

**DESIGN OF MULTI-STORY HOTEL “ALTYN FIESTA”
IN SAN ANTONIO, TEXAS, USA
(Capstone Project 2)**

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



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Declaration

We hereby declare that this report entitled “Design of Multi-Story Hotel “Altyn Fiesta” in San Antonio, Texas, 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

This Capstone 2 project presents the preliminary design of an eight-story hotel with three basement levels in San Antonio, Texas. The goal was to create a solid basis for the detailed architectural, structural, geotechnical, environmental and project management work. The building includes three podium levels for public and leisure functions, five hotel floors, and underground parking. The site's windy climate and shallow soft clay deposits introduce specific structural and geotechnical challenges.

Structurally, drift and internal forces were calculated; structural members were designed and serviceability was checked with the aim of designing a structurally safe and sound building. The geotechnical analysis covered borehole interpretation, soil classification, settlement and bearing capacity using common methods. Results showed that shallow and mat foundations cannot support the required loads, so deep foundations were utilized.

Environmental Engineering was focused on dealing with the Municipal Solid Waste (MSW) that was generated by residents of Altyn Fiesta Hotel and office workers for the Capstone II project. The sections below were designed to compute suitable waste management options in order for the system to operate smoothly.

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Figure 4.3.2.2.1. Structure geometry, bending, and shear force diagram

Figure 4.3.2.2.2. Structure design

Figure 4.3.2.2.3. Shear and bending analysis check

Figure 4.3.2.2.3. Structure stability check

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Figure 5.3.4. Lobby's three-stream sorting bin placement scheme

Figure 5.4.1. Waste Management Plan Flow Chart

Figure 6.2. Work breakdown structure of project

Figure 6.3. Gantt Chart of the project (Microsoft Project)

Figure 6.5. Modified Risk Matrix (MRM) for the Altyn Fiesta project

Figure 6.8. Stakeholders Hierarchy

1. Introduction

1.1. Project overview

The main aim was to prepare a foundation for carrying out a major endeavor of designing a 8-story hotel building with 3 floors of basement. The first 3 floors are dedicated to leisure, recreation and public functions, while the upper 5 floors accommodate guest rooms (including luxe and accessible units). The underground floors are used for parking and essential services.

The building is to be constructed in a windy area with a shallow soft clay layer which creates additional challenges to the design process. The selected project site is located in San Antonio, Texas, USA (29°25'53.4"N 98°29'45.6"W) which is suitable in terms of contextual integration, climatic and geotechnical conditions.

1.2. Project scope

Structural, geotechnical, environmental engineering closely collaborated with architectural planning and project management to achieve the objectives of the project.

Structural engineers and architects conducted a literature review on international codes and standards to ensure preliminary architectural design is safe, efficient and complies with global best practices. Site analysis was done to further improve the architectural planning. Then, LEED concepts were researched to be implemented in future work. Structural load calculations and preliminary sizing of structural members was conducted.

Geotechnical engineers have done a thorough soil investigation, constructing the soil profile and assessing soil characteristics which will be important in future foundation design.

Project management has created project charter, feasibility, and cost-benefit analysis based on RSMeans database and other sources in the case of San Antonio city, produced work breakdown structure and Gantt chart based on Primavera P6, and lastly, assessed all possible risks fully.

Environmental engineers took up the work of quantifying solid waste and designing an efficient collection system for it.

1.3. Roles & Responsibilities

Dariga Satybaldiyeva - BIM engineer, structural engineer

Aibibi Abuova - BIM engineer, architectural engineer

Dutbayev Nurtay - geotechnical engineer

Miras Kuzar - geotechnical engineer

Auyelkhan Abay - geotechnical engineer

Adilzhan Abralin - environmental engineer

Bauyrzhan Tyleukhanov - project manager

1.4. Design considerations

The structural and geotechnical engineers will consider the following conditions during design and calculation:

1. Strong wind
2. Corrosion environment
3. Large span requirement
4. Non-seismic zone with shallow soft clay layer

2. Architectural Design

2.1. Site analysis

2.1.1. History of location

San Antonio is one of the top tourist destinations in Texas, famous for its shopping scene and unique historical culture deriving from its Spanish, German, and American roots. The city is also known for featuring the largest Mexican market in the USA and having the iconic urban park with its path stretching for more than 24 kilometers. This park is called River Walk and its path connects hundreds of events, shops, restaurants, and also major landmarks such as San Antonio Museum of Art and La Villita Historic Arts Village (Visit The USA, 2025). It's one of the city's top attractions which draws millions of visitors each year.

Located in the very heart of downtown, Altyn Fiesta hotel will meet the tourism demand of the city. The key advantage of the location is that it takes only 5 minutes in car to get to the center of the key amenities, such as the River Walk, The Alamo, La Villita Historic Village, Briscoe Western Art Museum, Tobin Center for the Performing Arts, Hopscotch, and much more.

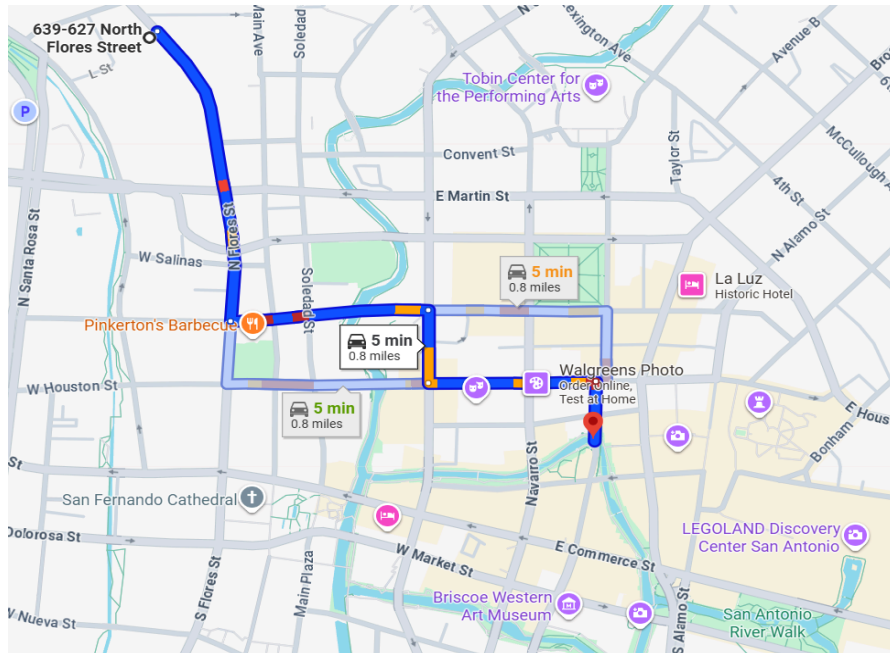


Figure 2.1.1.1. Proximity of tourist attractions (Google maps, 2025)

Since the city's urban development center is planning to launch a project on improving downtown and creating a new sports and entertainment district nearby, the value of the chosen location rises even more (Visit The USA, 2025).

Despite the dramatic decline of tourism during the pandemic in 2020, the city is gradually recovering its tourism sector. In 2023, a total of 37.7 million visitors visited San Antonio (Road Genius, 2023). Altyn Fiesta will help the city fully realize its touristic potential and support the local economy.

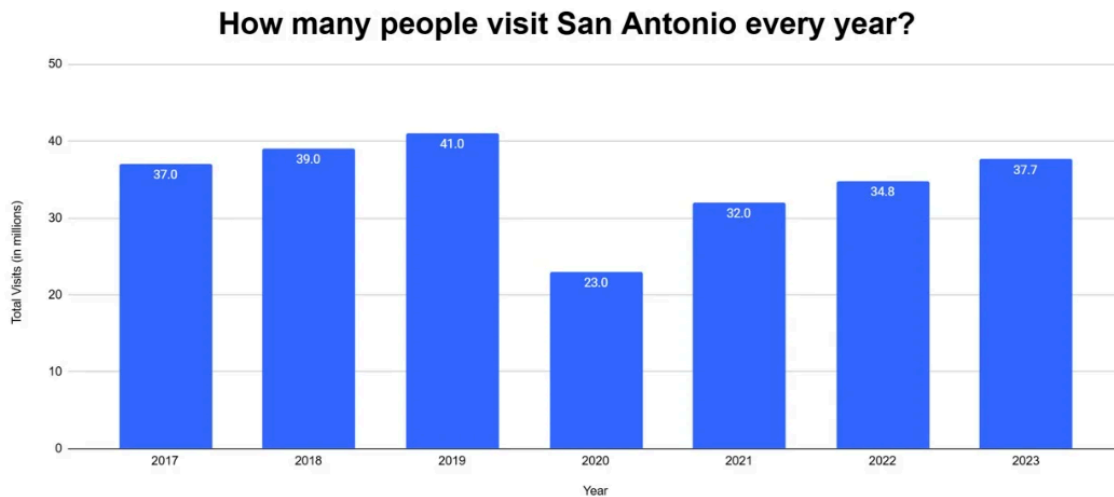


Figure 2.1.1.2. Number of visitors in San Antonio (Road Genius, 2023)

2.1.2. Climate responsive design and site-specific variables

San Antonio is considered to have a transitional humid subtropical climate, which means that the city has very hot, humid summers and mild winters. The hot season lasts from May to September, with the daily temperature being higher than 31.7°C and the peak of 35.6°C in August (Weatherspark, 2025). The cooler period extends from November to February, with the average temperature being around 5-7°C. San-Antonio is a windy city, with the windier part of the year lasting for 6 months from January to July with the highest average wind speed being around 5 m/s. Humidity in the city is low and comfortable in winter and spring, but rises to oppressive and muggy from June to September.

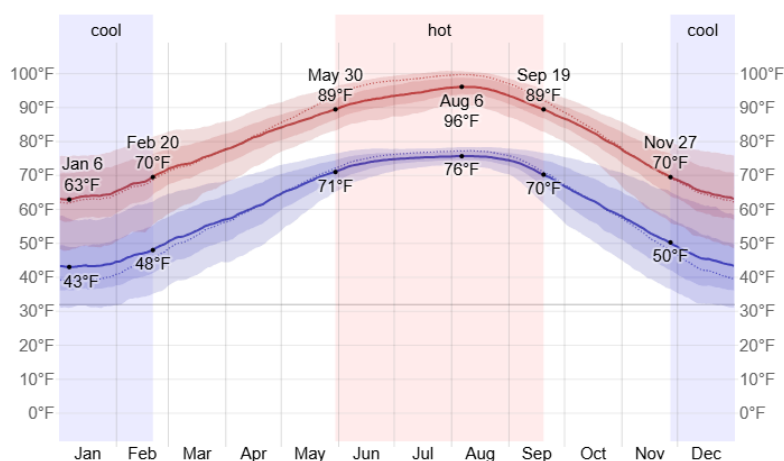


Figure 2.1.2.1. Temperature in San Antonio (Weatherspark, 2025)

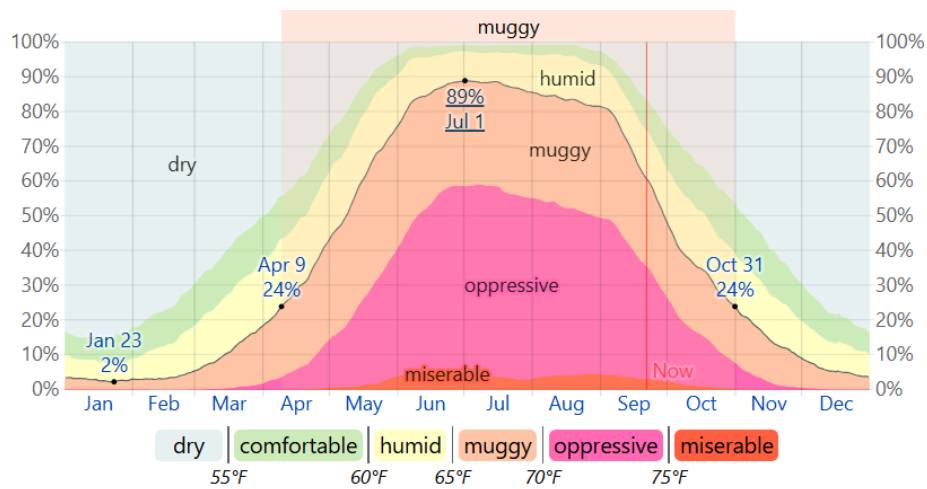


Figure 2.1.2.2. Humidity in San Antonio (Weatherspark, 2025)

These climate variables are significant in considering the design of the building. San Antonio's climate creates a challenge that should be effectively addressed in the design. It is the very hot and humid summers, which can cause overheating of the building and mold growth. For this reason the orientation of the building, solar access, wind direction plays a very important role.

2.1.3. Solar access

Figure 2.1.3.1 and 2.1.3.2 illustrate the solar orientation of our site. From figures we can see that the south side is mostly exposed to the sunlight throughout the year. This exposure has both advantages and disadvantages. On one hand, direct solar access in winter can be beneficial for naturally warming up the interior spaces when outdoor temperatures are mild. On the other hand, during summer, when tourist activity and building use are at their peak, the same southern exposure significantly increases the risk of overheating and discomfort. Therefore, the hotel building should be located in the way that no windows will be under the direct sun exposure facing the south side.

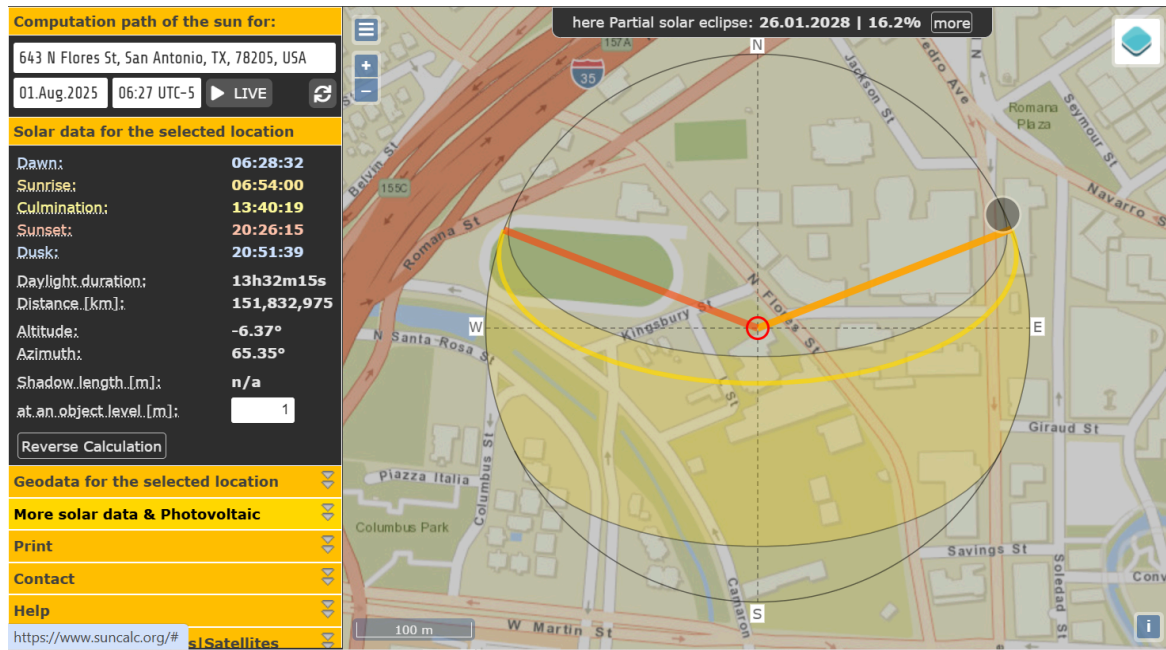


Figure 2.1.3.1. Solar access during summer

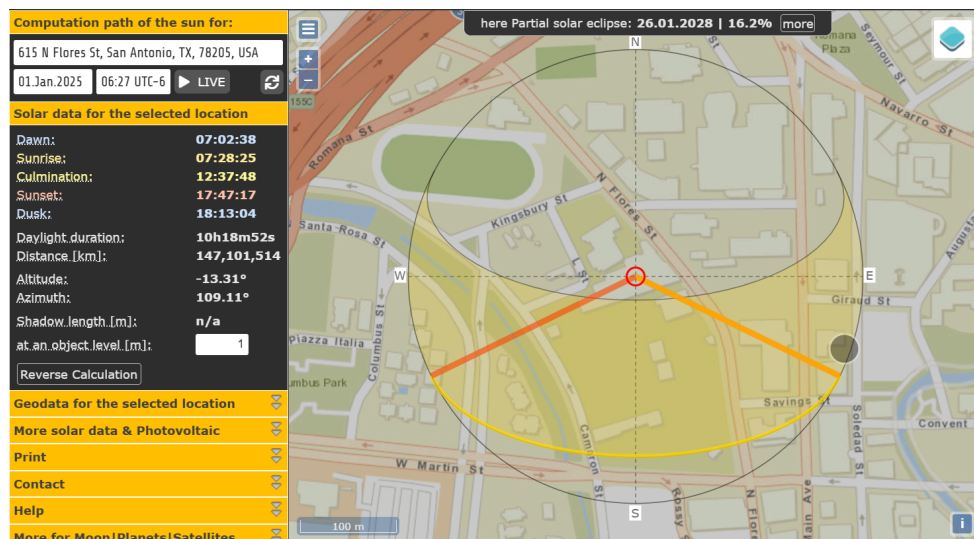


Figure 2.1.3.2. Solar access during winter

2.1.4. Cool breeze access

The wind in San Antonio generally comes from the south and southeast sides, moving to the northwest according to the figure 2.1.4.1. The wind speed is highest in spring (10 mph) and weakest in late summer (7 mph). This means that the natural ventilation can be taken care of in building design orientation. To effectively use the wind in the most efficient way, the longer facade of the building should be faced in the direction of the wind, increasing the number entry points through which air can enter the building. If openings and cross

ventilation corridors will be placed, then the wind can flow directly through the building removing accumulated hot and humid air resulting in comfortable conditions even in summertime. This design consideration also helps to avoid fully relying on mechanical systems.



Figure 2.1.4.1. Wind flow direction

2.1.5. Orientation

According to constraints explained in the previous two sections, the building shall be placed facing southeast rather than Cameron or Flores Streets which would be a more intuitive orientation for the facility that should be displayed clearly for people passing by. However, the comfort of hotel visitors is much more important than visual prominence from the street. Such orientation allows the best ventilation and gets the most sunlight but avoids the right-angled exposure.

2.1.6. Views

Windows looking out the city skyline and scenic urban landscape definitely will become an important part of hotel guests' pleasant stay in Altyn Fiesta.



Figure 2.1.6.1. View from southeast (Google Maps, 2025)

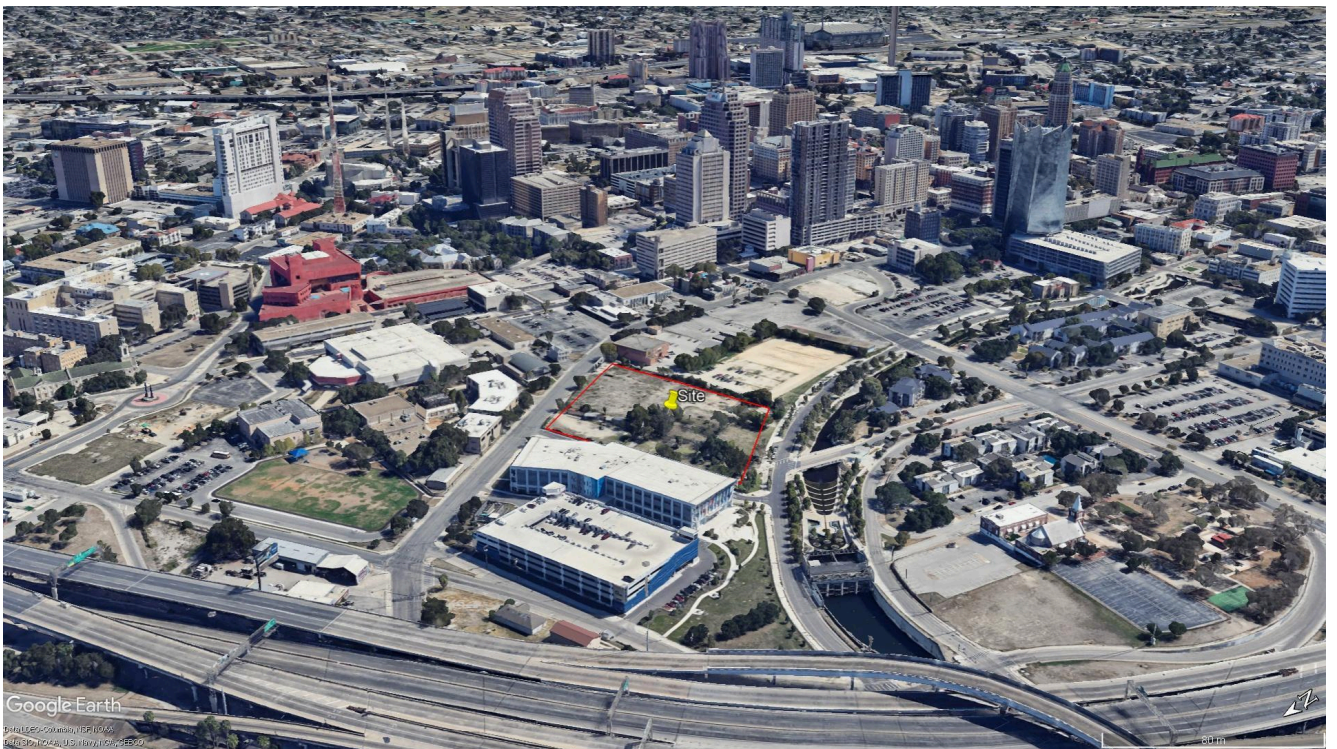


Figure 2.1.6.2. View from northwest (Google Maps, 2025)

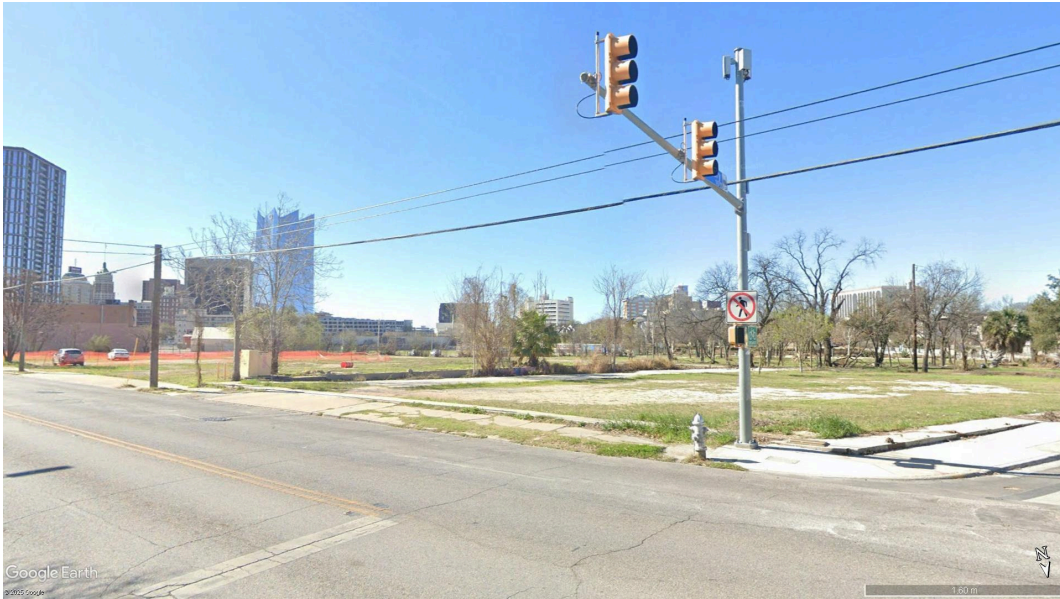


Figure 2.1.6.3. Street view (Google Maps, 2025)

2.1.7. Overshadowing

There is no significant overshadowing of the site by landforms, trees or buildings.

2.1.8. Access and transport

The site is very close to the city sightseeing tour route as well as several bus stops and bicycle rental spots. In addition to availability of all the major transportation means, the convenient location at the city center makes many facilities and tourist attractions easily accessible for hotel guests. The site is served by four-lane N Flores Street, two-lane Camaron Street and Kingsbury Street, meaning there will not be any congestion or entry problems.



Figure 2.1.8.1. Downtown area map

2.1.9. Bushfires risks

Texas Wildfire Risk portal defines San-Antonio as a large urban environment highly subject to wildfire risk. Extreme conditions with lots of dried vegetation and fuel loads, embers and spot fires are present. On average, wildfire risk in Texas is 76% higher than in other states (Wildfire Risk to Communities, 2025), making it a very important aspect to consider during selection of both structural and non-structural materials.

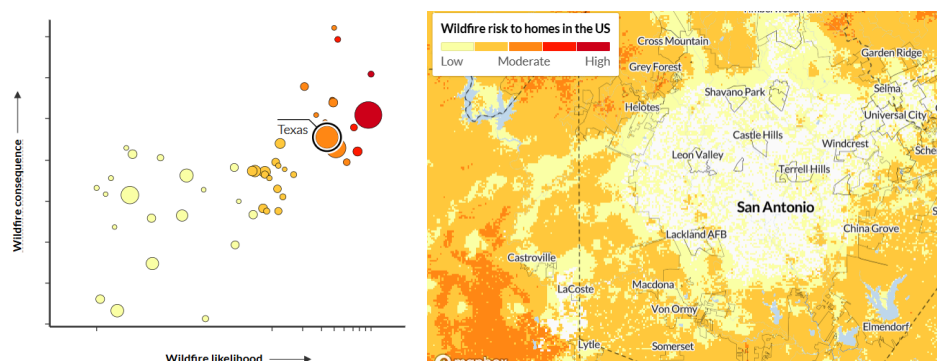


Figure 2.1.9.1. Wildfire risk map (Wildfire Risk to Communities, 2025)

2.1.10. Stormwater drainage

The terrain on the site is mainly flat. The city has a moderate amount of rainfall for which a stormwater drainage shall be incorporated into design.

Surface parking lots will be cross-sloped toward drains to collect rainwater in catch basins connected to storm sewer pipes. The underground levels, aside from floor drains and pump systems, should have enough waterproofing to avoid penetration of water from surrounding soil. The roof drainage design will be explained in the architectural part.

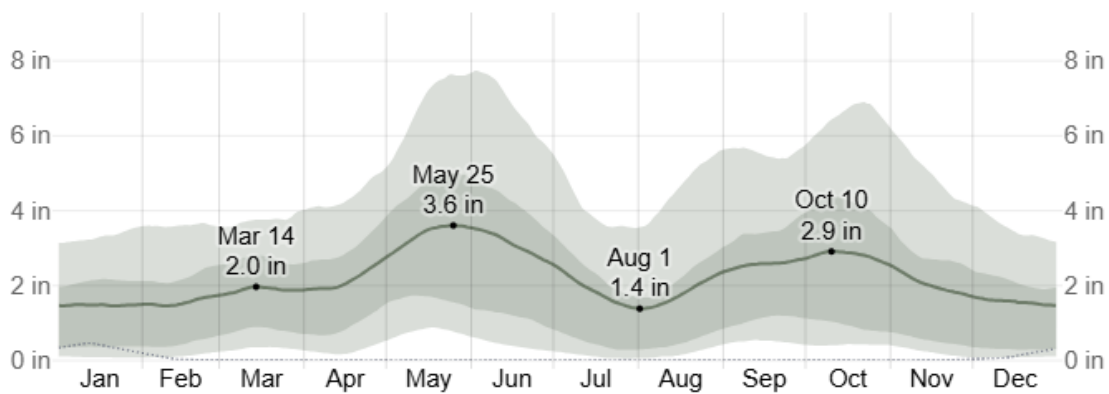


Figure 2.1.10.1. Average precipitation in San Antonio by months

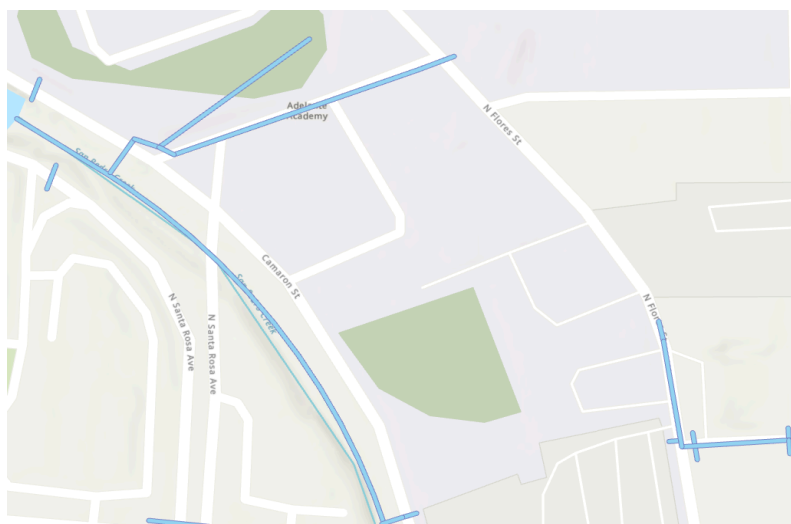


Figure 2.1.10.2. Map of the drainage systems in the area

2.1.11. Services

High humidity requires installation of dehumidifiers to prevent mold formation. Since the site is located in the city center, the power, gas, phone and water supply systems cover the area of the future hotel building.

2.2. Design concept

2.2.1. Design concept development

From the site analysis, our team decided to use the first three levels of the hotel as a long-span podium dedicated to amenities such as restaurants, banquet halls, spa, gym and office areas. These amenities will attract tourists and increase the hotel's long-term revenue. The long-span structural system provides large, column-free spaces that can accommodate high pedestrian traffic, especially in the banquet hall for events such as weddings. These large, open spaces also create a strong architectural identity for the hotel. In addition, their adaptability makes the design more economical, as the spaces can be easily reconfigured in the future, extending the building's useful life. Although large spans are usually used in public spaces, they are also used on the guest floors. Wider structural grid decreases the likelihood of awkward column intrusion in the bathrooms and hotel rooms, while creating clearer corridors and higher-quality guest environment. Moreover, denser column placement would constrain room planning.

Since the site is underlain by shallow soft clay, a three-level basement is appropriate because it allows the structure to reach a stiffer bearing stratum. With the expected high tourist flow and the inclusion of multiple amenities and residential floors, the basements will also be used for underground parking, which helps free up the ground level for more greenery and landscaping across the site.

Multi rise building consisting of 8 floors is suitable for the site, since very tall buildings are nearby but not immediately adjacent. The biggest challenge was to design the floors with sleeping units efficiently: rooms should have sensible dimensions and floors should not have too much empty unused space after room arrangement. Many of the potential designs were rejected because of this problem. Also, the building should have more length and less width to optimize its orientation as it was explained earlier.

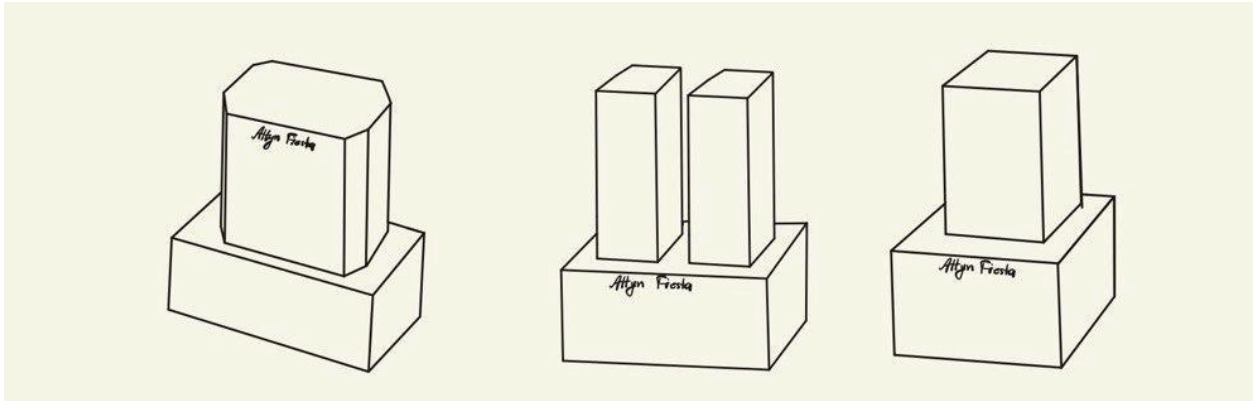


Figure 2.2.1.1. Preliminary design concepts

2.2.2. Design concept outcome

After analysing various alternatives, the selected final design has the upper 5 floors shaped like the letter “H” from plan view rising above the 3-story podium designated for recreation, business, and public functions, and 3 level basements for underground parking. This layout tackles the issue of efficient space use and answers the high tourism demand. It is also good for maximum cool breeze intake and improved ventilation.

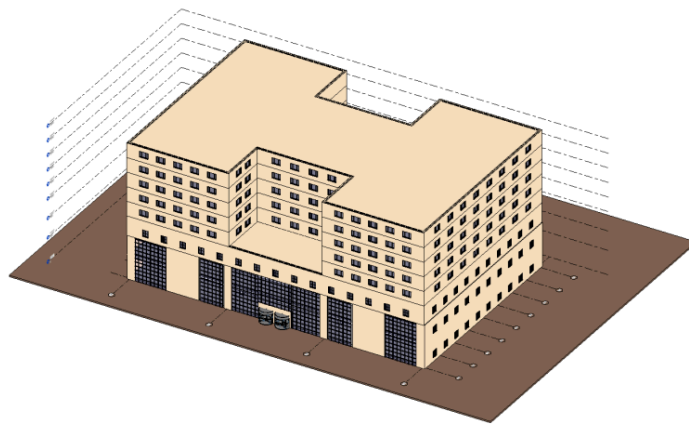


Figure 2.2.2.1. Preliminary 3D design of the building

2.3. Literature review on Design codes

The city of San Antonio adopts the International Building Code (IBC) of 2024 with some minor amendments (Municode, 2025). Altyn Fiesta hotel is planned to be a hybrid structure with the first 3 rectangular stories being a long-span podium and the next 5 stories forming a H-shaped tower.

2.3.1. Building classification

According to construction type classification, the building shall be of Type IB as this gives more flexible ranges on building height, area and floor numbers. Type I construction signifies the most fire-resistive buildings that can be built using non-combustible materials with some exceptions like fire-retardant-treated or sprayed fire-resistive materials, millwork, insulation with flame spread index no more than 25, etc. Its subtypes B differs in used materials' fire-resistance ratings of 2 hours (for Type IB).

According to occupancy classification of the code, the podium of Altyn Fiesta hotel belongs to Assembly group A-2, which includes portions of the building dedicated to gathering of people for civic, social and recreational purposes, and the upper 5 floors fall under the Residential Group, specifically R-1 category, with sleeping units for occupants of primarily transient nature. According to section 510.2 of the IBC on separate occupancies, there shall be a horizontal assembly with a fire-resistance rating of at least 3 hours separating slab between tower and podium.

2.3.2. Area, height and floor limits

The allowable height, number of stories and floor area for this project follow the rules in IBC Chapter 5. These limits depend on the construction type, occupancy group and whether the building has an automatic sprinkler system. Since our building uses Type IB construction and is fully sprinklered, we use the sprinklered values from Tables 504.3, 504.4 and 506.2. The podium contains Assembly Group A-2 spaces, and Type IB construction with sprinklers allows a greater height and number of stories compared to non-sprinklered buildings. The hotel tower above is classified as Residential Group R-1, so its height and area limits come from the Type IB sprinklered requirements for that occupancy.

The design also uses the separated podium approach allowed by IBC Section 510.2. In this setup, the floors above a 3-hour horizontal assembly are treated as a separate building for height and story calculations. This makes it possible for the podium and the hotel tower to meet their own height and area limits independently, while still functioning as one combined building.

2.3.3. Fire safety

Fire-resistance requirements for the building are based on the testing standards referenced by the IBC, mainly ASTM E119 for fire-resistance performance and ASTM E136 for checking whether materials are non-combustible. These standards make sure that structural and non-structural elements meet the fire protection levels expected for Type IB construction. The building includes all life-safety features required for its occupancies. A full automatic sprinkler system, designed according to NFPA 13, is installed throughout the building. This is required for both the A-2 assembly spaces in the podium and the R-1 residential floors above. Chapter 9 of the IBC also requires fire alarms, notification devices and automatic smoke detection in R-group sleeping units and corridors, and these systems are included in the design.

Exit-access travel distances follow the limits in IBC Table 1017.2. Because the building is sprinklered, the allowed distances are 400 ft (122 m) for A-2 spaces, 300 ft (91 m) for B occupancies, and 250 ft (76 m) for R-1 areas. The proposed floor layout keeps all travel distances within these limits.).

The following table shows the maximum distances to exit access analysed by Revit's path of travel tool.

Table 2.3.3.1. Maximum Travel distance per Level

Level	Time (s)	Length (mm)
B1		
B2	18.9	25412
B1	18.9	25412
L1	23.5	31558
L2	18.8	25240
L3	18.1	24262
L4-8	22.3	29895

2.3.4. Means of egress-occupant load

According to chapter 10 of IBC, for such multi-functional buildings as this one, means of egress requirements should be applied to each occupancy type according to its occupant load factor calculated independently based on the floor area of that space.

Table 2.3.4.1. Occupancy Load

Floor	Function of space	Rooms/spaces	Area (ft^2)	Occupant load factor ($ft^2/occupant$)	Occupant load per floor
1	Assembly unconcentrated (chairs and tables)	Banquet hall Restaurant with kitchen Lobby and lounge	22910	15 net	1527
2	Assembly unconcentrated (chairs and tables)	SPA Lounge Administration office	15030	15 net	1002
	Assembly concentrated (chairs only)	Changing room	2580	7 net	516
	Exercise rooms	Gymnasium	5810	50 gross	116
3	Assembly unconcentrated (chairs and tables)	Conference and meeting rooms Coworking Administration	23420	15 net	1562

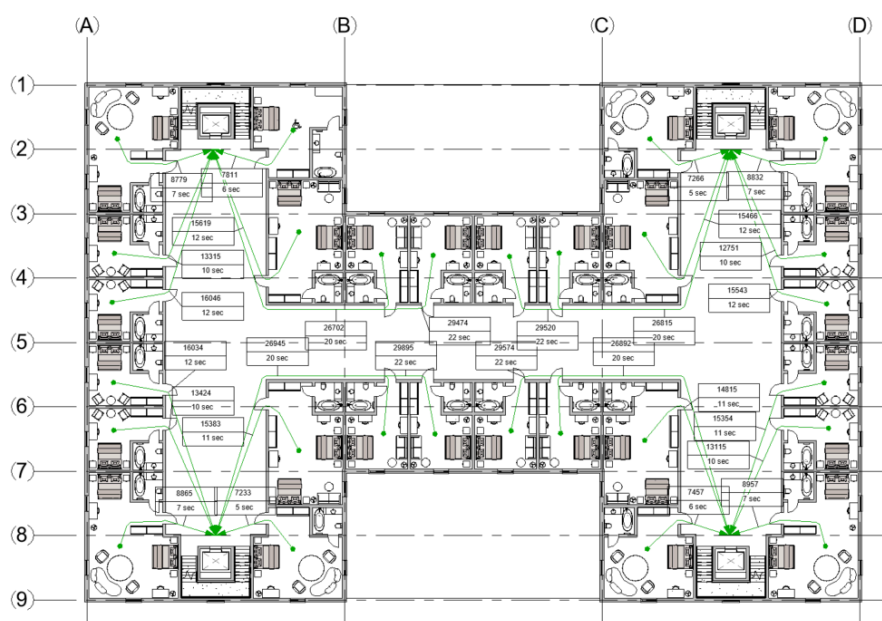
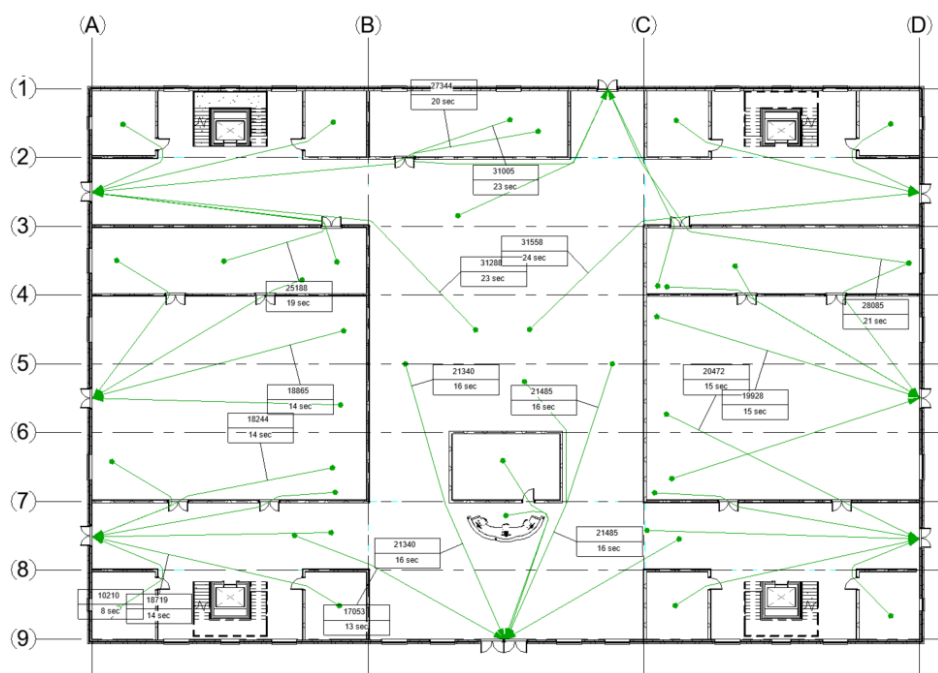
		office			
4-8	Residential	Hotel rooms	22000	200 gross	110
Total					5273

2.3.5. Exits

The occupant load per floor determines the required number of exits for that floor: if less than 501, two exits shall suffice; if it exceeds 1000, four or more exits should be provided. Two exits or exit access doorways should be provided on floors with hotel rooms and four or more shall be provided for the first 3 floors based on occupant load per floor. The distance between them shall not be less than half of the maximum diagonal dimension of the building.

According to section 1030, if occupant load is bigger than 300, a main exit should be provided for the Group A building portion. Assembly main exit should have the capacity to serve 100% of all means of egress leading to this exit and have a width of not less than 10 feet (3048 mm).

In our design the first floor has 8 exits to the outside, and floors from 2-8 have 4 exit access stairways.



2.3.6. Stairways

All stairway standards are summarized in the table below. The stair dimensions should be uniform but stair riser and depth variation of $\frac{3}{8}$ inch (9.5 mm) is allowed.

Table 2.3.6.1. Minimum requirements for stairway design

Dimension	Value
Stairway width	44 inch 1118 mm
Headroom clearance	80 inches 2032 mm
Stair riser height	4 inches 102 mm (min)
	7 inches 178 mm (max)
Tread depth	11 inches or 279 mm
Handrail height	34 inch 864 mm
	38 inch 965 mm
Clear space between handrail and wall	38 mm

The following figure is the staircase dimensions used in our project, following minimum requirements.

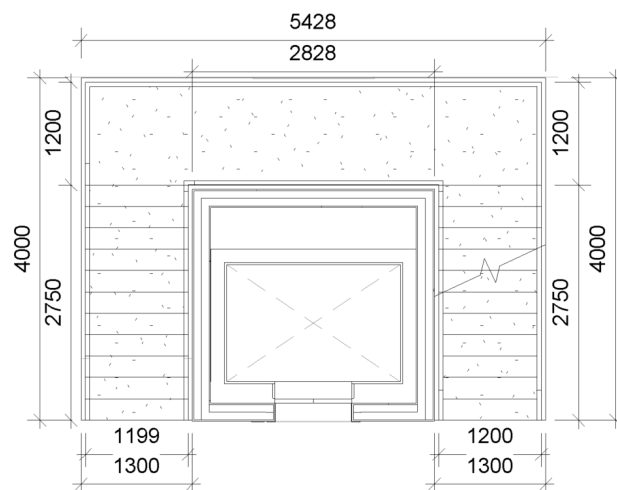


Figure 2.3.6.1. Staircase dimensions

2.3.7. Ceilings and corridors

A minimum value of 44 inches (1118 mm) is applicable for corridor width. Several requirements are also present for ceilings based on the function of space being designed. These standards are presented in Table 2.

