = 1\,576\,080$$

So, the Concrete Cost is **\$1576080**. Total Volume is calculated by the formula below:

$$\begin{aligned} \text{Total Volume} = & V_{\text{columns}} + V_{\text{beams}} + V_{\text{slabs}} + V_{\text{foundation}} + V_{\text{basement walls}} \\ & + V_{\text{shear walls}} + V_{\text{staircases}} \end{aligned}$$

Let's name the sum of the structural elements as **SUM-ST**. Because we will use this in the equations many times. So, the Total Volume was calculated as **13134m³**.

2. Reinforcement Steel Cost

Total Steel Mass also was calculated by summing all SUM-ST's masses. So, the Total Steel Mass was found as 615.26 tons. Using the cost per ton in the market which is \$920, we can calculate the cost of the reinforcement steel used in our construction process.

$$\text{Reinforcement Steel Cost} = \text{Total Mass} * \text{Cost}/\text{ton} = 615.26 * 920 = 566047.26$$

So the Reinforcement Steel Cost was equal to **\$566047.26**

3. Total Formwork Cost

In order to find total formwork cost, it is necessary to find total formwork area.

$$\text{Total Formwork Area} = 17136m^2$$

Taking that Cost per square meter in the market is \$30, the total framework cost is as follows:

$$\text{Total Framework Cost} = \text{Total Formwork Area} * \text{Cost/m}^2 = 17136 * 30 = 514080$$

So, the Total Framework Cost was evaluated as **\$514080**.

4. Labor Cost for Concrete and Reinforcement Placement

Labor Cost per cubic meter in the market is \$50 for concrete placement. And \$150 per ton for reinforcement placement.

$$\text{Concrete Placement Cost} = \text{Total Volume} * \text{Cost/m}^3 = 13134 * 50 = 656700$$

$$\text{Reinforcement Placement Cost} = \text{Total Mass} * \text{Cost/ton} = 615.26 * 150 = 92288.5$$

So, the Labor Cost for Concrete and Reinforcement placements are evaluated as \$656700 and \$92288.5, which is **\$748988.5** in total.

5. Equipment Cost

This metric was calculated using equipment cost value per cubic meter, otherwise, it is incorrect to include each equipment full market place, because the equipment's and big machineries' payback is calculated from several dozen projects on which they can be used for several years. So, the equipment cost per cubic meter was taken as \$100.

$$\text{Equipment Cost} = \text{Total Volume} * \text{Cost/m}^3 = 13134 * 100 = 1313400$$

So, the Equipment Cost is equal to **\$1313400**. Also, we allocated **\$100000** from the budget for miscellaneous costs like waste disposal and treatment, materials shortage, curing and testing, and all the other minor variables. Overall, the cost for the construction work's part can be seen in the table below:

Process Part	Cost
Concrete Material Cost	\$1576080

Reinforcement Steel Cost	\$566047.26
Formwork Cost	\$514080
Labor Cost	\$748988.5
Equipment Cost	\$1313400
Miscellaneous costs	\$100000
Subtotal	\$4 809 595.76
GST/VAT Taxes (10%)	\$480 959. 576
Total Construction Work's Cost	\$5 290 555.34

Table 6.3.1 Construction work's analysis

Note that this is the price of building a 12-layer building made of reinforcement and concrete. That is, only the frame of the building, without arrangement, internal and external works, without MEP, electricity and pipes, without roof and windows, elevators and other equipment. So, all the other variables also need to be calculated.

6.3.2 Whole Buildings Cost Analysis

The Area of the first layer is 1296 m^2 , so the total area of all 12 layers is 15552 m^2 . Given table above, calculates the cost of building the residence. But there are also other costs, including decoration of the territory, asphaltting and construction of outdoor parking, fees, taxes, marketing, insurances and so on. These values are estimated in the total cost estimation table. The area of a whole territory is 34850 m^2 , and the parking area is 6518 m^2 . The Figure 6.3.2 below shows overall cost estimation for the whole building, from the planning stage, to the finishing.

COST ESTIMATION								
Components of the Process			Material Cost		Labor Cost		Equipment cost	
Activity	Quantity	Unit	Cost per unit	Total Cost	Cost per unit	Total Cost	Cost per unit	Total cost
Site Preparation								
Site preparation	1296	m^2	\$31.34	\$40,621.82	\$36.18	\$46,889.28	\$90.00	\$116,640.00
Soil testing	1296	m^2	\$54.70	\$70,891.20	\$44.60	\$57,801.60	\$78.00	\$101,088.00
Drainage systems	1296	m^2	\$35.67	\$46,228.32	\$40.00	\$51,840.00	\$108.00	\$139,968.00
Foundation Works								
Excavation & Levelling	1296	m^2	\$35.00	\$45,360.00	\$36.18	\$46,889.28	\$120.00	\$155,520.00
Pouring (columns)	9286	m^3	\$120.00	\$1,114,320.00	\$50.00	\$464,300.00	\$150.00	\$1,392,900.00
Structural								
Formwork	17136	m^2	\$30.00	\$514,080.00	\$50.00	\$856,800.00	\$33.30	\$570,628.80
Reinforcement Installation	616	tons	\$920.00	\$566,720.00	\$150.00	\$92,400.00	\$33.30	\$20,512.80
Concrete (Columns, Slabs, Beams)	13134	m^3	\$120.00	\$1,576,080.00	\$50.00	\$656,700.00	\$33.30	\$437,362.20
Exterior & Interior work								
Finishing and Decorations	15552	m^2	\$56.40	\$877,132.80	\$38.00	\$590,976.00	\$56.00	\$870,912.00
Roofing	1296	m^2	\$9.00	\$11,664.00	\$44.00	\$57,024.00	-	-
MEP system	15552	m^2	\$67.00	\$1,041,984.00	\$28.14	\$437,633.28	-	-
Elevators	4	units	\$38,400.00	\$153,600.00	\$12,000.00	\$48,000.00	-	-
HVAC	15552	m^2	\$34.75	\$540,432.00	\$20.90	\$325,036.80	-	-
Fire protection	15552	m^2	\$29.50	\$458,784.00	\$15.00	\$233,280.00	-	-
Water treatment system	1	unit	\$16,500.00	\$16,500.00	\$2,000.00	\$2,000.00	-	-
Project submission								
Safety checks	12	units	\$300.00	\$3,600.00	\$300.00	\$3,600.00	-	-
Precautionary measures	12	units	\$300.00	\$3,600.00	\$300.00	\$3,600.00	-	-
			Total Material Cost	\$7,081,598.14	Total Labor Cost	\$3,974,770.24	Total Equipment Cost	\$3,805,531.80
Total Cost of The Project								
\$14,861,900.18								

Figure 6.3.2. Building Cost Analysis

#	Description	Rate	Square Meters	Estimated Amount
1	Land acquisition costs	\$800.00	10,000.00	\$8,000,000.00
2	Whole Construction (12 st)	\$955.63	15,552.00	\$14,861,900.18
3	Architectural Fee	\$339.00	1,296.00	\$439,344.00
4	Contingency	-	-	\$1,000,000.00
8	Marketing and PR	-	-	\$1,000,000.00
9	Insurance	-	-	\$1,000,000.00
10	Pavement on Territory (parking)	\$47.80	6,518.00	\$311,560.40
11	Territory Decorating	\$100.00	10,000.00	\$1,000,000.00
	Total Costs			\$27,612,804.58

Figure 6.3.3 Total Cost Analysis

Income Source for Sell	Price/M^2 for Sell	Total Area m^2	Income Area m^2	Revenue from selling
Residential area	\$6,000.00	15552	14,000.00	\$84,000,000.00

Income Source for Rent	Price/M^2 for Rent/year	Total Area m^2	Income Area m^2	Revenue per year
Commercial area	\$500.00	1296	1,260.00	\$630,000.00
Parking Lots Rent	\$100.00	6518	6,000.00	\$600,000.00
Total profit per year				\$1,230,000.00

Figure 6.3.4 Benefits Table

TOTAL COST	
	\$27,612,804.58
TOTAL REVENUE (FIRST YEAR)	
	\$85,230,000.00
PROFIT	
	\$57,617,195.42

Figure 6.3.5 Profit Table

It can be seen from Figure 6.3.3 that the total cost of building a 12-layer residential building in Cupertino city's downtown is \$27,612,804.58. Which is equal to the average cost in California, so the residence will payback itself if every apartment will be sold and get a profit of \$57,617,195.42. Including rent payment for the first year from the commercial area and parking area. Median selling price for 1 sq/m in Cupertino city is \$10000 according to realtor.com. But this price is for a fully designed apartment with furniture. In our case, the selling price will be \$6000 for 1 sq/m, as apartments will be sold in the rough finish.

6.4 Work Breakdown Structure

The work breakdown structure (WBS) is critical for organizing project tasks by breaking them down into a hierarchical structure, which simplifies task management. By defining individual work components and arranging them across different levels, it creates a clear interconnection between tasks. This enables more accurate preliminary cost estimates, effective risk analysis, and better project planning. The WBS for this project can be seen in Figure 6.4.1.

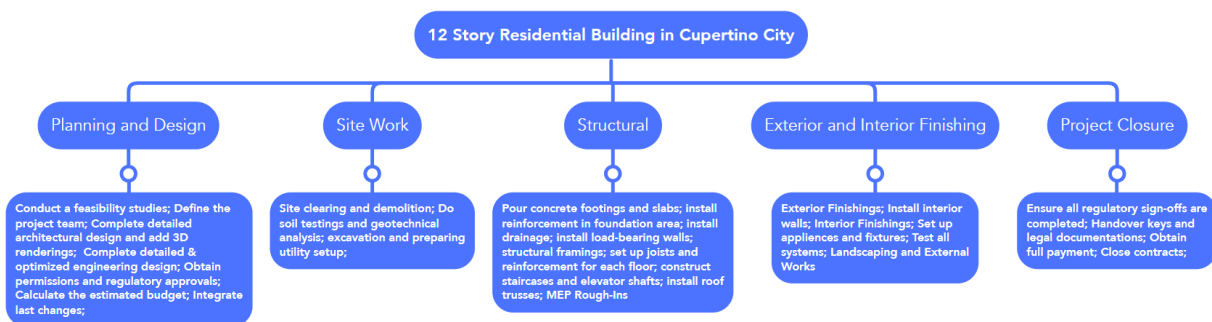


Figure 6.4.1 Work Breakdown Structure

Five main parts of the construction process are: Planning and Design, Site Work, Structural Work, Exterior and Interior Finishings, Project Closure. Overall project will be lasting 200 days, which gives 6160 working hours with a 9/5 working routine. Structural part is the most massive part here. It includes these subparts as well: Foundation Works (excavation, levelling and pouring); Structural Works (reinforcement installations, formworks, slabs & column building); 12-Layer Constructions (building each layer); MEP works.

6.5 Project Schedule

Project was scheduled to start in September 2024 and finish in May 2025. The Schedule is presented in a form of Gantt chart, which was done using ProjectLibre Software, the software used by construction managers around the world. Figure 6.5.1 presents the Project table with the names of the tasks, starting dates, ending dates and each task's predecessor, if it has it. The longest part of the construction process is Structural and Floor Construction parts. Both of them need 85 days to be completed. Days are calculated in a 5/2 work system, with 5 days of the week active and with 2 days of weekends. Each working day consists of 9 active hours, starting from 8 am, and finishing at 5 pm.

		Task Name	Number of days	Starting date	Ending date	Predecessor
1		Project Planning	2 1 ?	02.09.24 8:00	30.09.24 17:00	
2		Location determination	1 0 ?	02.09.24 8:00	13.09.24 17:00	
3		Architectural & Eng Design	1 5 ?	10.09.24 8:00	30.09.24 17:00	
4		Cost estimation	1 5 ?	10.09.24 8:00	30.09.24 17:00	
5		Site Works	2 2 ?	20.09.24 8:00	21.10.24 17:00	
6		Soil testing	1 1 ?	20.09.24 8:00	04.10.24 17:00	
7		Site preparation	1 1 ?	26.09.24 8:00	10.10.24 17:00	
8		Site clearing	1 1 ?	26.09.24 8:00	10.10.24 17:00	
9		Utility setup	8 ?	10.10.24 8:00	21.10.24 17:00	7;8
10		Drainage systems	8 ?	10.10.24 8:00	21.10.24 17:00	7;8
11		Foundation	2 3 ?	10.10.24 8:00	11.11.24 17:00	
12		Excavation	8 ?	10.10.24 8:00	21.10.24 17:00	
13		Levelling	9 ?	15.10.24 8:00	25.10.24 17:00	
14		Pouring	1 4 ?	23.10.24 8:00	11.11.24 17:00	
15		Structural	8 6 ?	11.11.24 8:00	10.03.25 17:00	11
16		Reinforcement Installation	8 1 ?	11.11.24 8:00	03.03.25 17:00	
17		Formwork	8 1 ?	11.11.24 8:00	03.03.25 17:00	
18		Placement of Concrete	6 6 ?	02.12.24 8:00	03.03.25 17:00	
19		Curing & Coating	7 1 ?	02.12.24 8:00	10.03.25 17:00	
20		Floor Construction	8 6 ?	11.11.24 8:00	10.03.25 17:00	11
21		Columns, beams, slabs	8 6 ?	11.11.24 8:00	10.03.25 17:00	
22		1-12 floor	8 6 ?	11.11.24 8:00	10.03.25 17:00	
23		Roof	1 1 ?	10.03.25 8:00	24.03.25 17:00	22
24		MEP	3 5 ?	24.03.25 8:00	09.05.25 17:00	23
25		Plumbing	3 5 ?	24.03.25 8:00	09.05.25 17:00	
26		Electrical	3 5 ?	24.03.25 8:00	09.05.25 17:00	
27		HVAC	3 5 ?	24.03.25 8:00	09.05.25 17:00	
28		Elevators	3 5 ?	24.03.25 8:00	09.05.25 17:00	
29		Finishing Works	1 9 ?	12.05.25 8:00	05.06.25 17:00	24
30		Windows & Doors	7 ?	12.05.25 8:00	20.05.25 17:00	
31		Interior Finishings	1 2 ?	12.05.25 8:00	27.05.25 17:00	
32		Safety Checks	8 ?	27.05.25 8:00	05.06.25 17:00	
33		Project Documentation	8 ?	27.05.25 8:00	05.06.25 17:00	

Capstone

Figure 6.5.1 Table for tasks

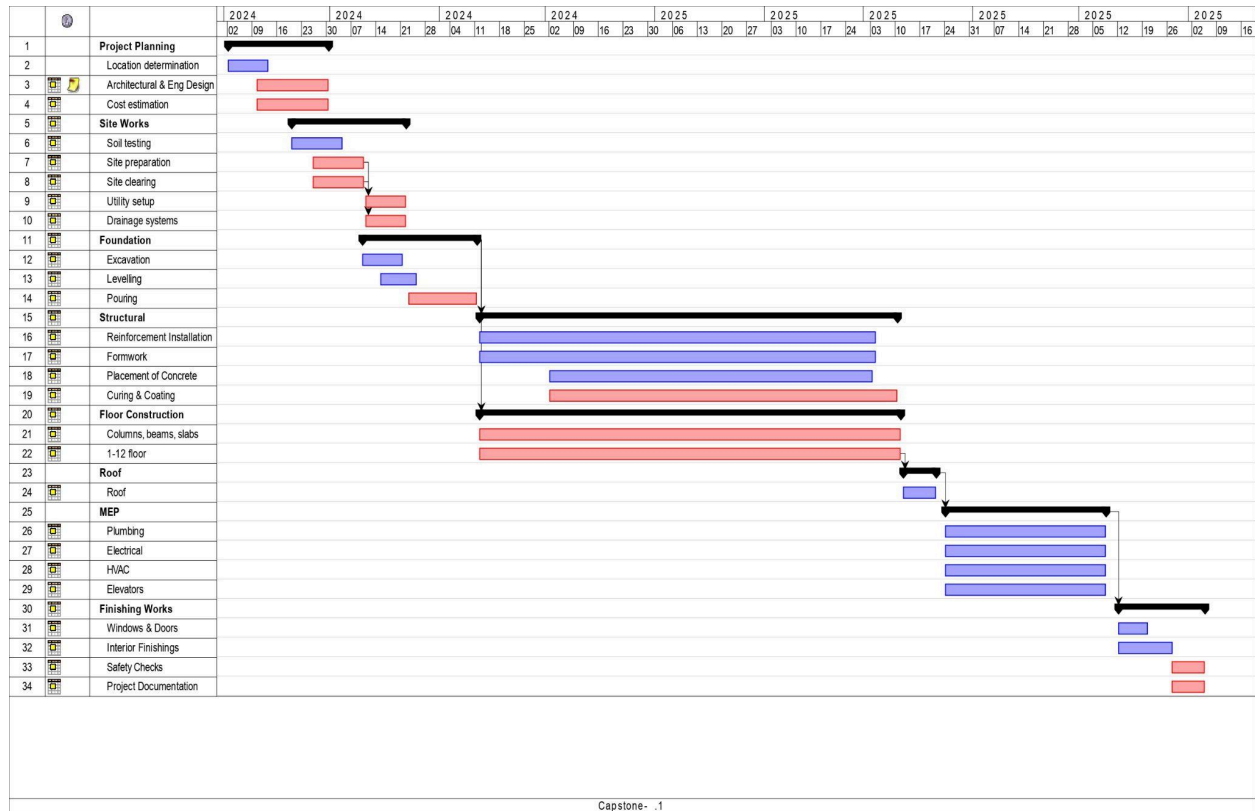


Figure 6.5.2 Gantt chart

Figure 6.5.2 presents a Gantt chart for the whole construction process. Colors are not indicators in this gantt chart. Arrows represent predecessors of each task. There are 8 main tasks, which are: Project Planning, Site Works, Foundation, Structural, Floor Construction, Roof, MEP, and Finishing Works. They are represented as a black intervals. On the top of the chart, year and dates are visible, so the viewer will understand the time frame for each task.

6.6 Risk Management

Risk Management is a crucial component of the entire construction process. By analyzing and predicting potential risks, as well as monitoring and controlling them throughout the construction phase, managers can successfully complete projects without incurring excessive costs or delays. To implement effective risk management, construction managers should identify risks, assess their impact, mitigate them, and monitor and report on resolved issues. Cupertino is situated in a highly seismic zone in one of the most expensive states in the U.S., coupled with a scarcity of workforce. Considering these identified risks and others that may arise, the following table has been created:

Risk Type	ID	Risk Description	Response	Severity	Likelihood
Environmental	EN1	Delays due to heavy snow, heavy rain.	Implement weather monitoring and construct the schedule based on it, Use covers.	Moderate	Unlikely
	EN2	Seismic risks and risks of ground falling	Do seismic assessments, use reinforced structural design. Strength connections and piles	Severe	Likely
	EN3	Soil or foundation issues. Soil erosion	Perform geotechnical analysis. Implement soil stabilization	Moderate	Unlikely
Financial	F1	Unexpected world trends that lead credit, interest rates, currency volatility in the US	Secure loans with fixed rates	Major	Unlikely
	F2	Risks of Budget overruns	Implement strict budget control and do regular financial reviews	Major	Possible
	F3	Increase of costs of material and labor	Negotiate long-term contracts with suppliers	Moderate	Possible
	F4	Unexpected expenses	Maintain 10% of the	Major	Unlikely

		or changes in budget	budget for unforeseen costs		
Safety & health	SH1	Accidents and injuries in construction site	Enforce safety trainings and use of PPE	Moderate	Possible
	SH2	Different unexpected incidents due to lack of safety systems (fire, flooding)	Install fire detection and suppression systems. Ensure flood control measures	Major	Rare
	SH3	Infection diseases and virus spreading	Implement sanitation protocols, medical checks	Moderate	Unlikely
Social	S1	Criminal activities, vandalism	Hire security personnel	Moderate	Rare
Political	P1	Government interference in private business	Engage in community outreach and maintain open communication with government	Moderate	Unlikely
	P2	Unstable political situation, protests	Monitoring the situation in the world and country	Major	Rare
	P3	Geopolitical factors that affect supply	Have plan B for suppliers	Moderate	Unlikely
Legal	L1	Legislation disagreements. Problems with contractors	Establish clear contracts, engage legal experts	Moderate	Possible
Workforce shortage	W1	Shortage of skilled professionals on construction site	Offer competitive wages, provide trainings, and consider bringing in labor from other regions	Major	Likely
	W2	Delays due to wrong distribution of human resources	Improve workforce planning, ensure adequate training and monitor resource allocation	Moderate	Possible

	W3	Productivity of workers	Provide incentives, improve working conditions and encourage some team buildings, breaks	Minor	Possible
Technical	T1	Malfunctioning equipment and machineries	Schedule regular maintenance and inspections. Keep backup equipment	Moderate	Possible
Design	D1	Improper building codes chosen	Review and ensure compliance with local building codes	Major	Unlikely
	D2	Changes of design during the construction	Implement strict design approvals before construction begins	Moderate	Possible
Operational	O1	Conflicts between workers	Promote a collaborative culture, provide conflict resolution training, and establish clear communication channels	Minor	Unlikely
	O2	Changes in project scope, deadlines, schedule. Delays in getting approvals	Set a clear project scope and timeline. Engage stakeholders early to expedite approvals	Moderate	Possible
	O3	Misusing of the resources and corruption	Implement strict resource tracking, perform regular audits, enforce a zero-tolerance policy for unethical practices	Major	Rare

Table 6.6.1 Risk Types Analysis

Once the types of possible risks are identified, the next step is to assess their likelihood and severity based on our specific situation. We use five levels to rate severity—Insignificant, Minor, Moderate, Major, and Severe—and five levels to rate likelihood: Rare, Unlikely, Possible, Likely, and Almost Certain. Using this matrix, risks are classified into five levels: low risk, medium risk, medium-high risk, high risk, and very high risk.

(ADD REF) For instance, Cupertino has a significant shortage of workforce resources, making this risk "likely" in terms of likelihood. The potential impact of this shortage could lead to considerable delays, so we have rated its severity as "major." More can be seen from the Risk Matrix below:

Risk Matrix		Severity				
		Insignificant	Minor	Moderate	Major	Severe
Likelihood	Almost Certain					
	Likely				W1	EN2
	Possible		W3	F3, SH1, L1, W2, T1, D2, O2	F2	
	Unlikely		O1	EN1, EN3, SH3, P1, P3	F1, F4, D1,	
	Rare			S1	SH2, P2, O3	
		Low	Medium	Medium - High	High	Very High

Figure 6.6.1 Risk Matrix

6.7 Quality Management

Quality management provides a standardized approach to evaluating all construction processes, ensuring that each action adheres to consistent quality assessment metrics. This plays a crucial role in the overall construction system, as it enables the assessment of project efficiency and helps predict future client (tenant) satisfaction. Below are the quality control measures used to evaluate construction quality, along with the key metrics that determine the overall quality of the process.

Quality control measures	
Certifications	Industry-recognized standards such as ISO 9001 and LEED. They help in ensuring that the construction project meets the set requirements of quality, safety, and environmental concerns, so improving reliability and credibility
Inspection, Audit and Testing	Regular site inspections, quality audits, and material testing aid in the detection of defects and ensure that specifications are met with construction integrity maintained.
Statistical Quality Control	It applies statistical methods, including control charts and sampling, in monitoring variations of construction processes for consistency to minimize defects.
BIM testing	BIM testing finds design clashes, structural inconsistencies, and a host of other likely issues before the beginning of construction, hence enhancing accuracy and efficiency.
Satisfaction checklists	Organized lists that determine if the project deliverables meet the client's expectation and quality standards to ensure overall satisfaction and compliance with agreed specifications.

Table 6.7.1 Quality Control Measures

Quality control metrics	
Compliance with the initial time schedule	Monitors the milestones and construction deadlines achieved according to plans in order to maintain quality by reducing rush work and making sure processes are done correctly
Rework percentage	The value for the work which has to be reworked due to defects, force majeure cases, natural disasters or bad workmanship, that directly affects project budget and timeline
Customer satisfaction score	Evaluates client feedback and satisfaction levels based on the quality, timeliness and overall project execution, influencing reputation and future contracts
Compliance with the initial budget	Assesses how well the initial budget was regulated and how much more/less budget is used in the construction process
Defect rate	The percentage of the defective work / errors concerning total output.

Table 6.7.2 Quality Control Metrics

Table 6.7.3 below is the sample for a quality control checklist which ensures consistent application of methods, procedures and requirements across all processes, activities and work components.

Issue	Answer		
Name of the inspected are			
Available documentation / certifications	☐ YES	☐ NO	☐ N/A
What documents are missing?			
Is the work fully finished?	☐ YES	☐ NO	☐ N/A

Does the work meet all quality standards?	☐ YES	☐ NO	☐ N/A
What are the sights where the work does not meet required specifications?			
Are there defects in the installation?	☐ YES	☐ NO	☐ N/A
Has the work been audited?	☐ YES	☐ NO	☐ N/A
Has the work been inspected?	☐ YES	☐ NO	☐ N/A
Has the work been tested?	☐ YES	☐ NO	☐ N/A
If not, when the audit/inspection/testing scheduled?			
Has the scope of the work has passed all Quality Control Metrics?	☐ YES	☐ NO	☐ N/A
Initials and Signature of the checklist filler	Initials _____ Sign _____		

Table 6.7.3 Quality Control Checklist

6.8 Procurement Planning

Defining roles and responsibilities of all the stakeholders refers to planning. On the other hand, for the project to be executed successfully, those relationships need to be established and maintained so as to be clear, well structured and realistic. The diagram below illustrated is an effective stakeholder relationship model according to Yang, Wei, and Zhang, (2023).

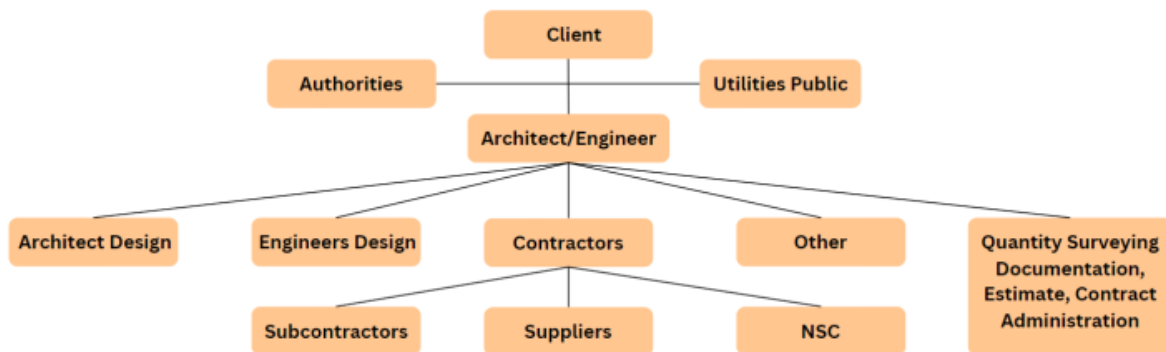


Figure 6.8.1 Stakeholders' relationship

Key Points of Managing Stakeholder Relationships:

1. Identifying all stakeholders. To begin, everyone involved in the project must be identified. The above diagram shows the common stakeholders in construction projects.
2. Understanding stakeholder expectations. When it comes to discovering what both sides are seeking from the project there is a need. The possibilities of successful outcomes come near when they get handled their interests in the most effective way. For example, whether residents care or not whether they get noise or not, government agencies are interested in the timelines, budgets, and all the other technicalities.
3. Good communication. Communication that is effective is needed to share progress reports, express concern and a way to resolve problems as they come up. In construction it is also certain that controversy is a given, hence a clear process to solve such controversy is also necessary.
4. Encouraging active involvement. And so this is an occasion in which you can actually get real feedback because you are engaging actively with the stakeholders through regular meetings and consultation. The role is evidenced, it means responsibility, it confirms trust, and therefore opens the project.

However, managing stakeholder successfully not only enables the future of the project but also enables the reputation of both the individuals as well as for the future, particularly future collaborations. A procurement matrix is presented in the table below showing the level of interest and influence of each party which can be used as a tool to prioritize the stakeholders during construction process. Good to know the expectations and likelihood impact by each stakeholder. In the case of the residential building construction, property owners and contractors are the most important key stakeholders.

Procurement	Role	Affection	Interest
Property Owners	responsible for initiating the project	High	High
Engineers	play a key role in both design and construction	Moderate	Moderate
Architects	tasked with designing the building	Moderate	High
Construction Contractors	in charge of managing the project	High	High
Subcontractors	carry out construction processes	Moderate	Moderate
Government, Financial Institutions	involved in granting permissions and inspections	High	Moderate
Utility Contractors	deliver services like electricity and water	Moderate	Low
Legalisation consultants	ensuring adherence to environmental regulations	Low	Moderate
Environmental consultants	deal with legal matters	Low	Moderate
Residents of the building	contribute preferences regarding the project	Low	Moderate

Table 6.8.1 Matrix of procurement

6.9 Construction Safety

The existence of construction safety regulations is important to ensure that the workplace is safe and in compliance for workers, visitors, and generally everybody engaged in construction work. The chances of accidents and injuries are greatly reduced when proper safety measure are in place. It not only protects people, yet it is also useful to keeping the project's reputation up and not to make additional charges where unwanted. Therefore, it becomes necessary to implement and adhere to the safety management practices on-site. The categories of safety management related in the construction process are specified in the table below.

Operational safety management measures	Site safety management measures	Worker safety and compliance
● Adoption of a documented safety policy ● Risks assessment and control of mitigation ● Training and response to incidents management ● Controlled access management through permit to enter ● Emergency planning and first aid protocol ● Traffic flow planning and management on the construction site	● Manage the entry of people by fencing of the building site ● Detailed site layout planning	● Employment of Personal Protective Equipment (PPE) ● Regular safety training sessions for staff ● Adherence to safety protocols in following established safety and security procedures ● Compliance with safety guidelines for safe storage of chemicals ● Development and dissemination of emergency plans in case of accidents or disasters.

Table 6.9.1 Safety management measures

Safety management measures diminish a large portion of occurrence risks in a construction site, however, no one can guarantee that we do not have such a risk, as well. In the event of incidents it is important to follow standard procedure and correctly complete the risk register. A model of the process of risk registration is presented on the table below, which could be used by teams to register actual risks and compare them with the original risk assessments.

Name, ID		Risk Type	
Probability (initial)		Probability (actual)	
Impact (initial)		Impact (actual)	
Risk evaluation (initial)		Risk evaluation (actual)	
Responsible person name, ID		Risk Description	
Mitigation process description			
Review date		Resolution date	

Table 6.9.2 Risk registration model

6.10 Construction site planning

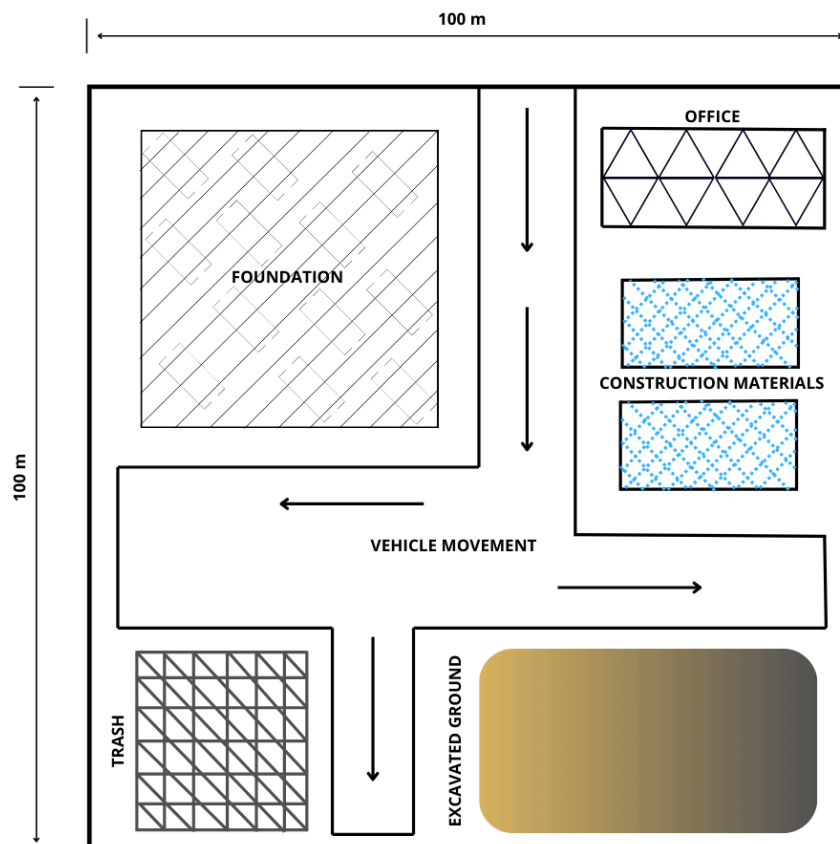


Figure 6.10.1 Construction site planning

The figure above indicates the construction site layout. Among other things, this includes designated areas for foundation work, vehicle access, waste disposal, excavated soil, material storage and site offices. With a total of 100 x 100 meters, the site area is easily accessible and movable for vehicles especially in the middle zone.