Table 2.3.7.1. Minimum requirements for ceiling heights

Dimension	Minimum value
Ceiling height	7 feet 6 inches (2286 mm)
Ceiling height in corridors within sleeping unit	7 feet (2134 mm)
Ceiling height in bathrooms, toilets, kitchen, and storage rooms	7 feet (2134 mm)

Adhering to these requirements, the ceiling height will be 4.5 in 1-3 floors, 3.5 in upper 5 floors, and 3.25 for basement floors.

2.3.8. Doors

One of the main principles states that continuity of egress shall be maintained with no interruption by building elements. Doors when fully opened should not decrease corridor width by more than 178 mm or reduce the required width by more than half. For floors with occupant load higher than 50, doors should be side-hinged swinging doors or pivoted doors.

Table 2.3.8.1. Requirements for door design

Dimension	Value
Clear opening width of doors	32 inches (813 mm) minimum
Clear opening width of doors in sleeping units (excluding accessible units for disabled people)	no limit

Clear opening height of doors	80 inches (2032 mm) minimum
Clear opening height of doors in sleeping units	78 inches (1981 mm) minimum
Threshold at doors	$\frac{3}{4}$ inch (19.1 mm) maximum
Spacing of doors	48 inches (1219 mm) minimum

Level	Height	Width	Thickness	Count
L+01	2110	910	38	9
910: 9				
L+01	2110	1510	44	18
1510: 18				
L+01	2110	1810	44	2
1810: 2				
L+01: 29				
L+02	2110	910	38	8
910: 8				
L+02	2110	1010	38	8
1010: 8				
L+02	2110	1510	44	8
1510: 8				
L+02: 24				
L+03	2110	910	38	8
910: 8				
L+03	2110	1010	38	8
1010: 8				
L+03	2110	1510	44	8
1510: 8				
L+03: 24				
L+04	2110	810	38	27
810: 27				
L+04	2110	910	38	29
910: 29				
L+04: 56				
L+05	2110	810	38	28
810: 28				
L+05	2110	910	38	28
910: 28				
L+05: 56				
L+06	2110	810	38	28
810: 28				
L+06	2110	910	38	28
910: 28				
L+06: 56				
L+07	2110	810	38	28
810: 28				
L+07	2110	910	38	28
910: 28				
L+07: 56				
L+08	2110	810	38	28
810: 28				
L+08	2110	910	38	28
910: 28				
L+08: 56				
357				

Figure 2.3.8.2. Used door dimensions and counts

2.3.9. Windows

The IBC does not specify exact window sizes, but it does set minimum requirements for natural light and ventilation based on the floor area of a room. However, IBC Section 1030 gives fixed dimensions for emergency escape and rescue openings in sleeping rooms. These openings must have a minimum clear width of 20 inches (508 mm), a minimum clear height of 24 inches (610 mm), and the bottom of the opening cannot be more than 44 inches (1117 mm) above the finished floor.

IBC Section 1204.2 also requires that at least 8% of a room's floor area is provided as glazed window area to ensure sufficient natural light. If the room does not use mechanical ventilation, the openable portion of the windows must be at least 4% of the floor area to provide adequate natural ventilation.

Level	Sill Height	Width	Height	Count
L+01	900	1810	2100	20
L+01	900	1360	910	4
L+01	1500	610	910	4
L+01: 28				
L+02	900	1810	2100	26
L+02	900	1360	910	4
L+02	1500	610	910	4
L+02: 34				
L+03	900	1810	2100	30
L+03	900	1360	910	4
L+03	1500	610	910	4
L+03: 38				
L+04	610	1200	1500	40
L+04	900	1360	910	4
L+04: 44				
L+05	610	1200	1500	40
L+05	900	1360	910	4
L+05: 44				
L+06	610	1200	1500	40
L+06	900	1360	910	4
L+06: 44				
L+07	610	1200	1500	40
L+07	900	1360	910	4
L+07: 44				
L+08	610	1200	1500	40
L+08	900	1360	910	4
L+08: 44				
Grand total: 320				

Figure 2.3.9.1. Used window dimensions and counts

2.3.10. Interior environment

According to IBC, habitable space should have a width of not less than 7 feet (2134 mm). Additionally, sleeping units should have a net floor area of at least 70 ft^2 (6.5 m^2). The separation walls for sleeping units should serve as fire partitions.

As for windows, their opening width should be at least 30 inches (762 mm). San Antonio officials also require that sill height above finished floors is no more than 44 inches (1117 mm) (Municode, 2025). Also, the openable area of windows shall be not less than 4% of total floor area being ventilated.

2.3.11. Accessibility

Buildings of R-1 group with more than 5 sleeping units should have at least 1 accessible route to each part of the facility for people with disabilities. It is also required to have minimum 1 power-operated door at the entrance for this purpose.

Since the total number of hotel rooms (140 units) falls within the range of 101-150, the total number of required accessible units equals 7. Also, if a hotel has more than 4 sleeping units, every unit should be a Type B unit meaning it should be easily adaptable to be used by disabled visitors. If these units are located 4 stories or more above the levels of exit discharge, an accessible elevator is required. The accessible elevators also should be able to fit wheelchairs. Their minimum design requirements are schematically shown in Figure 2.

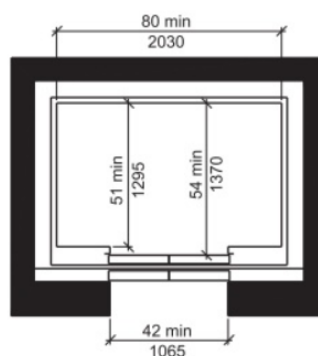


Figure 2.3.11.1. Accessible elevator requirements

According to ICC code for Accessible and Usable Buildings and Facilities (2017), the bathrooms of accessible units should be able to accommodate roll-in showers with dimensions of 60 inches (1525 mm) in width and 30 inches (760 mm) in depth [2].

For these sleeping units, clear opening width of doors should be at least 32 inches (815 mm), while clear floor space within the unit should be 48 inches (1220 mm) in length and 30 inches (760 mm) in width. This will allow free passage and maneuvering of the wheelchair.

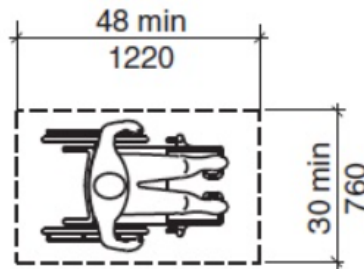


Figure 2.3.11.2. Wheelchair dimensions

The clear width of the interior accessible route should be at least 36 inches (915 mm) and exterior accessible route width should be at least 48 inches (1120 mm). The rise for ramps should be 30 inches (762 mm) maximum, while the width should be 36 inches (914 mm) minimum. The slope should be around 5% but not more than 8.3%.

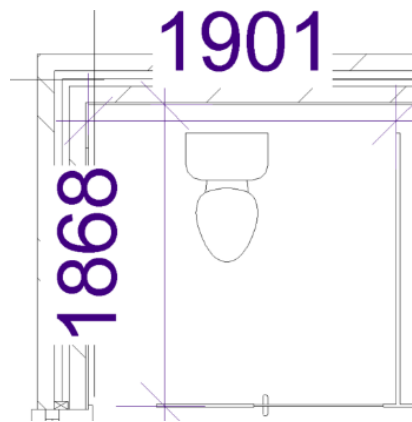


Figure 2.3.11.3. Accessible stalls in public restrooms

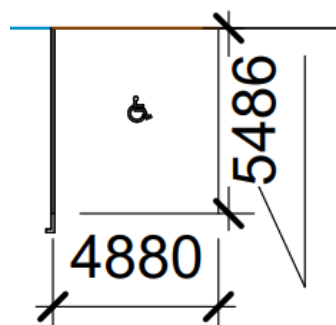


Figure 2.3.11.4. Accessible parking space

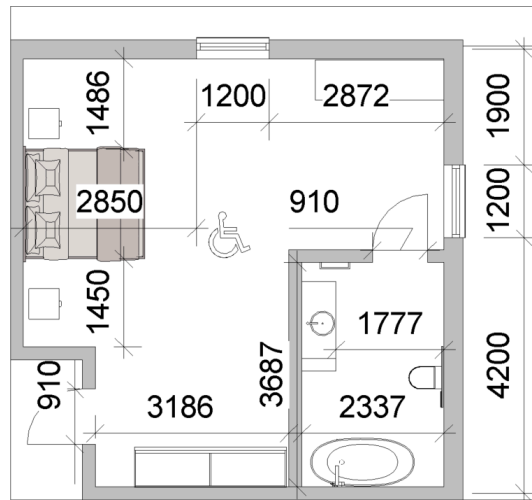


Figure 2.3.11.5. Accessible guest room layout

2.3.12. Parking

IBC or other international codes do not specify required parking spaces, except for accessible parking spaces. IBC 1106.2 presents the following table for accessible space and IBC 1106.5 requires at least 1 van-accessible space for every 6 accessible spaces.

Table 2.3.12.1. IBC 1106.2 table

TOTAL PARKING SPACES PROVIDED IN PARKING FACILITIES	REQUIRED MINIMUM NUMBER OF ACCESSIBLE SPACES
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 to 300	7
301 to 400	8
401 to 500	9
501 to 1,000	2% of total
1,001 and over	20, plus one for each 100, or fraction thereof, over 1,000

Since standard parking spaces are not regulated by IBC or IFC, the City of San Antonio Unified Development Code (UDC) will be used. Parking spaces should be calculated according to zoning rules. Calculated results will be taken only for reference and numerical estimation, and not as binding code requirement.

For a Hotel, UDC 35-526 requires 1 parking space per guest room and 1 parking space per every 2 employees. For the Altyn Fiesta Hotel, there will be a minimum of 150

spaces. Code does not specify the maximum number of parking spaces. Following table shows the parking space calculation for Altyn Fiesta Hotel.

Table 2.3.12.2. Parking space calculation

Occupancy	Code requirement	In Metric	Altyn Fiesta	Parking space
Hotel	1 space per guest room + 1 space per 2 employees	-	140 guest room + 20 employees	150
Restaurant	1 space per 100 sq ft	1 space per 9.29 m ²	293 m ²	32
Banquet	1 space per 4 occupants	-	212 occupants	53
Spa/Gym	1 space per 200 sq ft	1 space per 18.58 m ²	590 m ²	32
Office	1 space per 300 sq ft	1 space per 27.87 m ²	870 m ²	31

Table 2.3.12.3. Calculated parking space

Standard	Accessible	Van-Accessible
298	5	2

Given that a parking angle of 90 will be used, regular stalls should be 5486 in depth and 2743 mm in width. Width of an accessible parking lot should be not less than 2440 mm and include an access aisle of at least 1525 mm. So in total, it should be almost 4 meters in width (Municode, 2025).

Level	Count
B2	67
B1	64
L+01	47
Grand total: 178	

Figure 2.3.12.1. Total parking space

2.3.13. Ramp design

For Underground Parking, IBC 406.4.1 requires for Headroom to be at least 7 ft (2134 mm) through the ramp. It must have a 42 inch (1067 mm) tall guard is ramp edge is more than 30 in (762 mm) above surface. Ramps cannot be used as emergency egress routes, therefore separate stairs for evacuation should be present for underground levels. Ramp slopes are recommended to be no more than 20% by AASHTO design guides. The following figures show 3D views of ramps designed for our building.

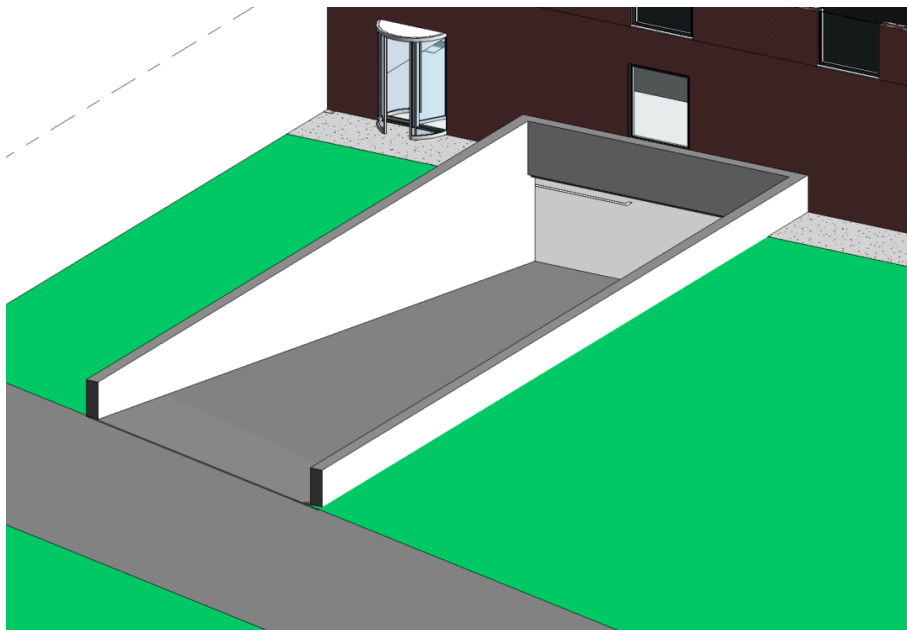


Figure 2.3.13.1. 3D view of ramp design going from level 1 to basement 1

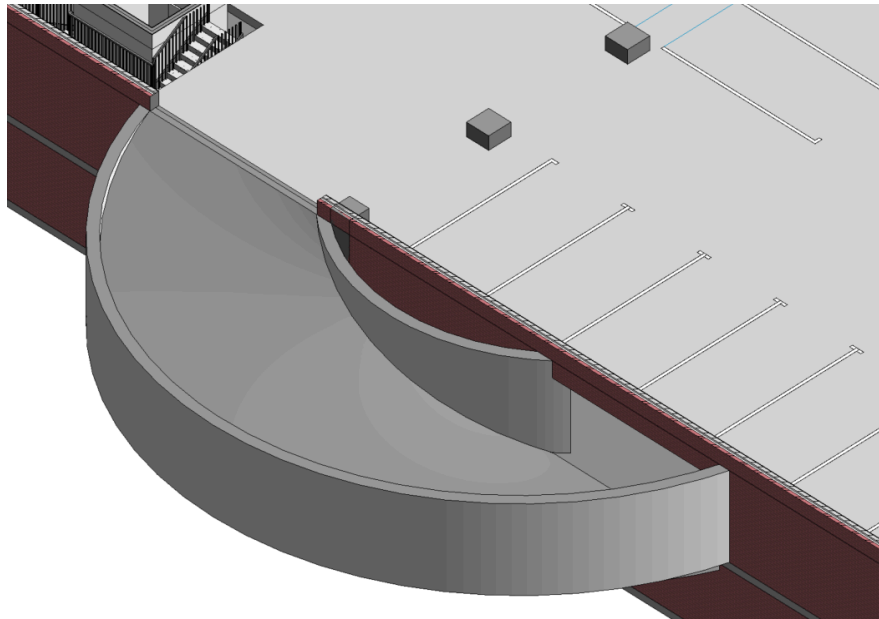


Figure 2.3.13.2. 3D view of ramp design going from basement 1 to basement 2

2.3.14. Roof design

For a modern look and better efficiency our building will have a flat roof. Roof drainage for flat roofs are governed by the International Plumbing Code (IPC). Primary drains should be placed on 1-2% slopes. Code requires every flat roof to have secondary emergency drainage that cannot connect to the same piping as the primary ones, and should be activated at 2 inches (51 mm) above the primary drain. The number of secondary drains should be the same as the number of primary drains. They are usually located adjacent to the primary drains.

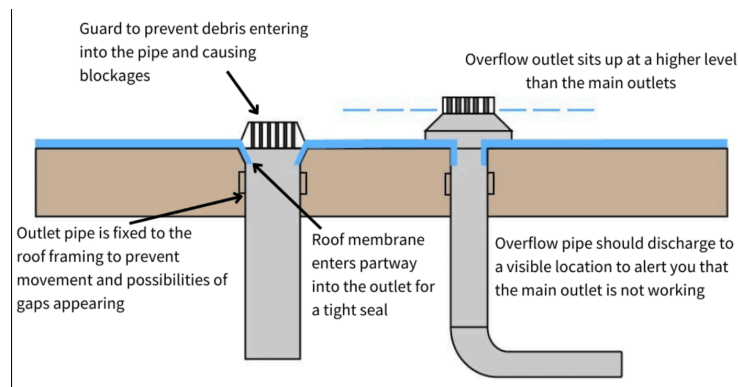


Figure 2.3.14.1. Primary and secondary drainage on flat roofs (Trusted Roofers, n.d.)

The exact number of drains are based on roof area and rainfall rate. Our building's roof is 2000m² and rainfall intensity for San Antonio, Texas is 3.5 in/hr from IPC table 1106.2.

Most common 4" diameter drains will be used with 10 L/s capacity each. It will result in 5 primary drains. For safety reasons, one extra drain will be used, giving a total of 6 primary drains and 6 secondary drains.

The following figure illustrates the proposed flat roof drainage system for our building.

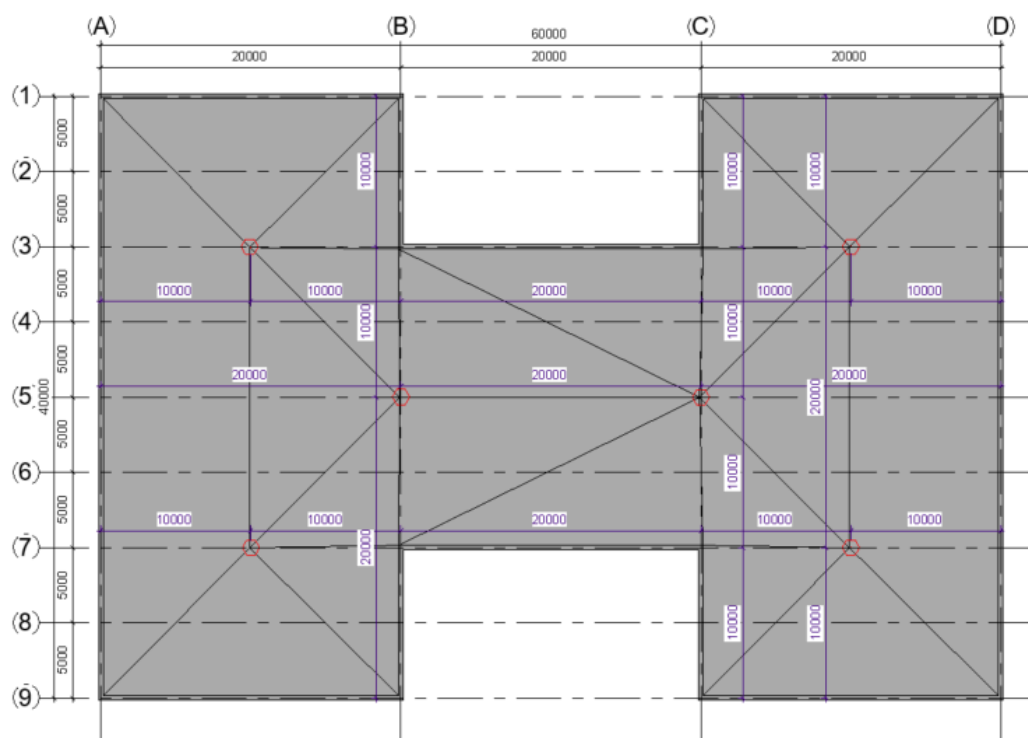


Figure 2.3.14.2. Roof drainage layout

2.4. Final design

After site analysis, design concept, and literature review, the final architectural design was developed. All drawings were done in Revit software and provided separately as “Technical Drawings” PDF file. Following figures are the 3D views of the finalized building.

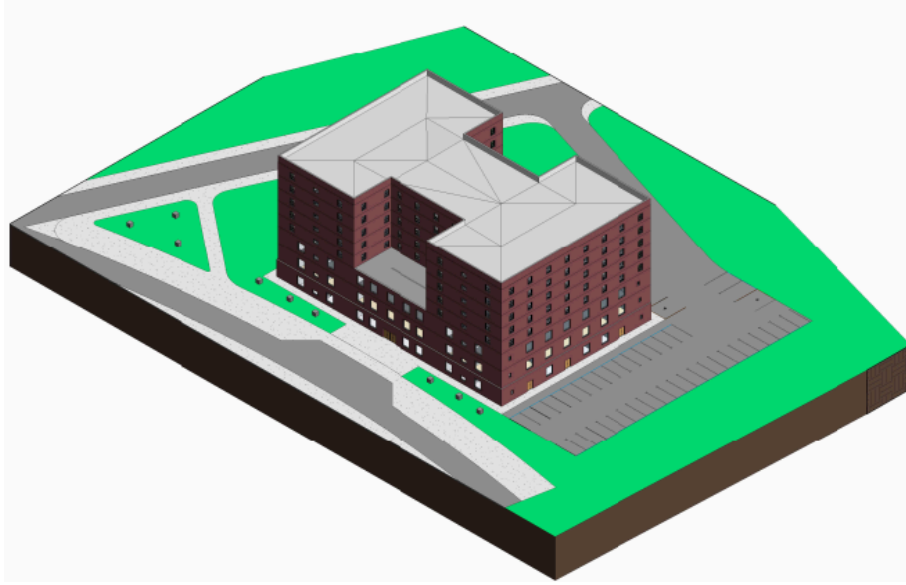


Figure 2.4.1.1. 3D view of final design



Figure 2.4.1.2. 3D view of final design

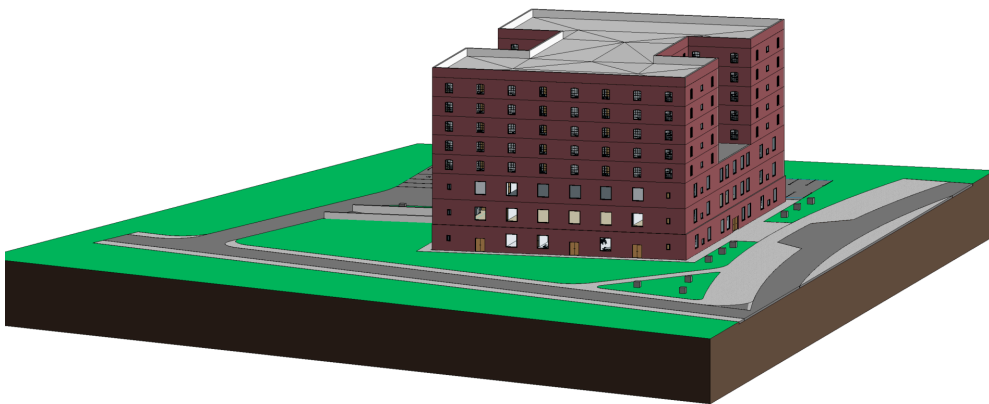


Figure 2.4.1.3. 3D view of final design

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

LEED certification is a globally accepted benchmark for healthy, energy-efficient and green buildings. Such buildings should contribute to reduction of CO_2 emissions, protect of water and bioresources, fight climate change, promote sustainability, enhance human health and quality of life (Lee & Park, 2024). Aside from environmental benefits, the LEED certified buildings have been proven to outperform their non-certified counterparts based on occupancy, guest satisfaction and revenue (Walsman et al., 2014; Goodarzi et al., 2025). It can be explained by the attraction of eco-conscious visitors, cost savings due to resource efficiency and increased comfort (Velaoras et al, 2025). As the research has shown, green certification can even positively affect employee performance, well-being and job satisfaction (Lee & Park, 2024).

The latest v5 version of LEED certification for new construction is obtained through a point-based system encompassing 9 categories. Based on awarded points, buildings get a corresponding level from certified up to platinum - the highest level of achievement in sustainable design and construction (USGBC, 2025).

A case study of 2 LEED platinum hotels in the USA has shown that sustainability and luxury can overlap and even complement each other (Yong et al., 2012). Based on their experience, several green practices and technologies that can be adopted for Altyn Fiesta hotel are listed in the table below.

Table 2.5.1. Possible implementations of green technologies

Sustainable practices	Significance	LEED category
Proximity to public transportation Bicycle racks Limited parking capacity	Reduce pollution from vehicle use	Location and transportation
Compost food waste	Minimize waste from dining	Sustainable sites

	facilities	
High-efficiency fixtures and fittings (e.g. dual flush toilets, low-flow showers)	Reduce water consumption	Water efficiency
Planting trees, avoiding turfgrass, drip irrigation system	Provide natural stormwater drainage	Water efficiency
Geothermal energy for refrigeration/heating Low-e glass High efficiency HVAC systems	Reduce energy consumption	Energy and atmosphere
Sensor technologies for guest rooms Regenerative elevator	Reduce energy consumption	Energy and atmosphere Innovation
Maximum daylighting Intelligent control of temperature and light	Reduce energy consumption Provide natural lighting	Energy and atmosphere Indoor environmental quality
Reflective roof materials Low-emitting materials Green furniture Construction materials with	Reduce urban heat island effect, reduce cooling energy needs	materials and resources

high percentage of recycled content		
Low volatile organic compound (VOC) emitting materials	Reduce indoor air contaminants	indoor environmental quality
Contract with local food makers, farmers and vendors	Support local economy Reduce transportation pollution	regional priority
Eco-friendly cleaning products	Limit exposure of staff and guests to hazardous chemicals	materials and resources
Integrated design approach (collaboration between engineers, architects and construction team)	Find optimal sustainable design at the lowest cost	Integrative approach
Photovoltaics on roof	On-site renewable energy	Sustainable cities

On the list of the areas of assessment, there are several essential categories:

- Energy and Atmosphere: It focuses on the use of highly efficient heating, ventilation, and air conditioning (HVAC) systems and clean energy sources.
- Water Efficiency: This involves the conservation of water by installing intelligent plumbing and recycling systems.
- Materials and Resources: The choice of building materials should be environmentally friendly, recycled or local.
- Indoor Environmental Quality: Making the indoor air more qualitative and utilization of interior materials that are non-toxic.

- Sustainable land use: management of rain water and abatement of the heat island effect in cities.

There are four levels of certification, depending on the points achieved; Certified (40-49), Silver (50-59), Gold (60-79) and Platinum (80 plus).

These sustainable building principles were adhered to in our Altyn Fiesta Hotel project even during the designing phase. Our project, according to the LEED assessment, has a score of 75 points. Consequently, Altyn Fiesta got the rating LEED Gold and that is how we understand that we are doing the best in terms of engineering and environmental friendliness.

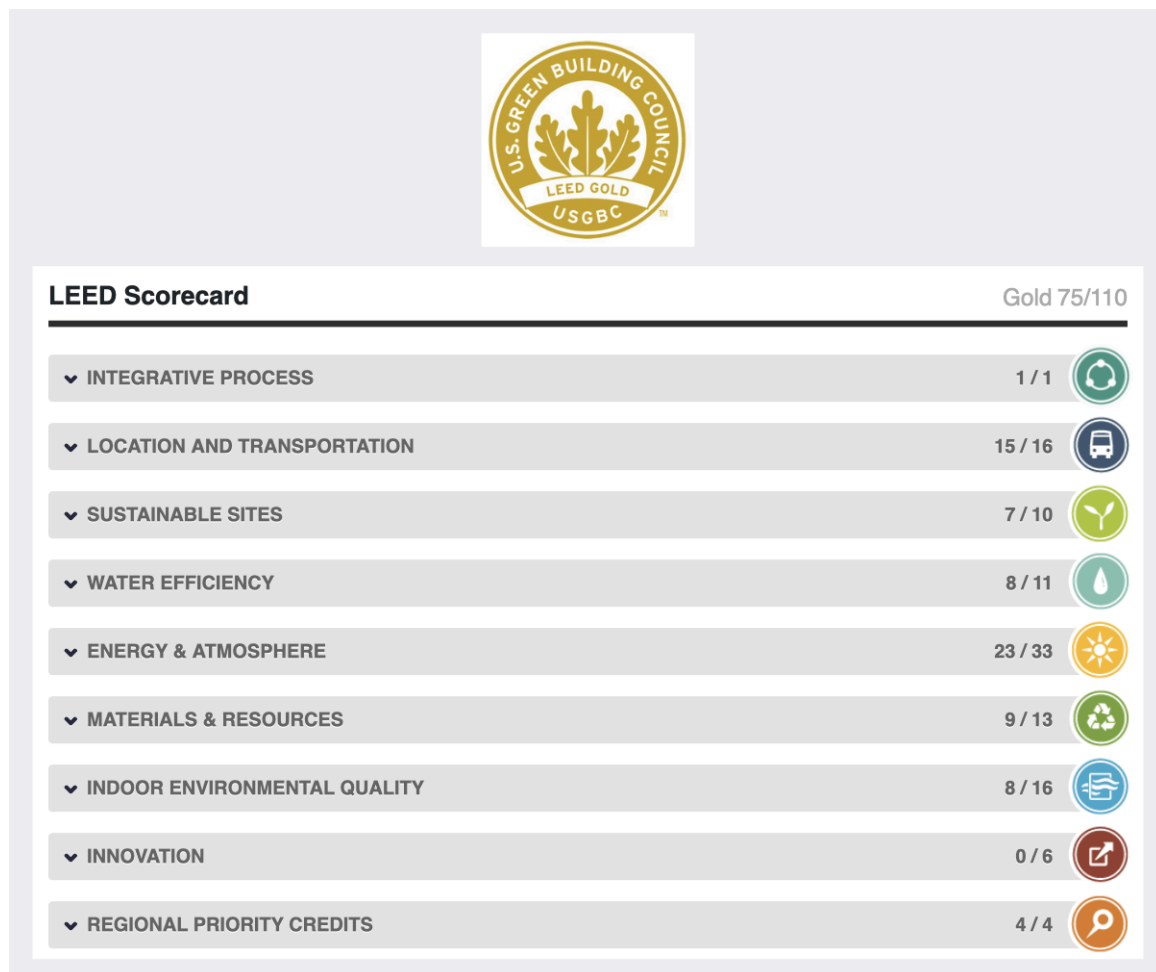


Figure 2.5.1. Leadership in Energy and Environmental Design Scorecard

Location and Transportation: The Altyn Fiesta project takes the full advantage of its urban location in San Antonio. Our choice of a location with high Surrounding Density will reduce the requirements of new infrastructure. The closeness to bus lines and walking-friendly areas resulted in the earning of the points on the Access to Quality Transit and Walkable Streets.

We also made the environment lighter by adopting Reduced Parking Footprint that would make people use Bicycle Facilities offered within the premises.

Location and Transportation (15/16)

☒
Sensitive Land Protection

1pt
This credit is intended to cultivate community resilience, avoid the development of environmentally sensitive lands that provide critical ecosystem services and reduce the environmental impact from the location of a building on a site. [Learn more about this credit.](#)

☒
High Priority Site and Equitable Development

2pts
This credit is intended to build the economic and social vitality of communities, encourage project location in areas with development constraints and promote the ecological, cultural, and community health of the surrounding area while understanding the needs and goals of existing residents and businesses. [Learn more about this credit.](#)

☒
Surrounding Density and Diverse Uses

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

☒
Access to Quality Transit

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

☒
Bicycle Facilities

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

☒
Reduced Parking Footprint

1pt
This credit is intended to minimize the environmental harms associated with parking facilities, including automobile dependence, land consumption, and rainwater runoff. [Learn more about this credit.](#)

☐
Electric Vehicles

1pt
This credit is intended to reduce pollution by promoting alternatives to conventionally fueled automobiles. [Learn more about this credit.](#)

Figure 2.5.2 Location and Transportation Scorecard

Sustainable Sites: The architecture is environmentally friendly to the local ecosystem. We adopted a stringent Construction Activity Pollution Prevention plan to avoid soil erosion. The site receives and treats 100 percent of the runoff through Rainwater Management. As a response to the Texas heat, we applied, in High-SRI (Solar Reflectance Index), materials to reduce the Heat Island to enhance the Open Space to guests and make the microclimate better.

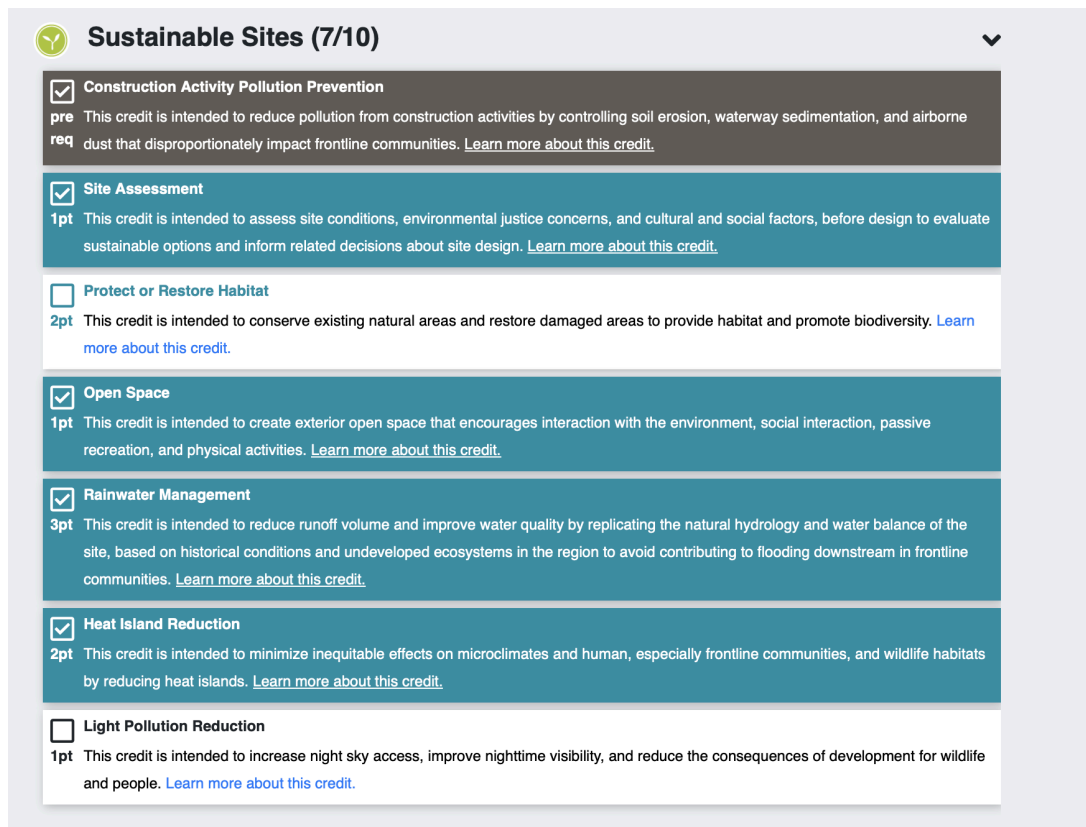


Figure 2.5.3. Sustainable Sites Scorecard

Water Efficiency: In Texas, water conservation is very important. We were able to reduce by 40 percent or more our Indoor Water Use by use of low-flow fixtures. In the case of Outdoor Water Use, we applied the use of xeriscaping using native plants which do not need permanent irrigation. The use of Cooling Tower Water is optimized using advanced treatment cycles, and all the systems are monitored using Water Metering to achieve efficiency in the long run.

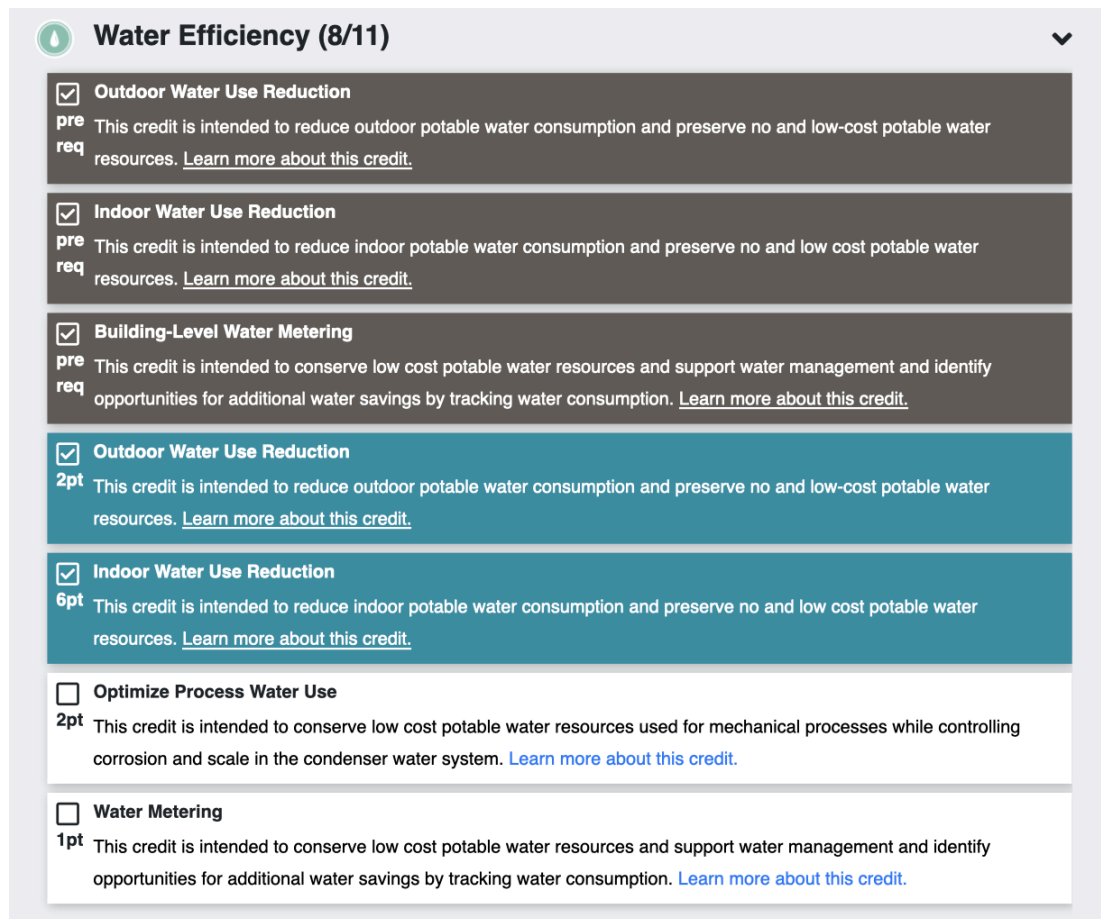


Figure 2.5.4. W&E Scorecard

Energy and Atmosphere: This is the most rated category of this project. With the addition of Phase Change Materials (PCM) into the building envelope we achieved a high level of Optimization of the Energy Performance through the reduction of the cooling loads (30-35 per cent). Renewable Energy Production through rooftop solar panels also takes place in the project. Basic Commissioning and Advanced Energy Metering make the HVAC systems work at their optimum level during the lifecycle of the building.

Energy and Atmosphere (23/33)

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

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

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

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

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

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

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

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

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

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

Figure 2.5.5. Energy and Atmosphere Scorecard

Materials and Resources: The supply chain was introduced with sustainability. We focused more on products that had Environmental Product Declarations (EPDs) and whose Material Ingredients were transparent. The Construction and Demolition Waste Management plan has been effective in diversion of more than 75 percent of debris out of landfills. We also carried out a Building Life-Cycle Impact Reduction study to reduce the overall carbon footprint of the reinforced concrete building.

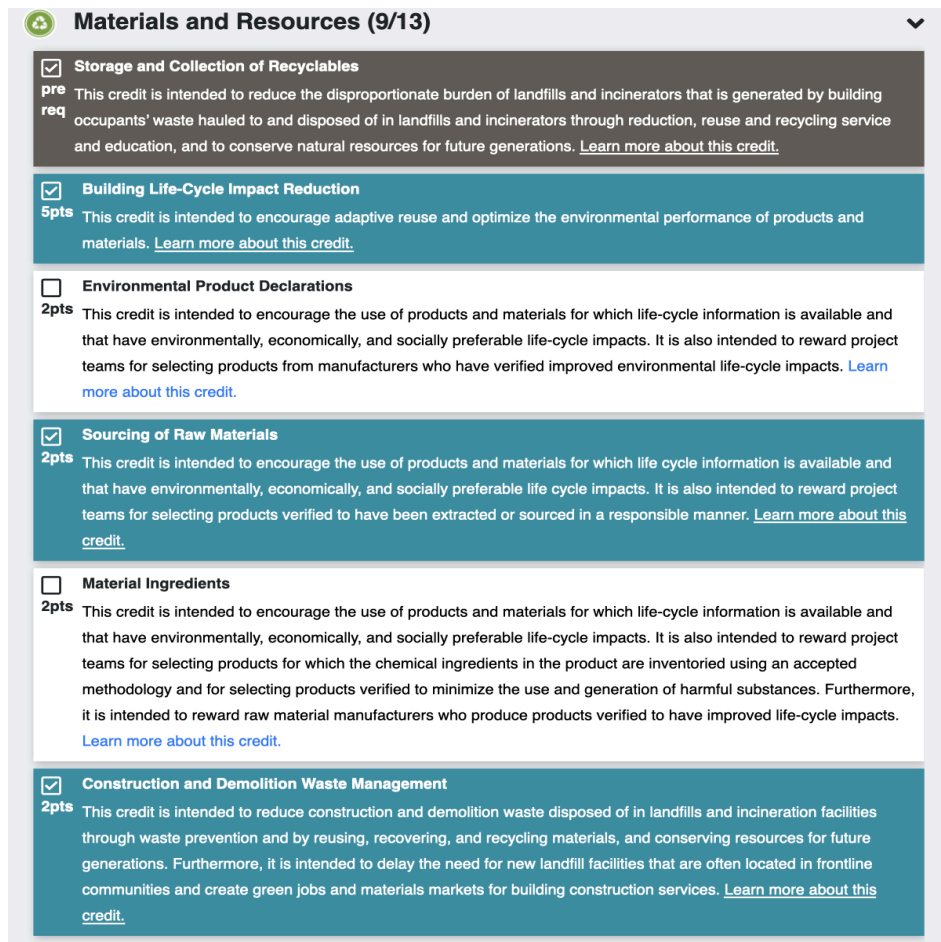


Figure 2.5.6. Materials and Resources Scorecard

Indoor Environmental Quality: The health of guests is one of the priorities. The tower design is H-shaped giving Quality Views and Daylight to more than 90% of the guest rooms. We exceeded Minimum IAQ Performance and only used Low-Emitting Materials (low-VOC paints and carpets). There is Zoned HVAC Control to maintain Thermal Comfort and automatic Interior Lighting that saves energy and offers optimum comfort.


Indoor Environmental Quality (8/16)


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

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

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

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

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

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

☒
Thermal Comfort
1pt This credit is intended to promote occupants' productivity, comfort, and well-being by providing quality thermal comfort. [Learn more about this credit.](#)

☐
Interior Lighting
2pts This credit is intended to promote occupants' productivity, comfort, and well-being by providing high-quality lighting. [Learn more about this credit.](#)

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

☒
Quality Views
1pt This credit is intended to give building occupants a connection to the natural outdoor environment by providing quality views. [Learn more about this credit.](#)

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

Figure 2.5.7. Indoor Environmental Quality Scorecard

Regional Priority: The project that won an Innovation credit on the design team was the one that had an LEED Accredited Professional and on the educational program about PCM technology. The regional Priority credits were reached through resolving the local environmental issues in San Antonio, especially on the extreme heat management and water shortage.


Regional Priority (4/4)


☒
Regional Priority Specific Credits
4pt These credits are intended to provide an incentive for the achievement of credits that address geographically specific environmental, social equity, and public health priorities. [Learn more about this credit.](#)

Figure 2.5.8. Regional Priority Scorecard

Life-365 Life-cycle cost analysis

The Life-365 software was used to conduct a service life evaluation of the reinforced concrete components. The tool was utilized to investigate the long-term behavior of durability of concrete when it is exposed to chlorides and determine the effects of various strategies on the material and protection. A reinforced concrete building in San Antonio, Texas was analyzed and the focus was on the performance trends of the alternatives that were being considered on a relative basis, not absolute economic numbers. The inputs in the form of costs were calculated with reference to the existing regional data on construction costs in the San Antonio market of Texas. The model of Life-365 had these values to consider realistic material and repair prices of the proposed hotel structure.

The screenshot displays the 'Cost Parameters' section of the Life-365 software. At the top, there are two dropdown menus: 'Base Units' set to 'US Units' and 'Concentration Units' set to '% wt. conc.'. Below these, the 'Cost Parameters' section is divided into three columns: 'Concrete & Steel', 'Barriers & Inhibitors', and 'Repairs'. Each column contains several input fields with numerical values and up/down arrows for adjustment.