The project has achieved all societal objectives of construction management, i.e., planning, scheduling, identification and allocation of risk and their controlling, as well as an assurance of safety and quality standards. As the project is highly profitable with a net income after construction and sale of another \$85,230,000.00 in commercial space rental, the total estimated construction cost comes up to be \$27, 612, 814.58.

7. Conclusion

This is a project in which design and planning is presented for creative elements of its requirements, the Truss Residences, 12-story reinforced concrete residential tower at 10101 N Wolfe Road, Cupertino, California. The building measures for a commercial ground level and one story basement, and 11 stories of 110 units of apartments. The project is a solution for a problem of a high demand of housing, population growth. In the case of the chosen site, which was an abandoned shopping mall previously, the residential building will be the most appropriate place for construction due to quick access to public transport, parks, schools, and business district. The purpose of the project was to design a suitable architectural and structural building, with an appropriate foundation and compliance with the Cupertino building code and Eurocode, ASTM, ASCE, ACI standards, and the California particular seismic code standards. The project also designed a suitable storm water and drainage system, including the topography and constructing a project feasibility with regard to cost and time.

Structurally, a two way slab with minor beams were the most economical and safe design decisions for the project. The frames were calculated with seismic and wind load resistance as well as torsional effect incorporated in them. Finally, for all members such as slabs and beams, and columns, iterations of load distribution to the ACI 318-19 standards were required. Stability assessment was then done using SAP2000 software models to verify stability as well by performing hand calculations of internal forces and lateral drifts.

The soil response analysis, in order to perform geotechnical design analysis, resulted in the identification of Class C soil which, in turn, affected design standards of foundations. Single piles and shallow foundations were outperformed by group pile foundations because their performance together with capacity of bearing load was higher, and the settlements calculated were lower. PLAXIS and GEO5 softwares were used which confirmed that the soil's foundation bearing capacity and settlement meet all safety requirements. As a result, it was decided that a deep foundation consisting of a group of piles penetrating into the soil to 12 m below each column would be used, and as compared to EC7, the calculated settlements were within the 25mm allowance. Softwares confirmed the calculated capacity with a factor of safety of 3, therefore the foundation can withstand safely three times the actual applied load. Manning's equation was used in the environmental engineering design of the drainage system for the stormwater and runoff calculations, which included the time of concentration of rainfall and

the intensity of the rainfall on the natural geography. Topography was analyzed to determine the suitable pipe layout and the water flow direction with minimum costs.

Construction management steps were completed to evaluate the cost-benefit of the project and its durations. Calculations confirmed that material which lasts a long time generates long term savings. To oversee possible shortcomings, work was done in schedule & risk management. Risk mitigation strategies were completed for completing a Work Breakdown Structure to ensure both safe and timely building operations. Scheduling, work distribution, detailed cost estimate, site safety and risk, logistics and feasibility were also considered in the construction management part. The project was projected to spend \$27.6 million and to make \$84 million in revenue.

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Appendices

Table 9.1. Calculations of stiffness of the moment frame.

Floor	Column size	Ic,cr, m4	Ib,cr, m4	Minor b, Ib,cr	D, kN/m	h, m	Cf, kN/rad
1	0,7	1,40E-02	1,89E-03	0,00035438	8503	3,5	178565
2	0,65	1,04E-02	1,89E-03	0,00035438	11446	3	206027
3	0,65	1,04E-02	1,89E-03	0,00035438	11446	3	206027
4	0,6	7,56E-03	1,89E-03	0,00035438	11097	3	199755
5	0,55	5,34E-03	1,89E-03	0,00035438	10607	3	190924
6	0,55	5,34E-03	1,89E-03	0,00035438	10607	3	190924
7	0,5	3,65E-03	1,89E-03	0,00035438	9915	3	178466
8	0,45	2,39E-03	1,89E-03	0,00035438	8949	3	161085
9	0,4	1,49E-03	1,89E-03	0,00035438	7646	3	137630
10	0,4	1,49E-03	1,89E-03	0,00035438	7646	3	137630
11	0,3	4,73E-04	1,89E-03	0,00035438	4162	3	74908
12	0,3	4,73E-04	1,89E-03	0,00035438	4162	3	74908
13	0,3	4,73E-04	1,89E-03	0,00035438	4162	3	74908

Estimations of total lateral seismic force including torsional effect for each floor are demonstrated in Tables 9.2-9.21

Table 9.2.. Calculation of seismic force on each frame considering the torsional effect (Floor 2).

Floors	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
2	A	1	35,45	0,3165	19,70	20,01
	B	2	35,45	0,2110	19,70	19,91
	C	3	35,45	0,1055	19,70	19,80
	D	4	35,45	0,0000	19,70	19,70

	E	5	35,45	-0,1055	19,70	19,59
	F	6	35,45	-0,2110	19,70	19,49
	G	7	35,45	-0,3165	19,70	19,38

Table 9.3. Calculation of seismic force on each frame including the torsional effect (Floor 3).

Floors	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
3	A	1	65,99	0,5892	36,66	37,25
	B	2	65,99	0,3928	36,66	37,05
	C	3	65,99	0,1964	36,66	36,86
	D	4	65,99	0,0000	36,66	36,66
	E	5	65,99	-0,1964	36,66	36,46
	F	6	65,99	-0,3928	36,66	36,27
	G	7	65,99	-0,5892	36,66	36,07

Table 9.4. Calculation of seismic force on each frame including the torsional effect (Floor 4).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
4	A	1	101,42	0,9055	56,35	57,25
	B	2	101,42	0,6037	56,35	56,95
	C	3	101,42	0,3018	56,35	56,65
	D	4	101,42	0,0000	56,35	56,35
	E	5	101,42	-0,3018	56,35	56,04
	F	6	101,42	-0,6037	56,35	55,74
	G	7	101,42	-0,9055	56,35	55,44

Table 9.5. Calculation of seismic force on each frame including the torsional effect (Floor 5).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
5	A	1	141,63	1,2646	78,68	79,95
	B	2	141,63	0,8430	78,68	79,53
	C	3	141,63	0,4215	78,68	79,11
	D	4	141,63	0,0000	78,68	78,68
	E	5	141,63	-0,4215	78,68	78,26
	F	6	141,63	-0,8430	78,68	77,84
	G	7	141,63	-1,2646	78,68	77,42

Table 9.6. Calculation of seismic force on each frame including the torsional effect (Floor 6).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
6	A	1	189,21	1,6894	105,12	106,81
	B	2	189,21	1,1263	105,12	106,24
	C	3	189,21	0,5631	105,12	105,68
	D	4	189,21	0,0000	105,12	105,12
	E	5	189,21	-0,5631	105,12	104,55
	F	6	189,21	-1,1263	105,12	103,99
	G	7	189,21	-1,6894	105,12	103,43

Table 9.7. Calculation of seismic force on each frame including the torsional effect (Floor 7).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
7	A	1	237,93	2,1244	132,18	134,31
	B	2	237,93	1,4162	132,18	133,60

	C	3	237,93	0,7081	132,18	132,89
	D	4	237,93	0,0000	132,18	132,18
	E	5	237,93	-0,7081	132,18	131,47
	F	6	237,93	-1,4162	132,18	130,77
	G	7	237,93	-2,1244	132,18	130,06

Table 9.8. Calculation of seismic force on each frame including the torsional effect (Floor 8).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
8	A	1	290,11	2,5902	161,17	163,76
	B	2	290,11	1,7268	161,17	162,90
	C	3	290,11	0,8634	161,17	162,03
	D	4	290,11	0,0000	161,17	161,17
	E	5	290,11	-0,8634	161,17	160,31
	F	6	290,11	-1,7268	161,17	159,44
	G	7	290,11	-2,5902	161,17	158,58

Table 9.9. Calculation of seismic force on each frame including the torsional effect (Floor 9).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
9	A	1	345,58	3,0855	191,99	195,07
	B	2	345,58	2,0570	191,99	194,05
	C	3	345,58	1,0285	191,99	193,02
	D	4	345,58	0,0000	191,99	191,99
	E	5	345,58	-1,0285	191,99	190,96
	F	6	345,58	-2,0570	191,99	189,93

	G	7	345,58	-3,0855	191,99	188,90
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Table 9.10. Calculation of seismic force on each frame including the torsional effect (Floor10).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
10	A	1	409,41	3,6555	227,45	231,11
	B	2	409,41	2,4370	227,45	229,89
	C	3	409,41	1,2185	227,45	228,67
	D	4	409,41	0,0000	227,45	227,45
	E	5	409,41	-1,2185	227,45	226,23
	F	6	409,41	-2,4370	227,45	225,01
	G	7	409,41	-3,6555	227,45	223,79

Table 9.11. Calculation of seismic force on each frame including the torsional effect (Floor 11).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
11	A	1	466,20	4,1625	259,00	263,16
	B	2	466,20	2,7750	259,00	261,77
	C	3	466,20	1,3875	259,00	260,38
	D	4	466,20	0,0000	259,00	259,00
	E	5	466,20	-1,3875	259,00	257,61
	F	6	466,20	-2,7750	259,00	256,22
	G	7	466,20	-4,1625	259,00	254,83

Table 9.12. Calculation of seismic force on each frame including the torsional effect (Floor12).

Floor	Longitudinal Frame	Transverse Frame	T, kN-m	Ftorsion, kN	Fdirect, kN	Ftotal, kN
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12	A	1	536,47	4,7899	298,04	302,83
	B	2	536,47	3,1933	298,04	301,23
	C	3	536,47	1,5966	298,04	299,63
	D	4	536,47	0,0000	298,04	298,04
	E	5	536,47	-1,5966	298,04	296,44
	F	6	536,47	-3,1933	298,04	294,84
	G	7	536,47	-4,7899	298,04	293,25

Table 9.13. Calculation of seismic force on each frame including the torsional effect (Floor 13).

Floor	Longitudinal				Transverse			
13	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
	A	4,8715	303,11	307,99	3	22,7336	303,11	325,85
	B	3,2477	303,11	306,36	4	0,0000	303,11	303,11
	C	1,6238	303,11	304,74	5	22,7336	303,11	325,85
	E	-1,6238	303,11	301,49				
	F	-3,2477	303,11	299,87				
	G	-4,8715	303,11	298,24				

Appendix B

Table 9.14. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 1).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
1	A	1	1,4865	90,37	91,86
	B	2	0,9910	90,37	91,36

	C	3	0,4955	90,37	90,87
	D	4	0,0000	90,37	90,37
	E	5	-0,4955	90,37	89,87
	F	6	-0,9910	90,37	89,38
	G	7	-1,4865	90,37	88,88

Table 9.15. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 2).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
2	A	1	1,1705	77,00	78,18
	B	2	0,7804	77,00	77,78
	C	3	0,3902	77,00	77,39
	D	4	0,0000	77,00	77,00
	E	5	-0,3902	77,00	76,61
	F	6	-0,7804	77,00	76,22
	G	7	-1,1705	77,00	75,83

Table 9.16. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 3).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
3	A	1	1,3046	84,49	85,79
	B	2	0,8697	84,49	85,36
	C	3	0,4349	84,49	84,92
	D	4	0,0000	84,49	84,49

	E	5	-0,4349	84,49	84,05
	F	6	-0,8697	84,49	83,62
	G	7	-1,3046	84,49	83,18

Table 9.17. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 4).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
4	A	1	1,4110	90,58	91,99
	B	2	0,9407	90,58	91,52
	C	3	0,4703	90,58	91,05
	D	4	0,0000	90,58	90,58
	E	5	-0,4703	90,58	90,11
	F	6	-0,9407	90,58	89,64
	G	7	-1,4110	90,58	89,17

Table 9.18. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 5).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
5	A	1	1,5005	97,94	99,44
	B	2	1,0003	97,94	98,94
	C	3	0,5002	97,94	98,44
	D	4	0,0000	97,94	97,94
	E	5	-0,5002	97,94	97,44
	F	6	-1,0003	97,94	96,94

	G	7	-1,5005	97,94	96,44
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Table 9.19. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 6).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
6	A	1	1,6475	102,51	104,16
	B	2	1,0983	102,51	103,61
	C	3	0,5492	102,51	103,06
	D	4	0,0000	102,51	102,51
	E	5	-0,5492	102,51	101,96
	F	6	-1,0983	102,51	101,41
	G	7	-1,6475	102,51	100,86

Table 9.20. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 7).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
7	A	1	1,6475	104,46	106,11
	B	2	1,0983	104,46	105,56
	C	3	0,5492	104,46	105,01
	D	4	0,0000	104,46	104,46
	E	5	-0,5492	104,46	103,91
	F	6	-1,0983	104,46	103,36
	G	7	-1,6475	104,46	102,81

Table 9.21. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 8).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
8	A	1	1,7101	108,19	109,90
	B	2	1,1401	108,19	109,33
	C	3	0,5700	108,19	108,76
	D	4	0,0000	108,19	108,19
	E	5	-0,5700	108,19	107,62
	F	6	-1,1401	108,19	107,05
	G	7	-1,7101	108,19	106,48

Table 9.22. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 9).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
9	A	1	1,7675	111,63	113,40
	B	2	1,1784	111,63	112,81
	C	3	0,5892	111,63	112,22
	D	4	0,0000	111,63	111,63
	E	5	-0,5892	111,63	111,04
	F	6	-1,1784	111,63	110,45
	G	7	-1,7675	111,63	109,86

Table 9.23. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 10).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
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10	A	1	1,8206	114,82	116,64
	B	2	1,2137	114,82	116,03
	C	3	0,6069	114,82	115,43
	D	4	0,0000	114,82	114,82
	E	5	-0,6069	114,82	114,21
	F	6	-1,2137	114,82	113,61
	G	7	-1,8206	114,82	113,00

Table 9.24. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 11).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
11	A	1	5,8180	117,80	123,62
	B	2	3,8786	117,80	121,68
	C	3	1,9393	117,80	119,74
	D	4	0,0000	117,80	117,80
	E	5	-1,9393	117,80	115,86
	F	6	-3,8786	117,80	113,92
	G	7	-5,8180	117,80	111,98

Table 9.25. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 12).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
12	A	1	1,9164	120,61	122,52
	B	2	1,2776	120,61	121,88

	C	3	0,6388	120,61	121,25
	D	4	0,0000	120,61	120,61
	E	5	-0,6388	120,61	119,97
	F	6	-1,2776	120,61	119,33
	G	7	-1,9164	120,61	118,69

Table 9.26. Calculation of wind force on each frame for Case 2 including the torsional effect (Floor 13).

Floor	Longitudinal				Transverse			
	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
13	A	0,6534	101,64	102,29	2	3,0492	101,64	104,69
	B	0,4356	101,64	102,08	3	0,0000	101,64	101,64
	C	0,2178	101,64	101,86	4	-3,0492	101,64	98,59
	E	-0,2178	101,64	101,42				
	F	-0,4356	101,64	101,20				
	G	-0,6534	101,64	100,99				

Table 9.27. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 1).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
1	A	1	1,1158	67,84	68,95
	B	2	0,7439	67,84	68,58
	C	3	0,3719	67,84	68,21
	D	4	0,0000	67,84	67,84

	E	5	-0,3719	67,84	67,47
	F	6	-0,7439	67,84	67,09
	G	7	-1,1158	67,84	66,72

Table 9.29. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 2).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
2	A	1	0,8787	57,80	58,68
	B	2	0,5858	57,80	58,39
	C	3	0,2929	57,80	58,10
	D	4	0,0000	57,80	57,80
	E	5	-0,2929	57,80	57,51
	F	6	-0,5858	57,80	57,22
	G	7	-0,8787	57,80	56,93

Table 9.30. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 3).

Floors	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
3	A	1	0,9793	63,42	64,40
	B	2	0,6529	63,42	64,07
	C	3	0,3264	63,42	63,75
	D	4	0,0000	63,42	63,42
	E	5	-0,3264	63,42	63,09
	F	6	-0,6529	63,42	62,77

	G	7	-0,9793	63,42	62,44
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Table 9.31. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 1).

Floor	Longitudinal Frame	Transverse Frame	Ftension, kN	Fdirect, kN	Ftotal, kN
3	A	1	0,9793	63,42	64,40
	B	2	0,6529	63,42	64,07
	C	3	0,3264	63,42	63,75
	D	4	0,0000	63,42	63,42
	E	5	-0,3264	63,42	63,09
	F	6	-0,6529	63,42	62,77
	G	7	-0,9793	63,42	62,44

Table 9.32. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 4).

Floor	Longitudinal Frame	Transverse Frame	Ftension, kN	Fdirect, kN	Ftotal, kN
4	A	1	1,0592	67,99	69,05
	B	2	0,7061	67,99	68,70
	C	3	0,3531	67,99	68,35
	D	4	0,0000	67,99	67,99
	E	5	-0,3531	67,99	67,64
	F	6	-0,7061	67,99	67,29
	G	7	-1,0592	67,99	66,94

Table 9.33. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 5).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
5	A	1	1,1263	73,52	74,64
	B	2	0,7509	73,52	74,27
	C	3	0,3754	73,52	73,89
	D	4	0,0000	73,52	73,52
	E	5	-0,3754	73,52	73,14
	F	6	-0,7509	73,52	72,77
	G	7	-1,1263	73,52	72,39

Table 9.34. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 6).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
6	A	1	1,2367	76,95	78,19
	B	2	0,8245	76,95	77,78
	C	3	0,4122	76,95	77,36
	D	4	0,0000	76,95	76,95
	E	5	-0,4122	76,95	76,54
	F	6	-0,8245	76,95	76,13
	G	7	-1,2367	76,95	75,71

Table 9.35. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 7).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
7	A	1	1,2367	78,41	79,65
	B	2	0,8245	78,41	79,24
	C	3	0,4122	78,41	78,83
	D	4	0,0000	78,41	78,41
	E	5	-0,4122	78,41	78,00
	F	6	-0,8245	78,41	77,59
	G	7	-1,2367	78,41	77,18

Table 9.36. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 8).

Floor	Longitudinal Frame	Transverse Frame	Ftorsion, kN	Fdirect, kN	Ftotal, kN
8	A	1	1,2837	81,22	82,50
	B	2	0,8558	81,22	82,07
	C	3	0,4279	81,22	81,65
	D	4	0,0000	81,22	81,22
	E	5	-0,4279	81,22	80,79
	F	6	-0,8558	81,22	80,36
	G	7	-1,2837	81,22	79,93

Table 9.37. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 9).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
9	A	1	1,3268	83,80	85,12
	B	2	0,8845	83,80	84,68
	C	3	0,4423	83,80	84,24
	D	4	0,0000	83,80	83,80
	E	5	-0,4423	83,80	83,36
	F	6	-0,8845	83,80	82,91
	G	7	-1,3268	83,80	82,47

Table 9.38. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 10).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
10	A	1	1,3667	86,19	87,56
	B	2	0,9111	86,19	87,10
	C	3	0,4556	86,19	86,65
	D	4	0,0000	86,19	86,19
	E	5	-0,4556	86,19	85,74
	F	6	-0,9111	86,19	85,28
	G	7	-1,3667	86,19	84,83

Table 9.39. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 11).

Floor	Longitudinal Frame	Transverse Frame	Ftortion, kN	Fdirect, kN	Ftotal, kN
11	A	1	0,0000	88,43	88,43

	B	2	2,9116	88,43	91,34
	C	3	1,4558	88,43	89,89
	D	4	0,0000	88,43	88,43
	E	5	-1,4558	88,43	86,97
	F	6	-2,9116	88,43	85,52
	G	7	-4,3673	88,43	84,06

Table 9.40. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 12).

Floor	Longitudinal Frame	Transverse Frame	Ftension, kN	Fdirect, kN	Ftotal, kN
12	A	1	1,4386	90,54	91,97
	B	2	0,9591	90,54	91,49
	C	3	0,4795	90,54	91,01
	D	4	0,0000	90,54	90,54
	E	5	-0,4795	90,54	90,06
	F	6	-0,9591	90,54	89,58
	G	7	-1,4386	90,54	89,10

Table 9.41. Calculation of wind force on each frame for Case 4 including the torsional effect (Floor 13).

Floors	Longitudinal				Transverse			
	Frame	Ftension, kN	Fdirect, kN	Ftotal, kN	Frame	Ftension, kN	Fdirect, kN	Ftotal, kN
13	A	0,4905	76,30	76,79	2	2,2889	76,30	78,59
	B	0,3270	76,30	76,62	3	0,0000	76,30	76,30
	C	0,1635	76,30	76,46	4	-2,2889	76,30	74,01

	E	-0,1635	76,30	76,13	
	F	-0,3270	76,30	75,97	
	G	-0,4905	76,30	75,81	

Appendix C

Table 9.42: Major beam details

Major beam for 1 floor			
Variables	End	Mid	End
SAP Tension As mm2	1794	1119	2230
Area in in2	2.78053	1.7343 5	3.4562 9
SAP Compression	947	714	1085
Area in in2	1.46776	1.1066 3	1.6816 5
Tension bar	3#9	3#8	3#10
Compression bar	4#6	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.43: Major beam details

Major beam for 2 floor			
Variables	End	Mid	End
SAP Tension As mm2	996	806	2531
Area in in2	1.54371	1.2492 3	3.9228 1
SAP Compression	645	806	1226
Area in in2	0.99969	1.2492 3	1.9001 9
Tension bar	3#7	3#6	3#11
Compression bar	4#5	4#6	4#7
Shear Reinforcement	3#130	4#300	3#130

Table 9.44: Major beam details

Major beam for 3 floor			
Variables	End	Mid	End
SAP Tension As mm2	1247	995	2477

Area in in2	1.93273	1.5421 6	3.8391 2
SAP Compression	995	789	1201
Area in in2	1.54216	1.2228 8	1.8614 4
Tension bar	3#8	3#7	3#11
Compression bar	4#6	4#5	4#7
Shear Reinforcement	3#130	4#300	3#130

Table 9.45: Major beam details

Major beam for 4 floor			
Variables	End	Mid	End
SAP Tension As mm2	1347	760	2133
Area in in2	2.08772	1.1779 3	3.3059 5
SAP Compression	1147	684	1039
Area in in2	1.77774	1.0601 4	1.6103 5
Tension bar	3#8	3#6	3#10
Compression bar	4#7	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.46: Major beam details

Major beam for 5 floor			
Variables	End	Mid	End
SAP Tension As mm2	1564	770	2073
Area in in2	2.42405	1.1934 3	3.2129 6
SAP Compression	1242	666	1011
Area in in2	1.92498	1.0322 4	1.5669 6
Tension bar	3#9	3#6	3#10
Compression bar	4#7	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.47: Major beam details

Major beam for 6 floor			
Variables	End	Mid	End

SAP Tension As mm2	947	782	2007
Area in in2	1.46776	1.2120 3	3.1106 6
SAP Compression	477	645	995
Area in in2	0.73931	0.9996 9	1.5421 6
Tension bar	3#7	3#6	3#10
Compression bar	4#4	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.48: Major beam details

Major beam for 7 floor			
Variables	End	Mid	End
SAP Tension As mm2	947	794	1924
Area in in2	1.46776	1.2306 3	2.9820 2
SAP Compression	388	619	995
Area in in2	0.60136	0.9593 9	1.5421 6
Tension bar	3#7	3#6	3#9
Compression bar	4#4	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.49: Major beam details

Major beam for 8 floor			
Variables	End	Mid	End
SAP Tension As mm2	1221	813	1777
Area in in2	1.89244	1.2600 7	2.7541 8
SAP Compression	741	573	995
Area in in2	1.14848	0.8881	1.5421 6

Tension bar	3#8	3#6	3#9
Compression bar	4#5	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.50: Major beam details

Major beam for 9 floor			
Variables	End	Mid	End
SAP Tension As mm2	1075	840	1596
Area in in2	1.66615	1.3019 2	2.4736 5
SAP Compression	643	517	995
Area in in2	0.99659	0.8013	1.5421 6
Tension bar	3#7	3#6	3#9
Compression bar	4#5	4#5	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.51: Major beam details

Major beam for 10 floor			
Variables	End	Mid	End
SAP Tension As mm2	947	866	1425
Area in in2	1.46776	1.3422 2	2.2086 2
SAP Compression	567	463	934
Area in in2	0.8788	0.7176 1	1.4476 1
Tension bar	3#7	3#7	3#8
Compression bar	4#5	4#4	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.52: Major beam details

Major beam for 11 floor			
Variables	End	Mid	End
SAP Tension As mm2	322	888	1226
Area in in2	0.49907	1.3763 2	1.9001 9
SAP Compression	161	400	806

Area in in2	0.24954	0.6199 6	1.2492 3
Tension bar	3#4	3#7	3#8
Compression bar	4#3	4#4	4#6
Shear Reinforcement	3#130	4#300	3#130

Table 9.53: Major beam details

Major beam for 12 floor			
Variables	End	Mid	End
SAP Tension As mm2	520	925	1010
Area in in2	0.80595	1.4336 6	1.5654 1
SAP Compression	258	330	665
Area in in2	0.39988	0.5114 7	1.0306 9
Tension bar	3#5	3#7	3#7
Compression bar	4#3	4#4	4#5
Shear Reinforcement	3#130	4#300	3#130

Table 9.54: Minor beam details

Minor beam for 1 floor			
Variables	End	Mid	End
SAP Tension As mm2	373	191	474
Area in in2	0.57812	0.2960 3	0.7346 6
SAP Compression	231	153	264
Area in in2	0.35803	0.2371 4	0.4091 8
Tension bar	2#5	2#4	2#6
Compression bar	2#4	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.55: Minor beam details

Minor beam for 2 floor			
Variables	End	Mid	End
SAP Tension As mm2	475	213	556

Area in in2	0.73621	0.3301 3	0.8617 5
SAP Compression	254	178	271
Area in in2	0.39368	0.2758 8	0.4200 2
Tension bar	2#6	2#4	2#6
Compression bar	2#5	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.56: Minor beam details

Minor beam for 3 floor			
Variables	End	Mid	End
SAP Tension As mm2	246	216	576
Area in in2	0.38128	0.3347 8	0.8927 5
SAP Compression	122	184	280
Area in in2	0.18909	0.2851 8	0.4339 7
Tension bar	2#4	2#4	2#7
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.57: Minor beam details

Minor beam for 4 floor			
Variables	End	Mid	End
SAP Tension As mm2	235	218	587
Area in in2	0.36423	0.3378 8	0.9098
SAP Compression	117	188	215
Area in in2	0.18134	0.2913 8	0.3332 3
Tension bar	2#4	2#4	2#7
Compression bar	2#3	2#4	2#4
Shear Reinforcement	3#71	3#150	3#71

Table 9.58: Minor beam details

Minor beam for 5 floor			
Variables	End	Mid	End

SAP Tension As mm2	231	218	591
Area in in2	0.35803	0.3378 8	0.916
SAP Compression	114	189	287
Area in in2	0.17669	0.2929 3	0.4448 2
Tension bar	2#4	2#4	2#7
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.59: Minor beam details

Minor beam for 6 floor			
Variables	End	Mid	End
SAP Tension As mm2	226	217	593
Area in in2	0.35028	0.3363 3	0.9190 9
SAP Compression	112	190	288
Area in in2	0.17359	0.2944 8	0.4463 7
Tension bar	2#4	2#4	2#7
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.60: Minor beam details

Minor beam for 7 floor			
Variables	End	Mid	End
SAP Tension As mm2	226	217	588
Area in in2	0.35028	0.3363 3	0.9113 5
SAP Compression	112	188	286
Area in in2	0.17359	0.2913 8	0.4432 7
Tension bar	2#4	2#4	2#7
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.61: Minor beam details

Minor beam for 8 floor			
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Variables	End	Mid	End
SAP Tension As mm2	237	214	572
Area in in2	0.36733	0.3316 8	0.8865 5
SAP Compression	117	183	278
Area in in2	0.18134	0.2836 3	0.4308 7
Tension bar	2#4	2#4	2#7
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.62: Minor beam details

Minor beam for 9 floor			
Variables	End	Mid	End
SAP Tension As mm2	256	211	544
Area in in2	0.39678	0.3270 3	0.8431 5
SAP Compression	127	175	265
Area in in2	0.19684	0.2712 3	0.4107 3
Tension bar	2#5	2#4	2#6
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.63: Minor beam details

Minor beam for 10 floor			
Variables	End	Mid	End
SAP Tension As mm2	264	207	515
Area in in2	0.40918	0.3208 3	0.7982
SAP Compression	136	165	264
Area in in2	0.21079	0.2557 3	0.4091 8
Tension bar	2#5	2#4	2#6
Compression bar	2#3	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.64: Minor beam details

Minor beam for 11 floor			
Variables	End	Mid	End
SAP Tension As mm2	264	207	467
Area in in2	0.40918	0.3208 3	0.7238 1
SAP Compression	151	151	264
Area in in2	0.23404	0.2340 4	0.4091 8
Tension bar	2#5	2#4	2#6
Compression bar	2#4	2#4	2#5
Shear Reinforcement	3#71	3#150	3#71

Table 9.65: Minor beam details

Minor beam for 12 floor			
Variables	End	Mid	End
SAP Tension As mm2	270	208	392
Area in in2	0.41847	0.3223 8	0.6075 6
SAP Compression	178	127	256
Area in in2	0.27588	0.1968 4	0.3967 8
Tension bar	2#5	2#4	2#5
Compression bar	2#4	2#3	2#5
Shear Reinforcement	3#71	3#150	3#71