Concrete & Steel	Barriers & Inhibitors	Repairs
Concrete (\$/cub. yd.) *	Membrane (\$/sq. ft.) *	Repair (\$/sq. ft.) *
150,00	4.50	65,00
Black Steel (\$/lb) *	Sealer (\$/sq. ft.) *	Area to repair (%) *
0,55	1,00	10,00
Epoxy Coated Steel (\$/lb) *	Inhibitor (\$/gal) *	Fixed repair intervals (yrs) *
0.95	10,00	10
Stainless Steel (\$/lb) *		
3.30		

Figure 2.5.9. Life 365 - Default Settings

Then, we chose “slabs and walls (1-D)” for type of structure:

The screenshot displays the 'Select Structure Type & Dimensions' section of the Life-365 software. It features a dropdown menu for 'Type Of Structure' set to 'slabs and walls (1-D)'. Below this are four input fields for 'Thickness (in)', 'Reinf. Depth (in)', 'Area (sq. ft.)', and 'Volume Of Concrete'. To the right of these fields is a diagram of a rectangular structure with dimensions 5.12 in (width) and 2.00 in (height). Below the diagram, the 'Chloride concentration units' are set to '% wt. conc.'.

Select Structure Type & Dimensions
Type Of Structure *
slabs and walls (1-D)
Thickness (in) *
5,12
Reinf. Depth (in) *
2,00
Area (sq. ft.) *
21527,00
Volume Of Concrete : 340.18 cub. yd.
Chloride concentration units : % wt. conc.

Figure 2.5.10. Life 365 - Settings

The climatic data used in this research paper used the settings of San Antonio, Texas, USA, which was the project site.

Temperature Cycle (°F) (Automatically set)

Month	Temperature (°F)
January	49.3
February	53.4
March	61.7
April	69.3
May	75.6
June	82.2
July	84.9
August	84.9
September	79.3
October	70.2
November	60.4
December	52.2

Figure 2.5.11. Life 365 - Temperature parametrs

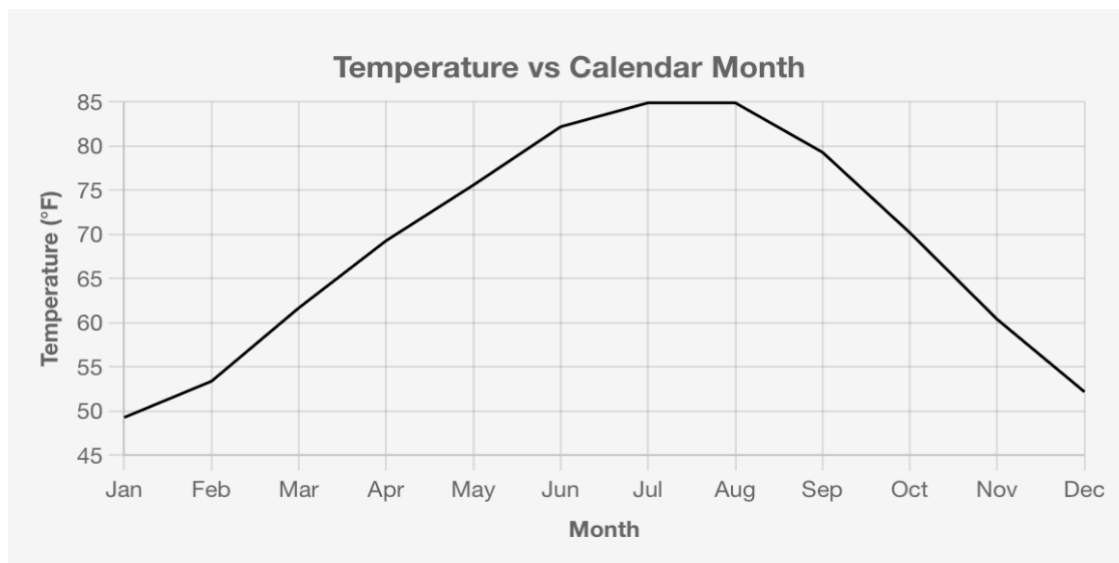


Figure 2.5.12. Life 365 - The graph of exposure parameters

Concrete Mixture Design for Service Life Modeling

The concrete mixture was set to meet the necessary compressive strength of the structural elements which is set at 42.7 MPa. The ratio between water-to-cementitious

material (w/cm) was chosen to be dependent on its simple correlation with compressive strength and durability performance.

Water to cementitious material ratio is given as:

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

where:

W_{water} - mass of mixing water ;

W_{cement} - mass of Portland cement;

W_{SCM} - mass of supplementary cementitious materials (e.g., slag).

In order to calculate the corresponding compressive strength, the empirical law of Abrams was applied:

$$f_c = \frac{K_1}{K_2^{w/cm}}$$

where:

K_1 - 96.5 MPa; K_2 - 7; w/cm - water-to-cementitious material ratio

By replacing these values:

$$f_c = \frac{96.5}{7^{0.42}} = \frac{96.5}{2.26} = 42.7 \text{ MPa}$$

This forecasted strength is higher than the design strength required of 42.7 MPa which verifies that the chosen mixture is both structurally strong and strong enough in terms of durability. Besides considerations of strength, lesser water to cementitious material ratio enhances durability since it reduces the permeability of concrete and chloride ion diffusion is slowed down. This directly raises the time taken to reach the concentration of chloride to the level of reinforcement, which is a very important factor in service life prediction.

Thus, the chosen value of:

$$w/cm = 0.42$$

is ideal combination of structural strength, durability and performance. The Life-365 software took these parameters of a mixture as input in simulating the chloride penetration

and estimating the service life of the reinforced concrete structural elements of the Altyn Fiesta Hotel in parking garage exposure conditions in San Antonio, Texas.

Base case

Mixture	Rebar	Barriers
w/cm * 0,45	Rebar Steel Type * Epoxy Coated	Select Barrier * <none>
Class F fly ash (%) * 0	Rebar % vol. concrete. * 1,2	
Slag (%) * 0	Inhibitor Select Inhibitor * <none>	
Silica Fume (%) * 0		

Figure 2.5.13. Settings for Base case

Alternative 1

Mixture	Rebar	Barriers
w/cm * 0,42	Rebar Steel Type * Epoxy Coated	Select Barrier * <none>
Class F fly ash (%) * 20	Rebar % vol. concrete. * 1,2	
Slag (%) * 0	Inhibitor Select Inhibitor * <none>	
Silica Fume (%) * 0		

Figure 2.5.14. Settings for Alternative 1 case

Alternative 2

Mixture	Rebar	Barriers
w/cm * 0,42	Rebar Steel Type * Epoxy Coated	Select Barrier * <none>
Class F fly ash (%) * 0	Rebar % vol. concrete. * 1,2	
Slag (%) * 0	Inhibitor Select Inhibitor * <none>	
Silica Fume (%) * 8		

Figure 2.5.15. Settings for Alternative 2 case

Alternative 3

Mixture	Rebar	Barriers
w/cm * 0,4	Rebar Steel Type * Epoxy Coated	Select Barrier * <none>
Class F fly ash (%) * 20	Rebar % vol. concrete. * 1,2	
Slag (%) * 0	Inhibitor Select Inhibitor * <none>	
Silica Fume (%) * 8		

Figure 2.5.15. Settings for Alternative 3 case

Thereby, by putting all these values to software, we obtain information about service life, which showing us our concrete mixes in one table with all important data and another table about diffusion properties & service lives.

Concrete Mixes						
Name	User?	w/cm	SCMs	Inhib.	Barrier	Rein
Base case	No	0.45		<none>	<none>	Epoxy Coa
Alternative 1	Yes	0.42	Class F Fly Ash (20%);	<none>	<none>	Epoxy Coa
Alternative 2	Yes	0.42	Silica Fume (8%);	<none>	<none>	Epoxy Coa
Alternative 3	Yes	0.4	Class F Fly Ash (20%); Silica Fume (8%);	<none>	<none>	Epoxy Coa

Figure 2.5.16. Concrete Mixes for all options

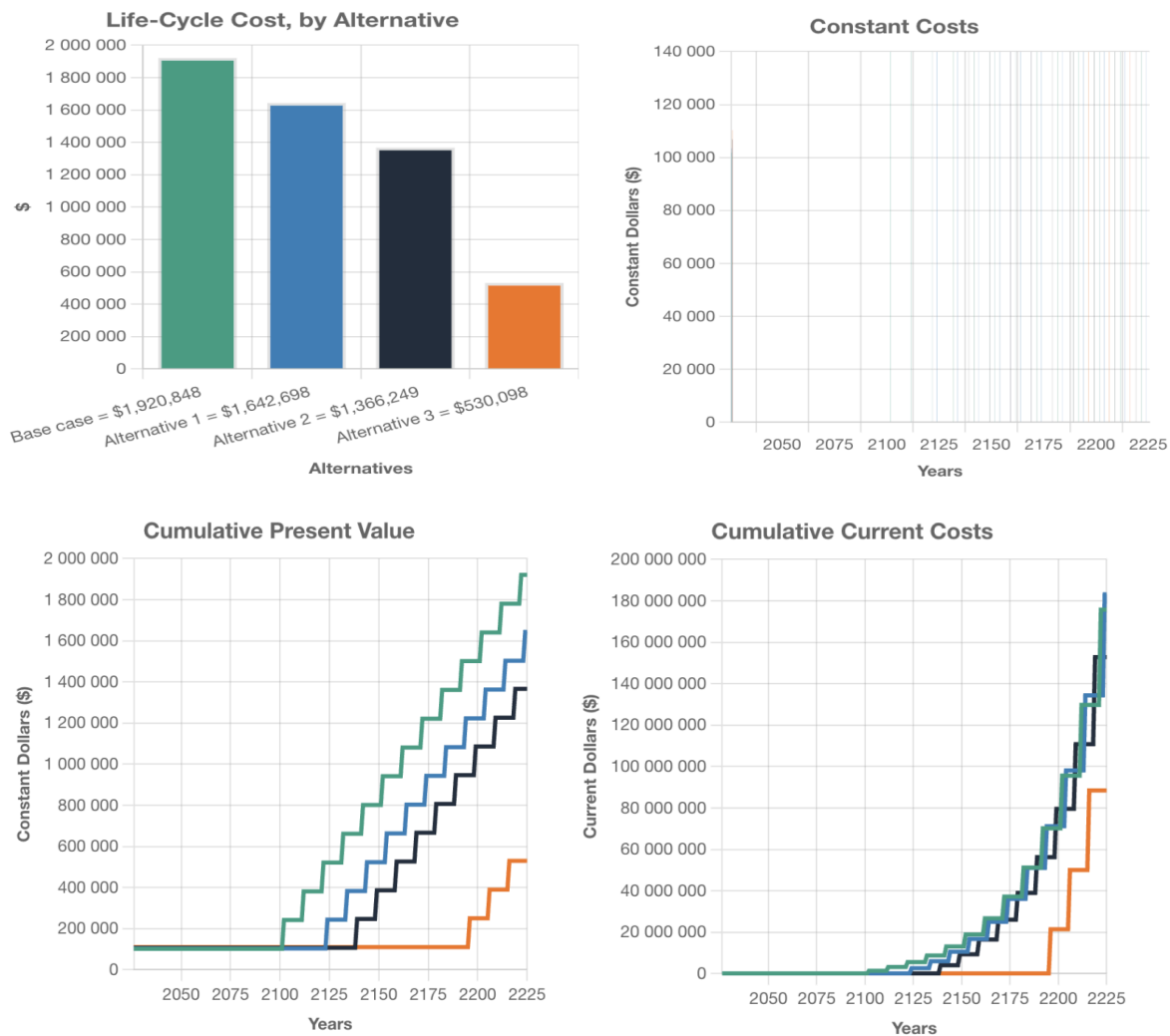
In this figure above, we can see that all of our options used epoxy coated bars for reinforcement. It was chosen because for our project we cannot use black steel, as in our project we have a corrosion environment. Because of that, we add additives to handle this issue and maximize all parameters. This procedure was beneficial, as we can see the difference and also compare how any additive can change the situation, worse or upgrade. By this we can see and choose the best option for our project.

Diffusion Properties & Service Lives						
Name	D ₂₈ (in ² /in/sec)	m	C _t (% wt. conc.)	Init.	Prop.	Service Life
Base case	1.62e-8	0.200	0.05	56.08 yrs	20.00 yrs	76.08 yrs
Alternative 1	1.38e-8	0.360	0.05	78.83 yrs	20.00 yrs	98.83 yrs
Alternative 2	3.67e-9	0.200	0.05	93.25 yrs	20.00 yrs	113.25 yrs
Alternative 3	3.29e-9	0.360	0.05	150.25 yrs	20.00 yrs	170.25 yrs

Figure 2.5.17. Service Lives for all options

Finally we have got results for this project, which include life cycle cost to all options, and 4 figures showing us important things.

Life-Cycle Costs				
Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Base case	\$101,817	\$0	\$1,819,032	\$1,920,848
Alternative 1	\$103,518	\$0	\$1,539,181	\$1,642,698
Alternative 2	\$106,920	\$0	\$1,259,330	\$1,366,249
Alternative 3	\$110,321	\$0	\$419,777	\$530,098



Figures 2.5.15.18. LLC Results from Life 365

After considering the information and data we have, we can say that we ought to select Option 3 because it is the cheapest alternative for repairs. Even though it is the most expensive alternative, with a construction cost of \$110,321, which many may find counter-intuitive and disagreeable, Option 3 is the least costly in the long run and is well worth it.

It may seem like a bad choice at first, but when we take into account the repair costs, Option 3 is much less expensive, with a repair cost of \$419,777, than the other options, which have much higher repair costs. For instance, Base Case has a repair cost of \$1,819,032, while Alternative 1 costs \$1,539,181 to repair.

Ultimately, Option 3 is more cost effective in terms of total life-cycle cost, with a total of \$530,098, compared to \$1,920,848 for the Base Case and \$1,642,698 for Alternative 1.

It's critical to take everything into account and invest in the right things now, rather than paying more in the future, which would be less economical. While the up-front cost is higher, it is far more economical to have a substantially lower cost of repair, making Option 3 the most practical and economical option.

Repair methods for Alternative 3

In terms of repair methods, for the future, we have identified a couple of repair methods that will help maintain the longevity of the structure and limit future costs. This will depend on the scenario and condition of the structure.

- Injection Grouting - this approach will be applied mainly where there are indications of soil movement or settling under the foundation. This type of grouting involves pumping grout into the soil to fill in open spaces, which will help to prevent further displacement. This method is suitable to secure the foundation in situations of slight to medium soil movement and is a cost-effective method that does not require major repairs.
- Replacing Damaged Members - in cases of significant damage or failure of a structural element, it may need to be replaced entirely. This may involve the replacement of damaged beams, columns or other primary elements. It's necessary when the damage is severe, and the element can't be repaired or reinforced to maintain the safety of the structure.
- Regular Monitoring and Maintenance - the structure will be continually checked and maintained. This involves looking out for early indicators of damage (cracking, moisture) and implementing simple repairs (crack sealing, surface treatments, drainage) as needed. This will be implemented on a regular basis to keep the structure in good shape and to avoid more severe problems.

These techniques will be used depending on the findings of inspections. For smaller problems, injection grouting and ongoing maintenance will be adequate. But for major structural problems, such as critical damage to structural elements, replacement will be preferred.

3. Structural design

3.1. Design consideration

The scope requirements and the ways they were addressed include:

- Strong wind zone. Since the building is located in a windy area, the wind load and 4 loading cases were evaluated. In the next part of the project, drift due to wind loading will be calculated and member sizes will be readjusted accordingly.
- Large span. To tackle the issue of large span design, a prestressed beam design was attempted. Suitability of large span for the building's function is explained in the design concept development.
- Corrosion environment. The resistance of the building to the corrosion is mainly provided by the choice of reinforced concrete as the main structural material. In the next part of the project, during detailing, more consideration will be given to this condition (e.g. epoxy coating of reinforcement, increased clear cover, etc.)

3.2. Material selection

3.2.1. Structural materials

The structural material chosen for this building is reinforced concrete (RC), which was selected after detailed evaluation. The decision is based on structural performance, durability, fire resistance, and on cost efficiency.

For a multi-rise 8-storey building located in San-Antonio, reinforced concrete provides a great balance between strength and economy. It is known for high compressive capacity, with steel reinforcement ensuring sufficient tensile strength. San- Antonio experiences high temperature variations, high humidity, and occasional rainfall. These climatic conditions can cause high weathering and corrosion, to which reinforced concrete resists better than steel structures.

Fire resistance is also a significant consideration in selecting the material. Unlike steel structures, reinforced concrete acts better in a fire. It retains its structural integrity under high temperature and does not rapidly deform when exposed to fire. This ensures better safety, especially for hotels with restaurants and saunas with likely fire emergencies. Additionally,

its thermal mass and acoustic insulation minimizes temperature fluctuations and reduces energy consumption for cooling in the Texas climate.

3.2.2. Non-structural materials

First, the non-structural materials suitable for each area were chosen. The floor composition is the same for all floors, with the topmost finish layer varying based on the functional and aesthetic needs of each area. Average thickness of floor finish is 245 mm.

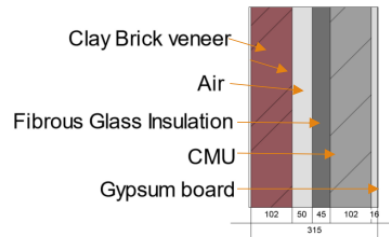


Figure 3.2.2.1. Exterior Wall Assembly

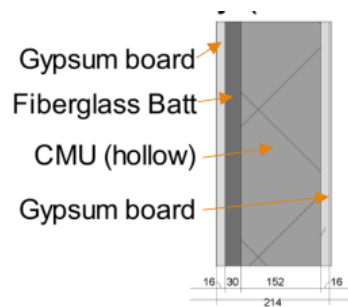


Figure 3.2.2.2. Interior Wall Assembly (1-3 floors)

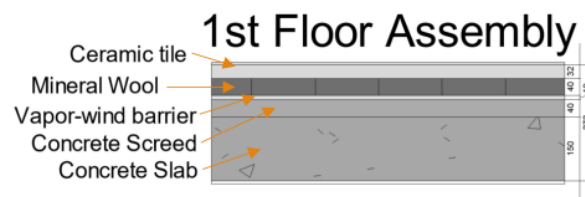


Figure 3.2.2.3. 1st floor assembly

3.3. Gravity loads calculation

3.3.1. Dead load

The design dead loads of non-structural materials were taken from Chapter 3 of ASCE 7-16. Tables 3.2.1-3.2.4 demonstrate the design dead loads for floor finishing, ceiling and roof.

Since the clear height of each floor will be reduced by floor-finish and ceiling layers by approximately 500 mm, the floor-to-floor height of each level was increased by 0.5 m to compensate for this loss.

Table 3.3.1.1. Dead load of floor finishing

Area	Function	Material Type	Thickness, mm	Load, kPa	Total Load, kPa
All rooms (without cover)	Structural Slab	Reinforced Concrete Slab	130	3.25	3.39
	Insulation	Mineral Wool Slab	40	0.088	
	Water-proofing	Vapor-wind barrier membrane	10	0.03	
	Leveling	Concrete Screed	40	0.023	
1 floor (lobby, restaurant, banquet hall)	Cover	Ceramic tile on mortar bed	32	0.77	4.16
2 floor (Gym + SPA)		Linoleum	6	0.05	3.44
3 floor (Business hub)		Carpet	11	0.02	3.41
5-8 floor (Hotel rooms)		Carpet	11	0.02	3.41

Table 3.3.1.2. Dead load of ceiling

Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Mechanical Duct Allowance	210	0.19	0.34
Suspended steel	25	0.1	

channel system			
Acoustical fiber board	20	0.05	

Table 3.3.1.3. Dead load of roof materials

Function	Selected Material	Thickness, mm	Load, kPa	Total Load, kPa
Cover	Three-ply ready roofing	10	0.05	3.482
Insulation	Insulation fiberboard	40	0.112	
Water-proofing and Leveling	Bituminous, smooth surface	-	0.07	
Structural Slab	Reinforced Concrete Slab	130	3.25	

Table 3.3.1.4. Wall compositions and their design dead loads

Exterior wall			
Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Clay brick veneer	102	1.87	3.1425
Air	50	0	
Fibrous Glass Insulation	45	0.0945	
Hollow CMU wall (normal-weight, ungrouted)	102	1.05	
Gypsum (Type X) Board	16	0.128	
Interior wall (1-3 floors)			
Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Gypsum Board	16	0.128	1.799

Fibrous Glass Insulation	30	0.063	
Hollow CMU wall (grout spacing 1219 mm)	152	1.48	
Gypsum Board	16	0.128	
Interior wall (4-8 floors)			
Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Gypsum Board	16	0.128	0.8229
Fibrous Glass Insulation	89	0.1869	
Steel studs, 13 mm gypsum each side	92	0.38	
Gypsum Board	16	0.128	
Parapetes (height = 1.2 m)			
Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Exterior Painting finish	1	0.0006	1.0512
Hollow CMU wall (normal-weight, ungrouted)	203	1.05	
Exterior Painting finish	1	0.0006	

For the final dead load applied at each level, the weight of partition walls was added by smearing their weight across the floor area.

$$DL_{\text{partition}} = \frac{\text{total load} \times \text{total length on the floor} \times \text{floor height}}{\text{floor area}}$$

For example, for floors 4-8 with lighter but longer walls:

$$DL_{\text{partition}} = \frac{0.8229 \times 682.4 \times 3.5}{2000} = 0.98 \text{ kPa}$$

The same calculations were done for the first three floors with varying wall lengths. The lengths of the interior walls were taken from the Revit model.

The exterior wall line load will be applied along the perimeter of the building only.

Table 3.3.1.5. Summary of dead loads for all floors

Location	Height, m	Total Area, m^2	Interior Wall	Final Dead	Exterior Wall
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			Load, kPa	Load, kPa	Load, kN/m
Basement slabs (-3, -2, -1)	3.25	2400	0	3.73	10.21
Slab 0	4.5	2400	0.82	5.32	14.14
Slab 1	4.5	2400	0.79	4.58	14.14
Slab 2	4.5	2400	1.01	4.76	14.14
Slab 3	3.5	2400	0.82	4.57	11.00
Slab 4-7	3.5	2000	0.98	4.73	11.00
Slab 8	1.2	2000	0	3.82	1.28

3.3.2. Live load

According to ASCE 7-16, live load reduction is not permitted in the following areas:

- Underground Parking floors: $L_0=1.92$ kPa
- Storage & Building Services floor $L_0=6$ kPa
- Assembly areas and corridors serving them (Lobby, Restaurant, Lounge, Conference Rooms) $L_0=4.79$ kPa
- Recreational areas (Gymnasium, Spa) $L_0=4.79$ kPa

However, the live load can be reduced for structural elements on the residential floors with hotel rooms and corridors serving them where $L_0=1.92$ kPa using corresponding live load element factor, K_{LL} , for each structural element and the formula:

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

Table 3.3.2.1. Reduced live load calculations for residential floors

Structural Element	$A_t (m^2)$	K_{LL}	L_0	Reduction	L (kPa)
Interior Column	100	4	1.92	0.479	0.92
Exterior Column	50	4	1.92	0.573	1.10

Corner Column	25	4	1.92	0.707	1.36
Central corner columns	75	4	1.92	0.514	0.99
Minor Beam	12.5	2	1.92	-	1.92
Exterior Beam (Large Span)	50	2	1.92	0.707	1.36
Exterior Beam (5 Span)	6.25	2	1.92	-	1.92
Interior Beam (Large Span)	100	2	1.92	0.573	1.10
Interior Beam (5 Span)	12.5	2	1.92	-	1.92

As for the roof live load, it can be reduced using the following formulas:

$$L_r = L_o R_1 R_2 \quad (0.58 \leq L_r \leq 0.96)$$

The reduction factor R_1 was determined as:

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

$$R_1 = 1.2 - 0.011A_t \quad (18.58 \text{ m}^2 < A_t < 55.74 \text{ m}^2)$$

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

The reduction factor $R_2 = 1$ since F (roof slope) = $0.12 \times 2 = 0.24 \leq 4$

Table 3.3.2.2. Reduced roof live load calculations

Structural Element	$A_t \text{ (m}^2\text{)}$	R_1	L_o	$L_r \text{ (kPa)}$
Interior Column	100	0.6	0.96	0.58
Exterior Column	50	0.65	0.96	0.624
Corner Column	25	0.925	0.96	0.888
Central corner columns	75	0.6	0.96	0.58
Minor Beam	12.5	1	0.96	0.96
Exterior Beam (Large Span)	50	0.65	0.96	0.624
Exterior Beam (5 Span)	6.25	1	0.96	0.96
Interior Beam (Large Span)	100	0.6	0.96	0.58
Interior Beam (5 Span)	12.5	1	0.96	0.96

3.3.3. Snow load

Flat roof snow load can be calculated using the following equation:

$$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$$

The calculation parameters include:

- Surface Roughness: B, Urban Area
- Roof Exposure: Partly Exposed
- Ground snow load, $p_g = 5 \text{ psf} = 0.24 \text{ kN/m}^2$ (ASCE Hazard Tool)
- Exposure factor, $C_e = 1.0$ (ASCE 7-10, Table 7-2, for Terrain Category B and partly exposed roof)
- Thermal factor, $C_t = 1.0$ (ASCE 7-10, Table 7-3)
- Importance Factor, $I_s = 1.0$ (Risk Category II, hotel building)

$$p_f = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 0.24 = 0.168 \text{ kN/m}^2$$

For $p_g < 0.96 \text{ kN/m}^2$, the minimum roof snow load is:

$$p_m = I_s \times p_g = 1.0 \times 0.24 = 0.24 \text{ kN/m}^2$$

Since $p_f < p_m$, the p_f will be taken as 0.24 kN/m^2

The snow density as estimated as:

$$\gamma_s = 0.426 \times p_g + 2.2 = 0.426 \times 0.24 + 2.2 = 2.3 \text{ kPa} < 4.7 \text{ kPa}$$

The height of the snow is the ratio of the flat roof snow load and snow density:

$$h_b = \frac{p_f}{\gamma_s} = \frac{0.24}{2.3} = 0.104 \text{ m}$$

To calculate the snow drift load on the slab at level 3, the following formulas were used:

$$h_c = h - h_b = 3.5 \times 5 - 0.16 = 17.34 \text{ m}$$

With length of the roof upwind of the drift $L_u = 40 \text{ m}$ and $L_u^* = 10 \text{ m}$, the height of snow drift was estimated as:

$$h_{d, leeward} = (0.43 \sqrt[3]{L_u \times 3.28} \times \sqrt[4]{p_g + 10} - 1.5) \div 3.28 = 0.58 \text{ m}$$

$$h_{d, windward} = 0.75 \times (0.43 \sqrt[3]{L_{u, windward}} \times \sqrt[4]{p_g + 10} - 1.5) \div 3.28 = 0.28 \text{ m}$$

The final value is taken as the maximum out of these 2 calculated drift heights, so:

$$h_d = 0.58 \text{ m}$$

$$p_d = h_d \times \gamma_s = 0.58 \times 2.3 = 1.34 \text{ kPa}$$

Since $h_d \leq h_c$, the drift width is:

$$w = 4 \times h_d = 4 \times 0.58 = 2.33 \text{ m}$$

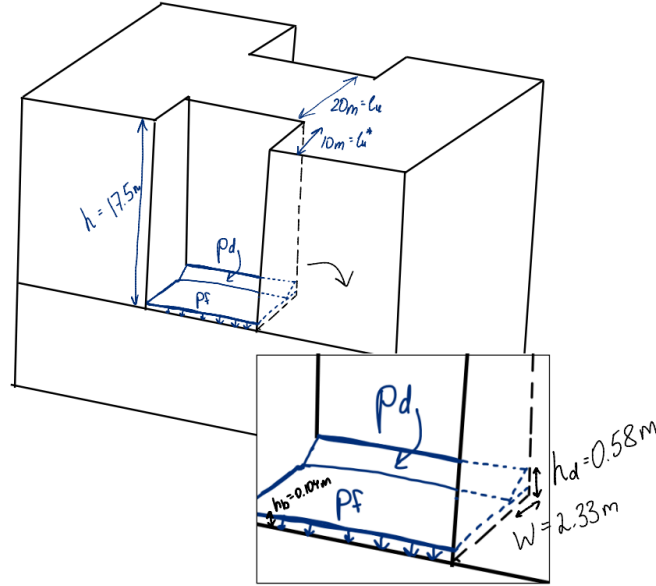


Figure 3.3.3.1. Snow drift illustration for slab at level 3 (not to scale)

During load assignment to SAP2000, the load due to snow drift was turned into load uniformly distributed across the slab area and added to p_f for simplification.

3.4. Member size estimation

3.4.1. Structural layout

It is required to have a large span of at least 20 meters in one direction, while in the other direction columns were chosen to be spaced at 5 meters. The large-span system makes it crucial to add minor beams. Without minor beams, the slab spanning 20 meters would have to be very thick which is impractical. To avoid punching shear, large deflections, serviceability problems and constructability issues, minor beams have to be added. This leads to the geometry and panel aspect ratio of a two-way slab system.

The major 5 meter beam's height and width are estimated as:

$$h_{beam} = L/12 = 5/12 = 0.42 \approx 0.45 \text{ m}$$

$$b_{beam} = 0.5 \times h_{beam} = 0.5 \times 0.45 = 0.2125 \approx 0.25 \text{ m}$$

The minor beam's height and width are estimated as:

$$h_{beam} = L/16 = 5/62 = 0.3125 \approx 0.35 \text{ m}$$

$$b_{beam} = 0.5 \times h_{beam} = 0.5 \times 0.35 = 0.156 \approx 0.175 \text{ m}$$

One minor beam would be insufficient as it still leads to large slab thickness, so the choice is between 2 or 3 minor beams.

When 3 minor beams are introduced, the large span slab is effectively divided into 4 two-way slab panels 5×5 meters. Two-way slab thickness estimation (for panel formed between minor beams) for the case with 3 minor beams:

The materials chosen for slabs and beams are concrete C30 grade ($f'_c = 30 \text{ MPa}$) and reinforcing steel Grade 60 ($f_y = 60000 \text{ ksi}$). Concrete's elastic modulus according to ACI 318 is:

$$E_c = 4700\sqrt{f'_c} = 25742.96 \text{ MPa} = E_{cb} = E_{cs}$$

For the 1st iteration, the slab thickness was assumed as $h_{slab} = 0.3 \text{ m}$

Cracked moment of inertia calculations:

$$I_{major} = \frac{0.35}{12} \times h_{beam}^3 \times b_{beam} = \frac{0.35}{12} \times 0.7^3 \times 0.35 = 0.0035 \text{ m}^4$$

$$I_{minor} = \frac{0.35}{12} \times 0.35^3 \times 0.175 = 0.000219 \text{ m}^4$$

$$I_{slab} = \frac{0.25}{12} \times h_{slab}^3 \times L = \frac{0.35}{12} \times 0.3^3 \times 5 = 0.002813 \text{ m}^4$$

Stiffness factor and stiffness ratio calculations:

$$\alpha_{f(major \text{ beam})} = \frac{E_{cb} I_b}{E_{cs} I_s} = \frac{25742.96 \times 0.0035}{25742.96 \times 0.002813} = 1.245$$

$$\alpha_{f(minor \text{ beam})} = \frac{25742.96 \times 0.000216}{25742.96 \times 0.002813} = 0.078$$

$$\alpha_{fm} = \alpha_{f1} + \alpha_{f2} + \alpha_{f3} + \alpha_{f4} = 2 \times \alpha_{f(major \text{ beam})} + 2 \times \alpha_{f(minor \text{ beam})} = 0.661$$

$$\beta = \frac{L_{yn}}{L_{xn}} = \frac{5}{5} = 1$$

Since $0.2 < \alpha_{fm} = 0.661 \leq 2$, minimum slab thickness is:

$$h_{min} = \frac{l_n(0.8 + \frac{f_y}{200000})}{36 + 5\beta(\alpha_{fm} - 0.2)} = \frac{4.825 \times (0.8 + \frac{60000 \text{ psi}}{200000})}{36 + 5 \times 1 \times (0.661 - 0.2)} = 138.6 \text{ mm} > 125 \text{ mm}$$

where clear slab span $l_n = L_{yn} - b_{minor} = 5 - 0.175 = 4.825$

This value was used for the 2nd iteration to obtain $\alpha_{fm} = 6.71 > 2$ and new minimum slab thickness:

$$h_{min} = \frac{l_n(0.8 + \frac{f_y}{200000})}{36 + 9\beta} = 118 \text{ mm} > 90 \text{ mm}$$

The same procedure was repeated for the case with 2 minor beams and the results are presented in Table 3.4.1 for comparison.

Table 3.4.2.1. Summary of slab thickness calculations for 2 cases

Case	3 minor beams		2 minor beams	
$\alpha_{f(major \text{ beam})}$	1.245	12.64	0.667	2.554
$\alpha_{f(minor \text{ beam})}$	0.078	0.79	0.056	0.523
α_{fm}	0.661	6.714	0.933	3.862
L_{xn} (m)	5.00	5.00	5.00	5.00
L_{yn} (m)	5.00	5.00	6.67	6.67
l_n (m)	4.83	4.83	6.46	6.46
β	1	1	1.334	1.334
h_{min} (m)	0.1386	0.118	0.187	0.148
Volume of slab	3.464	2.949	6.23	4.94
Volume of minor beam	0.306	0.306	0.31	0.31
Volume of 20-m major beam	4.900	4.900	4.90	4.90
Volume of 5-m major beam	0.563	0.563	0.56	0.56
Volume total	25.699	23.638	30.23	31.60
Weight, kN	642.47	590.95	755.83	789.88

Since the case with 3 minor beams requires less thickness and offers a more economical solution, this option was adopted for the GLRS design. The final slab thickness is 130 mm. The structural layout can be seen in the figure below.

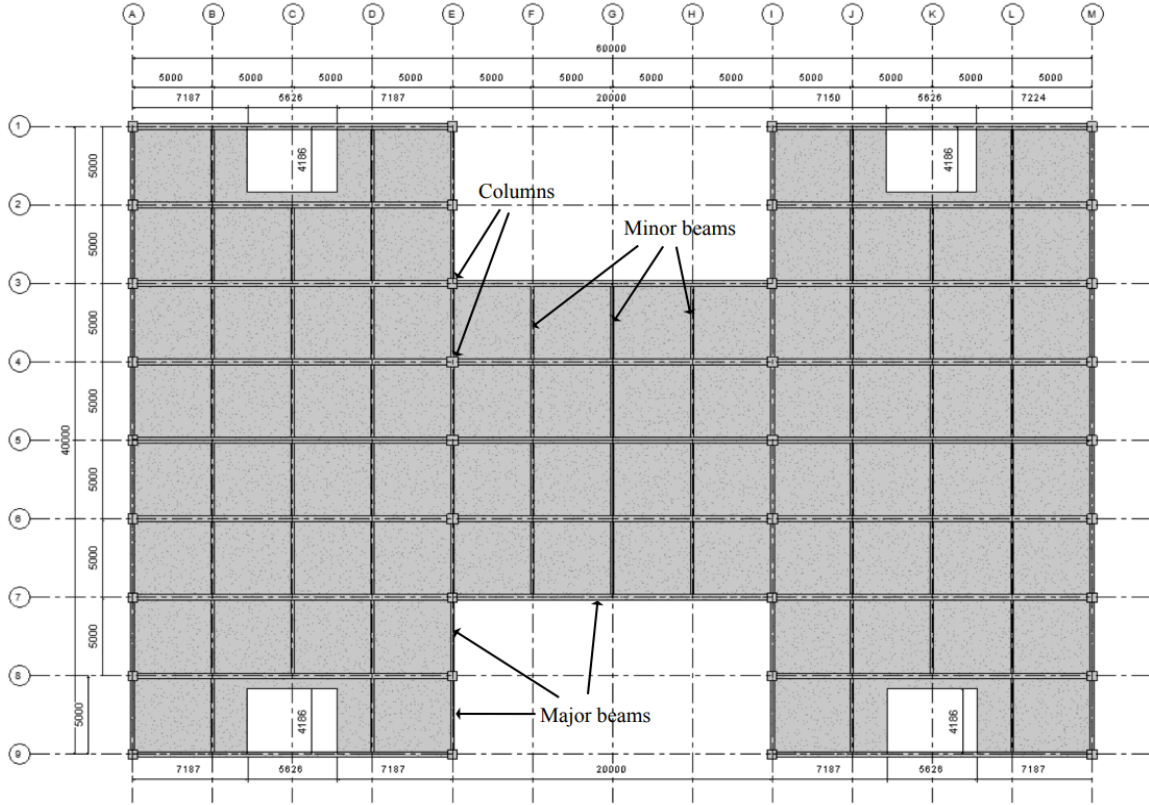


Figure 3.4.2.1. Structural layout

3.4.2. Column size estimation

According to ACI 318-19 (American Concrete Institute, 2022), the main formula used for reinforced concrete column's design axial compressive strength is:

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

where reinforcement area is assumed as 1% of the concrete area: $A_{st} = 0.01 \times A_g$.

The factored axial load pressure (kPa) and axial force (kN) based on tributary area are calculated as:

$$w_u = 1.2DL + 1.6LL + 0.5L_r$$

$$P_u = A_T \times P_u$$

Using $P_u \leq \phi P_n$ and rearranging produces formula for column's required cross-sectional area:

$$A_g = \frac{P_u}{0.8 \times \phi \times (0.99 \times \beta \times f'_c + 0.01 \times f_y)} \quad (B)$$

The self-weight of the column is added during the iterative column size estimation to the axial load demand:

$$W_1 = B^2 \times H \times \rho$$

Weight of beams carried by columns is calculated using beam's weight per unit length:

$$w_b = h_{beam} \times b_{beam} \times \rho_{conc}$$

Weight of beam carried by column:

$$W_b = w_b \times l$$

Where l - the total length of beam type carried by the column based on its tributary area (m)

The amount of beam weight carried by each column type was calculated separately. Example calculations of beam weight carried by internal columns is presented below.

Table 3.4.3.1. Weights of beams carried by internal columns

Major 20-meter beam			
w_b (kN/m)	l (m)	W_b (kN)	Factored W_b (kN)
6.13	20	122.50	147.0
Major 5-meter beam			
w_b (kN/m)	l (m)	W_b (kN)	Factored W_b (kN)
2.81	5	14.06	16.88
Minor 5-meter beam			
w_b (kN/m)	l (m)	W_b (kN)	Factored W_b (kN)
1.53	15	22.97	27.56

It is worth mentioning that for the exterior columns, the load from exterior walls was added to the final axial force demand as well.

Table 3.4.3.2. Column axial forces calculations for interior columns

Interior Columns							
Level	DL	L_r (kPa)	LL (kPa)	A_T (m ²)	w_u (kPa)	P_u (kN)	P_u (incl. weights)

	(kPa)						of beams) (kN)
-3	3.73	0.000	1.92	100.00	7.549	754.92	946.36
-2	3.73	0.000	1.92	100.00	7.549	754.92	946.36
-1	5.32	0.000	4.79	100.00	14.046	1404.64	1596.07
1	4.58	0.000	4.79	100.00	13.154	1315.40	1506.84
2	4.76	0.000	4.79	100.00	13.382	1338.20	1529.63
3	4.57	0.000	0.92	100.00	6.954	695.39	886.82
4	4.73	0.000	0.92	100.00	7.150	715.04	906.48
5	4.73	0.000	0.92	100.00	7.150	715.04	906.48
6	4.73	0.000	0.92	100.00	7.150	715.04	906.48
7	4.73	0.000	0.92	100.00	7.150	715.04	906.48
8	3.82	0.580	0.00	100.00	4.876	487.64	679.08

After estimating the axial loads at each floor level, they were summed up progressively downward to obtain the cumulative axial load on columns on each floor. They also included the self-weights of the columns from above floors.

Table 3.4.3.3. Cumulative axial load for interior columns

Interior Column		
Floor	P_u (cum) (kN)	Column self-weight (kN)
8	679.08	7.875
7	1593.43	7.875
6	2507.78	21.875
5	3436.14	21.875
4	4364.49	21.875
3	5273.19	47.53125
2	6850.35	47.53125

1	8404.72	47.53125
-1.00	10048.33	45.703125
-2.00	11040.39	45.703125
-3.00	12032.45	45.703125

Then, using the equation 3.11 and iterative approach, the interior columns on each floor were sized. Concrete C40 was used for columns since they have larger tributary areas and larger axial demand than usual columns due to the large span. Moreover, stronger concrete allows smaller column dimensions, preserving usable floor area. The following table shows the results for interior columns on the 3rd floor.

Table 3.4.3.4. Estimation of interior column sizes on the 3rd floor

Iterati on	b (mm)	H (mm)	ρ ($\frac{kN}{m^3}$)	W_1 (kN)	$W_1 + P_u$ (cum)	ϕ	f'_c (MPa)	f_y (MPa)	A_g (mm ²)	b (mm)	Final b (mm)
1	1000	4500	25	135.00	5408.19	0.65	40	420	249289.5	499.3	500
2	499.29	4500	25	33.65	5306.84	0.65	40	420	244618.0	494.6	
3	494.59	4500	25	33.02	5306.21	0.65	40	420	244588.9	494.6	

The same procedure was repeated for each type of column depicted in the figure below because they have different tributary areas. To differentiate between them, they were labelled as:

- Pink - edge columns
- Orange - corner columns
- Green - interior columns
- Yellow - transition columns
- Cyan - edge columns II
- Purple - corner columns II

The edge columns II and corner columns II were evaluated separately because their tributary areas differ between 1-3 floors and 4-8 floors, and it affects the cumulative axial load. The difference in tributary area comes from difference in floor plan geometry: the first 3 floors are rectangular and the upper 5 floors are H-shaped.

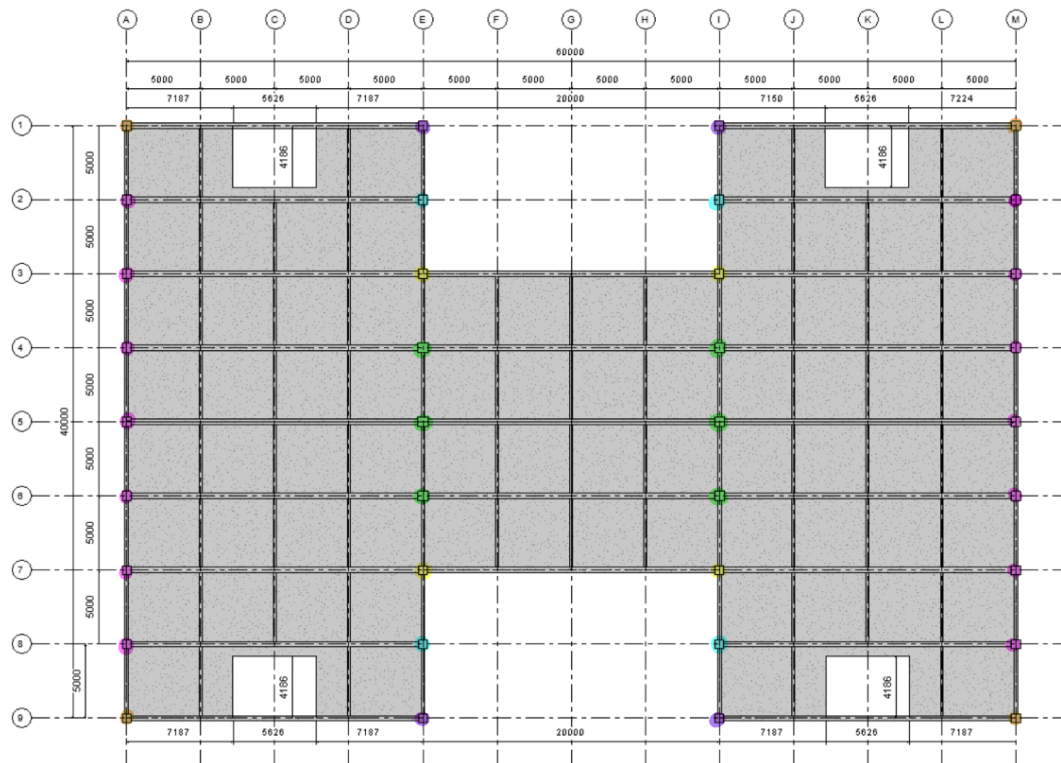


Figure 3.4.3.1. Types of columns used for size estimation

Table 3.4.3.5. Column size estimation results

Floor	Interior column (mm)	Edge column (mm)	Corner column (mm)	Edge column II (mm)	Corner column II (mm)	Transition column (mm)
-3	750	600	500	700	600	750
-2	750	550	500	700	600	750
-1	700	550	500	650	550	700
1	650	500	450	600	500	650
2	600	450	400	500	450	600
3	500	400	350	450	350	500
4	500	350	350	350	350	450
5	400	350	300	350	300	450
6	350	300	300	300	250	400
7	300	300	300	300	300	300

8	300	300	300	300	300	300
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Then, the appropriate column sizes were chosen per each 2-3 floors so that axial force on each type of column is safely resisted. The final column dimensions are listed below.