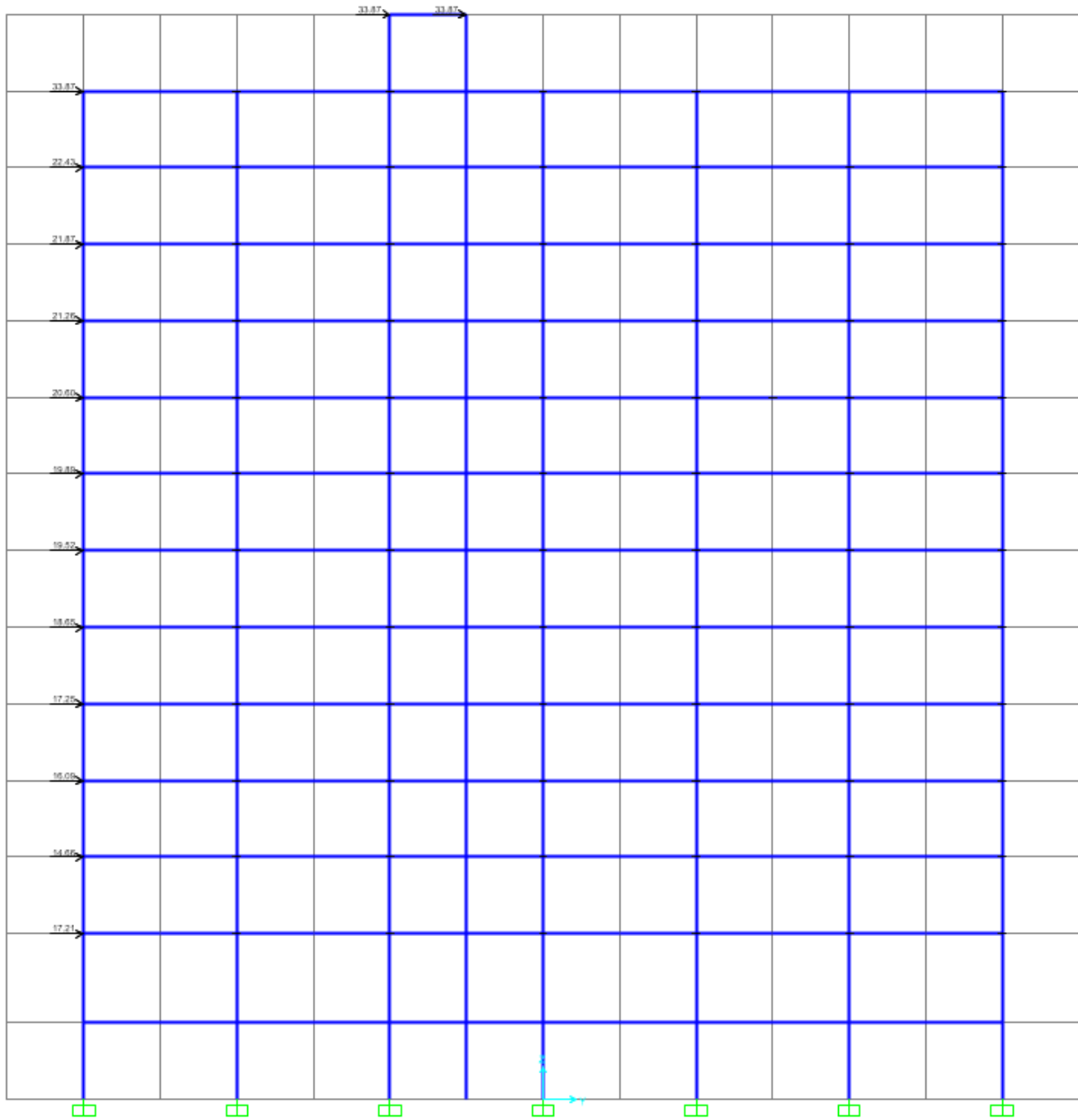


Figure 9.1. Assigned wind loads for Case 1

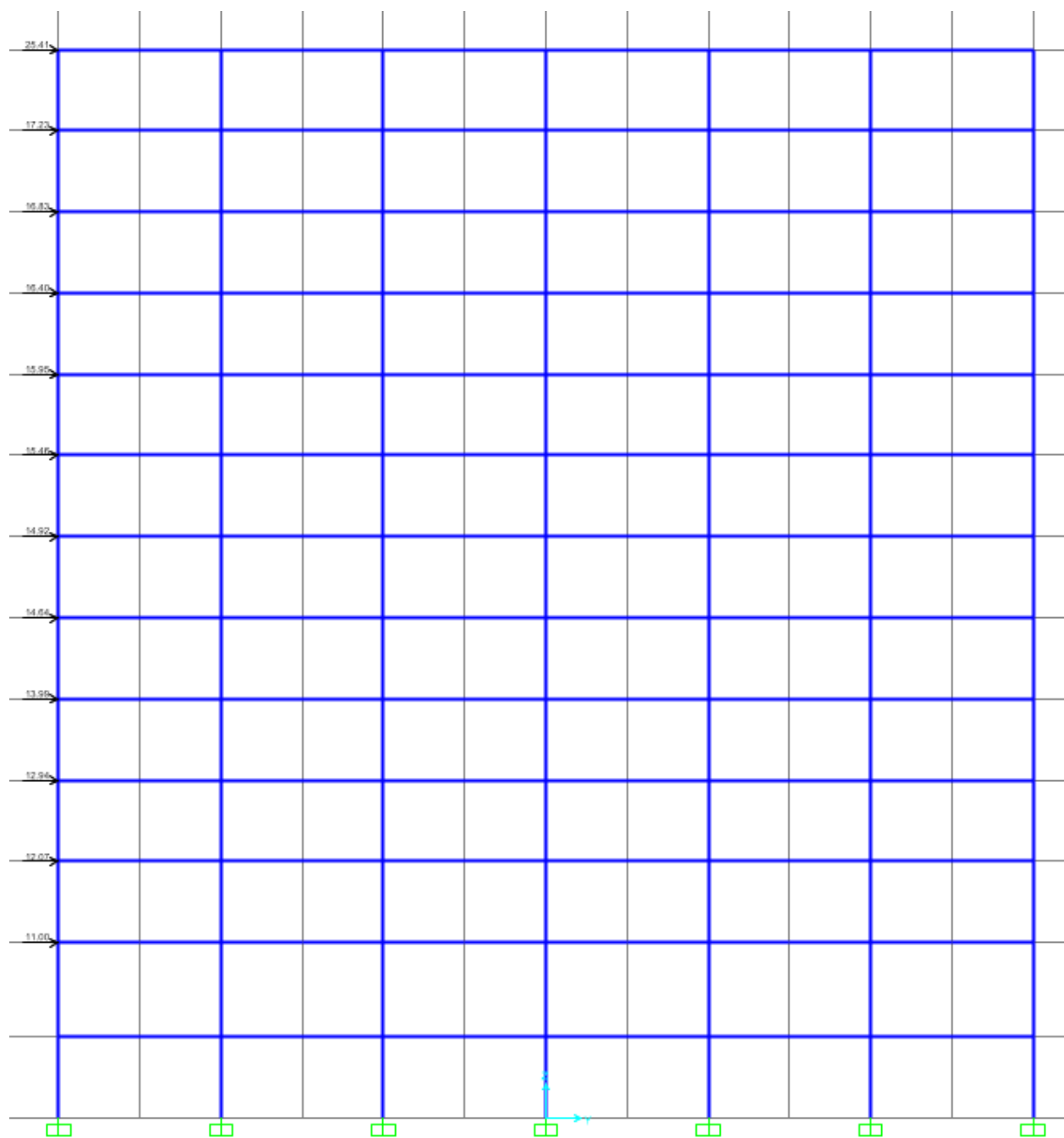


Figure 9.2. Assigned wind loads for Case 3

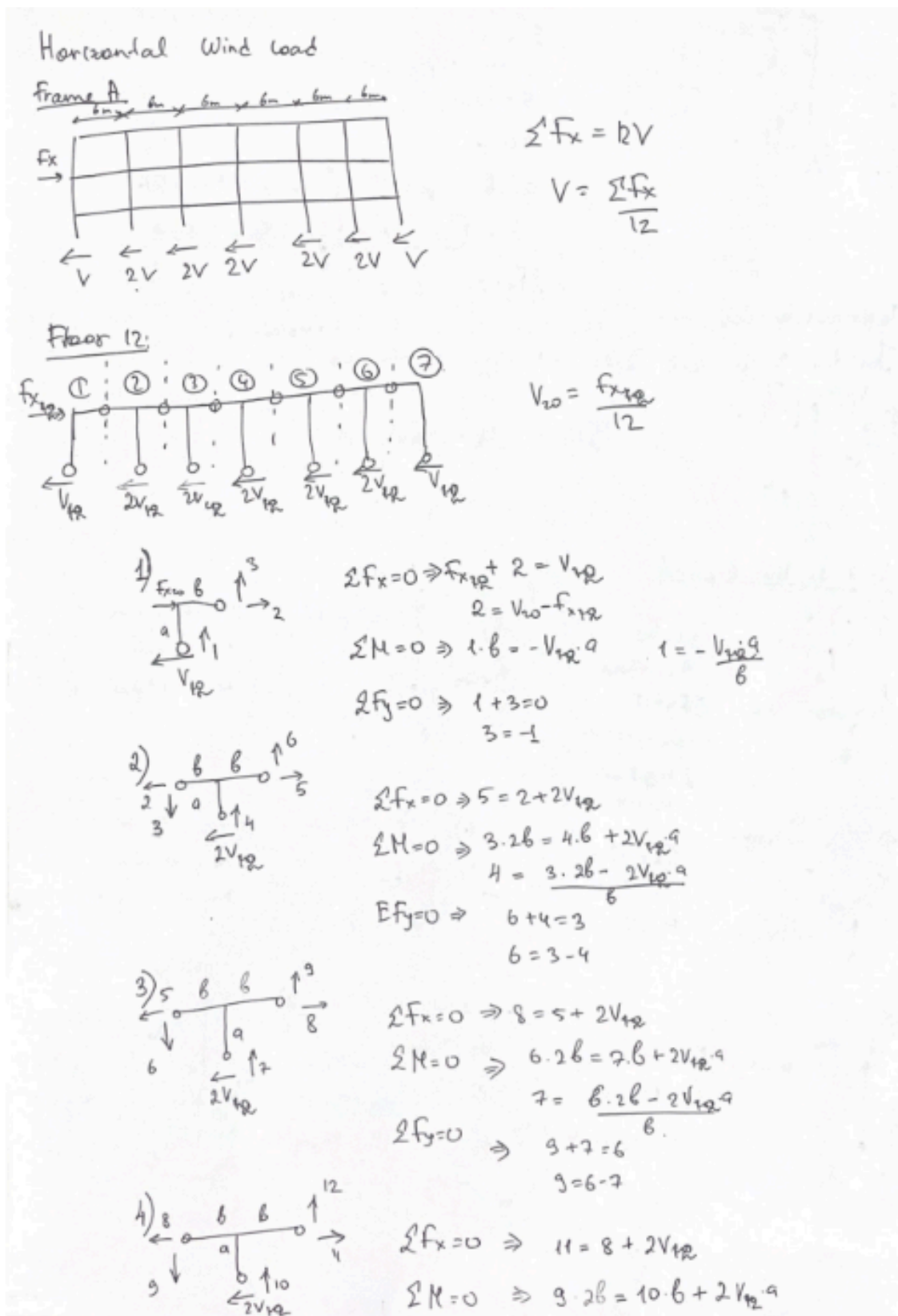


Figure 9.3. Hand calculations for Internal Forces under Wind Load (Part 1)

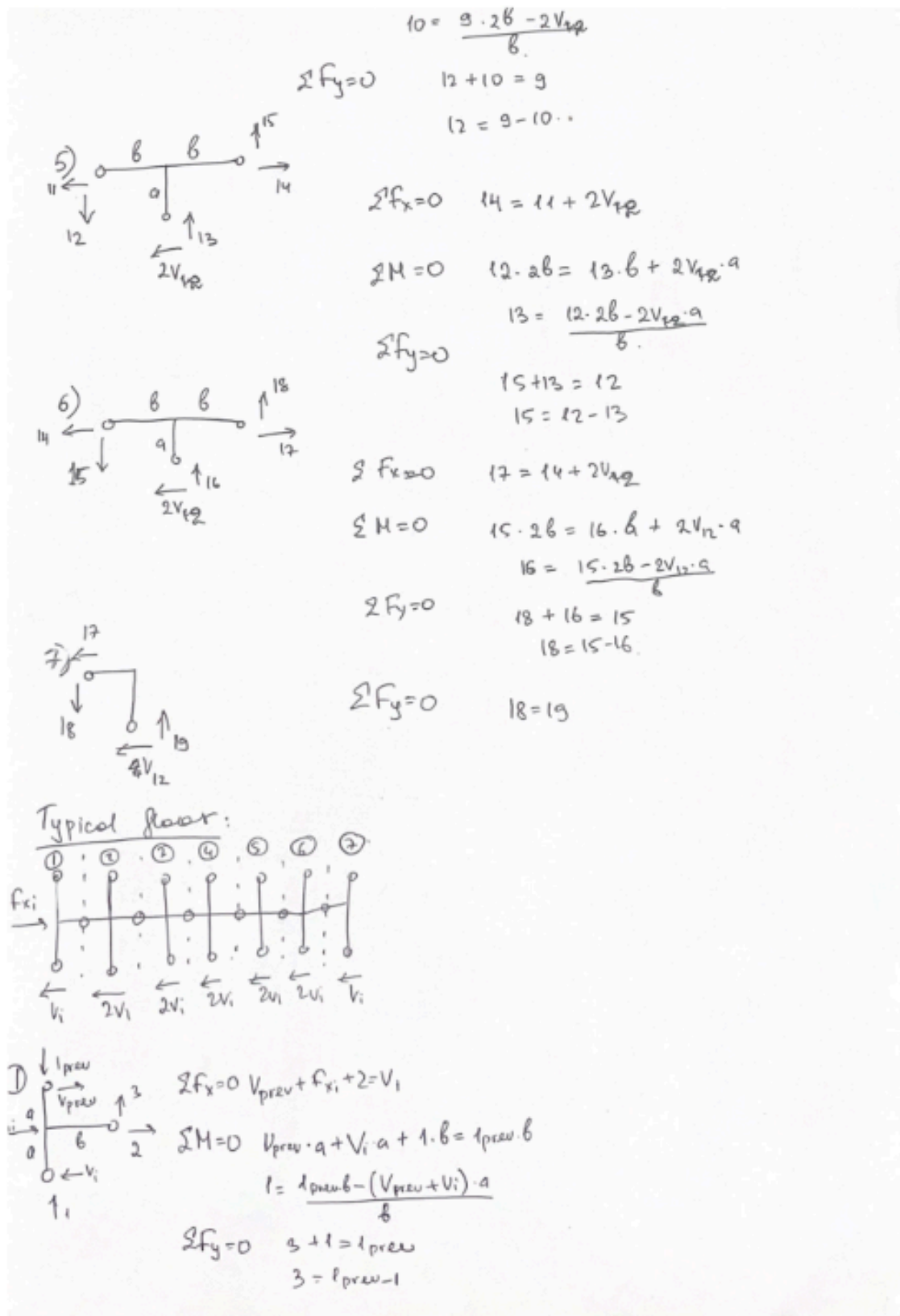


Figure 9.4. Hand calculations for Internal Forces under Wind Load (Part 2)

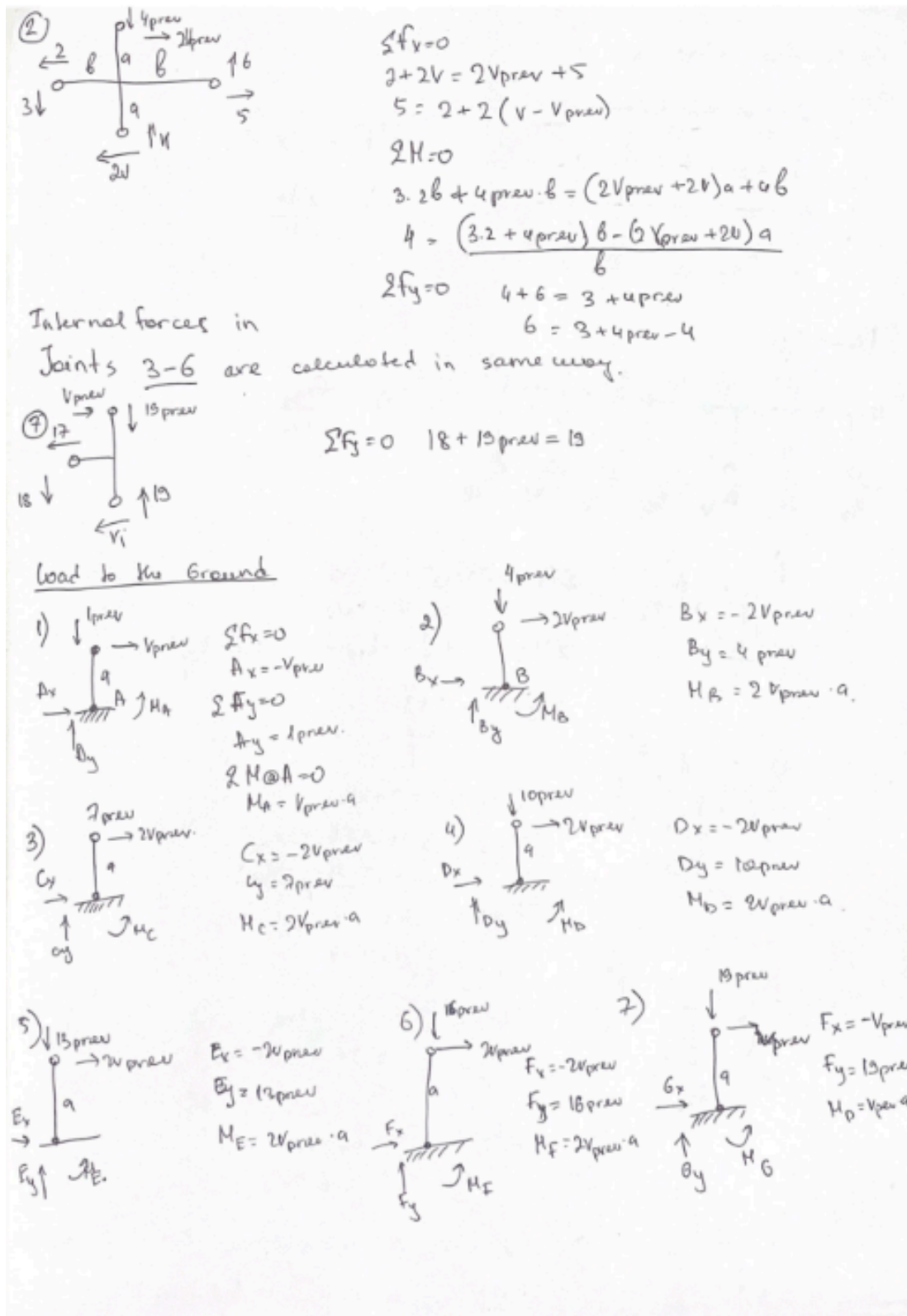
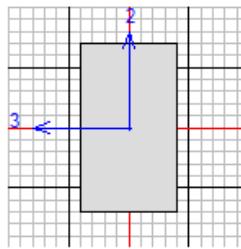