Table 3.4.3.6. Final column dimensions

Floor	Exterior column (mm)	Interior column (mm)
Basement	650	750
1-3	600	650
4-6	450	500
7-8	300	300

3.5. Wind loads

For the Altyn Fiesta building, Risk Category is II and the Importance Factor is $I_w = 1.0$. For San Antonio, TX, the basic wind speed used for calculations is $v = 51$ m/s (return period = 700 years; ASCE 7-10, Figure 26.5-1A & ASCE Hazard tool). The building is in the city center and surrounded by surface roughness B for more than 792 m, so the constants for exposure Category B were used. These coefficients are listed in the table below.

Table 3.5.1. 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	365.76	1/7	0.84	0.25	0.45	0.3	97.54	1/3	9.14

The building is located on the flat ground, so the topographical effect, $K_{zt} = 1.0$. For main wind load resisting system, the directionality factor is $K_d = 0.85$.

To consider the height effect, the following equations were employed:

$$K_z = 2.01 \left(\frac{4.572}{z_g} \right)^{2/\alpha} \text{ for } z \leq 4.572 \text{ m}$$

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} \text{ for } 4.572 \text{ m} \leq z \leq z_g$$

Table 3.5.2. Height effect coefficients

Floor #	z (m)	K_z (m)
1	4.5	0.5747
2	9.0	0.6974
3	13.5	0.7831
4	17.0	0.8364
5	20.5	0.8824
6	24.0	0.9230
7	27.5	0.9596
8	31.0	0.9930
Parapets	32.2	1.0040

Now, the windward and leeward wind velocity pressure for each floor level can be calculated using the following equations respectively:

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

$$q_h = 0.613v^2 K_d K_h K_{ht}$$

Table 3.5.3. Wind velocity pressure

Floor #	q_z (kPa)	q_h (kPa)
1	0.7789	1.3458
2	0.9452	1.3458
3	1.0613	1.3458
4	1.1335	1.3458
5	1.1958	1.3458
6	1.2509	1.3458
7	1.3005	1.3458
8	1.3458	1.3458

The next step includes the computation of the Gust factor for the cases when wind blows in 2 directions. For the case when wind forces act in y-direction:

$$n_a = \frac{43.5}{H^{0.9}} = \frac{43.5}{(32.2 \times 3.28)^{0.9}} = 0.656 < 1 \quad \& \quad h = 32.22 \text{ m} > 18 \text{ m} \Rightarrow \text{flexible structure}$$

$$\bar{z} = 0.6h = 0.6 \times 32.22 = 19.33 \text{ m} > z_{min} = 9.14 \text{ m}$$

$$I_z = c\left(\frac{10}{z}\right)^{\frac{1}{6}} = 0.3\left(\frac{10}{19.33}\right)^{\frac{1}{6}} = 0.2688$$

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V = 0.45\left(\frac{19.33}{10}\right)^{0.25} \times 51 = 27.06 \text{ m/s}$$

$$n_1 = n_a = 0.656, \quad L_z = l\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} = 97.54\left(\frac{19.33}{10}\right)^{0.33} = 121.51 \text{ m}$$

$$N_1 = \frac{n_1 L_z}{\bar{V}_z} = \frac{0.656 \times 121.51}{27.06} = 2.95$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47 \times 2.95}{(1 + 10.3 \times 2.95)^{5/3}} = 0.0706$$

$$\eta_h = \frac{4.6 n_1 H}{\bar{V}_z} = \frac{4.6 \times 0.656 \times 32.22}{27.06} = 3.59$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2} (1 - e^{-2\eta_h}) = 0.2396$$

$$\eta_b = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6 \times 0.656 \times 60}{27.06} = 6.69 \Rightarrow R_b = 0.1383$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4 \times 0.656 \times 36}{27.06} = 14.93 \Rightarrow R_L = 0.0647$$

$$R = \sqrt{\frac{R_n R_h R_b (0.53 + 0.47 R_L)}{\beta}} = \sqrt{\frac{0.0706 \times 0.2396 \times 0.1383 \times (0.53 + 0.47 \times 0.0647)}{0.02}} = 0.2561$$

$$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3600 n_1)}} = \sqrt{2 \ln(3600 \times 0.656)} + \frac{0.577}{\sqrt{2 \ln(3600 \times 0.656)}} = 4.09$$

$$g_v = g_Q = 3.4 \text{ (for flexible buildings)}$$

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{60+40}{121.51}\right)^{0.63}}} = 0.8086$$

$$G_f = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z} \right) = 0.925 \left(\frac{1 + 1.7 \times 0.2688 \sqrt{3.4^2 \times 0.8086^2 + 4.09^2 \times 0.2561^2}}{1 + 1.7 \times 4.09 \times 0.2688} \right) = 1.41$$

The same procedure was used for the case when wind forces act in x-direction.

Table 3.5.4. Summary of Gust effect calculations

Case 1: Wind in Y-direction		Case 2: Wind in X-direction	
B (m)	60	B (m)	40
L (m)	40	L (m)	60
Q	0.809	Q	0.829
$\eta (R_B)$	6.691	$\eta (R_B)$	4.461
$\eta (R_L)$	14.933	$\eta (R_L)$	22.400
R_B	0.138	R_B	0.199
R_L	0.065	R_L	0.044
R	0.256	R	0.305
G_f	1.412	G_f	1.435

External pressure coefficients for both cases were either taken directly or interpolated using the values from Figure 27.4-1 of ASCE 7-10 Chapter 27.

Table 3.5.5. External pressure coefficients

	Y-direction	X-direction
C_p^w	0.8000	0.8000
C_p^l	-0.5000	-0.4000

Using the values calculated above, the design wind pressure was calculated using these formulas:

$$p_w = q_w G_f C_p^w$$

$$p_l = q_l G_f C_p^l$$

$$p_{net} = |p_w| + |p_l| \geq 0.77 \text{ kPa}$$

Table 3.5.6. Design wind pressure at each floor

Floor #	Y-direction			X-direction		
	p_w (kPa)	p_l (kPa)	p_{net} (kPa)	p_w (kPa)	p_l (kPa)	p_{net} (kPa)
1	0.8798	-0.9501	1.8299	0.8943	-0.7726	1.6669
2	1.0676	-0.9501	2.0177	1.0852	-0.7726	1.8578
3	1.1987	-0.9501	2.1488	1.2185	-0.7726	1.9911
4	1.2804	-0.9501	2.2304	1.3015	-0.7726	2.0740
5	1.3507	-0.9501	2.3008	1.3730	-0.7726	2.1455
6	1.4129	-0.9501	2.3630	1.4362	-0.7726	2.2088
7	1.4690	-0.9501	2.4190	1.4932	-0.7726	2.2657
8	1.5201	-0.9501	2.4702	1.5452	-0.7726	2.3177

Using the height of each floor and net wind pressure, design wind forces were calculated for each floor as:

$$F_i = (p_i \frac{h_i}{2} + p_{i+1} \frac{h_{i+1}}{2})B$$

$$F_8 = p_{net8} (h_p + \frac{h_8}{2})B$$

Table 3.5.7. Design wind forces

Wind force, F (kN)		
Floor #	Y-direction	X-direction
1	519.41	317.22
2	562.48	346.40
3	524.29	324.38
4	475.78	295.37
5	489.70	304.80
6	502.11	313.22
7	513.37	320.84
8	439.89	275.16

3.5.1. Wind loading cases

Now using the calculated wind forces acting at each floor, four wind loading cases were considered and forces applied to each frame at each floor were computed.

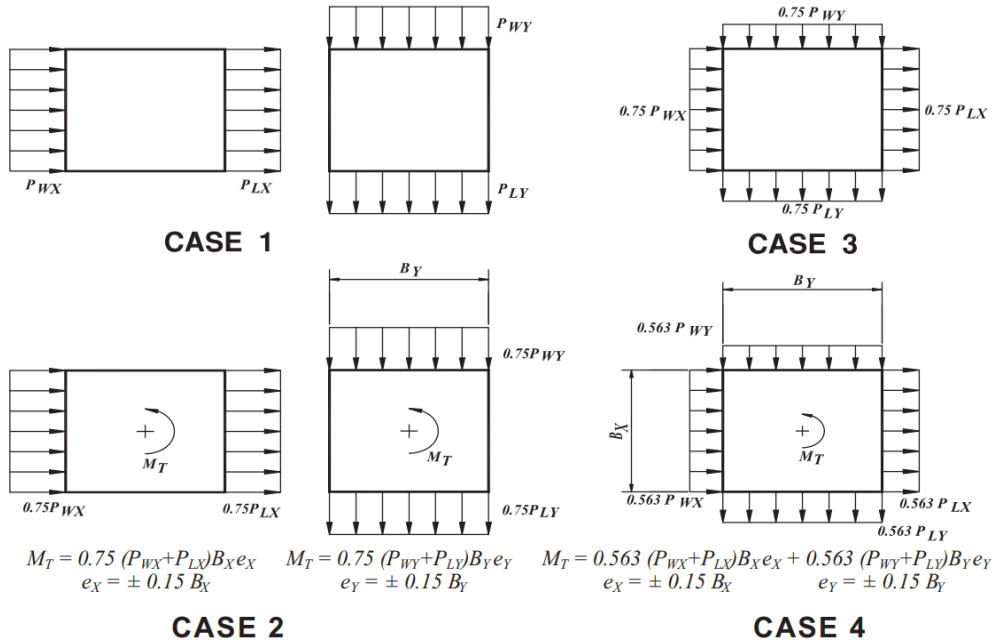


Figure 41. Design wind load cases (ASCE 7-10)

The direct wind force is distributed based on each frame's stiffness for rigid structures:

$$F_{direct} = F \frac{C_{fi}}{\sum C_{fi}} \quad (A)$$

The stiffness of the frame on the given floor is calculated as:

$$C_f = h \times \sum D$$

Stiffness for exterior and interior columns on the floor were evaluated using:

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

Table 3.5.1.1. Cracked moments of inertia of columns

Floor	Exterior column $I_{c,cr} \text{ (m}^4\text{)}$	Interior column $I_{c,cr} \text{ (m}^4\text{)}$
1-3	0.0076	0.0104
4-6	0.0024	0.0036

7-8	0.0005	0.0005
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Table 3.5.1.2. Cracked moments of inertia of beams

$I_{b,cr} (m^4)$	
20-meter major beam $L = 20 m, h = 0.75 m, b = 0.35 m$	5-meter major beam $L = 5 m, h = 0.45 m, b = 0.25 m$
For longitudinal frames	For transverse frames
0.00350	0.00066

Table 3.5.1.3. Lateral stiffness of the column-beam joint at each floor

Longitudinal frames		
Floor	D for exterior column (kN/m)	D for interior column (kN/m)
1-3	2792.9	2867.0
4-6	4058.3	2640.2
7-8	2219.5	1342.7
Transverse frames		
Floor	D for exterior column (kN/m)	D for interior column (kN/m)
1-3	573.9	576.9
4-6	922.6	937.5
7-8	776.3	776.3

Table 3.5.1.4. Stiffness of each frame at each floor (longitudinal frames)

$C_f (kN/rad)$			
Frame\Floor	1-3	4-6	7-8
1	37703.8	28408.2	15536.7

2	38371.0	28408.2	15536.7
3	38371.0	42612.3	23305.1
4	38371.0	32685.4	17167.1
5	38371.0	32685.4	17167.1
6	38371.0	32685.4	17167.1
7	38371.0	42612.3	23305.1
8	38371.0	28408.2	15536.7
9	37703.8	28408.2	15536.7
Total	344004.4	296913.3	160258.6

Table 3.5.1.5. Stiffness of each frame at each floor (transverse)

C_f (kN/rad)			
Frame\Floor	1-3	4-6	7-8
A	20659.3	25831.6	21737.8
E	20755.9	25988.5	21737.8
I	20755.9	25988.5	21737.8
M	20659.3	25831.6	21737.8
Total	82830.5	103640.2	86951.0

Now the wind forces for Case I (full wind pressure acting at each side considered separately) and Case III (75% of wind pressure acting at each side simultaneously) can be calculated using equation 3.24 and equations on Figure 3.1.1. The results are presented below.

Example calculation for wind force acting at frame A on the first floor:

$$\text{Case I: } F_{direct} = F_1 \frac{C_{fA}}{\sum C_{fi}} = 519.418 \times \frac{20659.3}{82830.5} = 129.55 \text{ kN}$$

$$\text{Case III: } F_{direct} = 0.75 F_1 \frac{C_{fA}}{\sum C_{fi}} = 0.75 \times 129.55 = 97.16 \text{ kN}$$

Table 3.5.1.6. Case I, wind forces in y-direction

Frame/Floor	1	2	3	4	5	6	7	8
A	129.55	140.29	130.77	118.58	122.05	125.15	128.34	109.97
B	130.16	140.95	131.38	119.30	122.79	125.91	128.34	109.97
C	130.16	140.95	131.38	119.30	122.79	125.91	128.34	109.97
D	129.55	140.29	130.77	118.58	122.05	125.15	128.34	109.97

Table 3.5.1.7. Case I, wind forces in x-direction

Frame/Floor	1	2	3	4	5	6	7	8
1	34.77	37.97	35.55	28.26	29.16	29.97	31.11	26.68
2	35.38	38.64	36.18	28.26	29.16	29.97	31.11	26.68
3	35.38	38.64	36.18	42.39	43.74	44.95	46.66	40.01
4	35.38	38.64	36.18	32.52	33.55	34.48	34.37	29.48
5	35.38	38.64	36.18	32.52	33.55	34.48	34.37	29.48
6	35.38	38.64	36.18	32.52	33.55	34.48	34.37	29.48
7	35.38	38.64	36.18	42.39	43.74	44.95	46.66	40.01
8	35.38	38.64	36.18	28.26	29.16	29.97	31.11	26.68
9	34.77	37.97	35.55	28.26	29.16	29.97	31.11	26.68

Table 3.5.1.8. Case III, wind forces in y-direction

Frame/Floor	1	2	3	4	5	6	7	8
A	97.16	105.22	98.07	88.94	91.54	93.86	96.26	82.48
B	97.62	105.71	98.53	89.48	92.10	94.43	96.26	82.48
C	97.62	105.71	98.53	89.48	92.10	94.43	96.26	82.48
D	97.16	105.22	98.07	88.94	91.54	93.86	96.26	82.48

Table 3.5.1.9. Case III, wind forces in x-direction

Frame/Floor	1	2	3	4	5	6	7	8
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r								
1	26.08	28.47	26.66	21.20	21.87	22.48	23.33	20.01
2	26.54	28.98	27.14	21.20	21.87	22.48	23.33	20.01
3	26.54	28.98	27.14	31.79	32.81	33.71	34.99	30.01
4	26.54	28.98	27.14	24.39	25.17	25.86	25.78	22.11
5	26.54	28.98	27.14	24.39	25.17	25.86	25.78	22.11
6	26.54	28.98	27.14	24.39	25.17	25.86	25.78	22.11
7	26.54	28.98	27.14	31.79	32.81	33.71	34.99	30.01
8	26.54	28.98	27.14	21.20	21.87	22.48	23.33	20.01
9	26.08	28.47	26.66	21.20	21.87	22.48	23.33	20.01

For cases II (75% of design wind pressure in combination with torsional moment acting at each side considered separately) and IV (56.3% of design wind pressure in combination with torsional moment acting at both sides simultaneously), the calculation of torsional moment and forces due to it were based on the equations from Figure 3.1.1 and formulas below.

Torsional force on the transverse and longitudinal frames respectively:

$$F_{torsion} = T \frac{x_i C_{fi}}{\sum x_i^2 C_{fi} + \sum y_i^2 C_{fi}}$$

$$F_{torsion} = T \frac{y_i C_{fi}}{\sum x_i^2 C_{fi} + \sum y_i^2 C_{fi}}$$

Total force applied to the frame:

$$F_{total} = F_{direct} + F_{torsion}$$

Eccentricity is equal to 15% of building dimension perpendicular to the force:

$$e_x = 0.15 \times 40 = 6 \text{ m}$$

$$e_y = 0.15 \times 60 = 9 \text{ m}$$

Example calculation of torsional moment on the first floor:

$$\text{Case II: } M_{Ty} = 0.75(p_{net})B_y e_y = 0.75 \times 1.83 \times 60 \times 9 = 741.09 \text{ kN-m}$$

$$M_{Tx} = 0.75(p_{net})B_x e_x = 0.75 \times 1.667 \times 40 \times 6 = 300.03 \text{ kN-m}$$

$$\text{Case IV: } M_T = 0.563(p_{net,y} B_y e_y + p_{net,x} B_x e_x)$$

$$= 0.563 \times (1.83 \times 60 \times 9 + 1.667 \times 40 \times 6) = 781.54 \text{ kN-m}$$

Table 3.5.1.10. Torsional moments for Case II and Case IV

Floor	Case II		Case IV
	M_{Ty} (kN-m) (y-direction)	M_{Tx} (kN-m) (x-direction)	M_T (kN-m) (both direction)
1	741.09	300.03	781.54
2	817.16	334.40	864.44
3	870.27	358.39	922.32
4	903.32	373.33	958.34
5	931.82	386.20	989.39
6	957.01	397.58	1016.85
7	979.71	407.83	1041.58
8	1000.43	417.19	1064.16

Table 3.5.1.11. Intermediate calculations for torsional force (Floor 1-3)

Frame #	Longitudinal frames			Frame #	Transverse frames		
	y_i (m)	C_{fi} (kN/rad)	$y_i^2 C_{fi}$ (kN · m ²)		x_i (m)	C_{fi} (kN/rad)	$x_i^2 C_{fi}$ (kN · m ²)
1	-20	37704	15081519.35	A	-30	20659	18593404.08
2	-15	38371	8633469.025	E	-10	20756	2075589.168
3	-10	38371	3837097.345	I	10	20756	2075589.168
4	-5	38371	959274.3362	M	30	20659	18593404.08
5	0	38371	0	sum			41337986.49
6	5	38371	959274.3362				
7	10	38371	3837097.345				
8	15	38371	8633469.025				
9	20	37704	15081519.35				

sum	57022720.12
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Table 3.5.1.12. Intermediate calculations for torsional force (Floor 4-6)

Frame #	Longitudinal frames			Frame #	Transverse frames		
	y_i (m)	C_{fi} (kN/rad)	$y_i^2 C_{fi}$ (kN · m ²)		x_i (m)	C_{fi} (kN/rad)	$x_i^2 C_{fi}$ (kN · m ²)
1	-20	28408.2	11363267.9	A	-30	25831.6	23248420.3
2	-15	28408.2	6391838.2	E	-10	25988.5	2598851.1
3	-10	42612.3	4261225.5	I	10	25988.5	2598851.1
4	-5	32685.4	817134.3	M	30	25831.6	23248420.3
5	0	32685.4	0.0	sum			51694543.0
6	5	32685.4	817134.3				
7	10	42612.3	4261225.5				
8	15	28408.2	6391838.2				
9	20	28408.2	11363267.9				
Frame #	Longitudinal frames						

Table 3.5.1.13. Intermediate calculations for torsional force (Floor 7-8)

Frame #	Longitudinal frames			Frame #	Transverse frames		
	y_i (m)	C_{fi} (kN/rad)	$y_i^2 C_{fi}$ (kN · m ²)		x_i (m)	C_{fi} (kN/rad)	$x_i^2 C_{fi}$ (kN · m ²)
1	-20	15536.7	6214696.8	A	-30	21737.8	19563978.9
2	-15	15536.7	3495766.9	E	-10	21737.8	2173775.4
3	-10	23305.1	2330511.3	I	10	21737.8	2173775.4
4	-5	17167.1	429178.5	M	30	21737.8	19563978.9
5	0	17167.1	0.0	sum			43475508.8

6	5	17167.1	429178.5
7	10	23305.1	2330511.3
8	15	15536.7	3495766.9
9	20	15536.7	6214696.8
sum			24940307.0

Example calculation of wind forces acting at frame 1 on the 3rd floor for Case II:

$$F_{direct} = 0.75 \times 324.38 \times \frac{37704}{344004.4} = 26.66 \text{ kN}$$

$$F_{torsion} = 358.39 \times \frac{-20 \times 37704}{57022720.12 + 41337986.49} = -2.75 \text{ kN}$$

$$F_{total} = F_{direct} + F_{torsion} = 26.66 - 2.75 = 23.92$$

Table 3.5.1.14. Example calculations of wind forces on each frame at 3rd floor (case II)

Frame #	Wind in X direction			Frame #	Wind in Y direction		
	F_{direct} (kN)	$F_{torsion}$ (kN)	F_{total} (kN)		F_{direct} (kN)	$F_{torsion}$ (kN)	F_{total} (kN)
1	26.66	-2.75	23.92	A	98.07	-5.48	92.59
2	27.14	-2.10	25.04	E	98.53	-1.84	96.70
3	27.14	-1.40	25.74	I	98.53	1.84	100.37
4	27.14	-0.70	26.44	M	98.07	5.48	103.56
5	27.14	0.00	27.14				
6	27.14	0.70	27.84				
7	27.14	1.40	28.53				
8	27.14	2.10	29.23				
9	26.66	2.75	29.41				

Table 3.5.1.15. Case II, resultant wind forces in y-direction

Frame/Floor	1	2	3	4	5	6	7	8
A	92.49	100.07	92.59	81.75	84.12	86.24	86.92	72.94
E	96.05	103.99	96.70	87.07	89.61	91.88	93.14	79.30

I	99.18	107.43	100.37	91.89	94.58	96.99	99.37	85.66
M	101.83	110.37	103.56	96.13	98.96	101.48	105.60	92.02

Table 3.5.1.16. Case II, resultant wind forces in x-direction

Frame/Floor	1	2	3	4	5	6	7	8
1	23.78	25.91	23.92	19.02	19.62	20.16	21.48	18.11
2	24.78	27.02	25.04	19.56	20.18	20.74	21.94	18.59
3	25.37	27.67	25.74	30.16	31.12	31.97	33.60	28.59
4	25.95	28.33	26.44	23.76	24.52	25.19	25.27	21.58
5	26.54	28.98	27.14	24.39	25.17	25.86	25.78	22.11
6	27.12	29.63	27.84	25.01	25.81	26.53	26.29	22.63
7	27.71	30.28	28.53	33.43	34.50	35.45	36.38	31.43
8	28.29	30.94	29.23	22.83	23.56	24.22	24.72	21.43
9	28.38	31.04	29.41	23.37	24.13	24.80	25.18	21.90

Table 3.5.1.17. Case IV, resultant wind forces in y-direction

Frame/Floor	1	2	3	4	5	6	7	8
A	68.01	73.54	67.81	59.13	60.84	62.36	62.33	51.77
E	71.63	77.53	72.02	64.61	66.49	15.59	68.95	58.53
I	74.93	81.18	75.91	69.73	71.77	21.02	75.57	65.30
M	77.86	84.43	79.43	74.39	76.59	31.96	82.18	72.06

Table 3.5.1.18. Case IV, resultant wind forces in x-direction

Frame/Floor	1	2	3	4	5	6	7	8
1	13.58	14.75	12.95	10.32	10.65	10.94	12.78	10.19
2	15.35	16.69	14.97	11.72	12.09	12.42	13.96	11.39
3	16.87	18.38	16.77	19.67	20.30	20.86	22.72	18.90

4	18.40	20.07	18.57	16.70	17.23	17.71	18.04	15.26
5	19.92	21.75	20.37	18.31	18.89	19.41	19.35	16.59
6	21.45	23.44	22.17	19.91	20.55	21.12	20.66	17.93
7	22.97	25.13	23.97	28.06	28.96	29.76	29.82	26.15
8	24.49	26.81	25.77	20.11	20.75	21.32	21.06	18.64
9	25.57	28.00	27.09	21.50	22.19	22.81	22.24	19.85

3.6. Seismic loads

From a geotechnical investigation report, it was identified that the soil class is D (stiff soil). The mapped acceleration parameters S_s and S_1 were taken from the ASCE Hazard Tool (2025) and site coefficients F_a and F_v were taken from Tables 11.4-1 and 11.4-2 of ACSE 7-10.

Table 3.6.1. Seismic calculation parameters

Site class	S_s	S_1	T_L	F_a	F_v
D	0.081	0.03	12	1.6	2.4

Adjusted spectral response acceleration parameters are calculated as follows:

$$S_{MS} = F_a S_s = 1.6 \times 0.081 = 0.1296$$

$$S_{M1} = F_v S_1 = 2.4 \times 0.03 = 0.072$$

Design acceleration parameters are calculated as follows:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.1296 = 0.0864$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.072 = 0.048$$

According to Table 11.6-1 and 11.6-2 of ACSE 7-10, since $S_{D1} < 0.067$ and $S_{DS} < 0.167$, the structure belongs to Seismic Design Category A.

As stated in Section 11.7 of ASCE 7-10, buildings that belong to Seismic Design Category A need to comply with the requirements of section 1.4 only, meaning that seismic design and detailing are not compulsory. The seismic loads were still calculated to confirm that they are not governing forces and to practice the seismic analysis procedure.

For the calculation of seismic loads, Equivalent Lateral Force (ELF) method was used since the building is located in a very low-seismicity region and belongs to seismic design category A as it will be proven below. According to Table 12.6-1 of ASCE 7-10 “Permitted Analytical Procedures”, it is permitted to use ELF method for all structures not belonging to categories D, E, F.

For the ordinary reinforced concrete frames, the design coefficients are detailed in the table below.

Table 3.6.2. Seismic design parameters

LFRS	Overstrength factor, Ω_0	Design amplification factor, C_d	Response modification coefficient, R	Seismic importance factor, I_e
Ordinary RC MF	3	2.5	3	1

For structures having number of stories $N < 12$ and the average story height of at least 3 m, the approximate fundamental period of the building is permitted to be evaluated as:

$$T_a = 0.1N = 0.1 \times 8 = 0.8 \text{ s}$$

For $S_{D1} = 0.048 < 0.1$, the coefficient for upper limit on calculated period

$C_u = 1.7$. Using it, the fundamental period is calculated as:

$$T = C_u \times T_a = 0.8 \times 1.7 = 1.36 \text{ s}$$

To determine T^* , modal analysis was performed using software (SAP2000).

Obtained value is $T^* = 3.94 \text{ s}$, which is bigger than the calculated $T = 1.36 \text{ s}$, so the latter will be used for the further.

The seismic response coefficient calculation is shown below:

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}} = \frac{0.0864}{3/1} = 0.0288$$

Since $T < T_L$,

$$C_{s,max} = \frac{S_{D1}}{T \times \frac{R}{I_e}} = \frac{0.048}{1.36 \times 3/1} = 0.012$$

For $S_1 < 0.6 g$,

$$C_{s,min} = \max[0.044S_{DS} \times I_e; 0.01] = \max[0.044 \times 0.0864 \times 1; 0.01] = 0.01$$

So the final value for seismic response coefficient is:

$$C_s = \max[\min[0.0288, 0.012], 0.01] = 0.012$$

To calculate base shear, the weight of the building W was estimated first.

Table 3.6.3. Beam weight contribution to seismic weight

Floor	Weight of 20-meter major beam (kN)	Weight of 5-meter major beam (kN)	Weight of minor beam (kN)	Number of 20-meter major beam	Number of 5-meter major beam	Number of minor beam	Total beam weight (kN)
1	122.500	14.06	7.66	27	32	72	4309
2	122.500	14.06	7.66	27	32	72	4309
3	122.500	14.06	7.66	27	32	72	4309
4	122.500	14.06	7.66	23	32	60	3727
5	122.500	14.06	7.66	23	32	60	3727
6	122.500	14.06	7.66	23	32	60	3727
7	122.500	14.06	7.66	23	32	60	3727
8	122.500	14.06	7.66	23	32	60	3727

Table 3.6.4. Column weight contribution to seismic weight

Floor	Weight of interior column (kN)	Weight of exterior column (kN)	Number of exterior column	Number of interior column	Total column weight (kN)
1	47.53	40.50	22	14	1556
2	47.53	40.50	22	14	1556
3	47.53	40.50	22	14	1556
4	21.88	17.72	30	6	663
5	21.88	17.72	30	6	663
6	21.88	17.72	30	6	663

7	7.88	7.88	30	6	284
8	7.88	7.88	30	6	284

Table 3.6.5. Effective seismic weight

Floor	Columns' weight (kN)	Non-structural materials & slab weight (kN)	Beams' weight (kN)	Partition wall (kN)	Exterior walls (kN)	Stairs (kN)	W_x (kN)
1	1556	12764.7	4309	1962.35	2828.3	234.96	23655
2	1556	10980.1	4309	1905.68	2828.3	234.96	21814
3	1556	11435.9	4309	2433.51	2828.3	234.96	22798
4	663	9139.8	3727	1965.41	2639.7	234.96	18370
5	663	9467.4	3727	1965.41	2639.7	234.96	18697
6	663	9467.4	3727	1965.41	2639.7	234.96	18697
7	284	9467.4	3727	1965.41	2639.7	234.96	18318
8	284	9467.4	3727	1965.41	2639.7	234.96	18318
Roof	0	7644.0	0	0.00	307.3	0	7951.29
Total							168618

Next, the seismic design base shear was calculated as:

$$V = C_s W = 0.012 \times 168618 = 1984 \text{ kN}$$

The force for each story was calculated using base shear and vertical distribution factor C_{vx} , weight contribution of the given floor W_x (kN) and its elevation above ground h_x (m):

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{W_x h_x^k}{\sum W_i h_i^k}$$

where k is an is interpolated as:

$$k = 1 + \frac{2-1}{2.5-0.5} \times (1.36 - 0.5) = 1.43$$

Table 3.6.6. Seismic force at each floor level

Floor	h_i (m)	W_x (kN)	$W_x h_x^k$	C_{vx}	F_x (kN)
1	4.5	23655	8089754.7	0.041	81.5
2	9	21814	14409155.4	0.073	145.2
3	13.5	22798	23020831.3	0.117	231.9
4	17	18370	21286866.4	0.108	214.5
5	20.5	18697	26326527.0	0.134	265.2
6	24	18697	30821300.0	0.157	310.5
7	27.5	18318	34296017.5	0.174	345.5
8	31	18318	38660965.1	0.196	389.5

3.6.1. Seismic loads including torsional effect

Since the building is fully symmetrical in both axes, only the accidental torsion of 5% has been considered.

$$e_{ax} = 0.05 \times 40 = 2 \text{ m}$$

$$e_{ay} = 0.05 \times 60 = 3 \text{ m}$$

The torsional moment was calculated using:

$$M_T = Fe$$

The forces due to torsion and the resultant force calculations were done using the formulas used for the wind force distribution calculations. The results are presented below.

Table 3.6.1.1. Example of calculations of seismic forces on the 1st floor

Frame #	Force in X direction			Frame #	Force in Y direction		
	F_{direct} (kN)	$F_{torsion}$ (kN)	F_{total} (kN)		F_{direct} (kN)	$F_{torsion}$ (kN)	F_{total} (kN)
1	8.93	-1.25	7.68	A	20.33	-1.54	18.79
2	9.09	-0.95	8.14	E	20.42	-0.52	19.91
3	9.09	-0.64	8.45	I	20.42	0.52	20.94

4	9.09	-0.32	8.77	M	20.33	1.54	21.87
5	9.09	0.00	9.09	sum			81.50
6	9.09	0.32	9.41				
7	9.09	0.64	9.73				
8	9.09	0.95	10.04				
9	8.93	1.25	10.18				
sum			81.50				

Table 3.6.1.2. Seismic forces acting in y-direction

Frame/Floor	1	2	3	4	5	6	7	8
A	18.79	24.41	39.00	34.97	43.25	50.63	54.90	61.89
E	19.91	26.36	42.12	38.61	47.76	55.91	61.49	69.32
I	20.94	28.20	45.05	42.05	52.00	60.88	68.08	76.74
M	21.87	29.90	47.77	45.21	55.91	65.46	74.66	84.17

Table 3.6.1.3. Seismic forces acting in x-direction

Frame/Floor	1	2	3	4	5	6	7	8
1	7.68	13.68	21.8630	18.02	22.28	26.08	30.36	34.22
2	8.14	14.49	23.1545	18.64	23.05	26.99	31.14	35.11
3	8.45	15.06	24.0593	28.90	35.74	41.84	47.89	53.99
4	8.77	15.63	24.9640	22.89	28.31	33.14	36.14	40.74
5	9.09	16.19	25.8687	23.61	29.20	34.18	37.01	41.72
6	9.41	16.76	26.7735	24.33	30.09	35.22	37.88	42.70
7	9.73	17.32	27.6782	32.65	40.39	47.28	52.60	59.29
8	10.04	17.89	28.5829	22.40	27.70	32.43	35.85	40.41
9	10.18	18.14	28.9749	23.02	28.47	33.33	36.63	41.30

3.7. Analytical model development

The following sections detail the definition and development of a 3D model of the structural system of Altyn Fiesta that will be used extensively in the next phase of the project.

3.7.1. Model geometry

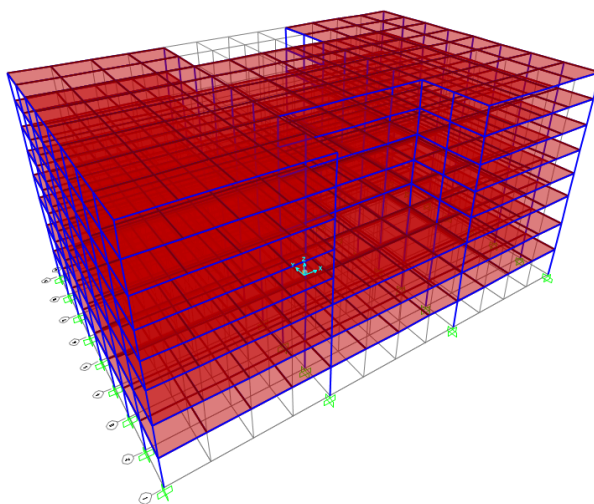


Figure 3.7.1.1. Model geometry

3.7.2. Material properties

The properties of the materials used in SAP2000 are the same as the ones used for calculations. They are listed in the table below for concrete and shown in Figure 32.1 for reinforcement.

Table 3.7.2.1. Properties of reinforced concrete

Structural member	f'_c (MPa)	ρ (kN/m ³)	E_c (MPa)
Beams, slabs	30	25	25742.96
Columns	40	25	29725.4
Pre-stressed beam	34.47	25	27594.2

Material Property Data

General Data

Material Name and Display Color: Steel_Gr60 ■

Material Type: Rebar

Material Notes: [Modify/Show Notes...](#)

Weight and Mass

Weight per Unit Volume: 76.9729

Mass per Unit Volume: 7.849

Units

KN, m, C

Uniaxial Property Data

Modulus of Elasticity, E: 1.999E+08

Poisson: 0.3

Coefficient of Thermal Expansion, A: 1.170E-05

Shear Modulus, G: 76903069

Other Properties for Rebar Materials

Minimum Yield Stress, Fy: 420000.

Minimum Tensile Stress, Fu: 600000.

Expected Yield Stress, Fye: 420000.

Expected Tensile Stress, Fue: 600000.

☐ Switch To Advanced Property Display

[OK](#) [Cancel](#)

Figure 3.7.2.1. Steel properties

3.7.3. Frame sections

Slabs were modeled using area (thin shell) sections.

Shell Section Data

Section Name: Slab

Section Notes: [Modify/Show...](#)

Display Color: ■

Type

☒ Shell - Thin

☐ Shell - Thick

☐ Plate - Thin

☐ Plate Thick

☐ Membrane

☐ Shell - Layered/Nonlinear

[Modify/Show Layer Definition...](#)

Thickness

Membrane: 0.13

Bending: 0.13

Material

Material Name: + Concrete_1

Material Angle: 0.

Time Dependent Properties

[Set Time Dependent Properties...](#)

Concrete Shell Section Design Parameters

[Modify/Show Shell Design Parameters...](#)

Stiffness Modifiers

[Set Modifiers...](#)

Temp Dependent Properties

[Thermal Properties...](#)

[OK](#) [Cancel](#)

Figure 3.7.3.1. Slab section

Each column section was added as per calculated dimensions.

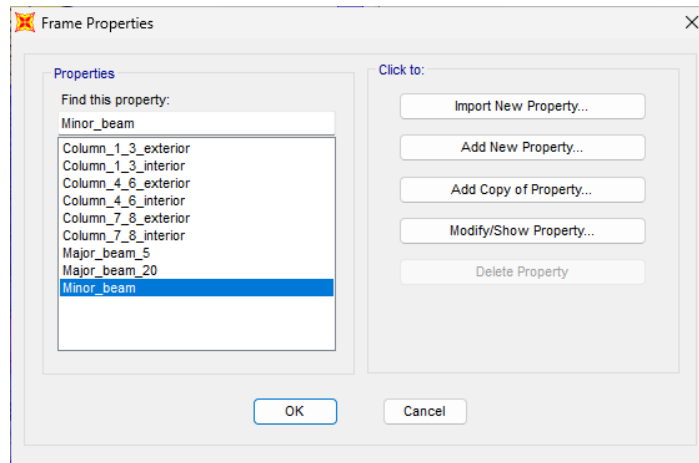


Figure 3.7.3.2. Frame sections

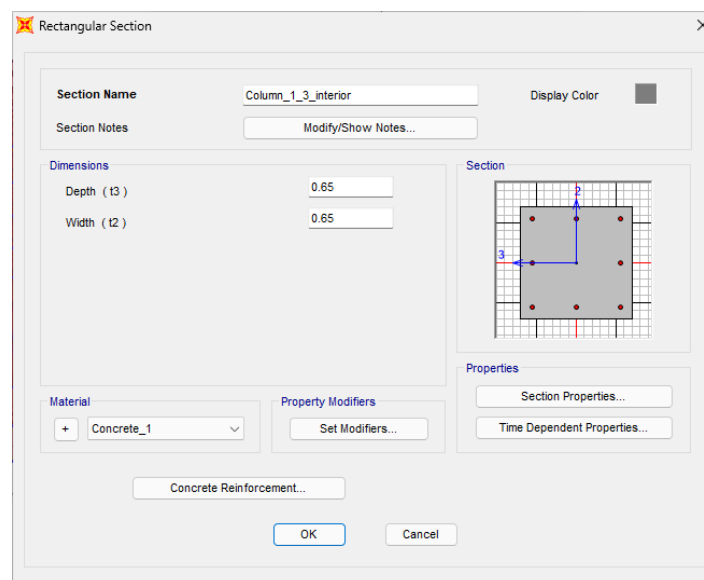


Figure 3.7.3.3. Example of column section definition

For each structural element, following modifiers of moment of inertia were applied:

- Columns - 0.7
- Beams - 0.35
- Slab - 0.25

3.7.4. Element type, connections and boundary conditions

Slabs were modeled with two-way configuration and all the joints were modeled as rigid. Columns and beams were modeled as frame elements. Diaphragm constraint was imposed for each floor so that each floor moves together rigidly when lateral loads are applied. Additionally, at the base, joints were fixed.