Figure 9.5. Hand calculations for Internal Forces under Wind Load (Part 3)



Units

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

Element	: 1457	D=700.000	B=400.000	bf=400.000
Section ID	: Major beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
252239.046	-504478.091	252239.046	-504478.091

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	2230.555	0.000	2230.555	995.448
Bottom (-2 Axis)	1085.426	1085.426	0.000	995.448

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.834	371.483	205.901	165.582	173.698

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	27052.022	57675.258	161896.229	1844.400

Figure 9.6: Major beam details taken from SAP 2000

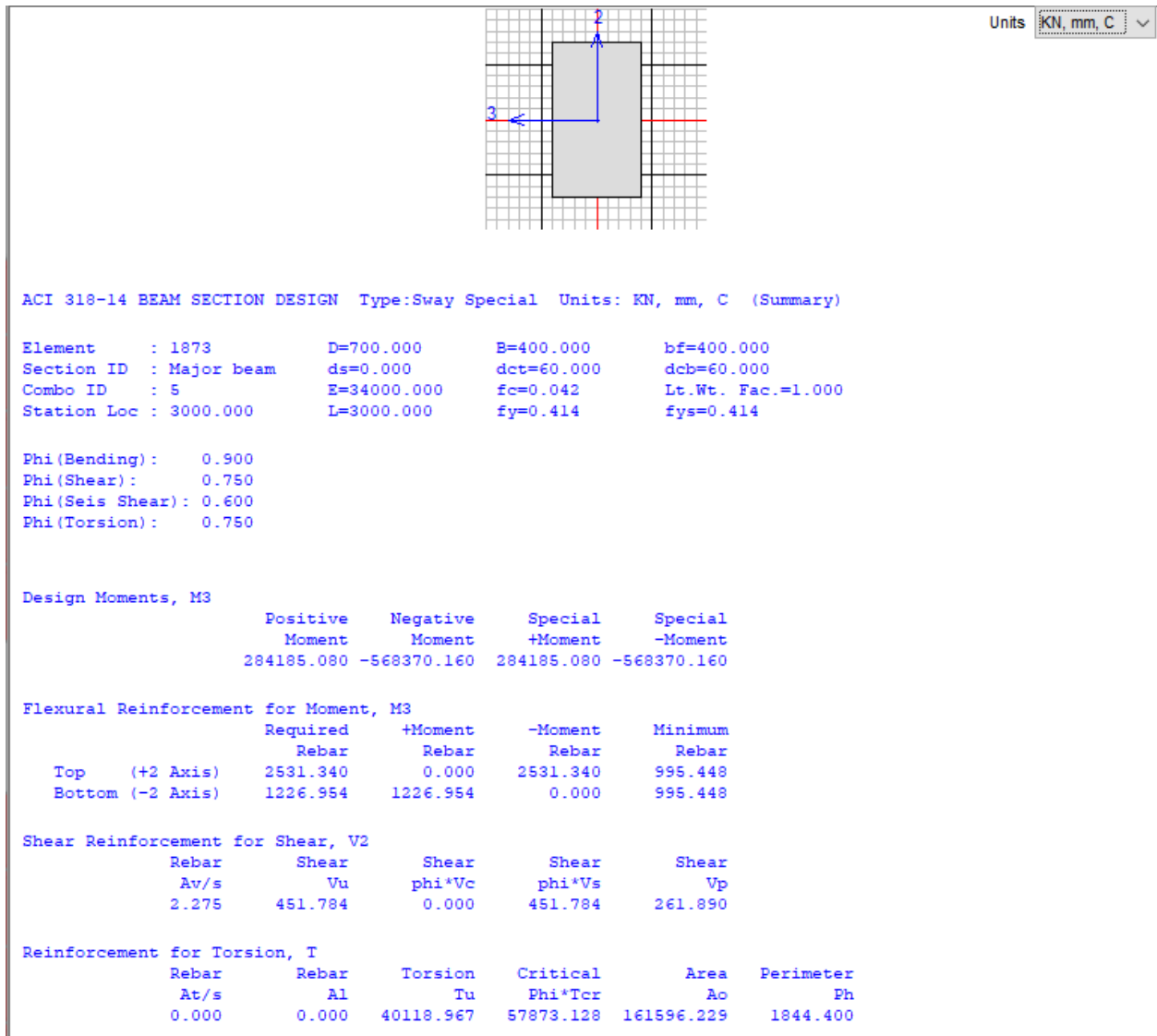


Figure 9.7: Major beam details taken from SAP 2000

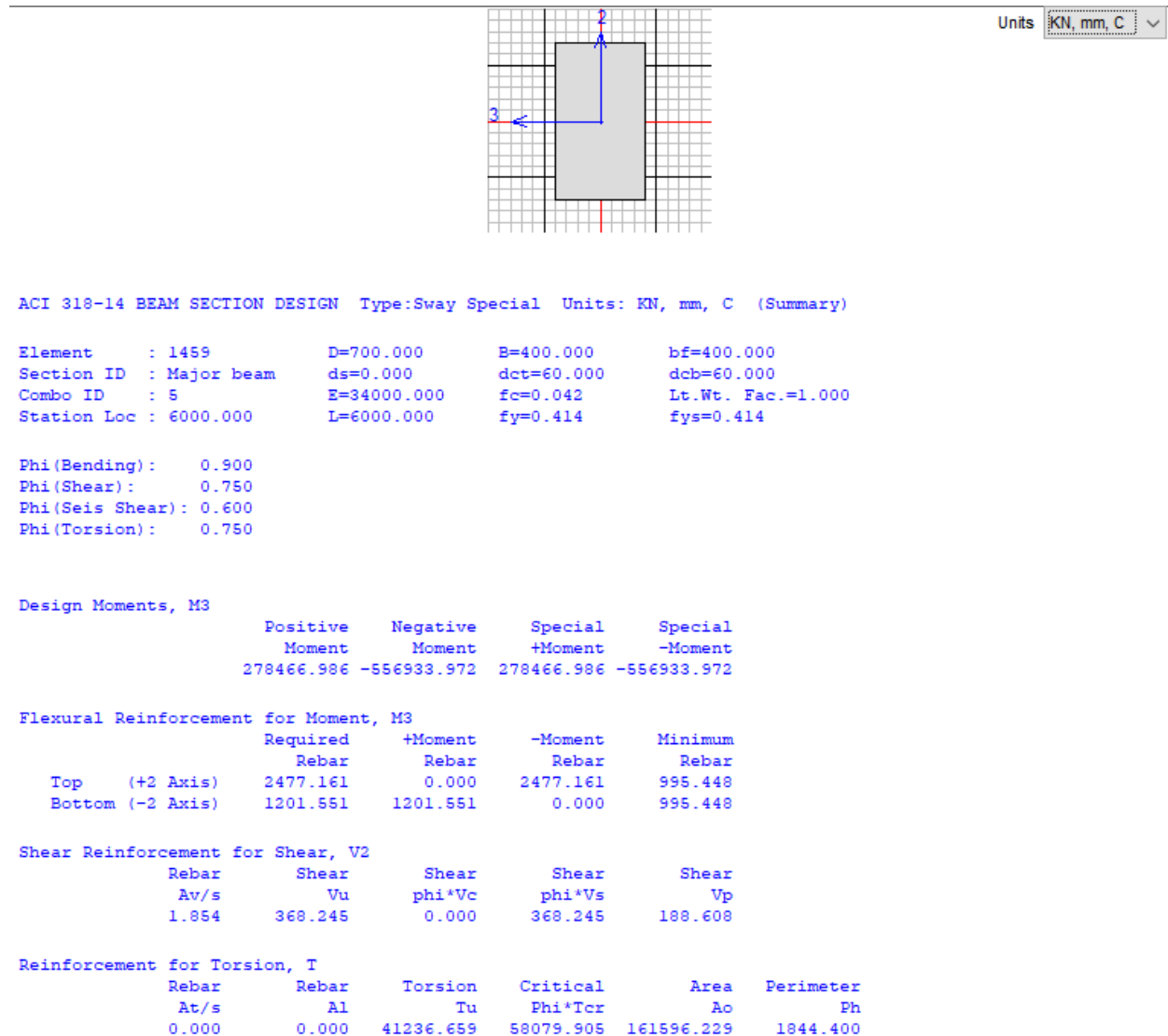
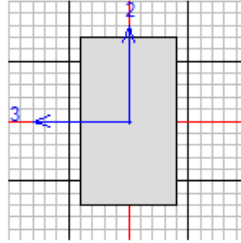


Figure 9.8: Major beam details taken from SAP 2000



Units KN, mm, C v

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

Element	: 1474	D=700.000	B=400.000	bf=400.000
Section ID	: Major beam	ds=0.000	dct=60.000	dcB=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
235410.843	-470821.685	235410.843	-470821.685

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	2073.942	0.000	2073.942	995.448
Bottom (-2 Axis)	1011.259	1011.259	0.000	995.448

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
1.585	314.793	0.000	314.793	167.163

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	43353.428	57960.776	161596.229	1844.400

Figure 9.9: Major beam details taken from SAP 2000

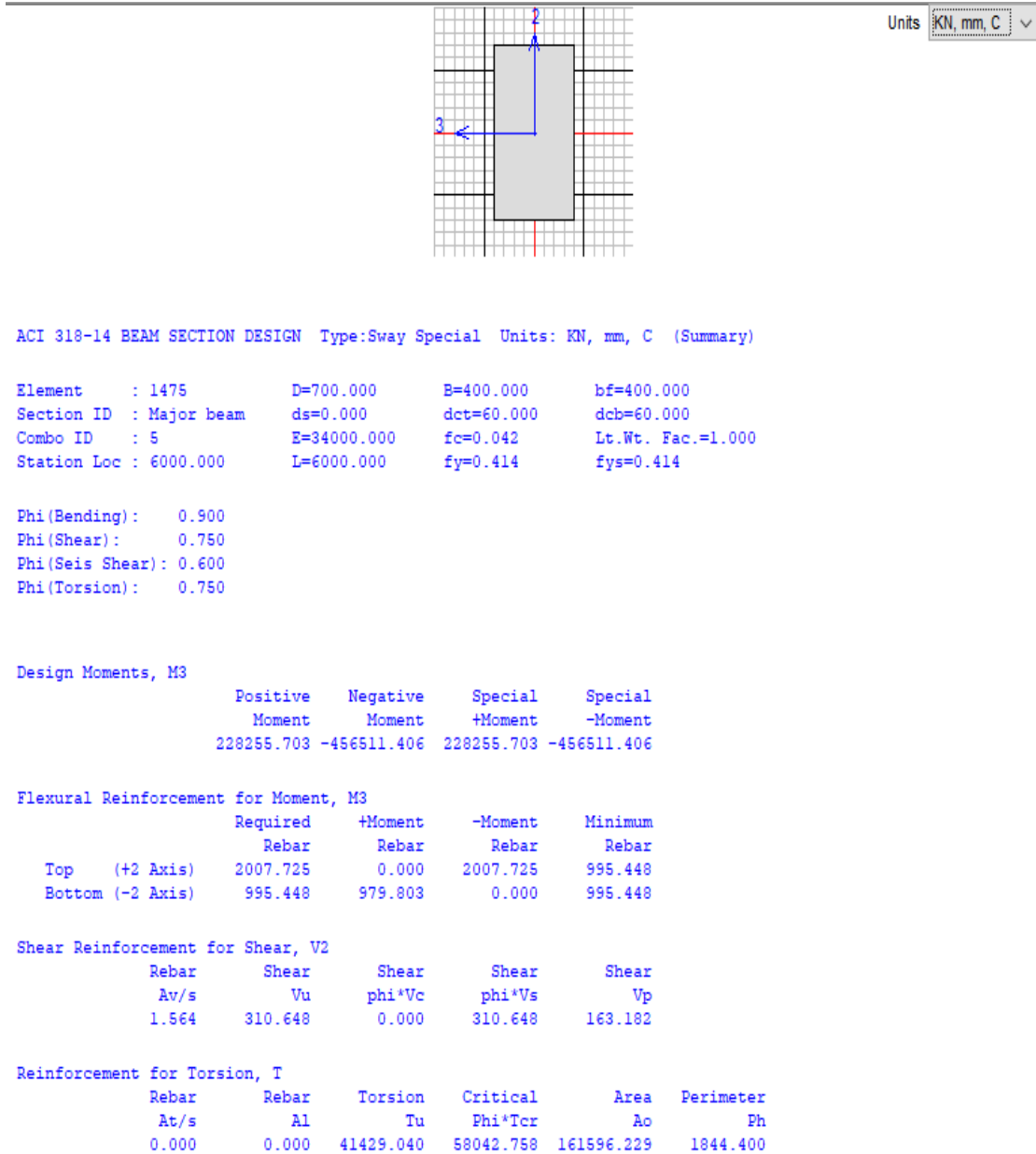
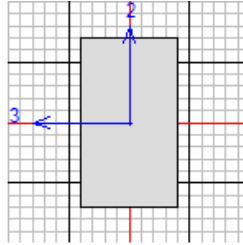


Figure 9.10: Major beam details taken from SAP 2000



Units kN, mm, C v

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

Element : 1476 D=700.000 B=400.000 bf=400.000
 Section ID : Major beam ds=0.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
219186.009	-438372.018	219186.009	-438372.018

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1924.105	0.000	1924.105	995.448
Bottom (-2 Axis)	995.448	939.998	0.000	995.448

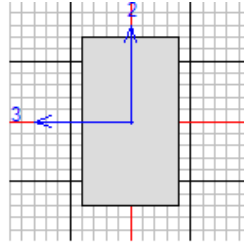
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
1.546	307.017	0.000	307.017	160.095

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	38993.054	58655.021	161596.229	1844.400

Figure 9.11: Major beam details taken from SAP 2000



Units

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

Element : 1477 D=700.000 B=400.000 bf=400.000
 Section ID : Major beam ds=0.000 dct=60.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
203147.390	-406294.779	203147.390	-406294.779

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1777.083	0.000	1777.083	995.448
Bottom (-2 Axis)	995.448	869.793	0.000	995.448

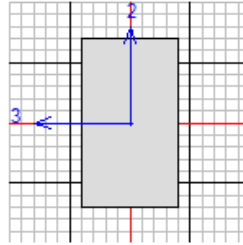
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.476	300.361	205.901	94.460	154.964

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	35048.396	59208.538	161596.229	1844.400

Figure 9.12: Major beam details taken from SAP 2000



Units

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

Element : 1478 D=700.000 B=400.000 bf=400.000
 Section ID : Major beam ds=0.000 dct=60.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
183321.215	-366642.430	183321.215	-366642.430