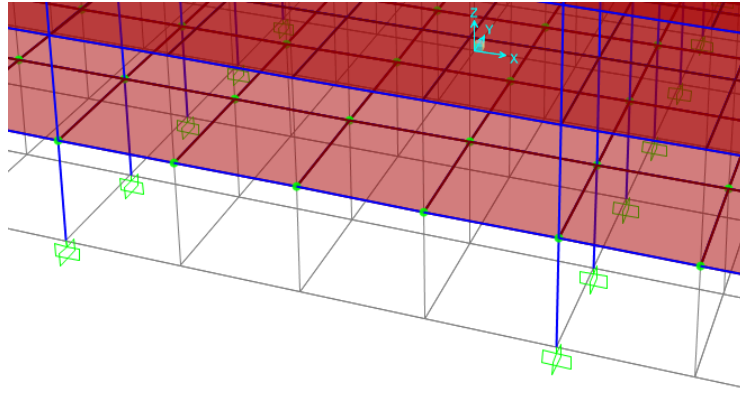


Figure 3.7.4.1. Boundary conditions at base and diaphragm constraints

3.7.5. Load assignment

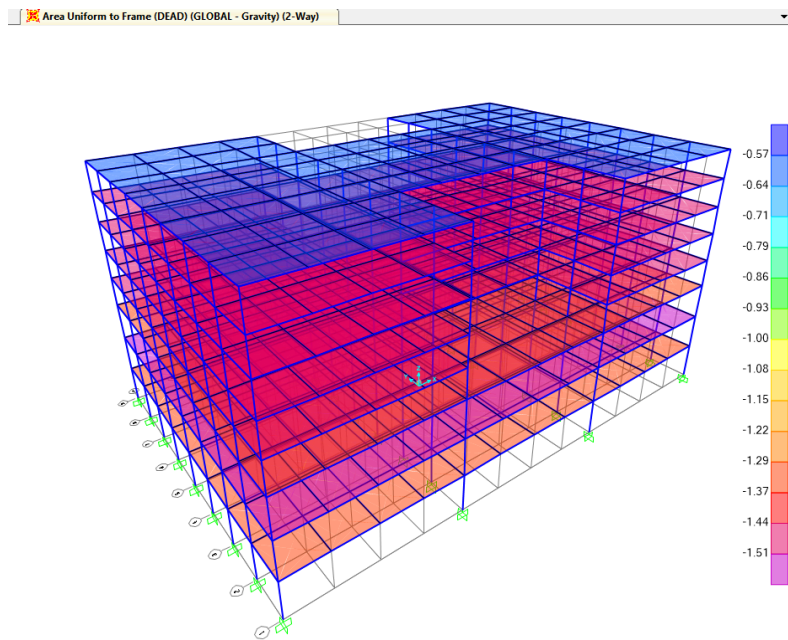


Figure 3.7.5.1. Assignment of dead load of non-structural materials

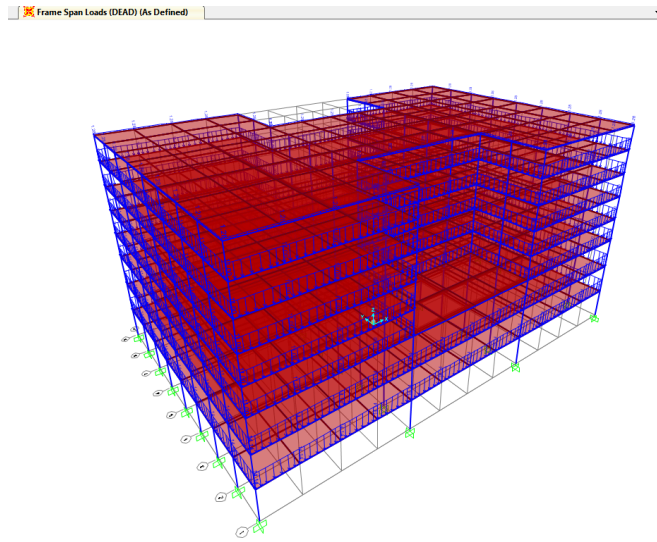


Figure 3.7.5.2. Dead load of exterior walls and parapetes applied to edge beams

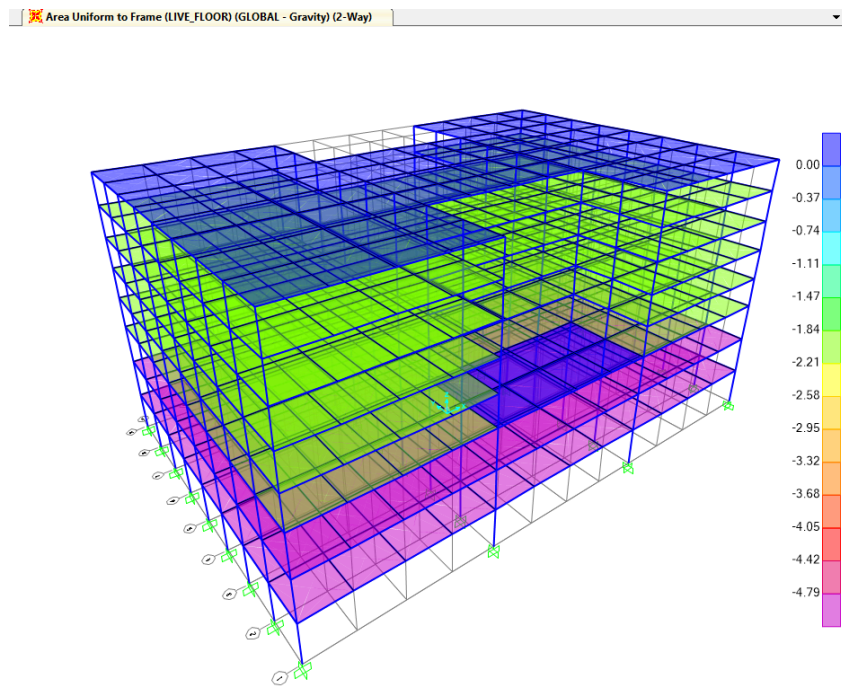


Figure 3.7.5.3. Floor live load assignment

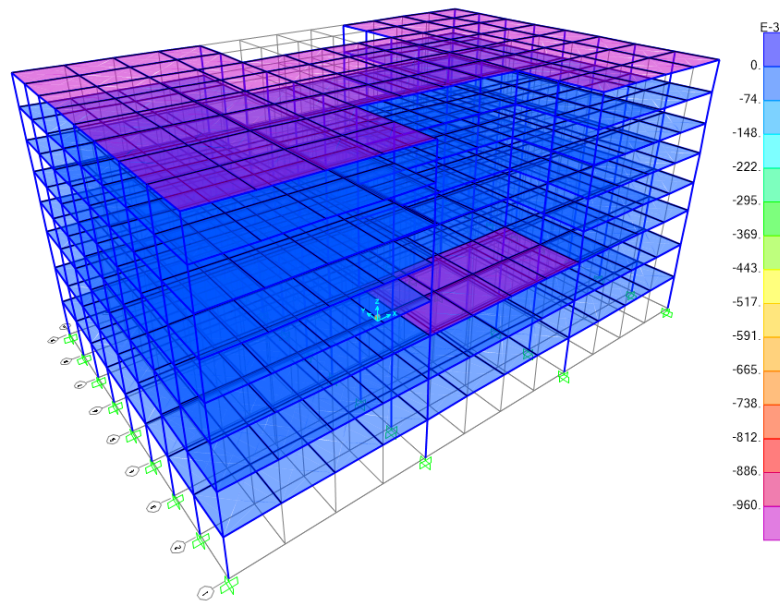


Figure 3.7.5.4. Roof live load assignment

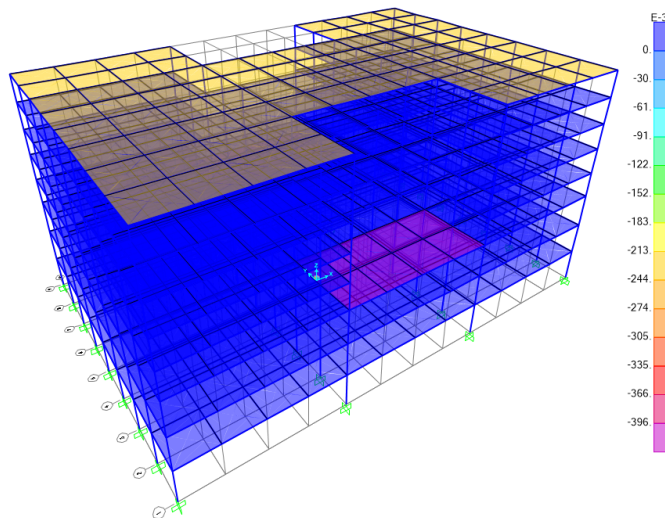


Figure 3.7.5.5. Snow load assignment

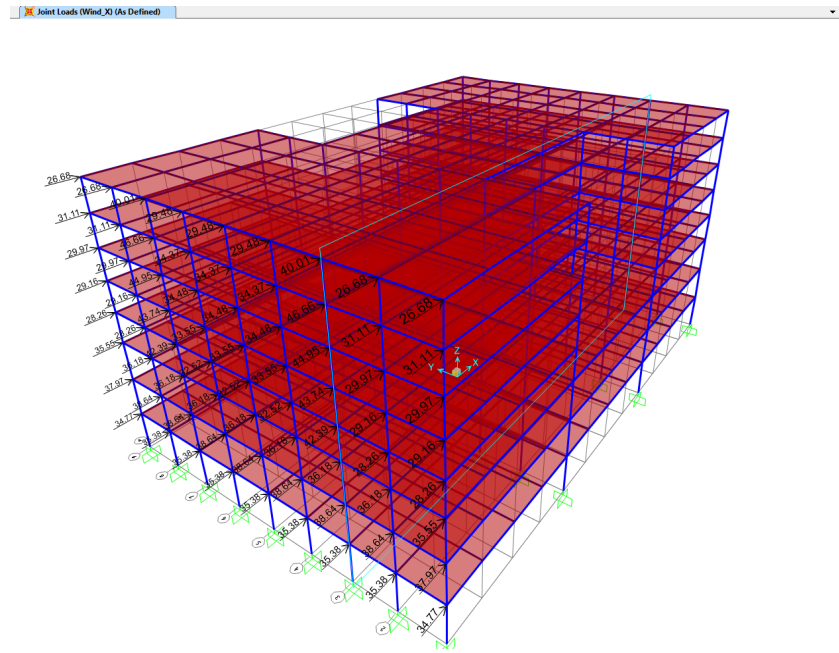


Figure 3.7.5.6. Assignment of wind forces in x-direction

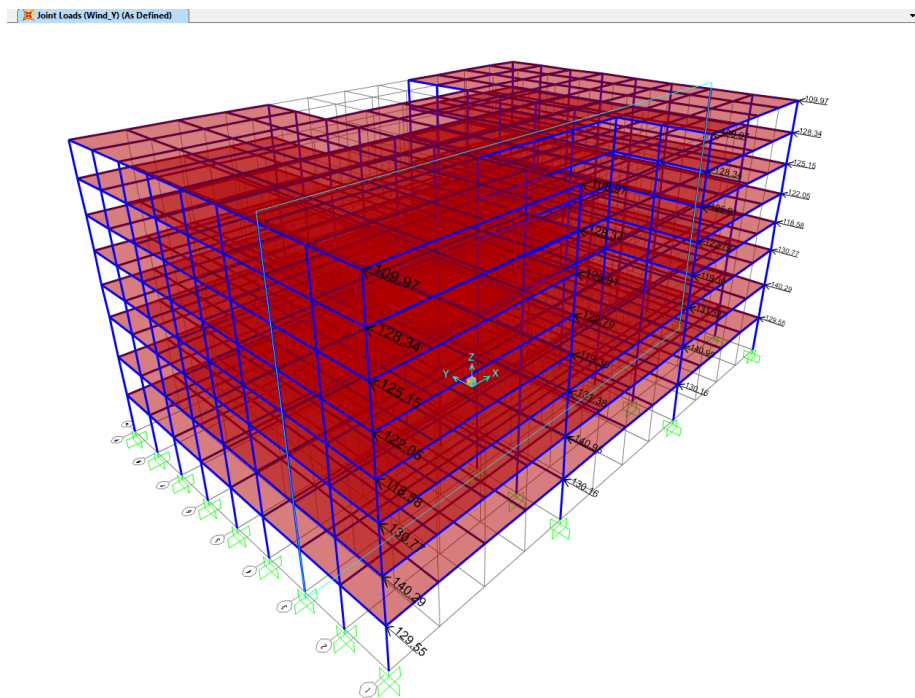


Figure 3.7.5.7. Assignment of wind forces in y-direction

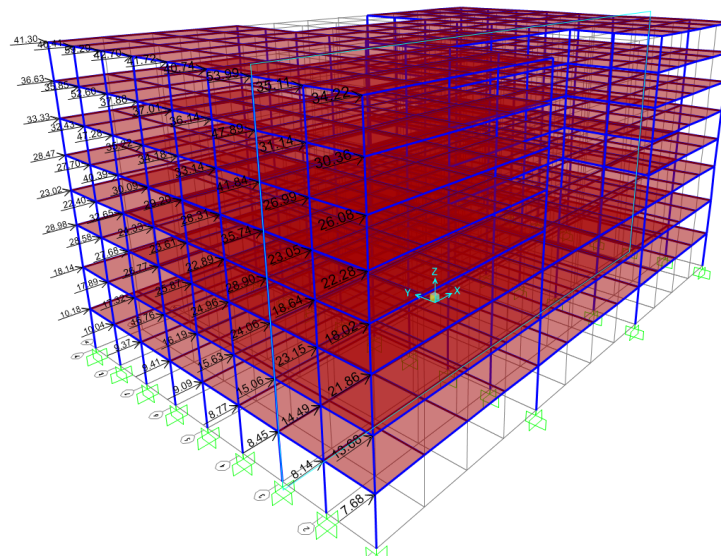


Figure 3.7.5.8. Assignment of seismic forces in x-direction

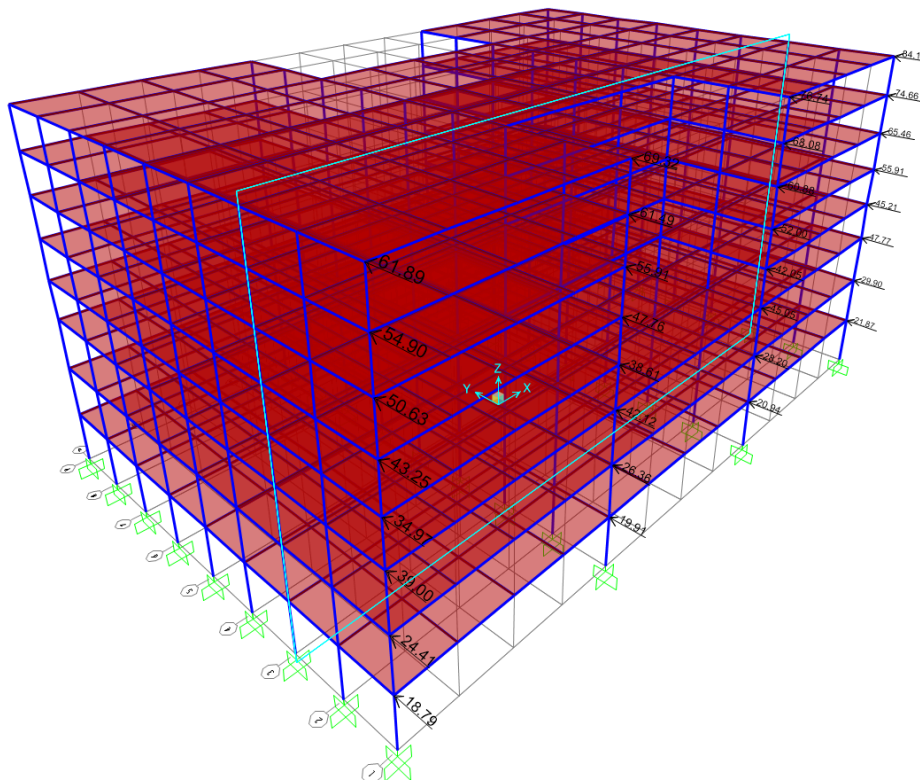


Figure 3.7.5.9. Assignment of seismic forces in y-direction

3.7.6. Load combinations

Table 3.7.6.1. Load combinations defined in SAP2000

Name	Combination
COMB1	$1.4D$
COMB2	$1.2D + 1.6L + 0.5L_r$
COMB3	$1.2D + 1.6L + 0.5S$
COMB4	$1.2D + 1.6L_r + 1L$
COMB5	$1.2D + 1.6L_r + 0.5W$
COMB6	$1.2D + 1.6S + 1L$
COMB7	$1.2D + 1.6S + 0.5W$
COMB8	$1.2D + 1W + 1L + 0.5L_r$
COMB9	$1.2D + 1W + 1L + 0.5S$
COMB10	$1.2D + 1E + 1L + 0.2S$
COMB11	$0.9D + 1W$
COMB12	$0.9D + 1E$

3.7.7. Modal analysis

Modal load case was defined using the mass source that includes dead weight of non-structural materials and self-weight of the structure. After running the analysis, T* value was obtained.

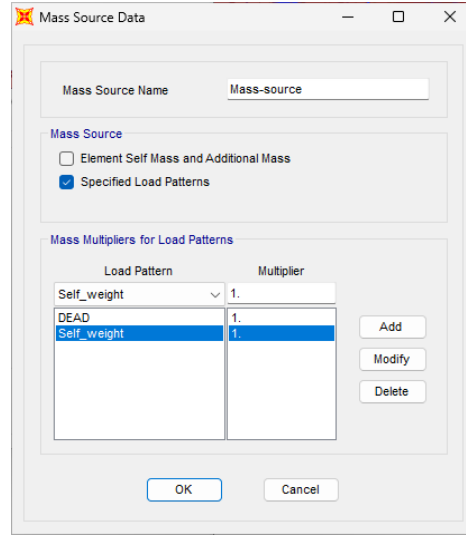


Figure 3.7.7.1. Mass source definition

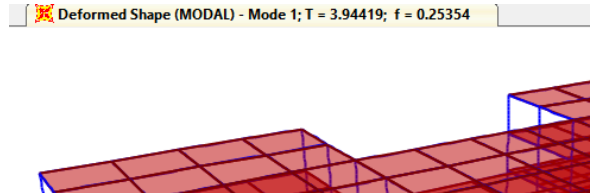


Figure 3.7.7.2. T^* result

3.8. Lateral drift analysis under wind and seismic loads

Lateral forces acting on the frames of choice E and 5 are taken from the previous section where wind and seismic forces were calculated.

Drifts for both longitudinal and transverse directions of the building were calculated using the following formulas for shear force:

$$V_i = F_i + V_{i+1}$$

$$V_{i-col} = \frac{V_i}{6} \times 2 = \frac{V_i}{3} \text{ (Frame 5)}$$

$$V_{i-col} = \frac{V_i}{16} \times 2 = \frac{V_i}{8} \text{ (Frame E)}$$

To calculate shear drift due to flexural deformation of beams and columns, the following formulas of interstory drift were used:

$$\delta_i^t = \delta_i^c + \delta_i^b = \frac{V_i h_i^3}{12EI_c} + \frac{V_i h_i^2 L}{12EI_b}$$

$$\delta_8^c = \frac{V_i h^3}{24EI_c} \text{ (for top floor)}$$

$$\delta_8^b = \frac{V_i h^2 L}{48EI_b} \text{ (for top floor)}$$

$$\delta_{inter,8} = \delta_8^t + \frac{\delta_7^t}{2} \text{ (for top floor)}$$

$$\delta_{inter,i} = \delta_i^t/2 + \delta_{i-1}^t/2$$

$$\delta_{inter,1} = \delta_1^t/2 + \delta_0^c = \delta_1^t/2 + \frac{V_i (h/2)^3}{3EI_c} \text{ (for ground floor)}$$

For both types of drift, the absolute drift is calculated as:

$$\delta_{abs,i} = \sum_i^{i=8} \delta_{inter,i}$$

Table 3.8.1. Shear drift calculations for frame E under wind loads

Story	h_i (mm)	h_{i-avg} (mm)	F_i (kN)	V_i (kN)	V_{i-col} (kN)	V_{avg} (kN)	δ_i^b (mm)	δ_i^c (mm)	δ_i^t (mm)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	3500	110.0	110.0	13.7	13.7	0.888	1.75	2.637	8.22	190.46
7	3500	3500	128.3	238.3	29.8	21.8	5.63	5.54	11.16	11.07	182.24
6	3500	3500	125.9	364.2	45.5	37.7	9.73	1.24	10.97	13.24	171.17
5	3500	3500	122.8	487.0	60.9	53.2	13.75	1.75	15.50	17.71	157.93
4	3500	3500	119.3	606.3	75.8	68.3	17.66	2.25	19.91	24.86	140.23
3	4500	4000	131.4	737.7	92.2	84.0	28.35	1.45	29.80	37.72	115.37
2	4500	4500	140.9	878.6	109.8	101.0	43.16	2.48	45.63	49.46	77.65
1	4500	4500	130.2	1008.8	126.1	118.0	50.39	2.89	53.29	28.19	28.19
0		2250						1.55	1.55	0	

Table 3.8.2. Shear drift calculations for frame 5 under wind loads

Story	h_i (mm)	h_{i-avg} (mm)	F_i (kN)	V_i (kN)	V_{i-col} (kN)	V_{avg} (kN)	δ_i^b (mm)	δ_i^c (mm)	δ_i^t (mm)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	3500	29.48	29.48	9.8	9.8	0.307	1.25	1.556	4.51	74.73
7	3500	3500	34.37	63.85	21.3	15.6	1.94	3.96	5.90	5.09	70.23
6	3500	3500	34.89	98.74	32.9	27.1	3.38	0.89	4.28	5.18	65.14

5	3500	3500	33.95	132.69	44.2	38.6	4.81	1.27	6.09	6.97	59.96
4	3500	3500	32.90	165.59	55.2	49.7	6.21	1.64	7.84	9.44	52.99
3	4500	4000	36.25	201.85	67.3	61.2	9.98	1.06	11.04	14.03	43.55
2	4500	4500	38.71	240.56	80.2	73.7	15.21	1.81	17.02	18.45	29.52
1	4500	4500	35.45	276.01	92.0	86.1	17.77	2.11	19.88	11.07	11.07
0		2250						1.13	1.13	0	

Table 3.8.3. Shear drift calculations for frame E under earthquake loads

Story	h_i (mm)	h_{i-avg} (mm)	F_i (kN)	V_i (kN)	V_{i-col} (kN)	V_{avg} (kN)	δ_i^b (mm)	δ_i^c (mm)	δ_i^t (mm)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	3500	78.04	78.04	9.8	9.8	0.630	1.24	1.871	5.48	93.19
7	3500	3500	69.23	147.28	18.4	14.1	3.64	3.58	7.22	6.87	87.71
6	3500	3500	62.97	210.25	26.3	22.3	5.77	0.74	6.51	7.57	80.85
5	3500	3500	53.79	264.04	33.0	29.6	7.66	0.98	8.64	9.52	73.27
4	3500	3500	43.62	307.66	38.5	35.7	9.23	1.18	10.41	12.55	63.75
3	4500	4000	47.24	354.90	44.4	41.4	13.98	0.71	14.69	17.79	51.20
2	4500	4500	29.78	384.67	48.1	46.2	19.75	1.13	20.88	21.61	33.41
1	4500	4500	22.16	406.83	50.9	49.5	21.13	1.21	22.35	11.80	11.80
0		2250						0.62	0.62	0	

Table 3.8.4. Shear drift calculations for frame 5 under earthquake loads

Story	h_i (mm)	h_{i-avg} (mm)	F_i (kN)	V_i (kN)	V_{i-col} (kN)	V_{avg} (kN)	δ_i^b (mm)	δ_i^c (mm)	δ_i^t (mm)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	3500	46.81	46.81	15.6	15.6	0.487	1.98	2.472	6.74	82.14
7	3500	3500	41.53	88.34	29.4	22.5	2.81	5.73	8.54	7.10	75.39
6	3500	3500	38.68	127.03	42.3	35.9	4.48	1.18	5.66	6.61	68.29
5	3500	3500	33.04	160.07	53.4	47.8	5.97	1.58	7.55	8.34	61.68

4	3500	3500	26.79	186.86	62.3	57.8	7.22	1.91	9.12	10.61	53.34
3	4500	4000	28.85	215.71	71.9	67.1	10.94	1.16	12.10	14.70	42.73
2	4500	4500	18.18	233.89	78.0	74.9	15.46	1.84	17.30	17.84	28.04
1	4500	4500	10.08	243.97	81.3	79.6	16.43	1.95	18.39	10.19	10.19
0		2250						1.00	1.00	0	

To calculate flexural drift due to axial deformation of columns, next formulas were used:

$$M_i = h_i V_i$$

$$M_i = M_{i+1} + h_i V_i$$

$$\phi_i = \frac{2M_i}{E(mL)^2 A_c}$$

$$\Delta\theta_i = \phi_i h_i$$

$$\theta_i = \sum_{i=8} \Delta\theta_i$$

$$\delta_{inter,i} = \theta_i h_i$$

Table 3.8.5. Flexural drift calculations for Frame E under wind loads

Story	h_i (mm)	F_i (kN)	V_i (kN)	M (N-mm)	A (mm^2)	ϕ_i	$\Delta\theta_i$ (rad)	θ_i (rad)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	110.0	110.0	3.85E+08	90000	1.80E-10	6.29E-07	1.01E-05	0.035	0.195
7	3500	128.3	238.3	8.34E+08	90000	3.90E-10	1.36E-06	9.49E-06	0.033	0.159
6	3500	125.9	364.2	1.27E+09	250000	2.14E-10	7.50E-07	8.13E-06	0.028	0.126
5	3500	122.8	487.0	1.70E+09	250000	2.87E-10	1.00E-06	7.38E-06	0.026	0.097
4	3500	119.3	606.3	2.12E+09	250000	3.57E-10	1.25E-06	6.38E-06	0.022	0.072
3	4500	131.4	737.7	2.95E+09	422500	2.94E-10	1.32E-06	5.13E-06	0.023	0.049
2	4500	140.9	878.6	3.95E+09	422500	3.94E-10	1.77E-06	3.80E-06	0.017	0.026
1	4500	130.2	1008.8	4.54E+09	422500	4.52E-10	2.03E-06	2.03E-06	0.009	0.009

Table 3.8.6. Flexural drift calculations for Frame 5 under wind loads

Story	h_i (mm)	F_i (kN)	V_i (kN)	M (N-mm)	A (mm^2)	ϕ_i	$\Delta\theta_i$ (rad)	θ_i (rad)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	29.5	29.5	1.03E+08	90000	2.14E-11	7.50E-08	1.24E-06	0.0044	0.0241
7	3500	34.4	63.8	2.23E+08	90000	4.64E-11	1.62E-07	1.17E-06	0.0041	0.0197
6	3500	34.9	98.7	3.46E+08	250000	2.58E-11	9.04E-08	1.01E-06	0.0035	0.0156
5	3500	34.0	132.7	4.64E+08	250000	3.47E-11	1.22E-07	9.17E-07	0.0032	0.0121
4	3500	32.9	165.6	5.80E+08	250000	4.33E-11	1.52E-07	7.95E-07	0.0028	0.0089
3	4500	36.3	201.8	9.08E+08	422500	4.02E-11	1.81E-07	6.44E-07	0.0029	0.0061
2	4500	38.7	240.6	1.08E+09	422500	4.79E-11	2.15E-07	4.63E-07	0.0021	0.0032
1	4500	35.5	276.0	1.24E+09	422500	5.49E-11	2.47E-07	2.47E-07	0.0011	0.0011

Table 3.8.7 Flexural drift calculations for Frame E under earthquake loads

Story	h_i (mm)	F_i (kN)	V_i (kN)	M (N-mm)	A (mm^2)	ϕ_i	$\Delta\theta_i$ (rad)	θ_i (rad)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
8	3500	78.04	78.04	2.73E+08	90000	1.28E-10	4.47E-07	5.13E-06	0.02	0.09
7	3500	69.23	147.28	5.15E+08	90000	2.41E-10	8.43E-07	4.69E-06	0.02	0.07
6	3500	62.97	210.25	7.36E+08	250000	1.24E-10	4.33E-07	3.84E-06	0.01	0.06
5	3500	53.79	264.04	9.24E+08	250000	1.55E-10	5.44E-07	3.41E-06	0.01	0.04
4	3500	43.62	307.66	1.08E+09	250000	1.81E-10	6.34E-07	2.87E-06	0.01	0.03
3	4500	47.24	354.90	1.42E+09	422500	1.41E-10	6.36E-07	2.23E-06	0.01	0.02
2	4500	29.78	384.67	1.73E+09	422500	1.72E-10	7.75E-07	1.60E-06	0.01	0.01
1	4500	22.16	406.83	1.83E+09	422500	1.82E-10	8.20E-07	8.20E-07	0.00	0.00

Table 3.8.9. Flexural drift calculations for Frame 5 under earthquake loads

Story	h_i (mm)	F_i (kN)	V_i (kN)	M (N-mm)	A (mm^2)	ϕ_i	$\Delta\theta_i$ (rad)	θ_i (rad)	$\delta_{inter,i}$ (mm)	$\delta_{abs,i}$ (mm)
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8	3500	46.81	46.81	90000	3.40E-11	1.19E-07	1.40E-06	0.0049	0.0248	90000
7	3500	41.53	88.34	90000	6.42E-11	2.25E-07	1.28E-06	0.0045	0.0199	90000
6	3500	38.68	127.03	250000	3.32E-11	1.16E-07	1.06E-06	0.0037	0.0155	25000 0
5	3500	33.04	160.07	250000	4.19E-11	1.47E-07	9.39E-07	0.0033	0.0118	25000 0
4	3500	26.79	186.86	250000	4.89E-11	1.71E-07	7.92E-07	0.0028	0.0085	25000 0
3	4500	28.85	215.71	422500	4.29E-11	1.93E-07	6.21E-07	0.0028	0.0057	42250 0
2	4500	18.18	233.89	422500	4.66E-11	2.10E-07	4.28E-07	0.0019	0.0029	42250 0
1	4500	10.08	243.97	422500	4.86E-11	2.19E-07	2.19E-07	0.0010	0.0010	42250 0

3.9. Comparison of drift for hand calculation and SAP2000 2D and 3D models

The final calculated total drift values and comparisons with SAP2000 model outputs are shown in tables below.

Table 3.9.1. Comparison of drift under wind forces

Level	Frame 5 (force acting in x-direction)			Frame E (force acting in y-direction)		
	Hand	2D	3D	Hand	2D	3D
8	5.24	5.10	4.62	8.25	7.43	6.02
7	6.60	11.33	9.66	11.10	15.73	12.58
6	7.50	10.56	9.07	13.27	14.52	10.35
5	10.06	14.07	11.55	17.73	18.55	13.50
4	13.98	18.53	14.13	24.88	23.45	17.32
3	21.10	26.71	18.91	37.74	32.84	23.87

2	27.72	25.56	17.86	49.48	30.32	23.50
1	16.06	13.24	9.24	28.20	15.09	12.78

Table 3.9.2. Comparison of drift under seismic forces

Level	Frame 5 (force acting in x-direction)			Frame E (force acting in y-direction)		
	Hand	2D	3D	Hand	2D	3D
8	6.98	5.72	6.23	4.88	4.44	3.69
7	8.19	10.77	11.69	6.11	8.43	6.89
6	8.53	8.89	10.15	6.74	7.18	5.23
5	10.75	10.89	12.21	8.47	8.69	6.43
4	14.02	13.14	13.99	11.15	10.31	7.71
3	19.75	17.65	17.69	15.80	13.71	10.03
2	23.95	15.89	15.90	19.20	12.11	9.37
1	13.23	7.79	7.86	10.48	5.81	4.87

Figures X and Y show plots based on the tables above. The general shape of the drift curve is similar in all cases, however there are visible differences between hand calculations and 2D, 3D models. These differences may be due to the assumptions made during each modeling method. Hand calculations assume linear elastic behavior, boundary conditions, and uniform lateral stiffness within each frame, while 2D SAP 2000 models allow redistribution of internal forces along the frame height, representing member stiffness and connectivity within the plane of loading. The 3D SAP 2000 models incorporate spatial interaction between frames, capturing out-of-plane stiffness contributions. Lateral loads are distributed among multiple elements, modifying the stiffness demand on individual frames.

For seismic loading, the elastic inter-story drifts were amplified in order to account for the inelastic structural behavior using the deflection amplification factor

$$\delta_x = \frac{C_d}{I_e} \delta_{xe}$$

In our case, seismic drifts were multiplied by 2.5. Allowable drift was calculated as 2% of story height.

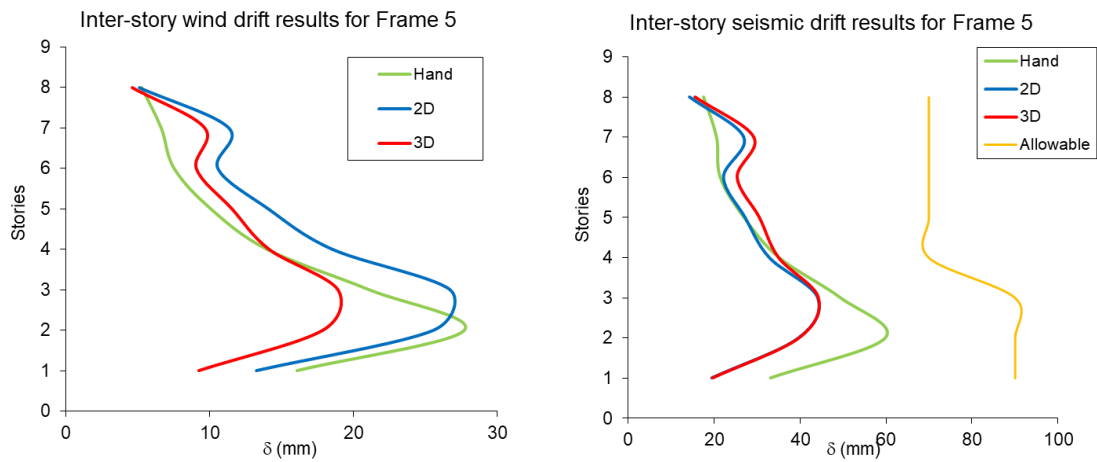


Figure 3.9.1. Inter Story Wind and Seismic drift results comparison for hand calculations, 2D and 3D SAP 2000 simulations for Frame 5 (x-direction)

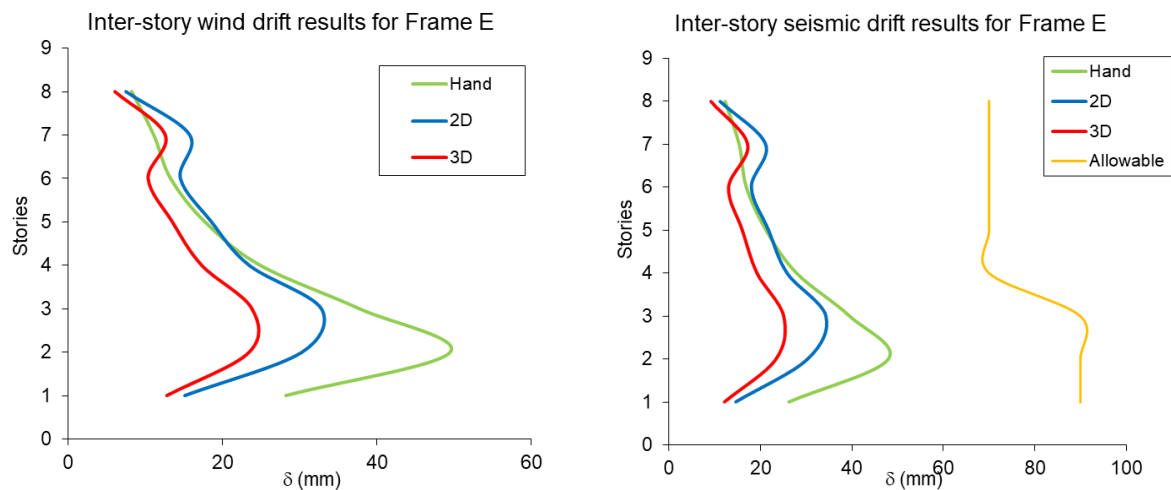


Figure 3.9.2. Inter Story Wind and Seismic drift results comparison for hand calculations, 2D and 3D SAP 2000 simulations for Frame E (y-direction)

3.10. Internal force analysis

3.10.1. Live load - vertical load case

Frame 5 was chosen to verify the results of internal force calculations produced by 2D and 3D SAP2000 models. Below are presented these results.

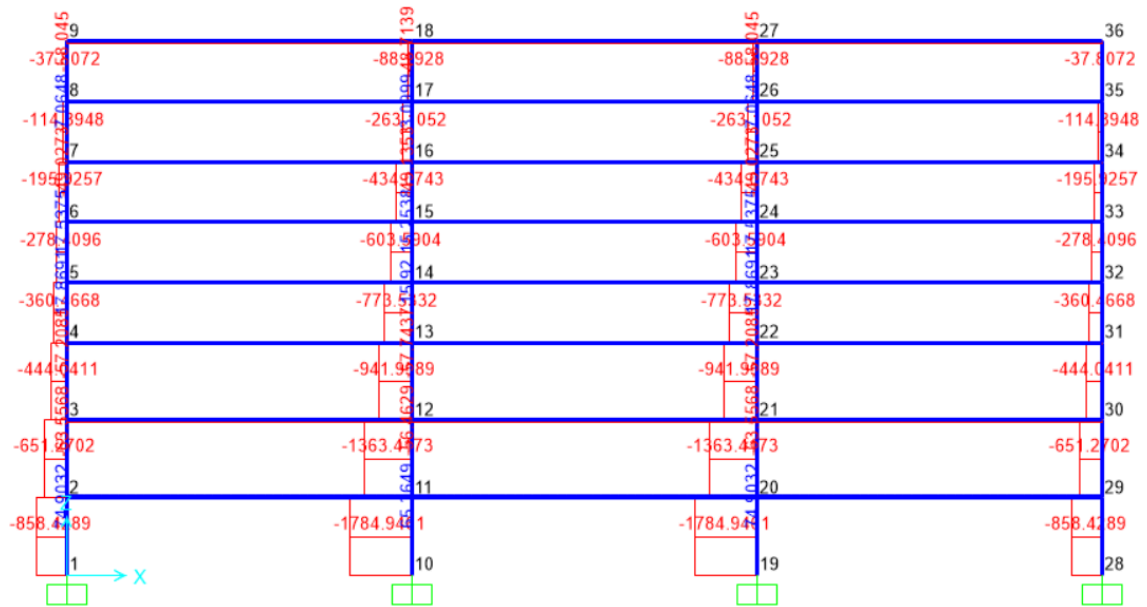


Figure 3.10.1.1. Axial force due to live load in SAP2000 2D model

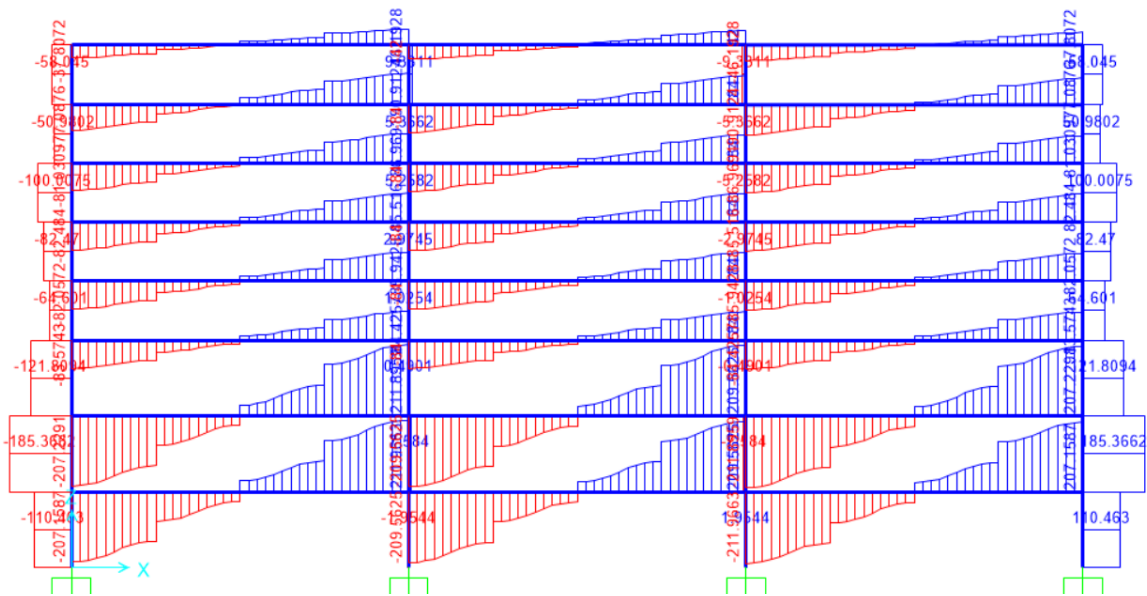


Figure 3.10.1.2. Shear force due to live load in SAP2000 2D model

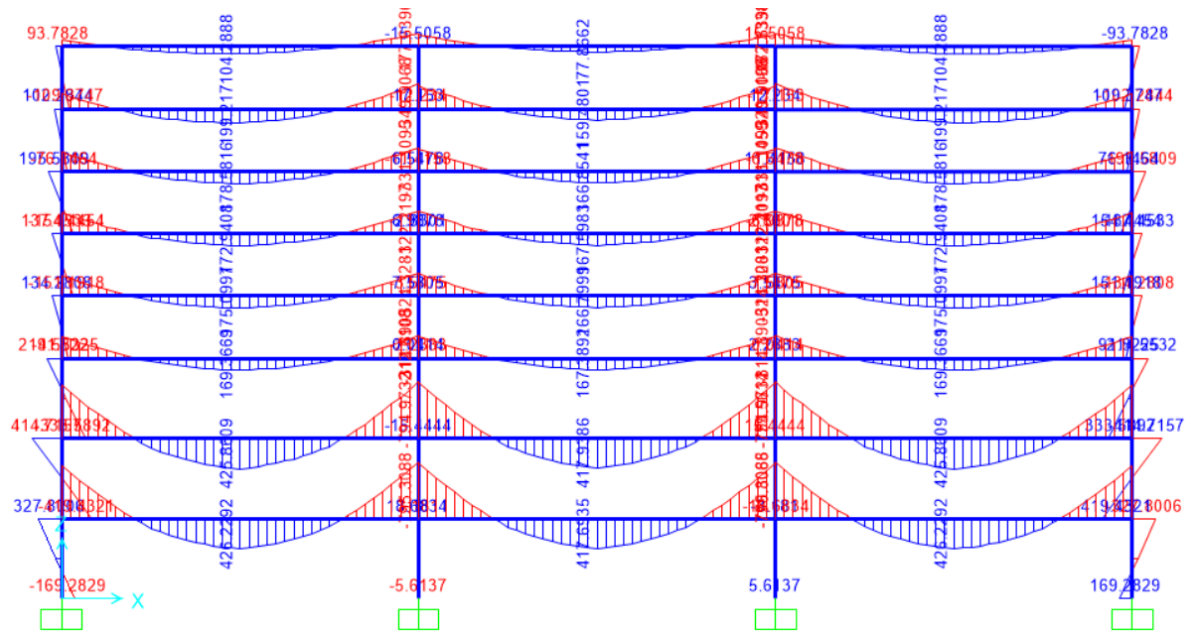


Figure 3.10.1.3. Bending moment due to live load in SAP2000 2D model

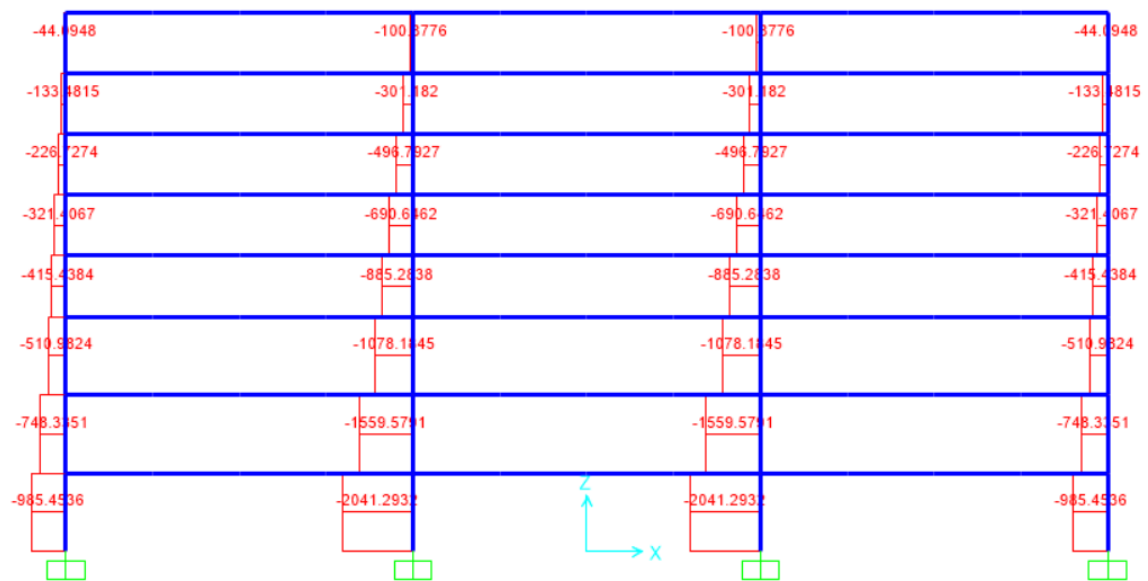


Figure 3.10.1.4. Axial force due to live load in SAP2000 3D model

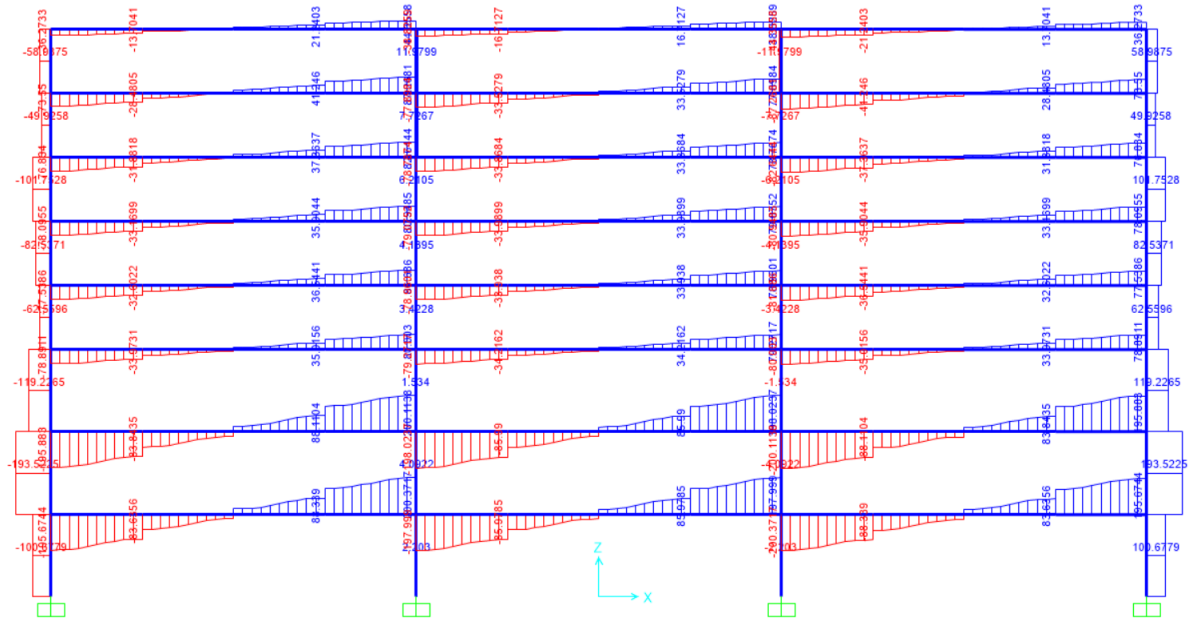


Figure 3.10.1.5. Shear force due to live load in SAP2000 3D model

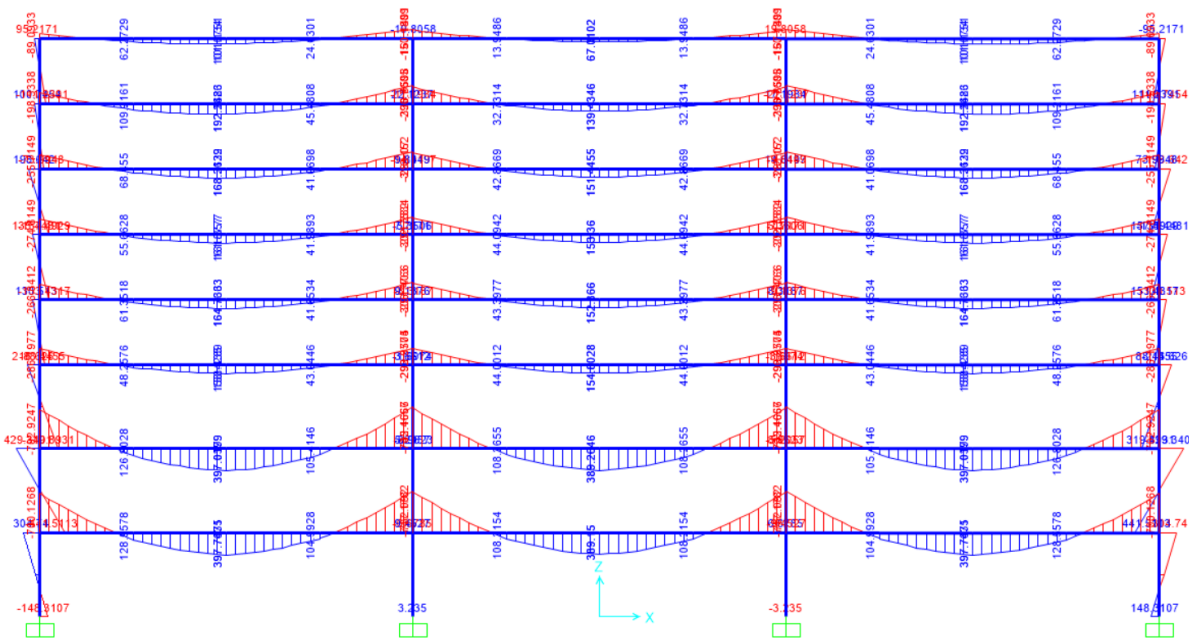


Figure 3.10.1.6. Bending moment due to live load in SAP2000 3D model

3.10.2. Hand calculation verification for internal forces under live load

The calculation of frame loading is as follows:

Floor live load acting on the slab: $L_{4-8} = 1.92 \text{ kPa}$

Triangular loads acting on major beam: $w = 1.92 \times 5 = 9.6 \text{ kN/m}$

Point load from 5-meter minor beams with tributary widths of 5 meters acting on major beam: $P_{beam} = (1.92 \times 5 \times 5 \times 0.5)/2 \times 2 = 24 \text{ kN}$

The same calculation apply to roof live loading $L_r = 0.96 \text{ kPa}$ and floor live loading on floors 1-3 with $L_{1-3} = 1.92 \text{ kPa}$. Frame 5 taken as an example for calculations is loaded as shown in the Figure below.

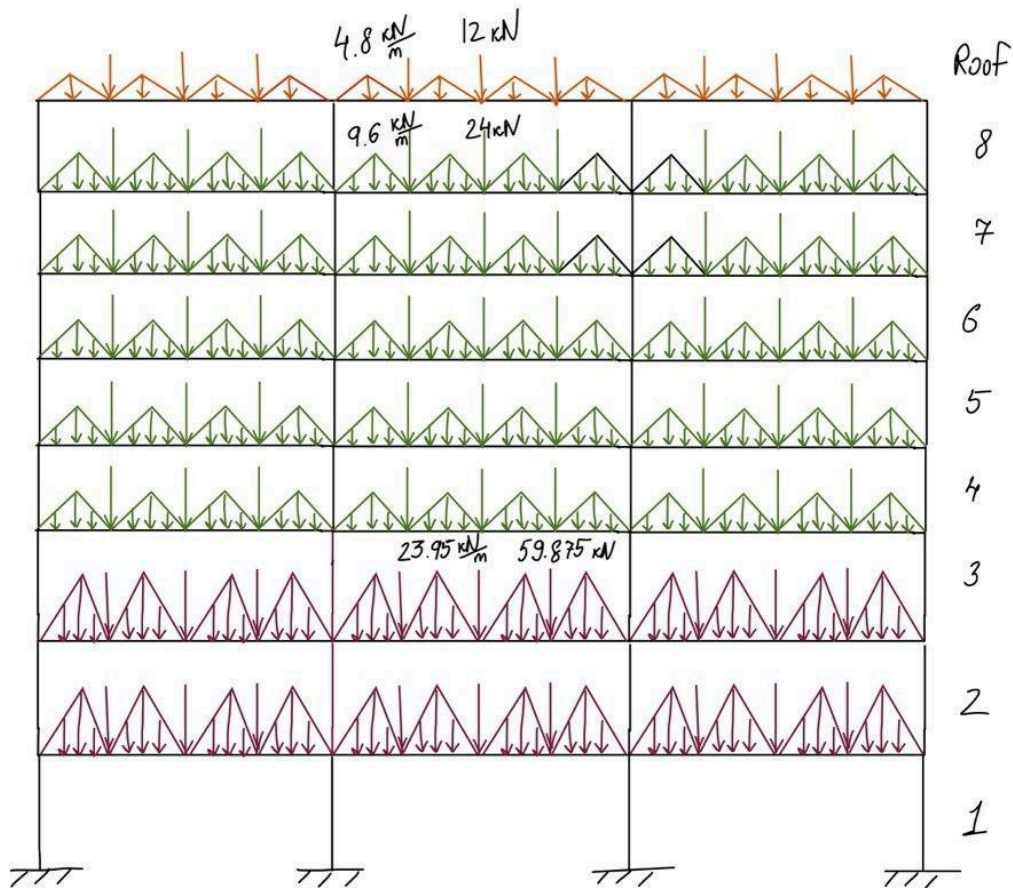


Figure 3.10.2.1. Live loads on frame 5

An approximate calculation of internal forces for statistically indeterminate structures under these calculated live loads was carried out. The standard inflection point of $0.1L$ usually used in this analysis is unreasonable for a 20-meter beam. To make the hand calculation results comparable to actual values, inflection points were adjusted according to $0.22L = 4.4 \text{ m}$, and the scheme of loading for the approximate analysis can be seen in Figure 3.10.2.1

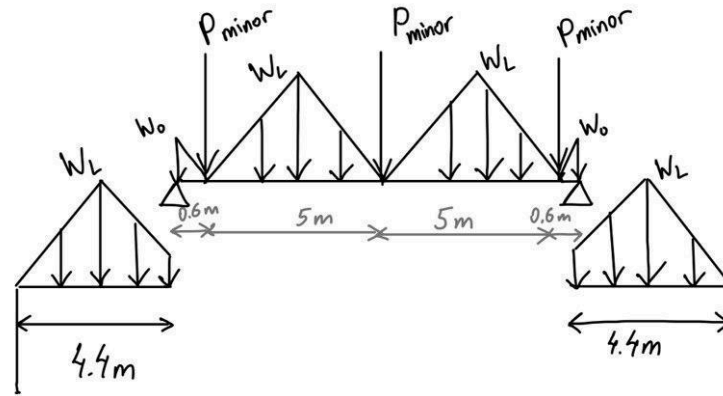


Figure 3.10.2.2. Loading scheme for approximate analysis

$L = 20 \text{ m}$; $0.22L = 4.4 \text{ m}$; $0.56L = 11.2 \text{ m}$

Example calculations for 4-8 floors

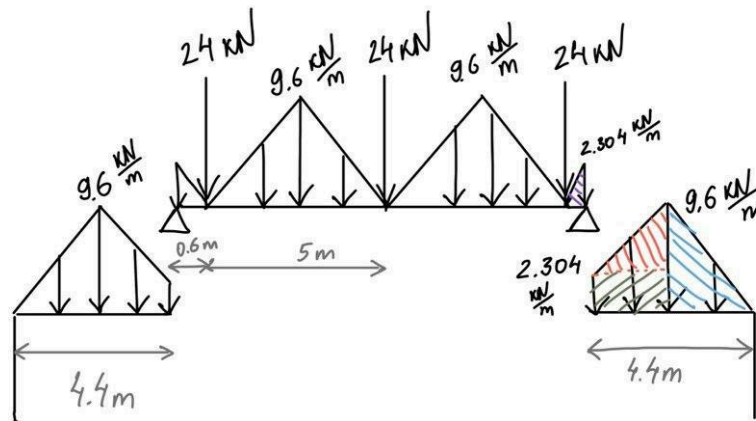


Figure 3.10.2.3. 4-8 floor live loading for approximate analysis

Converting the distributed loads into point loads:

$$9.6 \times 5 \times 0.5 = 24 \text{ kN (triangles)}$$

$$w_{4.4} = 9.6/2.5 \times 0.6 = 2.304 \text{ kN/m}$$

Purple triangle: $2.304 \times 0.6 \times 0.5 = 0.69 \text{ kN}$ acting 0.2 m from the edge

Red triangle: $(9.6 - 2.304) \times (4.4 - 2.5) \times 0.5 = 6.93 \text{ kN}$ acting at 3.133 m from the edge

Green rectangle: $2.304 \times (4.4 - 2.5) = 4.38 \text{ kN}$ acting at 3.45 m from the edge

Blue triangle: $9.6 \times 2.5 \times 0.5 = 12 \text{ kN}$ acting at 1.67 m from the edge

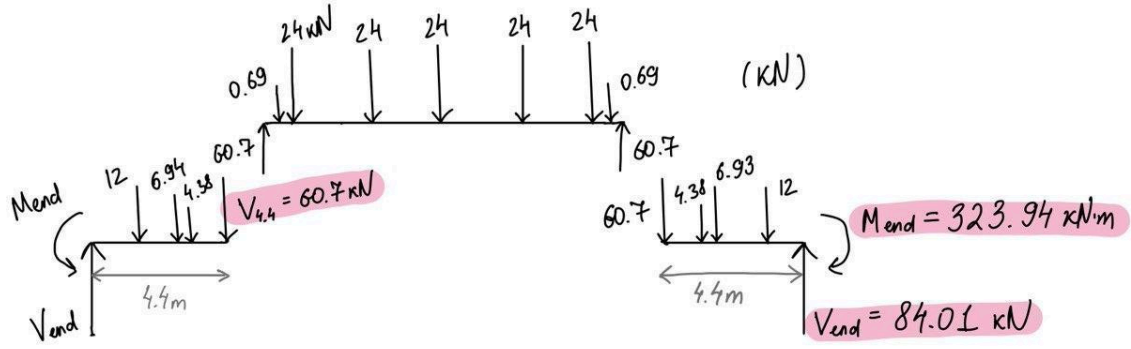


Figure 3.10.2.4. Internal force calculations for guest floor live loads

Now, the moments and shear forces can be calculated as:

$$V_{4.4} = (24 \times 5 + 0.69 \times 2)/2 = 60.7 \text{ kN}$$

$$M_{mid} = 60.7 \times 5.6 - 24 \times 2.5 - 24 \times 5 - 0.69 \times 5.4 = 156.2 \text{ kN} \cdot \text{m}$$

$$V_{end} = 60.7 + 4.38 + 6.93 + 12 = 84.01 \text{ kN}$$

$$M_{end} = 60.7 \times 4.4 + 6.93 \times 3.133 + 4.38 \times 3.45 + 12 \times 1.67 = 323.94 \text{ kN} \cdot \text{m}$$

The same calculations were repeated for the roof and 1-3 floors.

$$\text{For roof: } M_{mid} = 78.1 \text{ kN} \cdot \text{m}; M_{end} = 161.97 \text{ kN} \cdot \text{m}; V_{end} = 42 \text{ kN}$$

$$\text{For 1-3 floors: } M_{mid} = 389.52 \text{ kN} \cdot \text{m}; M_{end} = 808.06 \text{ kN} \cdot \text{m}; V_{end} = 209.56 \text{ kN}$$

To calculate the moments on top and bottom column relative to the floor, the following formulas were use:

$$M_{top} = \frac{\frac{E_1 I_1}{L_1}}{\frac{E_1 I_1}{L_1} + \frac{E_2 I_2}{L_2}} M \quad \& \quad M_{bot} = \frac{\frac{E_2 I_2}{L_2}}{\frac{E_1 I_1}{L_1} + \frac{E_2 I_2}{L_2}} M$$

For example, the exterior columns on floors 3 and 4 have different dimensions

($I_3 = 0.0108 \text{ m}^4$, $L_3 = 4.5 \text{ m}$ and $I_4 = 0.003417 \text{ m}^4$, $L_4 = 3.5 \text{ m}$); therefore:

$$M_3 = \frac{0.0108/4.5}{0.0108/4.5 + 0.003417/3.5} \times 323.94 = 230.27 \text{ kN} \cdot \text{m}$$

$$M_4 = \frac{0.003417/3.5}{0.0108/4.5 + 0.003417/3.5} \times 323.94 = 93.67 \text{ kN} \cdot \text{m}$$

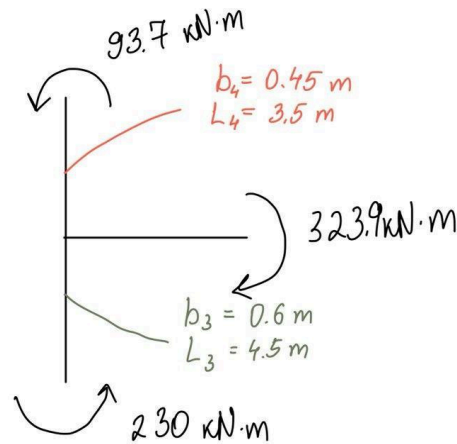


Figure 3.10.2.5. Moment distribution calculation example

This demonstrates that stiffer columns take more moments and the same ratio is later seen in moment diagrams produced by 2D and 3D SAP models.

Shear was calculated as:

$$V = \frac{M_t + M_b}{h_i}$$

For example, the shear force for the 7th floor was calculated as:

$$V_7 = \frac{161.97 + 129.2}{3.5} = 83.2 \text{ kN}$$

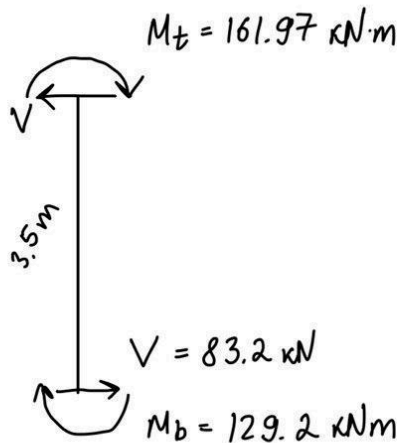


Figure 3.10.2.6. Shear force calculation example

The axial force in the columns was evaluated per each floor during approximate analysis. For the top floor it is equal V_{end} obtained from calculations. For all the floors

below, the axial force is obtained as the cumulative sum of the V_{end} from the floors above.

Results are shown in figures below.

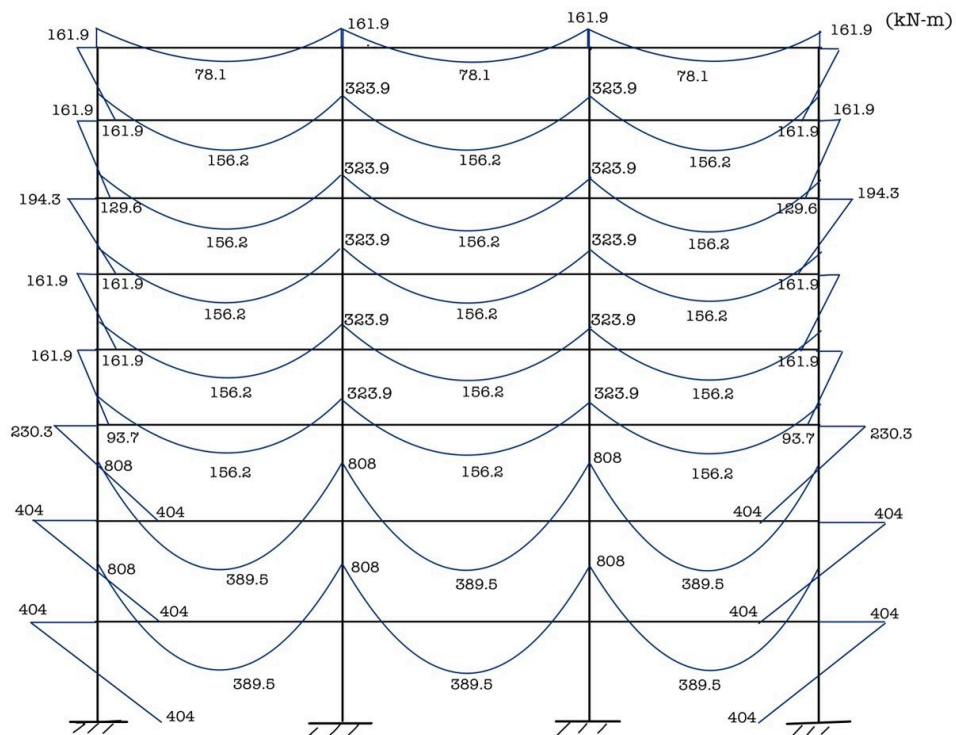


Figure 3.10.2.7. Bending moment diagram

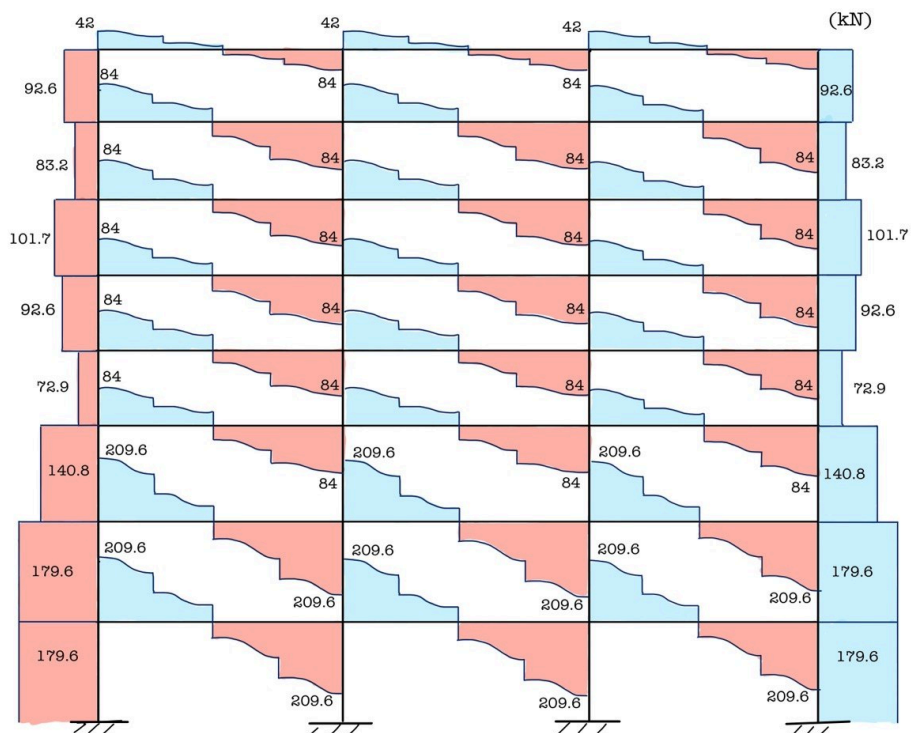


Figure 3.10.2.8. Shear force diagram

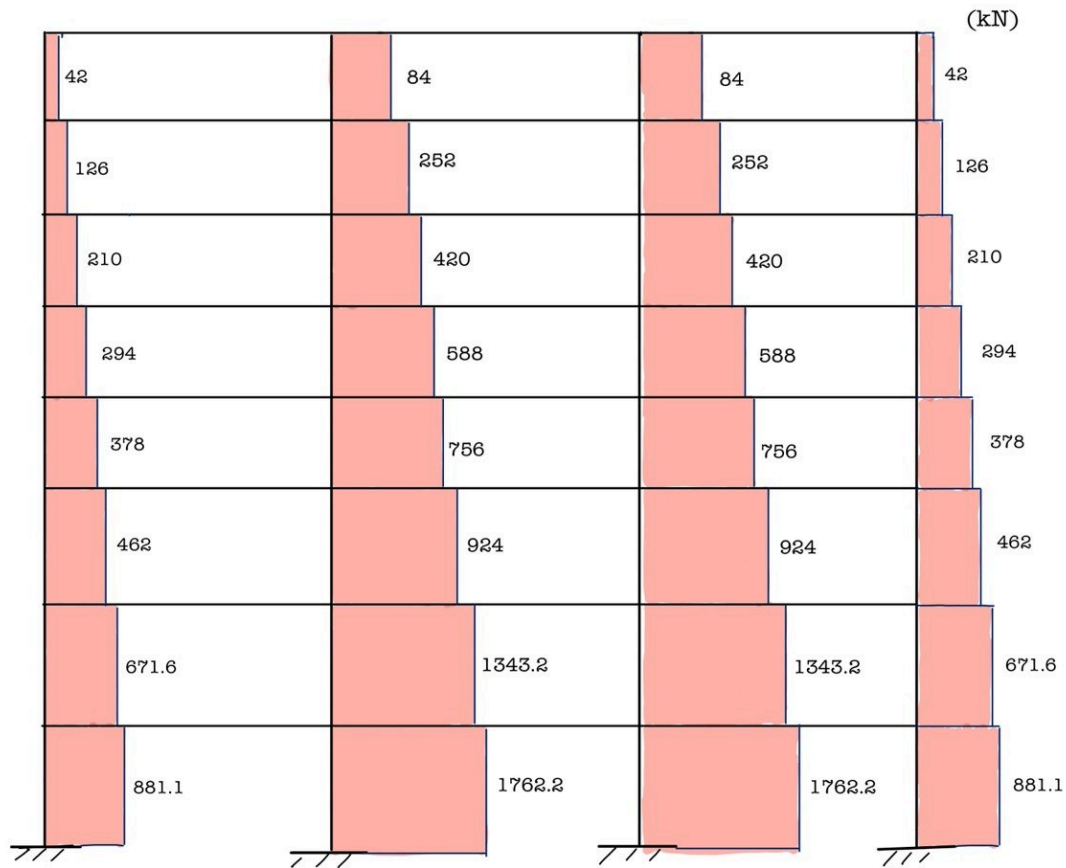


Figure 3.10.2.9. Axial force diagram

3.10.3. Comparison of hand calculations with software results for live load

Generally, the hand calculations and the 2D model results are in a very close alignment due to proper adjustment of inflection point in hand calculations.

The 2D model tends to overestimate the moment due to frame bending in one plane without full three dimensional stiffness interaction present in the 3D model. The same applies to the shear force. It is an indication that there is some redistribution of gravity load effects within the full structural system in the 3D model. However, the axial force for the 3D model turned out to be the highest among 3 cases, which can be explained by the selected members attracting more load due to their stiffness (columns stiffer than beams), adjacent members restraining deformation and stronger continuity. On the contrary, the 2D model is less restrained and thus predicts lower axial force values.

Table 3.10.3.1. Comparison of internal forces under live load

Level	Axial force beam (KN)			Shear force beam (KN)			Moment column (KNm)		
	Hand	2D	3D	Hand	2D	3D	Hand	2D	3D
8	42	37.8	44.1	43	42	38.8	78.1	76.8	67
7	126	114.9	133.5	85	84	77.9	156.2	157.2	140
6	210	196	226.7	85	84	77.8	156.2	166.9	151.4
5	294	278.4	321.4	85	84	79.1	156.2	167.6	153.4
4	378	360.5	415.4	85	84	79	156.2	166.8	152.4
3	462	444	510.98	85	84	79.3	156.2	167.4	154.8
2	671.6	651.3	743.3	210.6	209.6	198	389.5	417.9	389.3
1	881.1	858.4	985.4	210.6	209.6	198	389.5	417.7	389.2

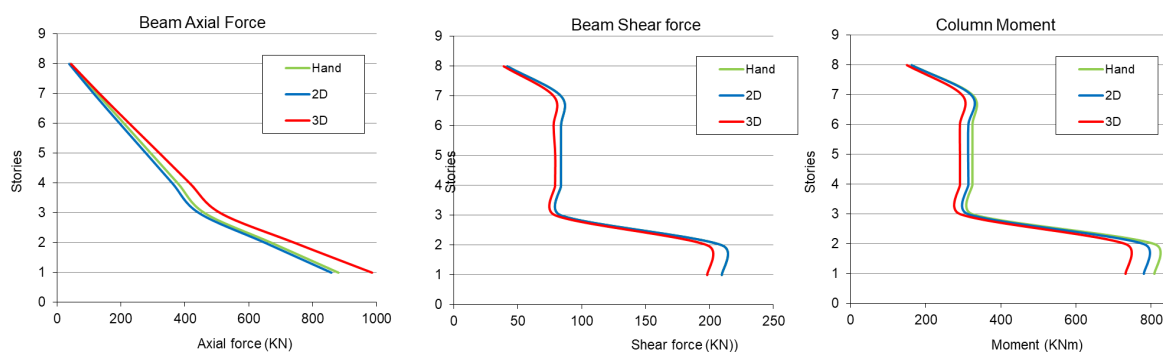


Figure 3.10.3.1. Internal forces comparison for Hand calculations, 2D, 3D SAP2000 models

3.10.4. Wind load - horizontal load case

Below presented are the results for internal forces of the same Frame 5 under wind loading in x-direction in 2D and 3D SAP models.

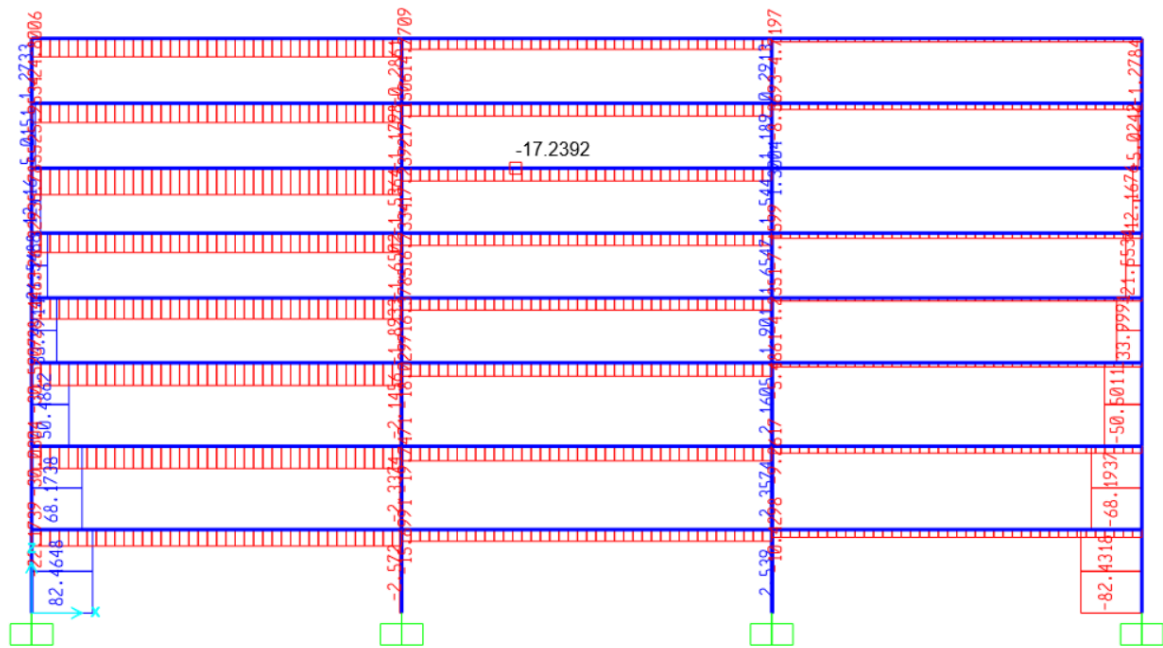


Figure 3.10.4.1. Axial force due to wind load in SAP2000 2D model

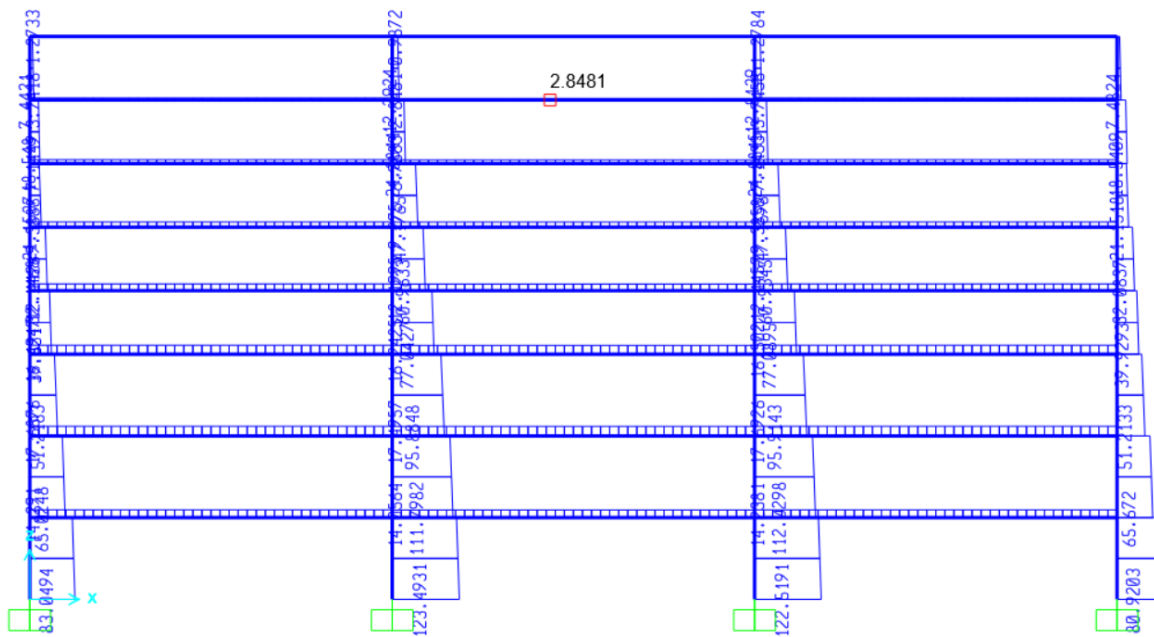


Figure 3.10.4.2. Shear force due to live load in SAP2000 2D model

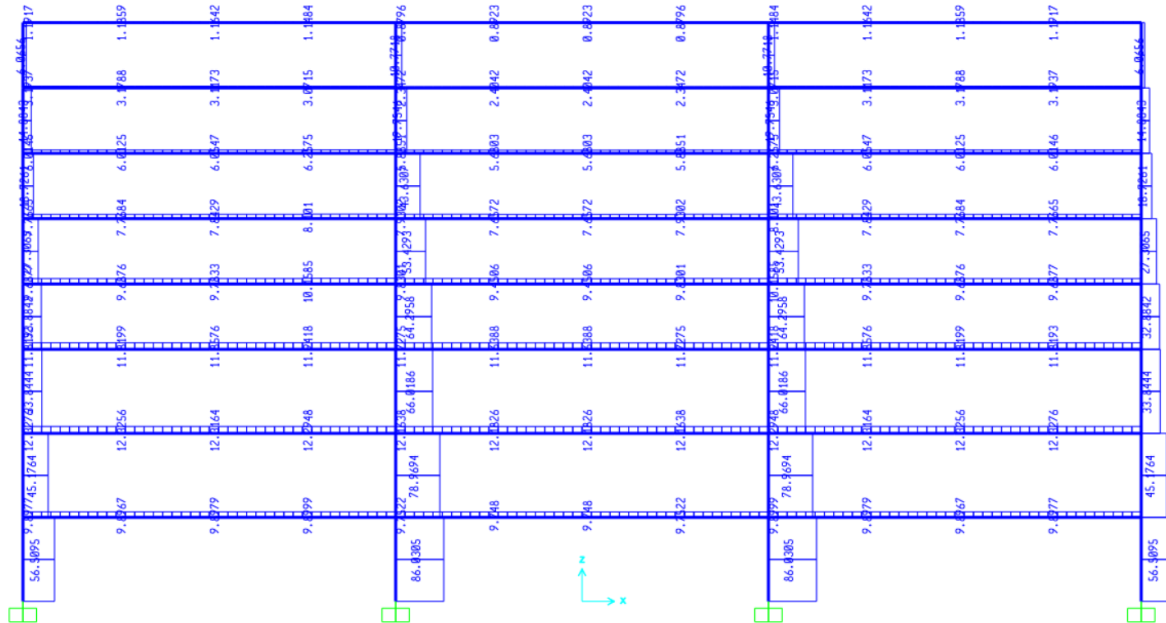


Figure 3.10.4.5. Shear force due to live load in SAP2000 3D model

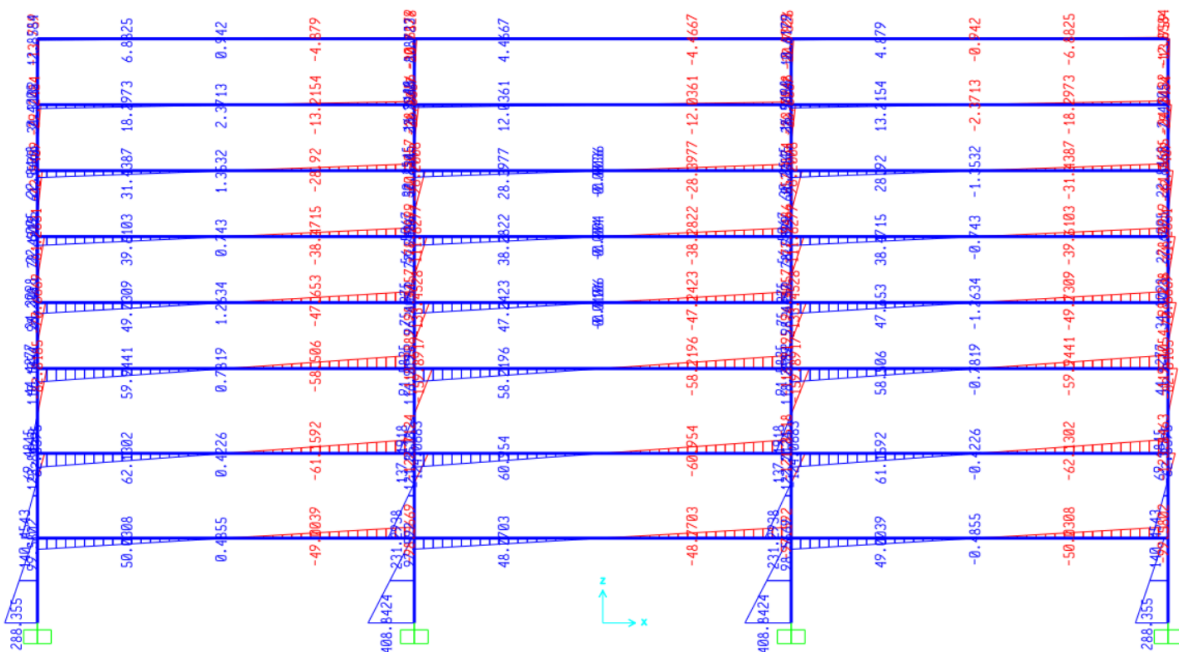


Figure 3.10.4.6. Bending moment due to wind load in SAP2000 3D model

3.10.5. Hand calculation verification for internal forces under wind load

The portal method was used to calculate the internal forces under the horizontal wind forces applied to Frame 5 shown in the figure below.

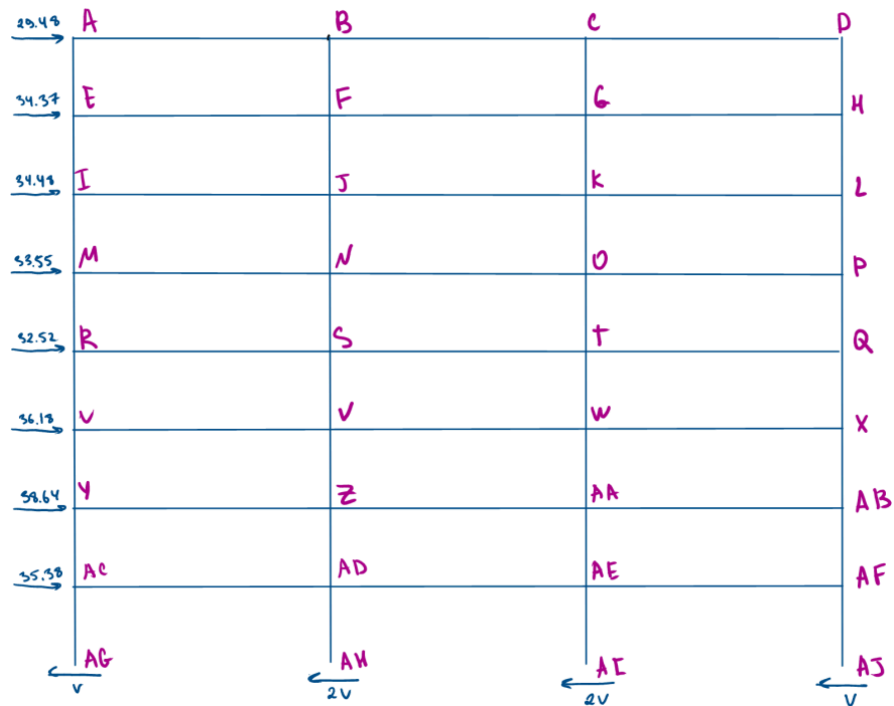
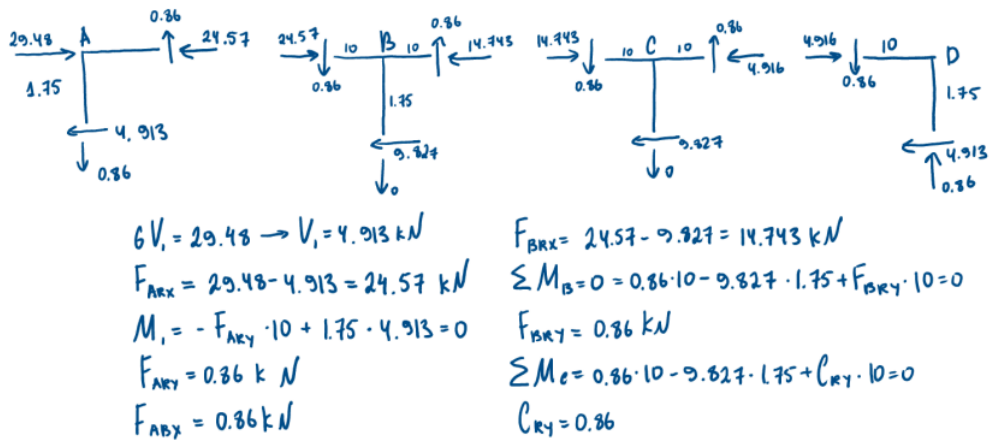


Figure 3.10.5.1. Wind forces on Frame 5

1. 8th floor $M \rightarrow$



2. 7th floor

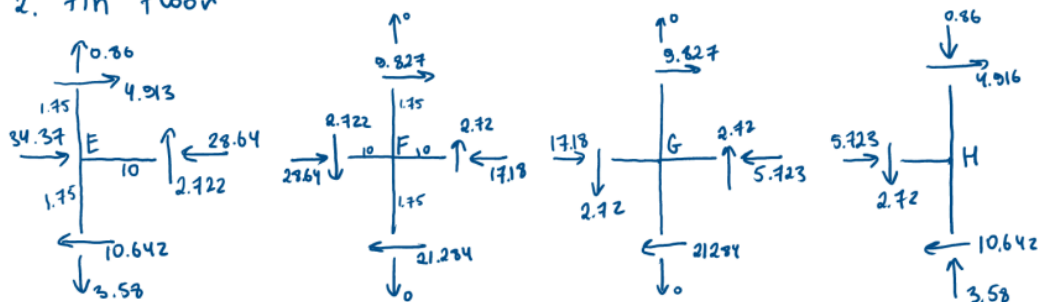
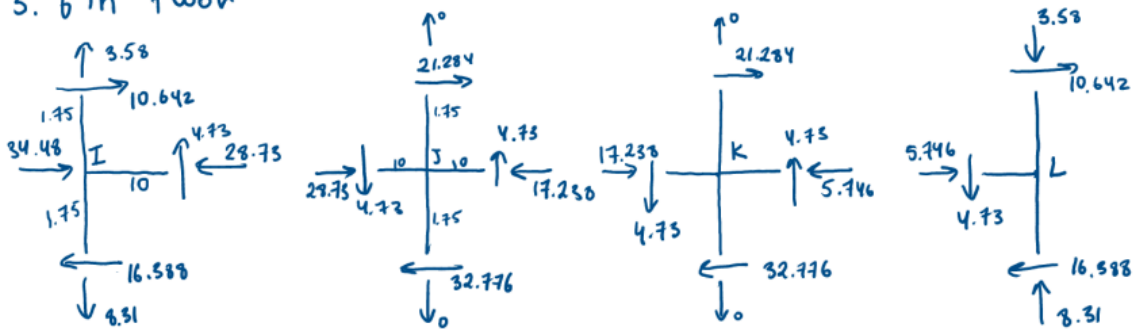
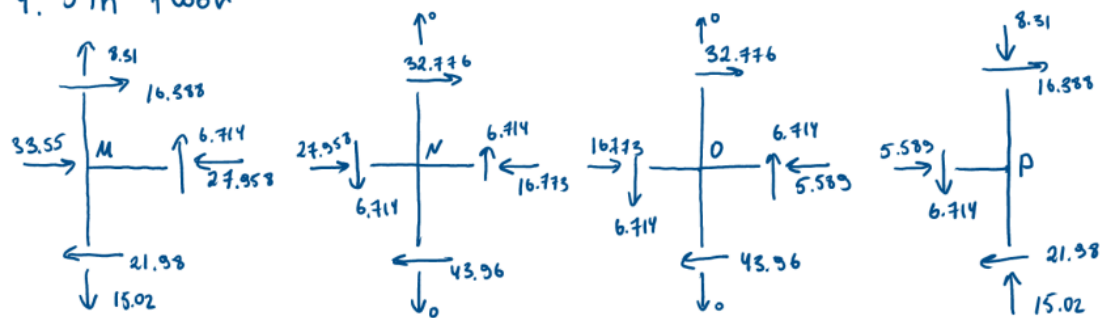


Figure 3.10.5.2. Hand calculations (part 1)

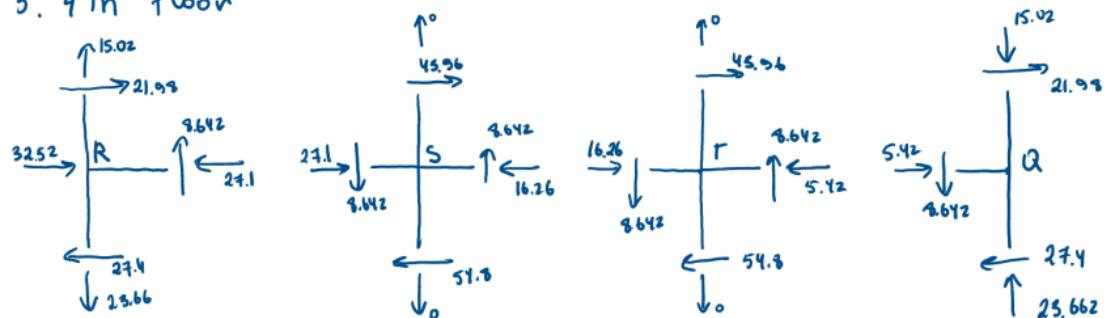
3. 6th Floor



4. 5th Floor



5. 4th Floor



6. 3rd Floor

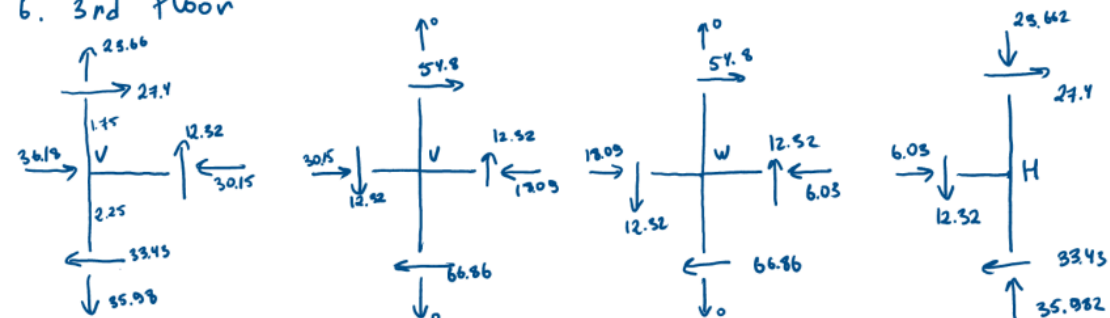
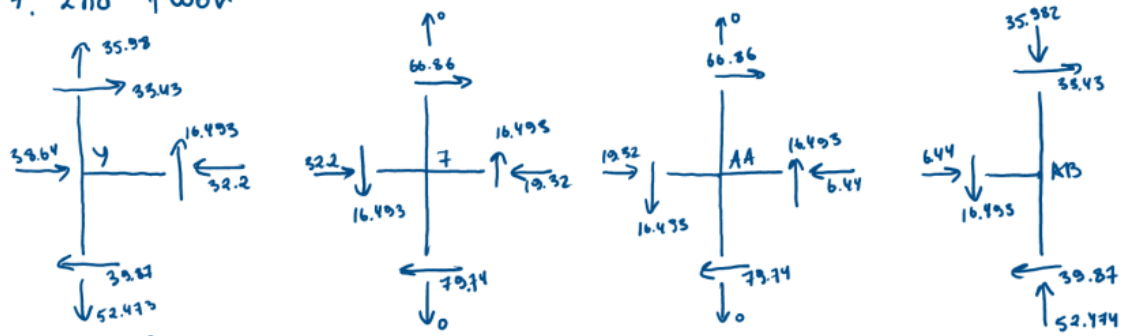


Figure 3.10.5.3. Hand calculations (part 2)

7. 2nd floor



8. 1st floor

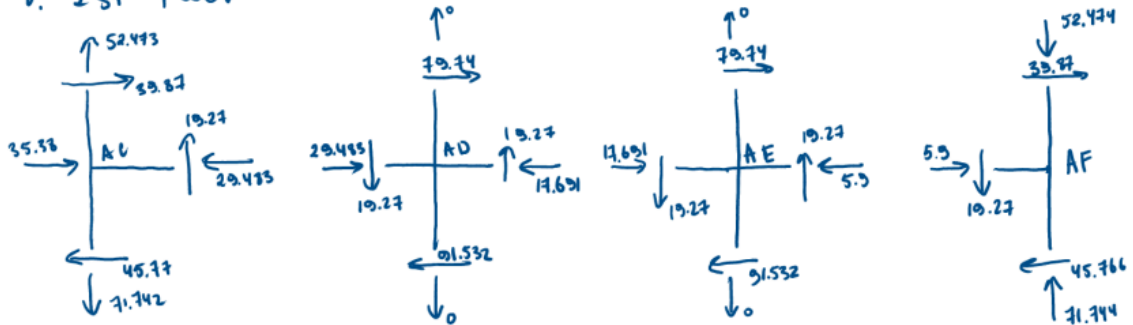


Figure 3.10.5.4. Hand calculations (part 3)

Based on these intermediate calculations, the following graphs were drawn.