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1596.809	0.000	1596.809	995.448
Bottom (-2 Axis)	995.448	783.330	0.000	995.448

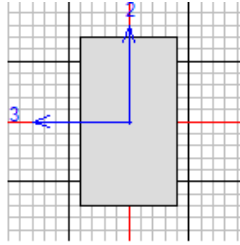
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.433	291.924	205.901	86.023	148.196

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	30146.326	59887.909	161596.229	1844.400

Figure 9.13: Major beam details taken from SAP 2000



Units kN, mm, C

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

Element	: 1479	D=700.000	B=400.000	bf=400.000
Section ID	: Major beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
164264.178	-328528.357	164264.178	-328528.357

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1425.023	0.000	1425.023	995.448
Bottom (-2 Axis)	934.072	700.554	0.000	934.072

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.389	280.591	205.901	77.213	136.907

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	24930.770	59882.022	161596.229	1844.400

Figure 9.14: Major beam details taken from SAP 2000

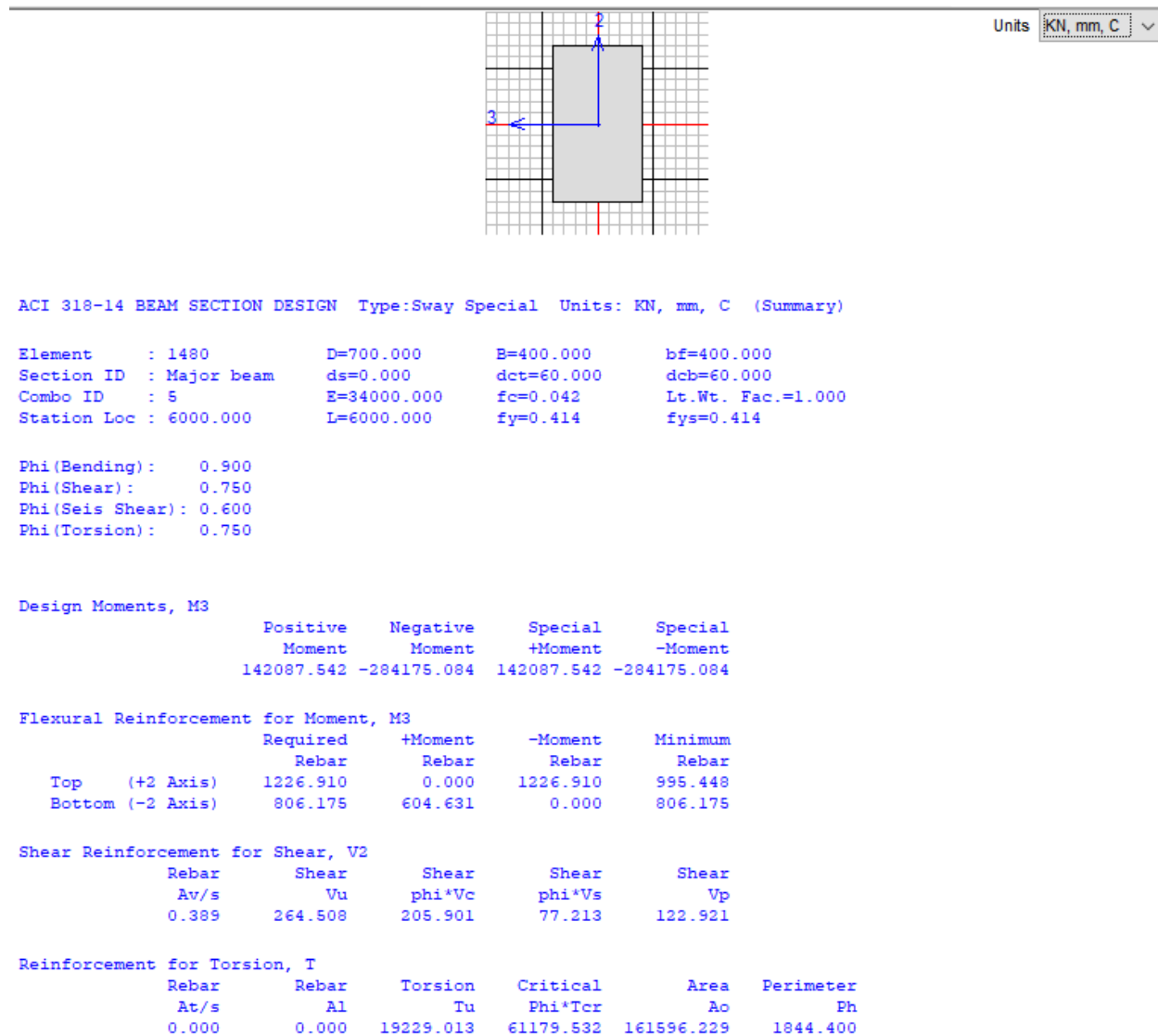
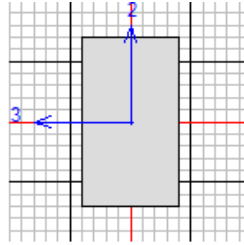


Figure 9.15: Major beam details taken from SAP 2000



Units kN, mm, C v

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

Element	: 1481	D=700.000	B=400.000	bf=400.000
Section ID	: Major beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
117606.911	-235213.823	117606.911	-235213.823

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1010.392	0.000	1010.392	995.448
Bottom (-2 Axis)	665.656	499.242	0.000	665.656

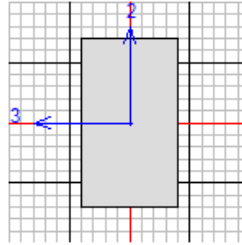
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.195	244.643	205.901	38.742	101.109

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	11556.152	60764.351	161596.229	1844.400

Figure 9.16: Major beam details taken from SAP 2000



Units KN, mm, C v

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

Element	: 1482	D=700.000	B=400.000	bf=400.000
Section ID	: Major beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
21363.871	-42727.741	21363.871	-42727.741

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	240.072	0.000	180.054	240.072
Bottom (-2 Axis)	119.789	89.842	0.000	119.789

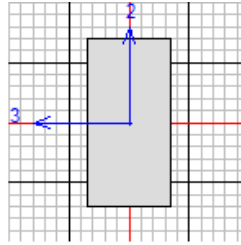
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.000	72.817	205.901	0.000	34.771

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	7777.294	58975.309	161596.229	1844.400

Figure 17: Major beam details taken from SAP 2000



Units

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

Element	: 3763	D=400.000	B=200.000	bf=200.000
Section ID	: Minor beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
28812.035	-57624.071	28812.035	-57624.071

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	474.535	0.000	474.535	264.416
Bottom (-2 Axis)	264.416	232.234	0.000	264.416

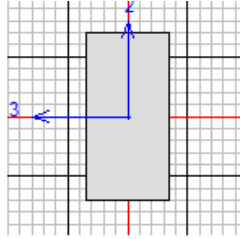
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	58.474	54.692	20.510	20.929

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	3881.479	8594.203	29378.729	844.400

Figure 9.18: Minor beam details taken from SAP 2000



Units:

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

Element : 3679 D=400.000 B=200.000 bf=200.000
 Section ID : Minor beam ds=0.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
33563.735	-67127.471	33563.735	-67127.471

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	556.899	0.000	556.899	264.416
Bottom (-2 Axis)	271.466	271.466	0.000	264.416

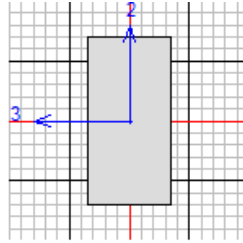
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	61.764	54.692	20.510	23.088

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4598.316	8590.967	29378.729	844.400

Figure 9.19: Minor beam details taken from SAP 2000



Units kN, mm, C v

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

Element	: 3686	D=400.000	B=200.000	bf=200.000
Section ID	: Minor beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
34685.026	-69370.053	34685.026	-69370.053

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	576.524	0.000	576.524	264.416
Bottom (-2 Axis)	280.765	280.765	0.000	264.416

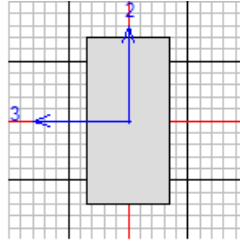
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	63.025	54.692	20.510	23.604

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4743.227	8578.434	29378.729	844.400

Figure 9.20: Minor beam details taken from SAP 2000



Units KN, mm, C v

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

Element	: 3693	D=400.000	B=200.000	bf=200.000
Section ID	: Minor beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	35304.779	-70609.557	35304.779	-70609.557

Flexural Reinforcement for Moment, M3

		Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top	(+2 Axis)	587.402	0.000	587.402	264.416
Bottom	(-2 Axis)	285.911	285.911	0.000	264.416

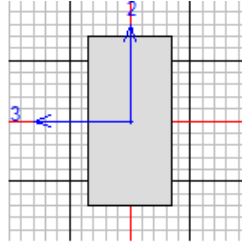
Shear Reinforcement for Shear, V2

Rebar	Shear	Shear	Shear	Shear
Av/s	Vu	phi*Vc	phi*Vs	Vp
0.194	63.920	54.692	20.510	23.892

Reinforcement for Torsion, T

Rebar	Rebar	Torsion	Critical	Area	Perimeter
At/s	Al	Tu	Phi*Tcr	Ao	Ph
0.000	0.000	4817.847	8564.148	29378.729	844.400

Figure 9.21: Minor beam details taken from SAP 2000



Units kN, mm, C v

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

Element	: 3700	D=400.000	B=200.000	bf=200.000
Section ID	: Minor beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
35542.548	-71085.095	35542.548	-71085.095

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	591.581	0.000	591.581	264.416
Bottom (-2 Axis)	287.886	287.886	0.000	264.416

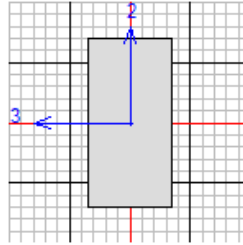
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	64.505	54.692	20.510	24.004

Reinforcement for Torsion, T

Rebar At/s	Rebar A1	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4819.036	8542.926	29378.729	844.400

Figure 9.22: Minor beam details taken from SAP 2000



Units KN, mm, C v

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

Element : 3707 D=400.000 B=200.000 bf=200.000
 Section ID : Minor beam ds=0.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
35649.842	-71299.684	35649.842	-71299.684

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	593.469	0.000	593.469	264.416
Bottom (-2 Axis)	288.778	288.778	0.000	264.416

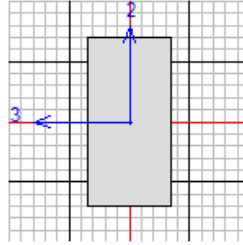
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	64.842	54.692	20.510	23.775

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4814.298	8536.190	29378.729	844.400

Figure 9.23: Minor beam details taken from SAP 2000



Units KN, mm, C v

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

Element	: 3714	D=400.000	B=200.000	bf=200.000
Section ID	: Minor beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
35394.032	-70788.064	35394.032	-70788.064

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	588.970	0.000	588.970	264.416
Bottom (-2 Axis)	286.652	286.652	0.000	264.416

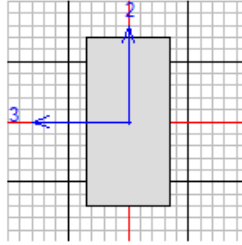
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	64.870	54.692	20.510	23.384

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4758.670	8499.062	29378.729	844.400

Figure 9.24: Minor beam details taken from SAP 2000



Units KN, mm, C v

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

Element	: 3721	D=400.000	B=200.000	bf=200.000
Section ID	: Minor beam	ds=0.000	dct=60.000	dcb=60.000
Combo ID	: 5	E=34000.000	fc=0.042	Lt.Wt. Fac.=1.000
Station Loc	: 6000.000	L=6000.000	fy=0.414	fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
34451.498	-68902.995	34451.498	-68902.995

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	572.431	0.000	572.431	264.416
Bottom (-2 Axis)	278.827	278.827	0.000	264.416

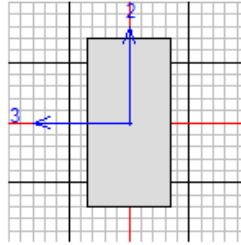
Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	64.358	54.692	20.510	22.761

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4582.550	8461.057	29378.729	844.400

Figure 9.25: Minor beam details taken from SAP 2000



Units

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

Element : 3728 D=400.000 B=200.000 bf=200.000
 Section ID : Minor beam ds=0.000 dct=60.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
32865.597	-65731.194	32865.597	-65731.194

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	544.718	0.000	544.718	264.416
Bottom (-2 Axis)	265.685	265.685	0.000	264.416

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	63.376	54.692	20.510	21.870

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	4331.212	8406.160	29378.729	844.400

Figure 9.26: Minor beam details taken from SAP 2000

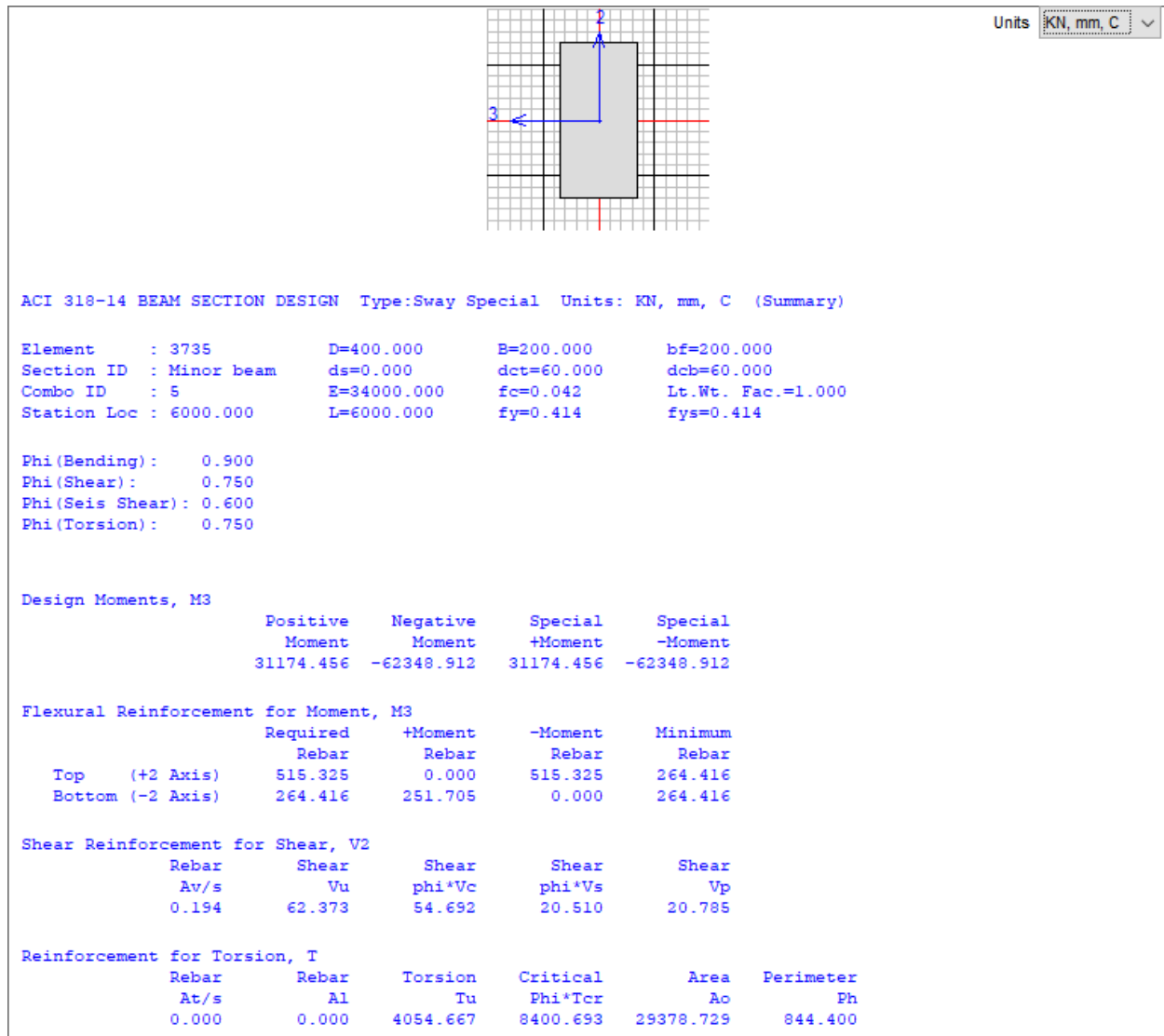
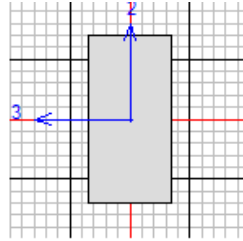