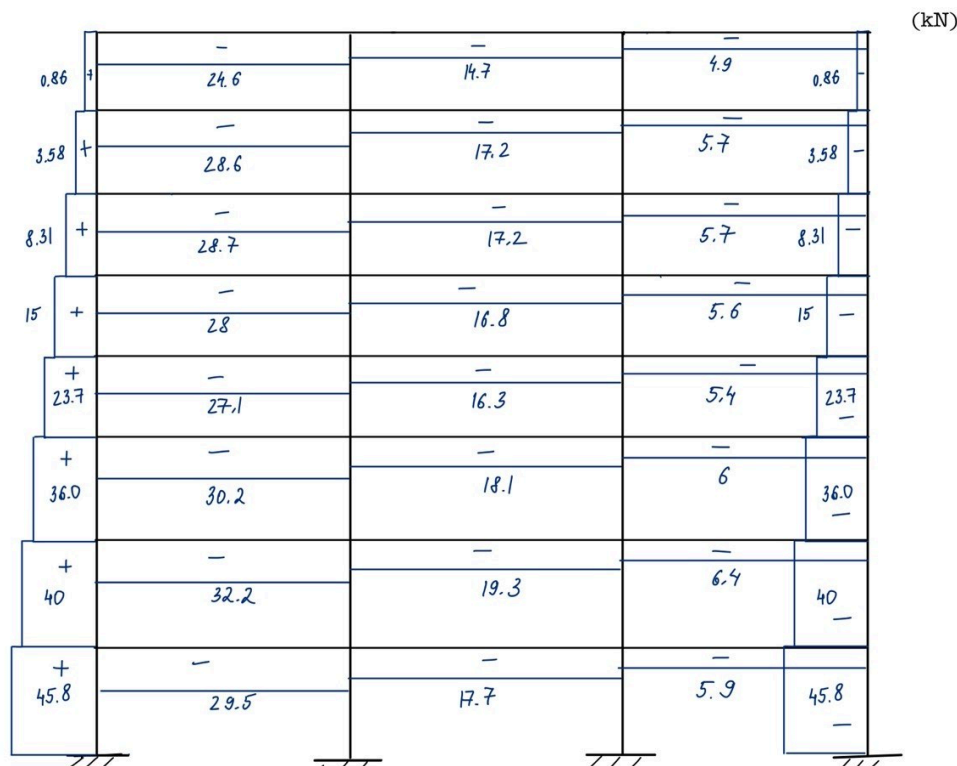


Figure 3.10.5.6. Axial force diagram

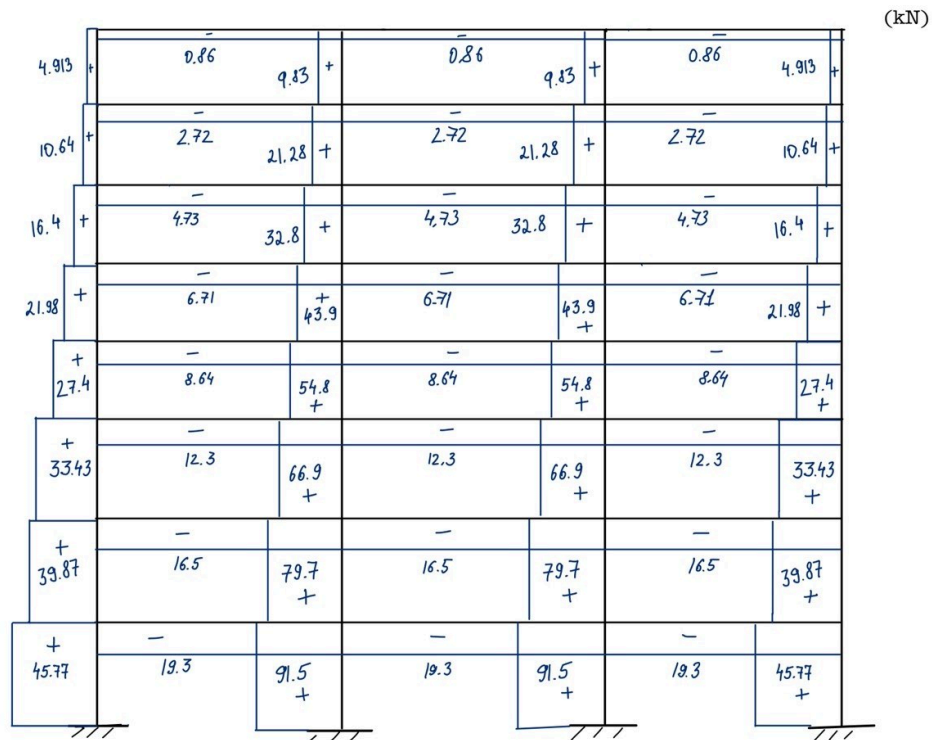


Figure 3.10.5.7. Shear force diagram

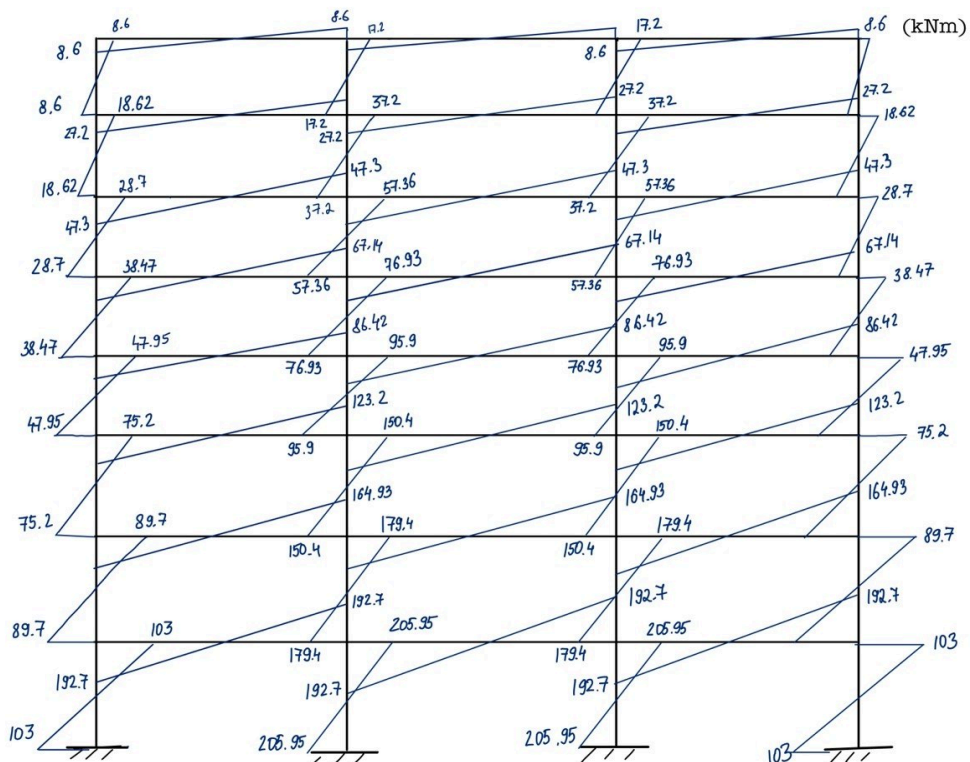


Figure 3.10.5.8. Moment diagram

3.10.6. Comparison of hand calculations with software results for wind load

Generally, hand calculations tend to underestimate the internal forces and always increase smoothly without any fluctuations unlike SAP2000 models. For example, shear force in beam and bending moment diagrams show sudden decrease at 1 floor level, which is most probably due to fixed supports at base that attracts more moment.

The reason for discrepancy between hand calculations and software results is that the portal method ignores stiffness differences and relies on an assumption that internal columns carry twice the shear the exterior columns carry. It also assumes that the moment is 0 at midpoints of beams and columns, which is not realistic especially when considering the moment diagram of the 3D model. The hand calculation produces an idealized smooth response based on assumed force patterns that probably lead to the resulting underestimation, while SAP2000 takes into account actual stiffness matrix and exact connectivity.

As for software results, it can be seen that the 2D model has higher results since it represents a single frame resisting the wind forces, whereas in the 3D model the frame is not working alone as it is supported by slabs (diaphragm action) and neighboring frames in resisting the loads. So, the 2D model restricts load transfer to a single plane, which increases the internal force values, while the 3D model transfers part of the load to adjacent frames .

Table 3.10.6.1. Comparison of internal forces under wind load

Level	Axial force column (KN)			Shear force beam (KN)			Moment beam (KNm)		
	Hand	2D	3D	Hand	2D	3D	Hand	2D	3D
8	0.56	1.27	1.26	0.86	0.987	0.892	8.6	9.87	8.87
7	3.58	5.01	4.63	2.72	2.85	2.4	27.2	28.5	23.8
6	8.31	12.16	11.05	4.73	6.78	5.68	47.3	67.88	57.8
5	15	21.55	19.37	6.71	9.28	7.66	67.14	92.75	78
4	23.7	34	29.72	8.64	12.2	9.45	86.42	121.99	96.4
3	36	50.49	42.37	12.3	16.24	11.64	123.2	162.4	116.9
2	40	68.17	55.55	16.5	17.5	12.18	164.93	174.95	121
1	45.8	82.47	66.14	19.3	14.05	9.75	192.7	140.6	97.6

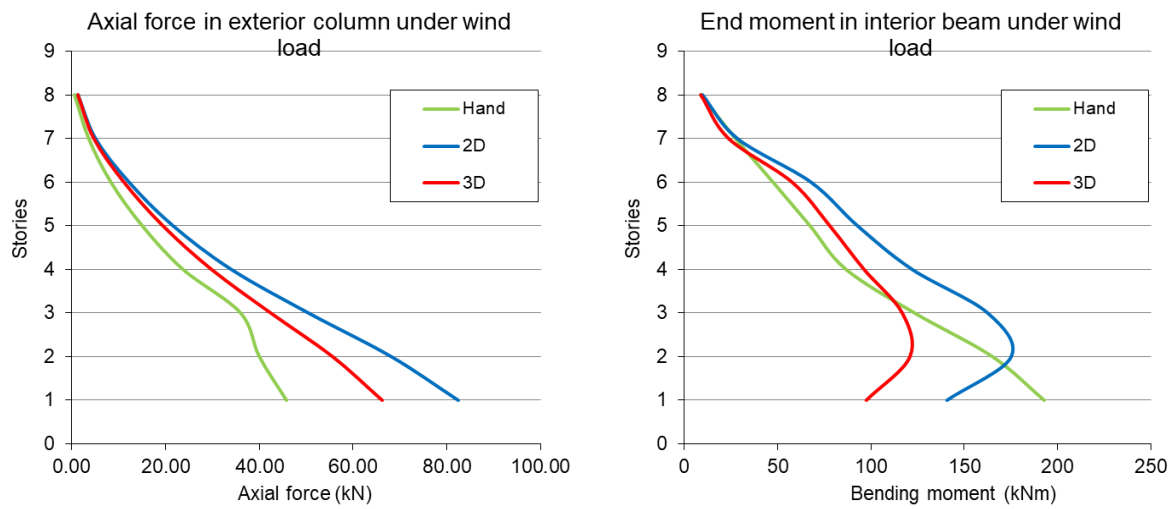


Figure 3.10.6.2. Comparative graphs for axial forces and bending moment under wind loads

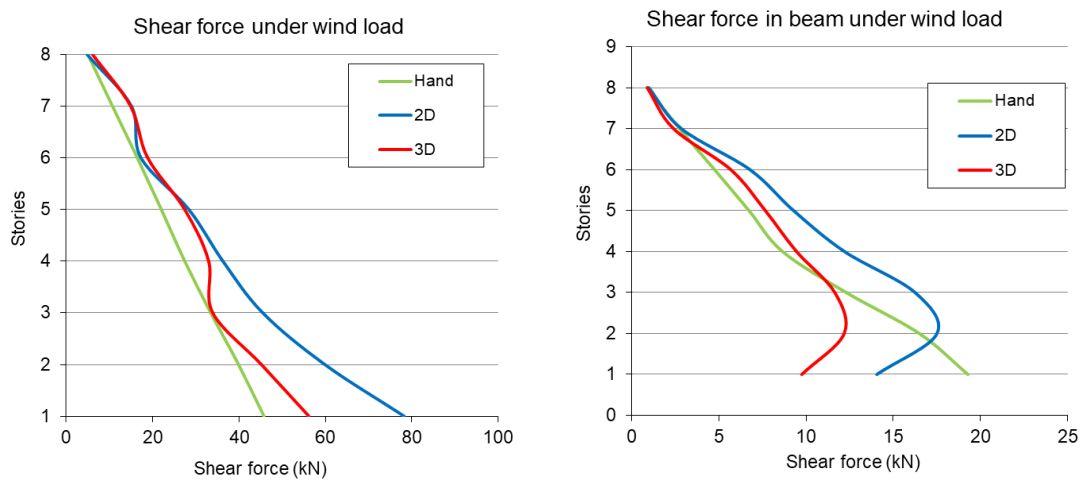


Figure 3.10.6.3. Comparative graphs for shear forces in columns and beams under wind loads

3.11. Structural member design

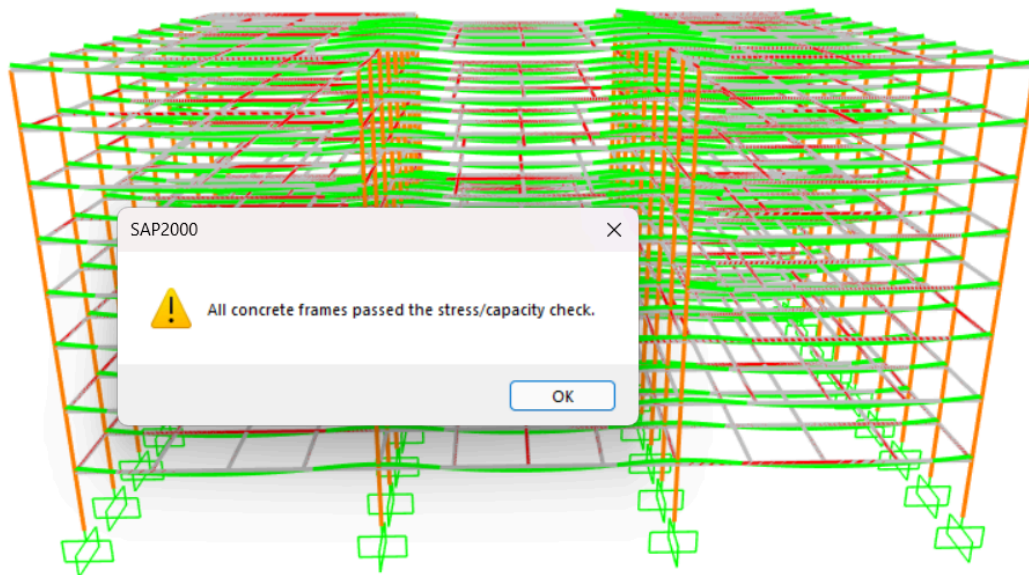


Figure 3.11.1. Structural integrity check

3.11.1. Prestressed beam

A large span beam cannot be designed as an ordinary beam due to large deflections. Design of the continuous post-tensioned beams is attempted following the procedure which is a synthesis of ACI 318-11 regulations, Prestressed/Precast Concrete Institute Design Handbook procedures (Sections 5.2.2, 5.7, 5.8) and specialist experience sourced online. Specifically, service stress limits and deflection checks, considering prestress losses and long-term deflection effects. The design accounts for the condition of continuity across 3 spans of 20 meters.

Section properties:

For the pre-stressed beam size estimation, the span-to-depth ratio is usually taken as 10 to 20. This is a guideline, not a limit. For a trial section, the section geometry parameters' calculations are as follows:

$$h = l/22 \approx 750 \text{ mm}$$

$$b \approx h/2 = 400 \text{ mm}$$

However, after modeling and running design checks, several iterations including resizing resulted in final prestressed beam dimensions of $h = 800 \text{ mm}$, $b = 500 \text{ mm}$.

For hand calculations of flexural stresses, an effective width is selected for bending effects. For axial effects, the entire cross-section is effective and the tributary of the structure (5000 mm) is taken as width.

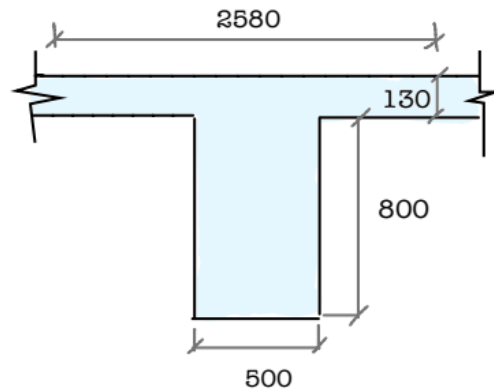


Figure 3.11.1.2. Prestressed beam cross section

Effective width is the smallest among:

- i) Sixteen times flange thickness and stem width: $16 \times 130 + 500 = 2580 \text{ mm}$
- ii) Quarter of a span: $0.25 \times 20000 = 5000 \text{ mm}$
- iii) Tributary = 5000 mm

So 2580 mm is taken as effective width to account for during flexural analysis.

Table 3.11.1.1. Section properties

	Axial effects	Bending effects
Area (mm^2)	1.05×10^6	7.35×10^5
I (mm^4)	-	6.13×10^{10}
Y_t (mm)	242.14	317.92
Y_b (mm)	687.86	612.08
S_{top} (mm^3)	-	1.93×10^8
S_{bot} (mm^3)	-	1×10^8

Where I - second moment of inertia; Y - distance from centroid to top/bottom section fiber; S - section modulus for top/bottom fiber ($S = I/Y$).

Material properties:

As per Section 2.3.5. of PCI Handbook (2004), the material usually used for pre-stressed concrete members is concrete with $f'_c = 40 \text{ MPa}$.

- $E_{ci} = 4700\sqrt{f'_c} = 29.7 \text{ GPa}$
- $w = 24 \text{ kN/m}^3$
- Creep coefficient, $t = 2$ (for long-term deflection)
- Strength at transfer, $f_{ci} = 28 \text{ MPa}$ (assuming 30% loss)

Nonprestressed reinforcement properties:

- $f_y = 420 \text{ MPa}$, $E_c = 200 \text{ GPa}$, $\phi = 0.9$

Prestressing reinforcement properties:

- Low relaxation, 7-wire ASTM A416 strand (Grade 270)
- Nominal strand diameter, $d_{str} = 15.7 \text{ mm}$
- Steel area of strand, $A_{str} = 150 \text{ mm}^2$
- $f_{pu} = 1860 \text{ MPa}$, $E_c = 200 \text{ GPa}$

Unbonded system:

- Angular coefficient of friction, $\mu = 0.07$
- Wobble coefficient of friction, $K = 0.003 \text{ rad/m}$
- Anchor set = 6 mm
- Stressing force = 80% of f_{pu}
- Eff. stress after all losses, $f_{se} = 1200 \text{ MPa}$

Minimum rebar cover = 40 mm $\Rightarrow$ take 70 mm to allow for installation of top slab bars and for corrosion mitigation.

Allowable stresses:

For sustained load condition: $Compr = 0.45f'_c = -18 \text{ MPa}$

For total load condition: $Compr = 0.6f'_c = -24 \text{ MPa}$

Targeting transition condition design (between uncracked and cracked), allowable range for tension in service condition:

$$= 0.62\sqrt{f'_c} < \sigma < 1\sqrt{f'_c} = 3.92 < \sigma < 6.32 \text{ (MPa)}$$

For initial condition:

$$\text{Compression} = 0.6f_{ci}' = 16.8 \text{ MPa}$$

$$\text{Tension} = 0.25\sqrt{f_{ci}'} = 1.32 \text{ MPa}$$

Design parameters' selection:

To carry on, 2 more inputs are needed:

i) level of precompression: ACI-318 requires a minimum of 0.86 MPa $\Rightarrow$

1 MPa is assumed based on specialist experience (online tutorial)

ii) target % of DL* to balance $\Rightarrow$ 60% [Aalami, et al, 2003]; after several iterations, the target was adjusted to 80%

Total dead load acting on a beam*: $w_{DL} = 30.13 \text{ kN/m}$ (includes super-imposed dead load and self-weight)

Design of Post-tensioning Force and Profile:

The design prestress force F_p will be chosen to match the whole number of prestressing strands.

$$F \text{ per strand} = f_{se} \times A = 1200 \times 150 \times 10^{-6} = 180 \text{ kN/strand}$$

$$\text{Required level of precompression: } 1 \text{ N/mm}^2 \times 9.5 \times 10^5 \text{ mm}^2 / 1000 = 950 \text{ kN}$$

$$N \text{ of strands} = 950 / 180 = 5.3 \Rightarrow 6 \text{ strands selected}$$

$$F_p = 6 \times 180 = 1080 \text{ kN (preliminary)}$$

Tendon profiles follow a simple parabolic curve and the position of low point in each of 3 spans is chosen so that a uniform upward force is generated. For 1 and 3 spans it follows a shape as demonstrated by the figure below. For span 2 (middle span), the lowest point coincides with the midspan. The real profile during construction will be smoother and resemble a reversed parabola rather than a sharp parabolic curve.

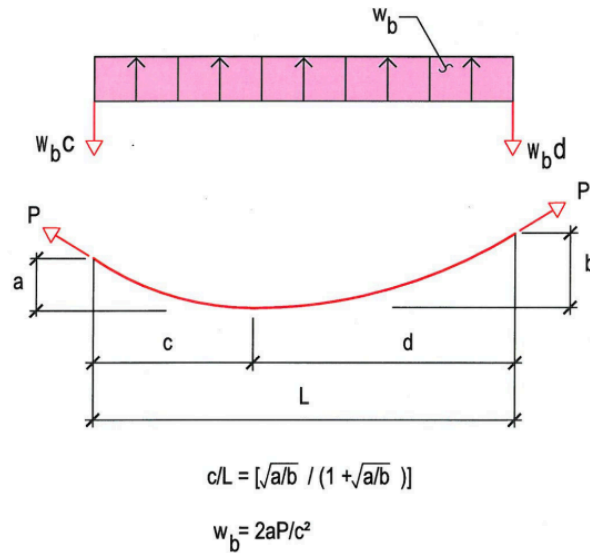


Figure 3.11.1.3. Geometry and actions of a parabolic tendon

Distance from the left highest tendon point to the lowest point located at CGS=70 mm:

$$a = Y_b - 70 = 620 \text{ mm (e=290 mm)}$$

$$b = 800 + 130 - 70 - 70 = 790 \text{ mm}$$

$$L = 20 \text{ m}$$

$$c = \left(\frac{\sqrt{620/790}}{1 + \sqrt{620/790}} \right) \times 20000 = 9386.3 \text{ mm} = 9400 \text{ mm}$$

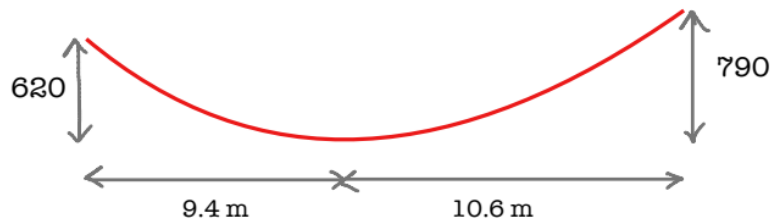


Figure 3.11.1.4. Preliminary tendon profile

Upward force due to post-tensioning: $w_b = 1080 \times 2 \times 0.590/9.4 = 15.1 \text{ kN/m}$

% balancing = $15.1/30.13 = 50\% < 80\%$ (target)

Adj. N of strands = $\frac{80\%}{50\%} \times 6 = 9.55 \Rightarrow \text{choose 10 strands}$

$$F_p = 10 \times 180 = 1800 \text{ kN}$$

$$w_b = 1800 \times 2 \times 0.59/9.4 = 25.25 \text{ kN/m (balances 84\% of DL)}$$

Since the loading is uniform across 3 spans and supports are fixed, b was equaled to a, resulting in a simple parabolic profile; additionally, the software calculated tendon profile suggested that the highest tendon point was at $a = 635 \text{ mm (e=305 mm)}$ and $c = 9 \text{ m}$, which is close to hand design. Following SAP2000 results, lowest point CGS was also adjusted from

70 mm to 95 mm. As a result of the synthesis of hand and software design, the final tendon profile is presented below:

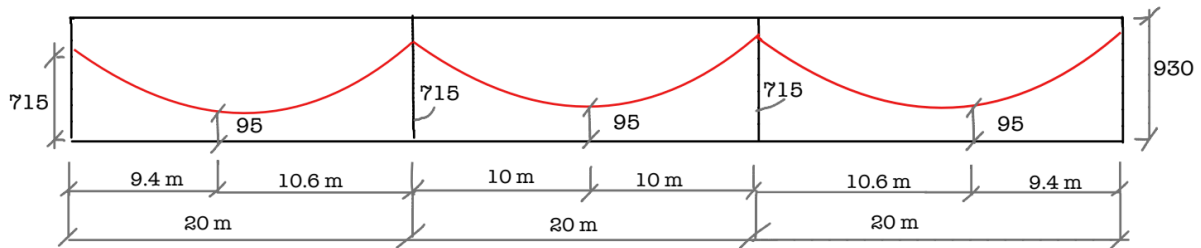


Figure 3.11.1.5. Final tendon profile

Stress check:

Using SAP2000, the moments required to conduct stress and deflection checks were extracted.

Load combinations for these checks based on ACI, IBC are as follows:

Total Load condition: $1DL + 1LL + 1PT$

Sustained Load Condition: $1DL + 0.3LL + 1PT$

Table 3.11.1.2. Actions due to Dead, Live and Post-Tensioning loads

Point	M_{DL}	M_{LL}	M_{PT}
A	538.6	428.1	-388.8
B	-1067.4	-848.5	770.5
C	-1007	-800	727.2
D	499	396.7	-360

Stress was calculated at selected points of interest shown in Figure 3.11.1.6 using the formulas below:

$$\sigma_{total-load} = (M_{DL} + M_{LL} + M_{PT})/S + F_p/A$$

$$\sigma_{sust-load} = (M_{DL} + 0.3M_{LL} + M_{PT})/S + F_p/A$$

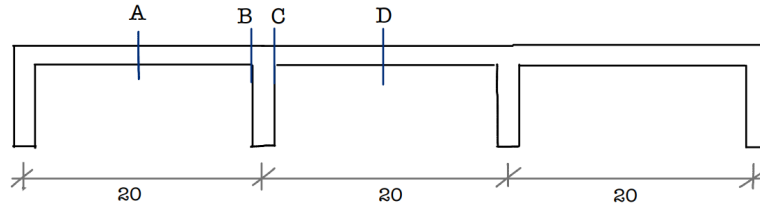


Figure 3.11.1.6. Points of stress check

Example calculation for point A:

$$\sigma_{top} = - (538.6 + 428.1 - 388.8) * 1000^2 / (1.93 \times 10^8) - 1800 / (7.35 \times 10^5)$$

$$= -6.48 \text{ MPa} < -18 \text{ MPa}$$

$$\sigma_{bot} = (538.6 + 428.1 - 388.8) * 1000^2 / (1 \times 10^8) - 1800 / (7.35 \times 10^5)$$

$$= 0.87 \text{ MPa} < 6.32 \text{ MPa}$$

Table 3.11.1.3. Stress check at critical points

	Total load combo		Sustained load combo	
Point	Stress at top fiber	Stress at bottom fiber	Stress at top fiber	Stress at bottom fiber
A	-6.48 (C)	4.92 (T)	-4.62 (C)	0.87 (T)
B	4.22 (T)	-18.4 (C)	0.53 (T)	-10.3 (C)
C	3.82 (T)	-17.5 (C)	0.34 (T)	-9.92 (C)
D	-6.2 (C)	4.35 (T)	-4.50 (C)	0.60 (T)
Allowable	$\sigma_c < 24 \text{ MPa}; \sigma_t < 6.32 \text{ MPa}$		$\sigma_c < 18 \text{ MPa}; \sigma_t < 6.32 \text{ MPa}$	

Since tensile stresses are within the transition condition range, the stress check is passed and now deflections calculated in SAP2000 can be used for further check. The instantaneous live load deflection and long term deflections (due to prestress, dead load and fraction of live load):

$$\Delta_{LT} = (1 + t)(\Delta_{DL} + \Delta_{PT} + 0.3\Delta_{LL}) = (1 + 2)(40.76 - 29.4 + 32.6) =$$

$$= 62.4 < L/240 = 83.33 \text{ mm}$$

$$\Delta_{LL} = 32.6 \text{ mm} < L/480 = 41.67 \text{ mm}$$

The design passes the deflection checks.

It is necessary to conduct a rough serviceability check to ensure that the section's cracking moment does not exceed its nominal capacity in flexure. This procedure involves the following formulas:

$$M_{cr} = (f_r + P/A) \times S ,$$

where $f_r = 0.625\sqrt{f'_c} = 0.625 \times \sqrt{40} = 3.95 \text{ MPa}$ and P/A is the average precompression.

The outcome of computation of cracking moment and design capacity of tendon is presented below.

Table 3.11.1.4. Rough estimate of capacity satisfied by tendon alone

Location along beam	End	Middle
CGS, mm	110	95
$d = h_{beam} + h_{slab} - CGS$, mm	820	835
$Moment Arm(d') = 0.9d$, mm	738	751.5
$\phi M_n = \phi A_{ps} f_{ps} d'$, kN-m	1482.5	1509.6

Table 3.11.1.5. Basic parameter analysis

Basic Parameter Analysis	Span's end faces	Span's middle
S_{top} , mm^3	1.93E+08	0
S_{bot} , mm^3	0	1.00E+08
P (kN)	1800	1800
A, mm^2	734500	734500
$f_r + (P/A)$, MPa	6.40	6.40
M_{cr} , kN-m	1233.13	640.51
$1.2M_{cr}$, kN-m	1479.76	768.61
ϕM_n	1482.5	1509.6
Status	OK	OK

Since the flexural design capacity provided by the tendon alone is more than cracking moment, no additional reinforcement is needed from this provision specifically. This check shows that cracking will occur before reaching ultimate strength and the beam's response is ductile.

Modeling in SAP2000

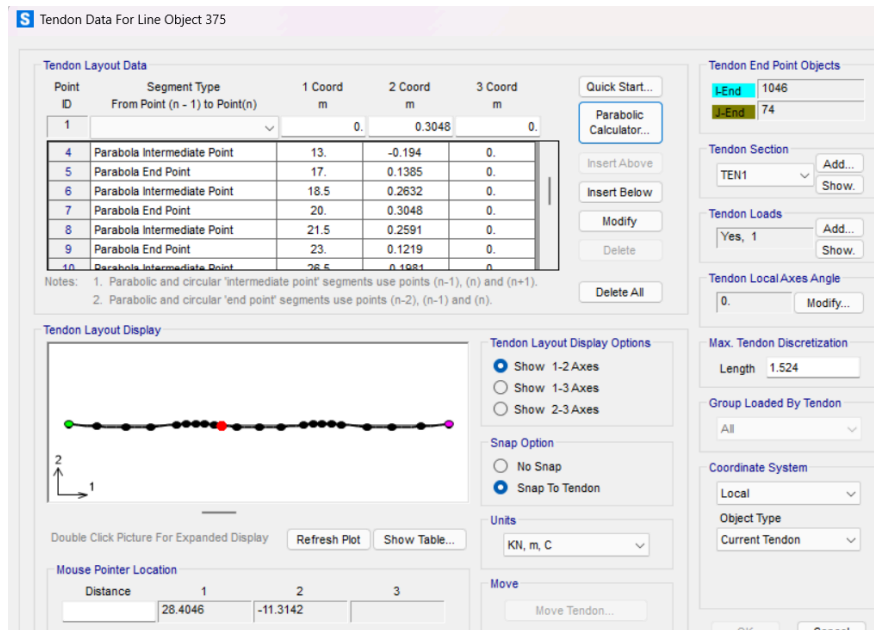


Figure 3.11.1.7. Tendon definition

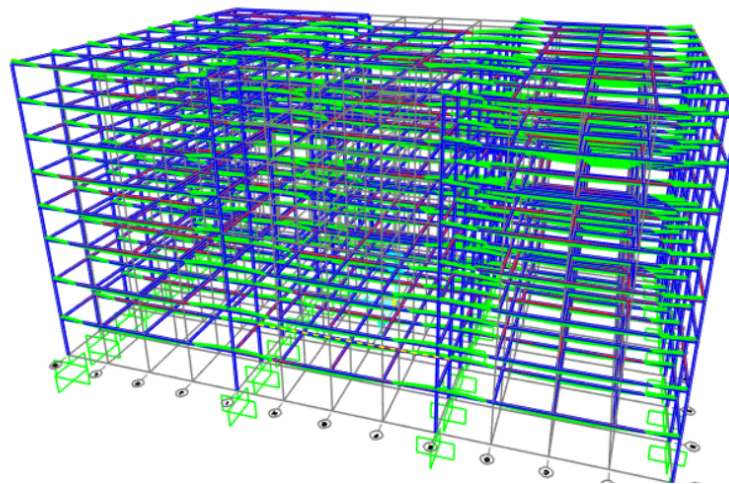


Figure 3.11.1.8. Modeling tendons

Reinforcement:

Now, to design the mild steel reinforcement required to resist design moment M_u created by factored loads, the minimum required passive reinforcement is equal to:

$$A_s = 0.004 \times A_{tens}$$

Top bars at supports:

$$A_{s-min} = 0.004 \times (130 \times 2580 + (317.92 - 130) \times 500) = 1717.5 \text{ mm}^2$$

Bottom bars at midspans:

$$A_{s-min} = 0.004 \times 612.08 \times 500 = 979.32 \text{ mm}^2$$

Since prestress force provided by tendons is already explicitly introduced as a separate load case and included in all of the load combinations, the design moments reported by the program are the residual (net) moments after accounting for the counteracting effects of the prestressing - secondary moments created by tendons. Therefore, the required reinforcement area displayed by SAP2000 corresponds only to passive reinforcement because the tendon's contribution is already incorporated.

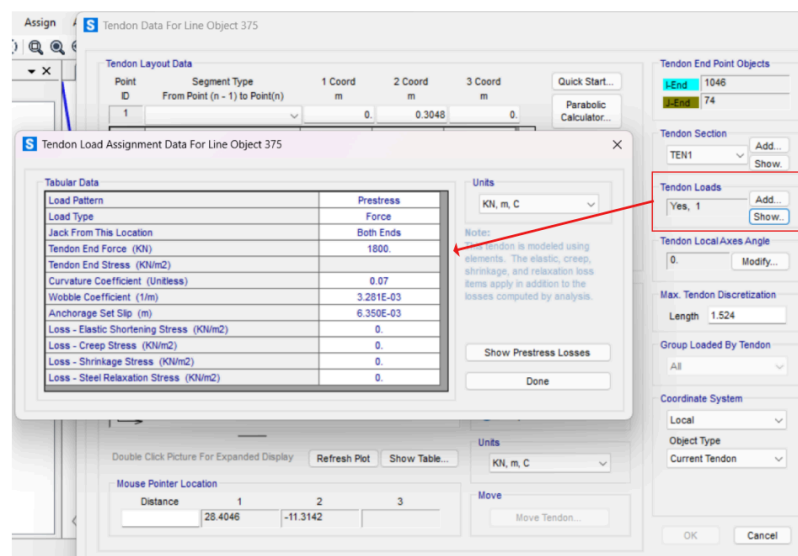


Figure 3.11.1.9. Definition of tendon (jacking) force as load pattern

Define Combination of Load Case Results			
Load Case Name	Load Case Type	Mode	Scale Factor
Prestress1	Linear Static		1.
DEAD_total	Linear Static		1.2
LIVE_FLOOR	Linear Static		1.6
LIVE_ROOF	Linear Static		0.5
Prestress1	Linear Static		1.

Figure 3.11.1.10. Adding prestress to loading combinations (e.g. COMB_2)

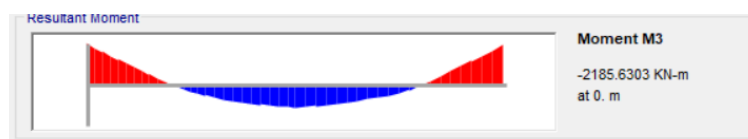


Figure 3.11.1.11. Critical moment demand with and without prestressing action

Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	886382.356	-1772764.71	886382.356	-1772764.71
Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	7203.968	0.	7203.968	1387.916
Bottom (-2 Axis)	3356.936	3356.936	0.	1387.916

Figure 3.11.1.12. Critical moment after introduction of the prestress and passive reinforcement requirement

Still, it was checked how the tendon and passive reinforcement resist the total moment together as follows:

$$f'_c = 5800 \text{ psi}, E_s = 4308 \text{ ksi}$$

$$f_{ps} = 270 \text{ ksi}, f_y = 60 \text{ ksi}$$

$$A_s = 7203 \text{ mm}^2 = 11.16 \text{ in}^2 \text{ (taken from SAP2000)}$$

$$A_{ps} = 10 \times 150 = 1500 \text{ mm}^2 = 2.33 \text{ in}^2$$

$$d_s = 800 - 60 = 740 \text{ mm} = 29.13 \text{ in}$$

$$d_{ps} = 800 - 95 = 705 \text{ mm} = 27.76 \text{ in}$$

$$T = A_s f_y + A_{ps} f_{ps} = 11.16 \times 60 + 2.33 \times 270 = 959.48 \text{ K}$$

$$a = \frac{T}{0.85 f'_c b_w} = \frac{959.48}{0.85 \times 5800 \times 19.69} = 9.89 \text{ in}$$

$$M_n = A_s f_s (d_s - \frac{a}{2}) + A_{ps} f_{ps} (d_{ps} - \frac{a}{2}) = 11.16 \times 60 \times (29.13 - \frac{9.89}{2}) + 2.33 \times 270 \times (27.76 - \frac{9.89}{2}) = 22345.3 \text{ K} - \text{in} = 2524.68 \text{ kN} - \text{m}$$

$$\phi M_n = 0.9 \times 2524.68 = 2272.21 \text{ kN} - \text{m} > M_u = 2185 \text{ kN} - \text{m}$$

Table 3.11.1.6. Passive reinforcement selection for prestressed major beam (Floor 1-2)

Span	1 Span			2 Span			3 Span		
Loc.	End	Mid	End	End	Mid	End	End	Mid	End
Top	8#10	5#10	9#10	9#10	2#10	9#10	9#10	5#10	8#10
As (provided)	10.16	5.08	11.43	11.43	2.54	11.43	11.43	5.08	10.1600
Bottom	4#10	6#10	5#10	5#10	5#10	5#10	5#10	6#10	4#10

As (provided)	5.08	7.62	6.35	6.35	6.35	6.35	6.35	7.62	5.08
Stirrups	#4@ 170 mm	#4@ 370 mm	#4@ 170 mm	#4@ 200 mm	#4@ 550 mm	#4@ 200 mm	#4@ 170 mm	#4@ 370 mm	#4@ 170 mm

Table 3.11.1.7. Passive reinforcement selection for prestressed major beam (Floor 3-8)

Span	1 Span			2 Span			3 Span		
Loc.	End	Mid	End	End	Mid	End	End	Mid	End
Top	5#10	2#10	6#10	6#10	2#10	6#10	6#10	2#10	5#10
As (provided)	6.35	2.54	7.62	7.62	2.54	7.62	7.62	2.54	6.35
Bottom	3#10	5#10	3#10	3#10	3#10	3#10	3#10	5#10	3#10
As (provided)	3.81	5.08	3.81	3.81	3.81	3.81	3.81	5.08	3.81
Stirrups	#4@ 300 mm	#4@ 550 mm	#4@ 250 mm	#4@ 250 mm	#4@ 550 mm	#4@ 250 mm	#4@ 250 mm	#4@ 550 mm	#4@ 300 mm

Permissible range for reinforcements is given by:

$$A_{s,min} = \max(0.25\sqrt{f'_c}bd/f_y; 1.4bd/f_y) = 2.087 \text{ in}^2$$

$$A_{s,max} = 4\% * bd = 22.13 \text{ in}^2$$

As can be seen from the tables, all reinforcement selections are acceptable and are not governed by minimum.

3.11.1.1. Cutting length

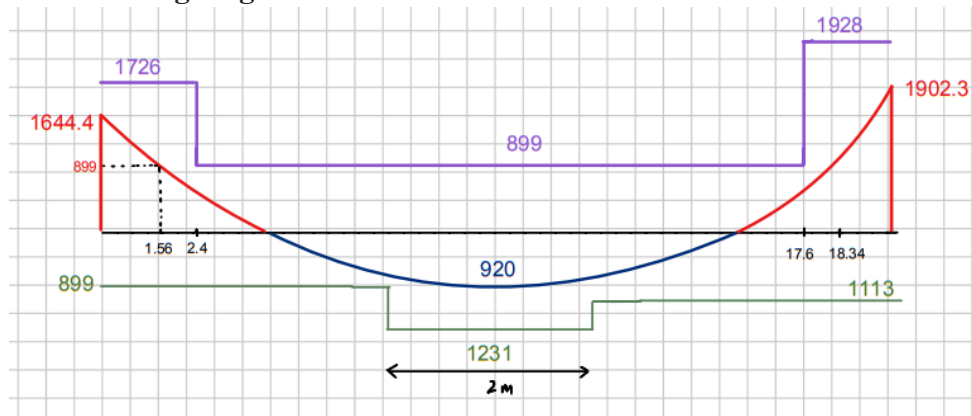


Figure 3.11.1.1.1 Moment demand and capacity diagram

The demand is taken from software. The capacity was estimated using selected bar area and formula:

$$\phi M_n = A_s f_s (d_s - \frac{a}{2})$$

Additional top reinforcement is provided only where negative moment demand is highest, and that is over supports. The bars are not terminated at the theoretical cut-off point; they are extended beyond by not less than:

$$l_{cutoff} = x + \max(d; 12d_b) = x + \max(740, 12 \times 32.26) = x + 740 \text{ mm}$$

The length x was found from SAP2000 and cutting lengths were calculated for top reinforcement as:

$$l_{cut-left} = 1.563 + 0.74 = 2.303 \approx 2.4 \text{ m}$$

$$l_{top-right} = (20 - 18.34) + 0.74 = 2.4 \text{ m}$$

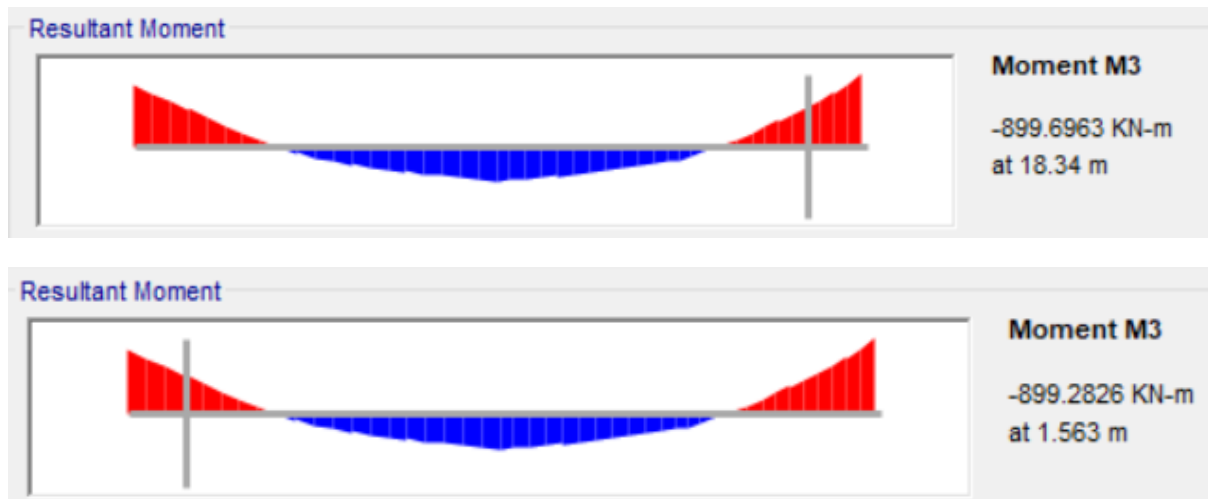


Figure 3.11.1.1.2. Theoretical cutoff point

Similarly, for bottom reinforcement, the cutting length was found to be slightly less, 2 meters.