Figure 9.27: Minor beam details taken from SAP 2000



Units

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

Element : 3742 D=400.000 B=200.000 bf=200.000
 Section ID : Minor beam ds=0.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
28415.490	-56830.981	28415.490	-56830.981

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	467.719	0.000	467.719	264.416
Bottom (-2 Axis)	264.416	228.972	0.000	264.416

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.194	60.580	54.692	20.510	19.404

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	3725.869	8279.948	29378.729	844.400

Figure 9.28: Minor beam details taken from SAP 2000

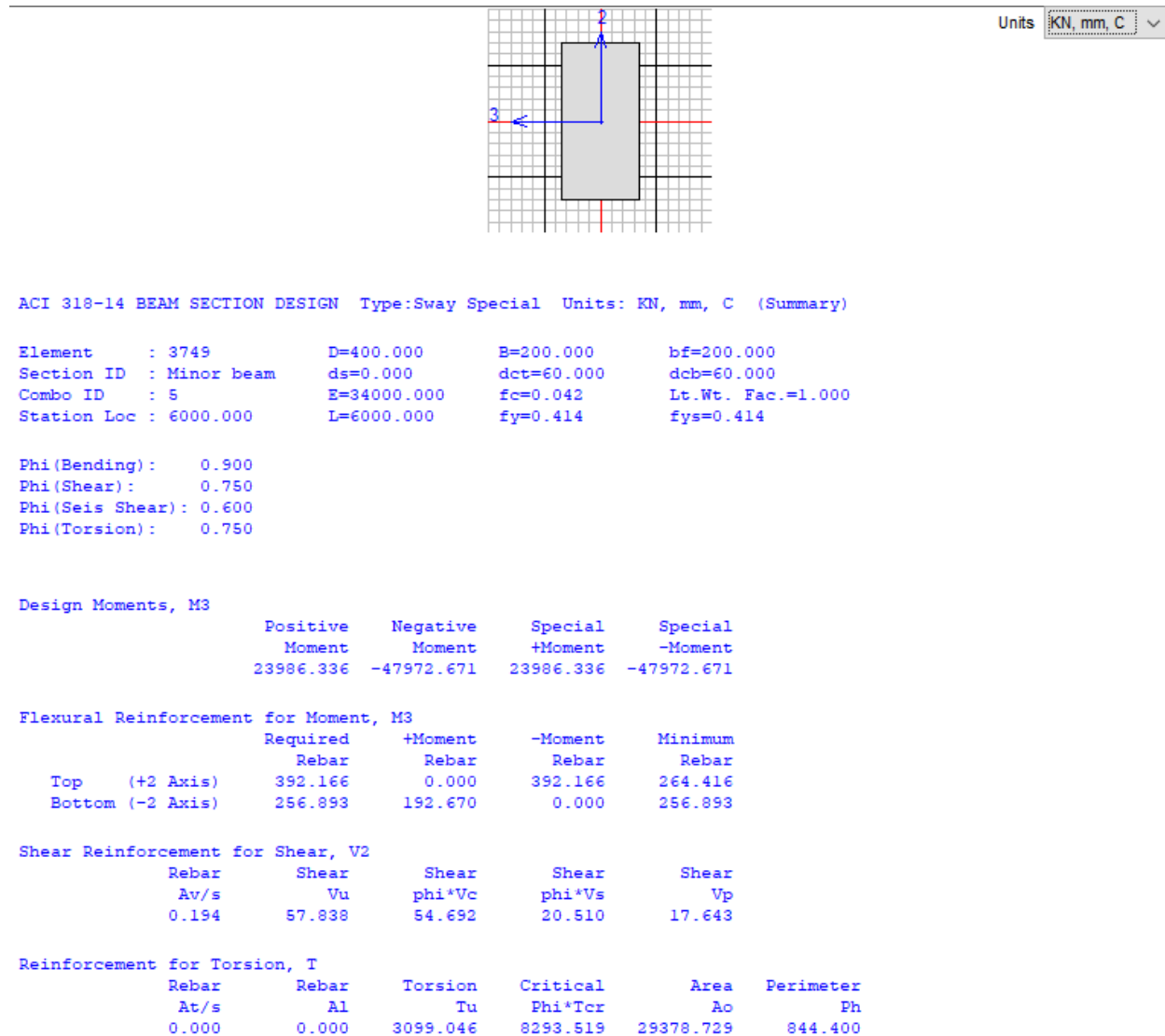
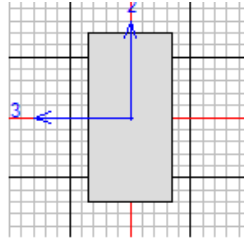


Figure 9.29: Minor beam details taken from SAP 2000



Units

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

Element : 3756 D=400.000 B=200.000 bf=200.000
 Section ID : Minor beam ds=0.000 dcb=60.000
 Combo ID : 5 E=34000.000 fc=0.042 Lt.Wt. Fac.=1.000
 Station Loc : 6000.000 L=6000.000 fy=0.414 fys=0.414

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

Design Moments, M3

Positive Moment	Negative Moment	Special +Moment	Special -Moment
16171.098	-32342.196	16171.098	-32342.196

Flexural Reinforcement for Moment, M3

	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	264.416	0.000	261.354	264.416
Bottom (-2 Axis)	172.238	129.178	0.000	172.238

Shear Reinforcement for Shear, V2

Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
0.000	25.483	54.692	0.000	10.908

Reinforcement for Torsion, T

Rebar At/s	Rebar Al	Torsion Tu	Critical Phi*Tcr	Area Ao	Perimeter Ph
0.000	0.000	1515.684	8027.348	29378.729	844.400

Figure 9.30: Minor beam details taken from SAP 2000

Table 9.66. Portal Frame Method for Internal Forces analysis (Part 1)

Story	h,m	Fxi (kN)	Fx (kN)	V (kN)	a (m)	b (m)	1 (kN)	2 (kN)	3 (kN)
12	3	22.97	22.97	1.91	1.5	3	-0.96	-21.06	0.96
11	3	22.44	45.41	3.78	1.5	3	-3.81	-20.57	2.85
10	3	21.87	67.28	5.61	1.5	3	-8.50	-20.05	4.70

9	3	21.26	88.54	7.38	1.5	3	-14.99	-19.49	6.49
8	3	20.61	109.15	9.10	1.5	3	-23.23	-18.89	8.24
7	3	19.9	129.05	10.75	1.5	3	-33.16	-18.24	9.93
6	3	19.53	148.58	12.38	1.5	3	-44.72	-17.90	11.57
5	3	18.65	167.23	13.94	1.5	3	-57.88	-17.10	13.16
4	3	17.25	184.48	15.37	1.5	3	-72.54	-15.81	14.65
3	3	16.09	200.57	16.71	1.5	3	-88.58	-14.75	16.04
2	3	14.67	215.24	17.94	1.5	3	-105.91	-13.45	17.33
1	3.5	17.21	232.45	19.37	1.75	3	-127.67	-15.78	21.76
0 (Ground)	A_x (kN)	A_y (kN)	M_a (kN-m)	B_x (kN)	B_y (kN)	M_b (kN-m)	C_x (kN)	C_y (kN)	M_c (kN-m)
	-19.3	-127.6	33.90	-38.74	0.00	67.80	-38.74	0.00	67.80

Table 9.67. Portal Frame Method for Internal Forces analysis (Part 2)

4 (kN)	5 (kN)	6 (kN)	7 (kN)	8(kN)	9 (kN)	10(kN)	11 (kN)	12 (kN)
0.00	-17.23	0.96	0.00	-13.40	0.96	0.00	-9.57	0.96
0.00	-16.83	2.85	0.00	-13.09	2.85	0.00	-9.35	2.85
0.00	-16.40	4.70	0.00	-12.76	4.70	0.00	-9.11	4.70
0.00	-15.95	6.49	0.00	-12.40	6.49	0.00	-8.86	6.49
0.00	-15.46	8.24	0.00	-12.02	8.24	0.00	-8.59	8.24
0.00	-14.93	9.92	0.00	-11.61	9.93	0.00	-8.29	9.92
0.00	-14.65	11.57	0.00	-11.39	11.57	0.00	-8.14	11.57
0.00	-13.99	13.16	0.00	-10.88	13.16	0.00	-7.77	13.16

0.00	-12.94	14.65	0.00	-10.06	14.65	0.00	-7.19	14.65
0.00	-12.07	16.04	0.00	-9.39	16.04	0.00	-6.70	16.04
0.00	-11.00	17.33	0.00	-8.56	17.33	0.00	-6.11	17.33
0.00	-12.91	21.76	0.00	-10.04	21.76	0.00	-7.17	21.76
Dx (kN)	Dy(kN)	Md(kN -m)	Ex	Ey	My	Fx	Fy	My
-19.37	0.00	33.90	-38.741	0.00	67.797	-38.741	0.00	67.80

Table 9.68. Portal Frame Method for Internal Forces analysis (Part 2)

13(kN)	14(kN)	15(kN)	16(kN)	17(kN)	18(kN)	19(kN)
0.00	-5.74	0.96	0.00	-1.91	0.96	0.96
0.00	-5.61	2.85	0.00	-1.87	2.85	3.81
0.00	-5.47	4.70	0.00	-1.82	4.70	8.50
0.00	-5.31	6.49	0.00	-1.77	6.49	14.99
0.00	-5.15	8.24	0.00	-1.72	8.24	23.23
0.00	-4.97	9.93	0.00	-1.66	9.92	33.16
0.00	-4.88	11.57	0.00	-1.63	11.57	44.72
0.00	-4.66	13.16	0.00	-1.55	13.16	57.88
0.00	-4.31	14.65	0.00	-1.44	14.65	72.54
0.00	-4.02	16.04	0.00	-1.34	16.04	88.58
0.00	-3.67	17.33	0.00	-1.22	17.33	105.91
0.00	-4.30	21.76	0.00	-1.43	21.76	127.67
Gx	Gy	Mg				
-38.74	127.67	67.797				

Table 9.69. Wind and seismic drifts in 3D SAP2000 model for Frame A (both directions)

	Total drift under wind load, mm			Total drift under seismic load, mm				
Level	Inter-story displacement, mm	Absolute displacement, mm	Inter-story drift, %	Inter-story displacement, mm	Absolute displacement, mm	Inter-story drift, %	Amplified inter-story deflection, mm	Allowable limit, mm
12	0.7	18.534	0.023	3.89	82.14	0.130	21.40	60
11	1.101	17.834	0.037	5.67	78.25	0.189	31.18	60
10	1.273	16.733	0.042	6.18	72.58	0.206	34.00	60
9	1.34	15.46	0.045	6.73	66.40	0.224	37.00	60
8	1.58	14.12	0.053	7.00	59.67	0.233	38.50	60
7	1.71	12.54	0.057	7.18	52.67	0.239	39.50	60
6	1.83	10.83	0.061	7.41	45.49	0.247	40.75	60
5	1.97	9	0.066	7.73	38.08	0.258	42.53	60
4	2.05	7.03	0.068	8.09	30.35	0.270	44.50	60
3	2.2	4.98	0.073	8.41	22.26	0.280	46.25	60
2	1.6	2.78	0.053	8.58	13.85	0.286	47.18	60
1	1.18	1.18	0.039	5.27	5.27	0.176	29.00	70

Table 9.70. Wind and seismic drifts in 2D SAP2000 model for Frame A (both directions)

	Total drift under wind load, mm	Total drift under seismic load, mm		
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Level	Inter-story displacement, mm	Absolute displacement, mm	Inter-story drift, %	Inter-story displacement, mm	Absolute displacement, mm	Inter-story drift, %	Amplified inter-story deflection, mm	Allowable limit, mm
12	0.86	20.54	0.029	4.7171	98.5456	0.157	25.94	60
11	1.32	19.68	0.044	7.0445	93.8284	0.235	38.75	60
10	1.45	18.36	0.048	7.3735	86.7839	0.246	40.55	60
9	1.6	16.91	0.053	8.2483	79.4104	0.275	45.37	60
8	1.7	15.31	0.057	8.3936	71.1622	0.280	46.16	60
7	1.85	13.61	0.062	8.6891	62.7686	0.290	47.79	60
6	1.95	11.76	0.065	8.7588	54.0795	0.292	48.17	60
5	2.13	9.81	0.071	9.0818	45.3207	0.303	49.95	60
4	2.3	7.68	0.077	9.2782	36.2389	0.309	51.03	60
3	2.38	5.38	0.079	9.5335	26.9607	0.318	52.43	60
2	1.8	3	0.060	10.2109	17.4273	0.340	56.16	60
1	1.2	1.2	0.036	7.2164	7.2164	0.219	39.69	70

Table 9.71. Wind and seismic drifts by hand calculations for Frame A (both directions)

	Total drift under wind load, mm			Total drift under seismic load, mm				
Level	Inter-story displacement, mm	Absolute displacement, mm	Inter-story drift, %	Inter-story displacement, mm	Absolute displacement, mm	Inter-story drift, %	Amplified inter-story deflection, mm	Allowable limit, mm

12	0.86	23.17	0.029	4.77	95.69	0.159	26.23	60
11	1.34	22.31	0.045	7.33	90.92	0.244	40.30	60
10	1.46	20.97	0.049	7.57	83.59	0.252	41.63	60
9	1.65	19.51	0.055	8.15	76.02	0.272	44.83	60
8	1.8	17.86	0.060	8.42	67.87	0.281	46.30	60
7	1.89	16.06	0.063	8.37	59.45	0.279	46.05	60
6	2.04	14.17	0.068	8.52	51.08	0.284	46.88	60
5	2.23	12.13	0.074	8.74	42.56	0.291	48.08	60
4	2.36	9.90	0.079	8.76	33.82	0.292	48.18	60
3	2.49	7.54	0.083	8.71	25.06	0.290	47.93	60
2	3.17	5.05	0.106	10.35	16.35	0.345	56.95	60
1	1.88	1.88	0.057	5.99	5.99	0.182	32.95	70

Table 9.72. Stability coefficient calculations for Frame A (both directions).

	Shear		Flexural		Total		Vu	Pu	theta
Story	Interstory	Absolute	Interstory	Absolute	Interstory	Absolute			
12	4.77	95.69	0	1.41	4.77	97.10	1251.9	2086.3	0.0005
11	7.33	90.92	0.22	1.41	7.55	92.33	2906.8	3899.2	0.0006
10	7.57	83.59	0.22	1.19	7.79	84.78	4964.6	5491.4	0.0005
9	8.15	76.02	0.19	0.97	8.34	76.99	7428.1	6835.3	0.0005
8	8.42	67.87	0.17	0.78	8.59	68.65	10300.5	7963.5	0.0004
7	8.37	59.45	0.15	0.61	8.52	60.06	13581.7	8888.8	0.0003
6	8.52	51.08	0.12	0.47	8.64	51.55	17275.5	9624.6	0.0003

5	8.74	42.56	0.1	0.35	8.84	42.91	21381.8	10175.4	0.0003
4	8.76	33.82	0.08	0.24	8.84	34.06	25488.0	10569.8	0.0002
3	8.71	25.06	0.06	0.16	8.77	25.22	30427.3	10826.4	0.0002
2	10.35	16.35	0.05	0.1	10.40	16.45	35783.0	10964.3	0.0002
1	5.99	5.99	0.03	0.05	6.02	6.04	41562.3	11016.6	0.0001

Table 9.73. Torsional irregularity check.

	Ratio
Level	Both directions
12	0.049
11	0.078
10	0.080
9	0.086
8	0.088
7	0.088
6	0.089
5	0.091
4	0.091
3	0.090
2	0.107
1	0.062