3.11.2. Non-prestressed major beams

3.11.2.1. Flexural analysis

ACI 318-14 BEAM SECTION DESIGN Type:Sway Special Units: KN, mm, C (Summary)				
Element : 2057	D=450.	B=400.	bf=400.	
Section ID : Major_beam_5	ds=0.	dct=60.	dcb=60.	
Combo ID : COMB4_3	E=29.725	fc=0.04	Lt.Wt. Fac.=1.	
Station Loc : 5000.	L=5000.	Fy=0.42	fys=0.42	
Phi(Bending):	0.9			
Phi(Shear):	0.75			
Phi(Seis Shear):	0.6			
Phi(Torsion):	0.75			
Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	78719.824	-157439.647	78719.824	-157439.647
Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	1117.402	0.	1117.402	585.175
Bottom (-2 Axis)	585.175	545.777	0.	585.175
Shear Reinforcement for Shear, V2				
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs
	0.381	92.866	122.887	46.083
				Shear Vp
				66.496
Reinforcement for Torsion, T				
	Rebar At/s	Rebar Al	Factored Tu	Design Tu
	0.	0.	879.887	879.887
				Threshold Phi*Tth
				7506.662
				Area Ac
				95487.479
				Perimeter Ph
				1344.4

Figure 3.12.2.1.1. Theoretical cutoff point

For negative moment: $M_u = -157.439 \text{ kN} - m$

$$b = 400 \text{ mm}, h = 450 \text{ mm}, d = 450 - 60 = 390 \text{ mm}$$

$$f'_c = 40 \text{ MPa (5801 psi)}, f_y = 420 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{157.439}{0.9 \cdot 0.4 \cdot 0.39^2} = 2875.29 \text{ kN/m}^2$$

$$\rho = \frac{0.85 f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2 R_n}{0.85 f'_c}} \right] = 0.007163$$

$$A_s = \rho b d = 0.007163 \cdot 400 \cdot 390 = 1117.4 \text{ mm}^2$$

The calculated value matches the SAP2000 value exactly; choose #9 bars

$$A_s f_y = 0.85 f'_c a b \rightarrow a = 34.51 \text{ mm}$$

$$\beta_1 = 0.85 - 0.05\left(\frac{5801-4000}{1000}\right) = 0.76 > 0.65$$

$$c = \frac{a}{\beta_1} = 45.41 \text{ mm}$$

$$\varepsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{390-45.41}{45.41}\right) = 0.0227 > 0.005 \rightarrow \text{tension controlled and ductile}$$

$$A_{s,min} = 0.25\sqrt{f'_c}bd/f_y \geq 1.4bd/f_y = 587.3 \text{ mm}^2 > 520 \text{ mm}^2$$

$$A_s = 1117.4 \text{ mm}^2 > 587.3 \text{ mm}^2$$

Similarly, for positive moment: $A_s = 585.175 \text{ mm}^2 \Rightarrow$ Choose #6 bars

3.11.2.2. Shear analysis

Maximum shear force, $V_u = 92.86 \text{ kN}$, $\lambda = 1$

$$\phi V_c = \phi \cdot 2\lambda\sqrt{f'_c}b_w d = 0.75 \times 2 \times \sqrt{5801} \times 15.748 \times 15.35$$

$$= 27.623 \text{ kip} = 122.87 \text{ kN} > V_u = 92.86 \text{ kN}$$

$$V_u > \frac{\phi V_c}{2} = 61.43 \text{ kN} \Rightarrow \text{minimum shear reinforcement (\#3 bar)}$$

$$V_{c1} = 4\sqrt{f'_c}b_w d = 327.7 \text{ kN}, V_{c2} = 8\sqrt{f'_c}b_w d = 655.31 \text{ kN}$$

$$\frac{A_{v-min}}{s} = 0.062 \frac{\sqrt{f'_c} \times b_w}{f_y} = 0.373$$

$$V_s = \frac{A_{v-min}}{s} f_y d = 45.88 \text{ kN} < V_{c1} \Rightarrow s = \frac{d}{2} = \frac{390}{2} = 195 \text{ mm} \approx 200 \text{ mm}$$

Choose #3 bars at 200 mm

3.11.2.3. Torsional analysis

$$T_n = \phi \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) = 1.03 \text{ kN} - m > T_u = 0.88 \rightarrow \text{no torsional reinforcement is needed}$$

Table 3.12.2.3.1. Bar selection for 5-meter major beam

			END	MID	END
1-2 floors	Top	A_s (required)	2.604	0.744	2.604
		A_s (provided)	3.0	2.0	3.0
	Bottom	A_s (required)	1.256	0.744	1.256
		A_s (provided)	1.3	1.3	1.3
	Top	Selected bars	3#9	2#9	3#9
	Bottom	Selected bars	3#6	3#6	3#6
3-8 floors	Top	A_s (required)	1.860	0.671	1.860
		A_s (provided)	2.0	2.0	2.0
	Bottom	A_s (required)	0.946	0.671	0.946
		A_s (provided)	1.2	1.2	1.2
	Top	Selected bars	2#9	2#9	2#9
	Bottom	Selected bars	2#7	2#7	2#7

Permissible range for reinforcements is given by:

$$A_{s,min} = \max(0.25\sqrt{f'_c}bd/f_y; 1.4bd/f_y) = 1 \text{ in}^2$$

$$A_{s,max} = 4\% * bd = 9.7 \text{ in}^2$$

Some parts are governed by minimum requirements.

3.11.3. Minor beam

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

Element	: 1873	D=350.	B=350.	bf=350.
Section ID	: Minor_beam	ds=0.	dct=60.	dcb=60.
Combo ID	: COMB2	E=29.725	fc=0.04	Lt.Wt. Fac.=1.
Station Loc	: 2500.	L=5000.	Fy=0.42	fys=0.42

Phi(Bending):	0.9
Phi(Shear):	0.75
Phi(Seis Shear):	0.6
Phi(Torsion):	0.75

Design Moments, M3				
	Positive Moment	Negative Moment	Special +Moment	Special -Moment
	71363.002	-16011.922	4043.576	-16011.922

Flexural Reinforcement for Moment, M3				
	Required Rebar	+Moment Rebar	-Moment Rebar	Minimum Rebar
Top (+2 Axis)	196.519	0.	147.389	196.519
Bottom (-2 Axis)	679.064	679.064	0.	380.739

Shear Reinforcement for Shear, V2					
	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
	0.	0.095	79.955	0.	28.987

Reinforcement for Torsion, T							
	Rebar At/s	Rebar Al	Factored Tu	Design Tu	Threshold Phi*Ith	Area Ao	Perimeter Ph
	0.186	583.473	6681.751	6681.751	4221.773	57947.228	1044.4

Figure 3.11.3.1. Minor beam section design (middle)

3.11.3.1. Flexural analysis

For negative moment: $M_u = -16 \text{ kN} - m$

$$b = 350 \text{ mm}, h = 350 \text{ mm}, d = 350 - 60 = 290 \text{ mm}$$

$$f'_c = 40 \text{ MPa (5801 psi)}, f_y = 420 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{16}{0.9 \cdot 0.35 \cdot 0.29^2} = 603.97 \text{ kN/m}^2$$

$$\rho = \frac{0.85 f'_c}{f_y} \left[1 - \sqrt{1 - \frac{2 R_n}{0.85 f'_c}} \right] = 0.00145$$

$$A_s = \rho b d = 0.00145 \cdot 350 \cdot 350 = 147.9 \text{ mm}^2$$

$$A_{s,min} = 0.25 \sqrt{f'_c} b d / f_y \geq 1.4 b d / f_y = 196.5 \text{ mm}^2$$

$$\Rightarrow A_s = 196.5 \text{ mm}^2 \Rightarrow \text{Choose 2\#4 bars (251.6 mm}^2)$$

Similarly, **for positive moment:** $A_s = 675.5 \text{ mm}^2 \Rightarrow \text{Choose 2\#7 (774 mm}^2)$

3.11.3.2. Shear analysis

Maximum shear force, $V_u = 0.1 \text{ kN}$, $\lambda = 1$

$$\phi V_c = \phi \cdot 2\lambda \sqrt{f'_c} b_w d = 79.95 \text{ kN} > V_u$$

$$V_u < \frac{\phi V_c}{2} \Rightarrow \text{no shear reinforcement required}$$

3.11.3.3. Torsional analysis

$$A_{cp} = 350 \times 350 = 0.1225 \text{ m}^2, P_{cp} = 350 \times 4 = 1.4 \text{ m}$$

$$T_n = \phi \lambda \sqrt{f'_c} \left(\frac{A_{cp}^2}{P_{cp}} \right) = 0.75 \times 1.0 \times \sqrt{5800} \times \left(\frac{189.87^2 \text{ in}^2}{55.12 \text{ in}} \right) = 37.36 \text{ K} - \text{in} =$$

$$= 4.22 \text{ kN} - \text{m} < T_u = 6.681 \text{ kN} - \text{m} \rightarrow \text{torsional reinforcement is needed}$$

$$A_o = [13.7795 - 2 \times (2.36 + 0.375)] \times [13.7795 - 2 \times (2.36 + 0.375)] =$$

$$= 88.93 \text{ in}^2 \text{ or } 57372.22 \text{ mm}^2$$

$$P_h = 1.04 \text{ m}$$

Section can be designed for torsion since it satisfies the requirement:

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

$$\frac{A_t}{s} = \frac{T_n}{2 A_o f_{yt} \cot \theta} = \frac{59.132}{2 \times 88.93 \times 60 \times 0.75} = 0.0074 \text{ in}^2 / \text{in} = 0.1876 \text{ mm}^2 / \text{mm}$$

The spacing for #3 stirrups:

$$s = \frac{2 \times 71}{0.1876} = 760 \text{ mm} \rightarrow 140 \text{ mm} \text{ since minimum is given by:}$$

$$s_1 = 305 \text{ mm}, s_2 = P_h / 8 = 140 \text{ mm}$$

Provided reinforcement is more than minimum:

$$A_{vt} = 0.75 \sqrt{f'_c} \frac{b_w s}{f_{yt}} \geq 50 \frac{b_w s}{f_{yt}} \rightarrow 0.0723 \text{ in}^2 > 0.062 \text{ in}^2$$

$$A_v = 0.22 \text{ in}^2 > 0.072 \text{ in}^2 \rightarrow \text{sufficient}$$

3.11.4. Columns

Reinforced concrete column design was carried out using the ACI 318 Code. One example column was selected to show how the calculations were performed, as provided below. For this purpose, an exterior column measuring 900 mm × 900 mm was chosen. In the figure 3.11.4.2, a SAP2000 screenshot is presented showing the given moments and shear forces used for the design.

One important step in designing the column is to check whether the column is to be designed as a slender or short column, to magnify moments if necessary. Effective length of the column will be determined by using alignment charts from ACI 318 as in figure below. Effective inertia for the beam is 0.35I_g and for column 0.7 I_g. The calculations will be done assuming non-sway frames.

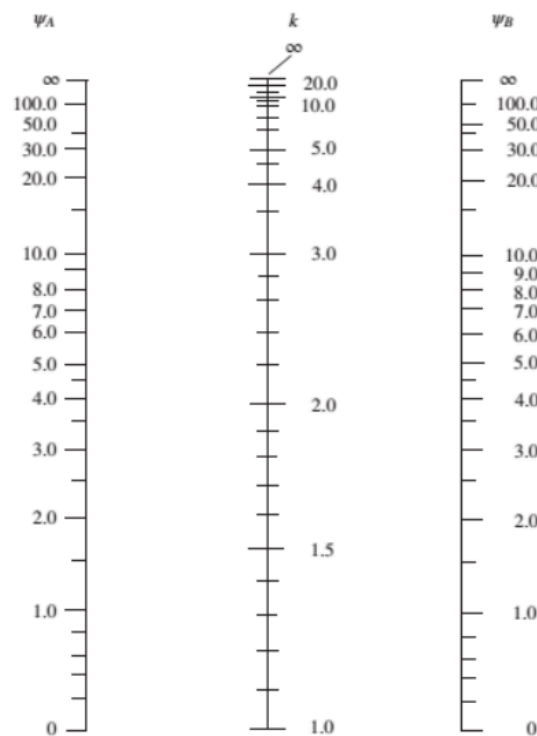
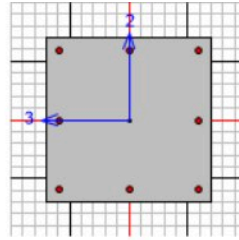


Figure 3.11.4.1. Columns beginning

Hand calculations will be done for the following column example from figure 3.11.4.2.



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

Element : 9 B=900. D=900. dc=67.026
 Section ID : Column_1_3_exterior E=29.725 fc=0.04 Lt.Wt. Fac.=1.
 Combo ID : COMB6_2 L=4500. Fy=0.42 fys=0.42
 Station Loc : 4500. RLLF=1.

Phi(Compression-Spiral): 0.75
 Phi(Compression-Tied): 0.65
 Phi(Tension Controlled): 0.9
 Phi(Shear): 0.75
 Phi(Seismic Shear): 0.6
 Phi(Joint Shear): 0.85

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

Rebar Area	Design Pu	Design M2	Design M3	Minimum M2	Minimum M3
8100.	3223.702	346787.603	236809.202	136169.179	136169.179

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Cm Factor	Delta_ns Factor	Delta_s Factor	K Factor	L Length
Major Bending(M3)	0.4	1.	1.	1.	4500.
Minor Bending(M2)	0.726	1.	1.	1.	4500.

SHEAR DESIGN FOR V2,V3

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
Major Shear(V2)	0.	100.531	760.989	0.	453.866
Minor Shear(V3)	0.	168.129	760.989	0.	80.948

Figure 3.11.4.2. Column section design

Slender/Short Column Check

- a. Exterior Column (900x900 mm, $L_c = 4500\text{mm}$)

$$I_c = 0.7 \frac{900 \cdot 900^3}{12} = 38.27 \times 10^9 \text{ mm}^4$$

$$\left(\frac{EI}{L} \right)_c = \frac{38.27 \times 10^9}{4500} = 8.505 \times 10^6$$

- Beams

$$Mj_1 \text{ beam } (800 \times 500 \text{ mm}, L_c = 20000 \text{ mm})$$

$$Mj_2 \text{ beam } (450 \times 400 \text{ mm}, L_c = 5000 \text{ mm})$$

$$I_{b1} = 0.35 \frac{500 \cdot 800^3}{12} = 7.467 \times 10^9 \text{ mm}^4$$

$$\left(\frac{EI}{L}\right)_{b1} = \frac{7.467 \times 10^9}{20000} = 3.733 \times 10^5$$

$$I_{b2} = 0.35 \frac{400 \cdot 450^3}{12} = 1.063 \times 10^9 \text{ mm}^4$$

$$\left(\frac{EI}{L}\right)_{b2} = \frac{1.063 \times 10^9}{5000} = 2.126 \times 10^5$$

- Effective length factor

$$\psi_a = \psi_b = \frac{2(EI/L)_c}{(EI/L)_{b1} + 2(EI/L)_{b2}} = 21.30$$

$$l_u = 4500 - 0.5(450 + 800) = 3875 \text{ mm}$$

$$r = 0.3$$

$$h = 0.3(900) = 270 \text{ mm}$$

$$k \text{ (chart)} = 4.1$$

The slenderness ratio:

$$\frac{kl_u}{r} = \frac{4.1 \times 3875}{270} = 58.84 > 22$$

The column is treated as Slender.

3.11.4.1. Moment Magnification

For concrete strength $f'_c = 40 \text{ MPa}$,

$$E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{40} = 29725 \text{ MPa}$$

$$I_g = \frac{900 \cdot 900^3}{12} = 54.675 \times 10^9 \text{ mm}^4$$

$$\beta_{dns} = 0.6$$

$$(EI)_{eff} = \frac{0.4E_c I_g}{1 + \beta_{dns}} = \frac{0.4(29725)(54.675 \times 10^9)}{1.6} = 4.063 \times 10^{14} \text{ N}\cdot\text{mm}^2$$

$$P_c = \frac{\pi^2 (EI)_{eff}}{(kl_u)^2} = 15887 \text{ kN}$$

From the SAP output, the non-sway moment coefficients are taken as

$$C_{m,x} = 0.726, C_{m,y} = 0.40$$

$$\delta_x = \frac{C_{m,x}}{1 - \frac{P_u}{0.75P_c}} = \frac{0.726}{1 - \frac{3223.702}{0.75(15887)}} = 0.995$$

$$\delta_y = \frac{C_{m,y}}{1 - \frac{P_u}{0.75P_c}} = \frac{0.40}{1 - \frac{3223.702}{0.75(15887)}} = 0.548$$

ACI requires $\delta > 1$, so 1 will be used and the governing moments remain

$$M_{cx} = 346.788 \text{ kNm}$$

$$M_{cy} = 236.809 \text{ kNm}$$

3.11.4.2. Flexural & Axial Capacity

$$\gamma = \frac{900-127}{900} \sim 0.859$$

$$P_n = \frac{P_u}{\phi} = \frac{3223.702}{0.65} = 4959.54 \text{ kN}$$

$$K_n = \frac{P_n}{f'_c A_g} = 0.153$$

$$R_n = \frac{P_n}{f'_c A_g} \frac{\delta e}{h} = 0.183$$

Using column interaction diagram, we obtain:

$$A_g = 12.70 \text{ in}^2 = 8194 \text{ mm}^2$$

Thus 8#10 vertical reinforcement should be used

3.11.4.3. Shear Design & Capacity

$$\phi V_n = \phi(V_c + V_s) > V_u$$

$$V_u = 166.95 kN$$

$$\phi = 0.75$$

$$V_c = 2(f'_c)^{0.5} A_g = 467 kN$$

$$V_s = \left(d \frac{f_y A_v}{s}, 4 * V_c \right) = 333.2 kN$$

$$\phi V_n = 600.5 kN \text{ (safe)}$$

$$A_v = 0.11 in^2 = 71 mm^2$$

$$d = 9.53 mm$$

$$s = 10 in = 254 mm$$

$$(16d_b, 48d_{tie}, b) = 457.4$$

$$254 mm < 457.4 mm$$

Stirrup design 2 legs # 3 @ 254 mm

3.11.4.4. Biaxial check

The nominal concentric axial capacity is

$$P_0 = 0.85 f'_c A_g + A_s (f_y - 0.85 f'_c)$$

$$P_0 = 0.85(40)(810000) + 8194(420 - 0.85 \times 40) = 30702.9 kN$$

Using Bresler's reciprocal-load equation,

$$\frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_0} \frac{1}{P_n} = \frac{1}{29270} + \frac{1}{29665} - \frac{1}{30702.9}$$

$$P_n \approx 28325 kN$$

The maximum concentric compression cap:

$$\phi P_{n,max} = 0.65 * 0.80 < P_u = 3223.7 kN$$

$$P_0 = 0.65 * 0.80(30702.9) = 15965.5 kN$$

The Column is safe in biaxial bending

3.11.4.5. Final Columns Design

The design approach was applied to all columns. On each floor, the column with the highest load and moment was selected. The following table presents the design:

Table 3.11.4.5. Design

Floor	P _u , kN	M ₃ , kNm	M ₂ , kNm	V ₂ , kN	V ₃ , kN	b, mm	d, mm	L, mm	Vertical reinforc ement	Shear reinforcem ent
1	3224	346788	236809	101	168	900	900	4500	10#10	2 legs, #3@10 in
2	2777	-117327	360809	185	153	900	900	4500	10#10	2 legs, #3@10 in
3	2660	-299950	112365	16	144	850	850	3500	10#10	2 legs, #3@10 in
4	1208	-147678	301751	125	41	850	850	3500	10#10	2 legs, #3@10 in
5	939	-175700	-23347 5	152	29	850	850	3500	10#10	2 legs, #3@10 in
6	703	-155225	-24590 8	147	11	800	800	3500	8#10	2 legs, #4@10 in
7	450	-123042	-20174 6	137	3	800	800	3500	8#10	2 legs, #4@10 in
8	319	104580	870459	425	53	800	800	3500	8#10	2 legs, #4@10 in

Following figures are reinforcement detail drawings made for columns.

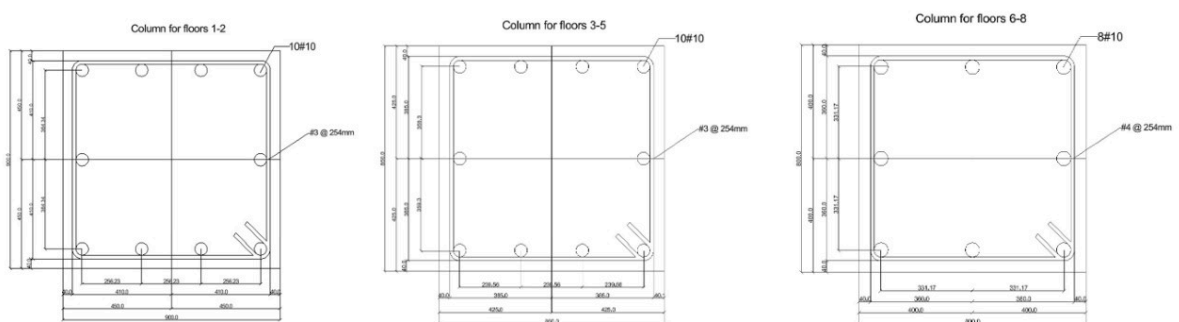


Figure 3.11.4.5.1 Reinforcement detail drawings for columns

3.11.5. Two way slab

Previously, the thickness of the slab was found as 130 mm using average value of α_{fm} of beams supporting the slab panel. The slab panel bound by beams is 5000 mm x 5000 mm.

$$\bar{y} = \frac{102700 \times 65 + 77000 \times 240}{102700 + 77000} = 140 \text{ mm}$$

$$I_{beam} = \frac{790 \times 130^3}{12} + 102700(140 - 65)^2 + \frac{350 \times 220^3}{12} + 77000(240 - 140)^2 = 1.8 \times 10^9 \text{ mm}^4$$

$$I_{slab} = \frac{5000}{2} * \frac{130^3}{12} = 4.57 \times 10^8 \text{ mm}^4$$

$$\alpha_{fm} = \alpha_f = \frac{EI_b}{EI_s} = \frac{4.57 \times 10^8}{1.8 \times 10^9} = 3.94$$

$$h_{min} = \frac{l_n(0.8 + f_y/1400)}{36 + 9\beta} = \frac{4825 \times (0.8 + 420/1400)}{36 + 9 \times 1} = 118 \text{ mm}, \text{ therefore } h = 130 \text{ mm can be kept}$$

$$\alpha_{f1} = \alpha_s = 3.94 \geq 1.0; \quad \frac{l_2}{l_1} = 1$$

3.11.5.1. Flexural analysis

The section 5.3.4 Direct Design Method for two-way beam-supported slabs of ACI 318-19 was employed for these calculations. Since there are no major column offsets, 3 or more consecutive spans in each direction and uniformly distributed loads satisfy the condition $LL/DL \leq 2$, the DDM can be used. The total factored static moment, M_o , in each span is evaluated as:

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(1.2 \times 4.76 + 1.6 \times 4.79) \times 5 \times 4.1^2}{8} = 140.53 \text{ kN} - \text{m}$$

This moment was distributed among slab strips according to Table 5.19 of ACI 318-19 of section 5.3.4 Direct Design Method. These coefficients are shown below.

Span Ratio, ℓ_2 / ℓ_1	Slab and Beam Moments		End Span			Interior Span	
			①	②	③	④	⑤
			Exterior Negative	Positive	First Interior Negative	Positive	Interior Negative
	Total Moment		$0.16M_o$	$0.57M_o$	$0.70M_o$	$0.35M_o$	$0.65M_o$
1.0	Column Strip	Beam*	$0.10M_o$	$0.37M_o$	$0.45M_o$	$0.22M_o$	$0.42M_o$
		Slab	$0.02M_o$	$0.06M_o$	$0.08M_o$	$0.04M_o$	$0.07M_o$
	Middle Strip		$0.04M_o$	$0.14M_o$	$0.17M_o$	$0.09M_o$	$0.16M_o$

Figure 3.11.5.1.2. Design Moment Coefficients for Two-Way Beam-Supported Slabs

Effective depth of the slab:

$$d = 130 - 25.4 - 12.7/2 \approx 100 \text{ mm}$$

Spacing calculations:

$$\text{For \#4 bars} \Rightarrow d_b = 12.7 \text{ mm}; A_b = 129 \text{ mm}^2$$

$$s_{max} = \min(3h; 457, 2) \approx 390 \text{ mm}$$

$$s = \min\left(\frac{A_b \times 1000}{A_s}; s_{max}\right) = \min\left(\frac{129 \times 1000}{234}; 390\right) = 390 \text{ mm}$$

$$\text{Quantity of \#4 bars} = 2500/390 = 7$$

Table 3.11.5.1.1. Interior slab parameters

Interior	Minor beam strip		Middle Strip	
	Negative	Positive	Negative	Positive
M_u to strips (kN – m)	68.86	36.54	22.49	12.65
M_u beam (kN – m)	59.02	30.92	-	-
M_u slab (kN – m)	9.84	5.62	-	-
Strip width, m	2.5	2.5	2.5	2.5
M_u slab per unit length (kNm)/m	3.93	2.25	8.99	5.06
R_n (kPa)	437.21	99.93	399.73	224.85
ρ	0.00105	0.00024	0.00096	0.00054

$A_s(m^2)$	104.78	23.83	95.74	53.71
$A_{s\min}(mm^2)$	234	234	234	234
Final $A_s(mm^2)$	234.0	234.0	234.0	234.0
Spacing, mm	390	390	390	390
Quantity of #4 bars	7.00	7.00	7.00	7.00

Table 3.11.5.1.2. Exterior slab parameters

Exterior	Column Strip			Middle Strip		
	Exterior Negative	Positive	First Interior Negative	Exterior Negative	Positive	First Interior Negative
M_u to strips ($kN - m$)	16.86	60.43	74.48	6	19.67	23.89
M_u beam ($kN - m$)	14.05	52.00	63.24	-	-	-
M_u slab ($kN - m$)	2.81	8.43	11.24	-	-	-
Strip width, m	1.25	2.5	2.5	2.5	2.5	2.5
M_u slab per unit length (kNm)/m	2.25	3.37	4.50	2.25	7.87	9.56
$R_n(kPa)$	249.83	374.75	499.67	249.83	874.42	1061.79
ρ	0.00060	0.00090	0.00120	0.00060	0.00211	0.00257
$A_s(m^2)$	59.70	89.72	119.86	59.70	210.94	256.88
$A_{s\min}(mm^2)$	234	234	234	234	234	234
Final $A_s(mm^2)$	234.00	234.00	234.00	234	234	257
Spacing, mm	390	390	390	390	390	390
Quantity of #4 bars	4	7	7	7	7	7

3.11.5.2. Shear analysis

First, applied shear force is evaluated as:

$$V_u = [l_1 l_2 - (b_{col} + d_{ave})(h_{col} + d_{ave})] \times W_u = 321 \text{ kN}$$

$$d_{ave} = h - \text{clear cover} - d_b = 130 - 25.4 - 12.7 \approx 100 \text{ mm}$$

Punching shear calculations

Basic punching shear capacity:

$$\phi V_{c1} = \phi * 4 * \lambda_s * \lambda * \sqrt{f'_c} * b_0 * d = 3236.2 \text{ kN} > V_u \Rightarrow OK$$

$$\text{Size effect factor: } \lambda_s = \sqrt{\frac{2}{1 + d_{ave}/10}} = 0.426 \leq 1.0 \Rightarrow OK$$

$$\lambda = 1 \text{ (normal-weight concrete)}$$

Column shape effect:

$$\phi V_{c2} = \phi \times (2 + \frac{4}{\beta}) \lambda_s \lambda \sqrt{f'_c} b_0 d_{ave} = 4854.2 \text{ kN} > V_u \Rightarrow OK$$

Column location effect:

$$\phi V_{c3} = \phi * (2 + \frac{\alpha_s d_{ave}}{b_0}) \lambda_s \lambda \sqrt{f'_c} b_0 d_{ave} = 2427.12 \text{ kN} > V_u \Rightarrow OK$$

$$\alpha_s = 40 - \text{location factor for two-way shear for (interior columns)}$$

For exterior column: $\alpha_s = 30$

$$\phi V_{c3} = 2224.86 \text{ kN} > V_u \Rightarrow OK$$

Perimeter of critical section for two-way shear:

$$b_0 = 2 * (c_1 + d_{ave}) + 2 * (c_2 + d_{ave}) = 4000 \text{ mm}$$

One-way shear check:

$$\phi V_{c1} = \phi (0.17 \lambda \sqrt{f'_c} + \frac{N_u}{6A_g}) b_w d = 322.5 \text{ kN} > V_u \Rightarrow OK$$

$$\phi V_{c2} = \phi (0.66 \lambda_p^{1/3} \sqrt{f'_c} + \frac{N_u}{6A_g}) b_w d = 451.3 \text{ kN} > V_u \Rightarrow OK$$

3.11.6. Joint Design

As per ACI-318 Section 11.4.1, strength requirements for beam-column joints must be satisfied.

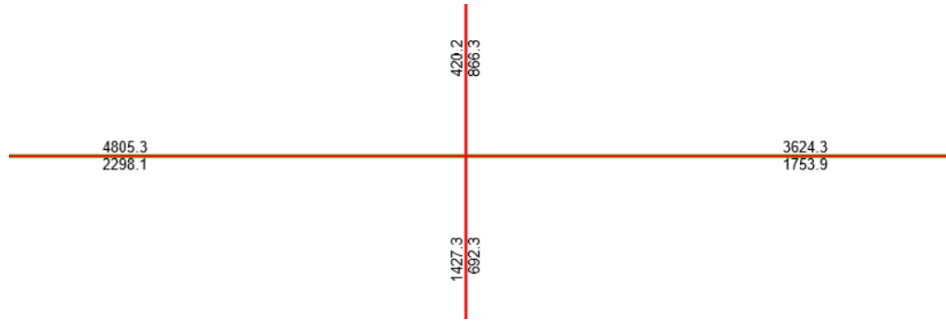


Figure 3.11.6.1. Joint Design

Summation of forces acting in horizontal direction on a free-body diagram of a joint allows evaluating the horizontal shear force as follows:

$$V_{u,j} = T_1 + T_2 - V_{col}$$

where:

$$T_1 = A_s^- f_y = 3624 \times 0.420 = 1522.08 \text{ kN}$$

$$T_2 = A_s^+ f_y = 2298.1 \times 0.420 = 965.202 \text{ kN}$$

$$V_{col} = 146 \text{ kN (from SAP2000)}$$

$$V_{u,joint} = 1522.1 + 965.202 - 146 = 2341.3 \text{ kN}$$

The nominal shear strength, according to ACI 318-19 table 15.4.2.3. for internal joint and confined transverse beams:

$$w = \text{lesser of } (b + c_1; b + 2x) = \text{lesser of } (500 + 850; 500 + 175) = 675 \text{ mm}$$

Effective cross-sectional area within a joint:

$$A_j = c_1 w = 850 \times 675 = 573750 \text{ mm}^2 = 889.3 \text{ in}^2$$

$$V_{n,j} (\text{lbs}) = 24\lambda\sqrt{f'_c} A_j = 24 \times \sqrt{5801} \times 889.3 = 1,625,589.8 \text{ kip} = 7230 \text{ kN}$$

Finally:

$$V_{u,j} < \phi V_{n,j} (\phi = 0.75)$$

$$2341.3 \text{ kN} < 0.75 \times 7230 = 5422.5 \text{ kN} \Rightarrow \text{shear strength req} - t \text{ satisfied}$$

For #10 bars or smaller, ties of bar #3 will be used with $A_s = 71 \text{ mm}^2$. The spacing in interior joint:

$$s = \min\left(\frac{A_v f_{yt}}{0.062\sqrt{f'_c} b_c}, \frac{A_v f_{yt}}{0.35b_c}\right) = \min\left(\frac{2 \times 71 \times 420}{0.062 \times \sqrt{40} \times 900}, \frac{2 \times 71 \times 420}{0.35 \times 900}\right) = 170 \text{ mm}$$

3.11.7. Development length and Hook design

For bar tension to be properly transferred to column concrete through bond stress, the bars should be extended by a distance called development length necessary for the bar to develop yield strength. Since primarily #7 bars and larger are used; clear cover and spacing are larger than beam diameters, the following simplified equation is utilized:

$$\frac{l_d}{d_b} = \frac{f_y \Psi_t \Psi_e}{20 \lambda \sqrt{f'_c}} \rightarrow l_d = \frac{60000 \times 1.3 \times 1.5}{20 \sqrt{5800}} d_b = 76.8 d_b$$

Therefore, for #9 bars: $l_d = 76.8 \times 1.13 = 86.784 in = 2204.3 mm$

and for #10 bars: $l_d = 76.8 \times 1.27 = 97.536 in = 2477.4 mm$

Since columns cannot accommodate such development length, hooks shall be used:

$$l_{dh} = \frac{\Psi_e \Psi_r \Psi_o \Psi_c f_y}{55 \lambda \sqrt{f'_c}} d_b^{1.5} \rightarrow l_{dh} = 17.2 d_b^2$$

Table 3.11.7.1. Modification factors for development of hooked bar in tension

λ	Ψ_r	Ψ_e	Ψ_c	Ψ_o
Normal-weight concrete	$A_{th} > 0.4 A_{hs}$	Epoxy-coated bars	$f'_c < 6000 psi$	For #11 and smaller hooked bar with cover in column $> 2.5 in$
1	1	1.2	0.9877	1

Therefore, for #9 bars: $l_{dh} = 17.2 \times 1.13^2 in = 560 mm$

and for #10 bars: $l_{dh} = 17.2 \times 1.27^2 in = 700 mm$

$D = 8d_b = 8 \times 1.13 = 9 in = 230 mm$

$r = D/2 = 115 mm$

Hook distance = $12d_b = 13.56 in = 344.24 mm \approx 350 mm$

Similarly, for #10 bars: $D = 260 mm$, $r = 130 mm$, Hook distance = $390 mm$

For compression reinforcement:

$$I_{dc} = \left(\frac{f_y \Psi_r}{50 \lambda \sqrt{f'_c}} \right) d_b \geq 0.0003 f_y \Psi_r d_b \rightarrow l_{dc} = 15.8 d_b$$

$I_{dc} = 450 mm$ (#9 bars) & $I_{dc} = 510 mm$ (#10 bars) $<$ smallest column section $\Rightarrow$ no need for hooks

3.11.8. Lap splices

Lap splices are required for a safe transfer of force from one bar to another since a single bar cannot run across the whole member and reinforcement layout changes along the section.

For beams in compression (top bars middle) and columns (middle height):

$$l_{sc} = 0.0005f_y d_b = 860 \text{ mm (\#9 bars)} \text{ and } 970 \text{ mm (\#10 bars)} > 12 \text{ in}$$

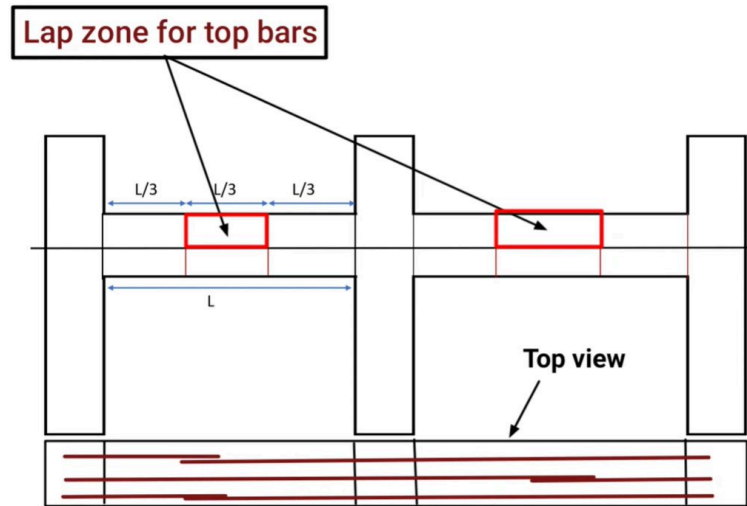


Figure 3.11.8.1. Lap zone for top bars

Columns are spliced at the middle of height where the moment is zero.

3.12. Structural serviceability design

3.12.1. Deflection

ACI 318 code regulations exempt members designed following minimum thickness requirements from deflection check. For members with steel with $f_y = 420 \text{ MPa}$ this requirement applies:

$$h_b = \frac{L}{21} = \frac{5000}{21} = 238 \text{ mm}$$

Since height of both major and minor beam sections far exceed this value, they do not have to be checked for deflection. The slab also satisfies the minimum thickness requirement. For prestressed major beam, all the serviceability checks were incorporated in the design workflow outlined in more detail in section 3.11.1 above.

3.12.2. Crack width

The Gergely-Lutz equation for crack width estimation is:

$$d_c = 60 \text{ mm} = 2.36 \text{ in}$$

$$A = \frac{19.68 \times 5.91}{8} = 14.54 \text{ in}^2$$

$$w = 0.076\beta_h f_s \sqrt[3]{d_c A} = 0.076 \times 1.2 \times (0.6 \times 60) \times \sqrt[3]{2.36 \times 14.54} = 0.011 \text{ in} < 0.012 \text{ in}$$

As per calculations, the crack width is within acceptable limits.

Maximum spacing for longitudinal reinforcement is:

$$s(in) = [15(\frac{40}{f_s}) - 2.5C_c] = 15 \times \frac{40}{0.6 \times 60} - 2.5 \times 2.36 = 10.7 \text{ in} = 270 \text{ mm}$$

4.0 Geotechnical part

Literature review

The project will be located in Bexar County, San Antonio, Texas. It must comply with the 2018 International Building Code (IBC) and its Texas changes, taking into account the site location. The project must also comply with the Texas Department of Infrastructure and Transportation (TxDOT) Geotechnical Design Guide and ASCE 7-16 standards for design loads. Collectively, these guidelines and codes provide recommendations for foundation design, load calculations, and soil investigations.

This project uses the results of a geotechnical study conducted by Aviles Engineering Corporation, Report G157-19 (Revision 1, 2022). The project complies with American Society for Testing and Materials (ASTM) standards and includes comprehensive data obtained through laboratory testing and field investigations. The study included 54 boreholes, soil sampling, and testing, ensuring the completeness and accuracy of the report.

The report is primarily responsive to our project requirements because it contains site-specific information, complies with all legal requirements, and classifies soils using the Unified Soil Classification System (USCS) in accordance with ASTM D2487.

4.1 Site Description

The project site will be located in Bexar County in downtown San Antonio, Texas. The approximately 3,500 m^2 site is located within urban areas with paved streets and adjacent residential and commercial buildings. The site is already zoned for urban use, and the land surface is predominantly level.

The project is planned for a nine-story, mixed-use hotel with three basement levels. In this area of San Antonio, soft clay soils are typically found near the surface, while dense soils and limestone layers lie deeper. Although limestone layers provide stronger foundation support, clay soils can collapse under heavy loads. Groundwater can also impact the site

depending on seasonal fluctuations, so it must be considered in the foundation and excavation design.

These geological and hydrogeological conditions further illustrate the importance of careful geotechnical design. Properly selected foundation systems and construction methods will ensure reliable support for the proposed building and compliance with current building codes.

4.1.1 Site analysis

The chosen boring B-8 has a depth of 30.5 meters (100 feet), which is located along the road in San Antonio, Texas. This boring log provides data about soil stratigraphy, Unified Soil Classification System (USCS) group symbols, relative density, etc. The results in Table A-8 demonstrate that the upper 21 meters (70 feet) of soil consists of predominantly high plasticity fat clays (CH) with interbedded silts (ML), silt clays (CL) (Aviles Engineering Corporation, 2019). They are related to Type A based on classification by OSHA due to soft, swelling, and sedimentation properties. Beneath this, there are dense silty sands (SM) and sandy lean clays (CL) prolonged to 21-22m. These layers are categorised as Type B due to intermediate strength and layering. They are followed by stiff lean clay (CL) to a depth of 30.5 meters. This layer has a feature of high density and can be considered as Type A soil and as the main load bearing stratum at this site. The data for the soil profile is provided in the table below.

4.1.2 Soil profile

Table 4.1.1 Soil profile

Soil Type	USCS	Depth, m	Unit weight $\gamma(\text{kN/m}^3)$	Undrained shear strength c_u (kPa)	Friction angle, ϕ	Strain
Fill	GC	0-0.91	18.85	9.6	0	0.02
Soft clay	CH	0.91-5.18	19.47	76.6	0	0.07
Compacted Silt	ML	5.18-6.7	19.95	0	26	-

Stiff Clay	CH	6.7-9.75	19.63	110	0	0.005
Stiff Clay	CL	9.75-11.28	21.36	144	0	0.005
Soft Clay	CL-ML	11.28-12.8	18.85	38.4	0	0.01
Stiff Clay	CL	12.8-18.89	19.79	96	0	0.007
Stiff Clay	CL-ML	18.89-20.42	18.85	57.6	0	0.007
Stiff Clay	CH	20.42-21.94	18.85	48	0	0.01
Silty Sand	SM	21.94-23.46	19.63	0	34	-
Stiff Clay	CH	23.46-25	19	144	0	0.005
Lean Clay	CL	25-26.51	18.85	57.6	0	0.007
Lean Clay	CL	26.51-30.48	20.9	144	0	0.005

This area is covered with a layer of soft clay which poses a potential problem. Soft clays typically have low shear strength, high compressibility, and low permeability. Building on top of it can be problematic as it can lead to excessive settlement, low bearing capacity, and potential failure at some point in the future.

The low cutting strength of soft clay means that it cannot withstand high cutting stresses and may fail under load. The high compressibility of the soil can also lead to settlement and differential settlement of the foundations. Soft clay also has low permeability due to its water-retaining properties, which can lead to settlement and instability.

However, we mitigated these potential problems. We conducted in-depth geotechnical investigations and soil tests to determine the behavior of the soil under different loads. These studies allowed us to learn about soil treatments such as improving the drainage properties of the soil and foundation design that would allow for settlement.

Such measures will minimize the risks arising from soft clay and ensure the structural integrity of the building. Soft clay is a challenge, but with careful planning and technical measures, we can overcome it. With proper design and construction, the foundation will perform well.

4.1.3 Groundwater Table

According to boring B-8, advanced in this project, the groundwater table was encountered approximately at 25 feet (7.8 meters) of drilling. After 10 minutes of drilling completed, it was stabilised at 19.1 (5.82 meters) feet below ground level (Aviles Engineering Corporation, 2019). This observation demonstrates that the actual level of the groundwater table is between 19-25 feet. Since the basement excavation will extend to approximately 10 meters and piles about to 12 meters, the construction requires lowering of groundwater level and groundwater control system to provide stability of the excavation. It includes consistent pumping of the water and deep well systems depending on the soil encountered. The groundwater level should be reduced to a minimum of 3 meters from excavation bottom due to excessive seepage risks, boiling in granular layers, and base heave. The presence of risk factors such as swelling and sedimentation of high plasticity soils layers such as fat and silt clays during the alteration of moisture should be considered (Aviles Engineering Corporation, 2019). Also, if underground parking levels are planned, the temporary dewatering, waterproofing are required to prevent humidity issues (Trushko, Trushko, & Demenkov, 2024).

4.1.4 Seismicity

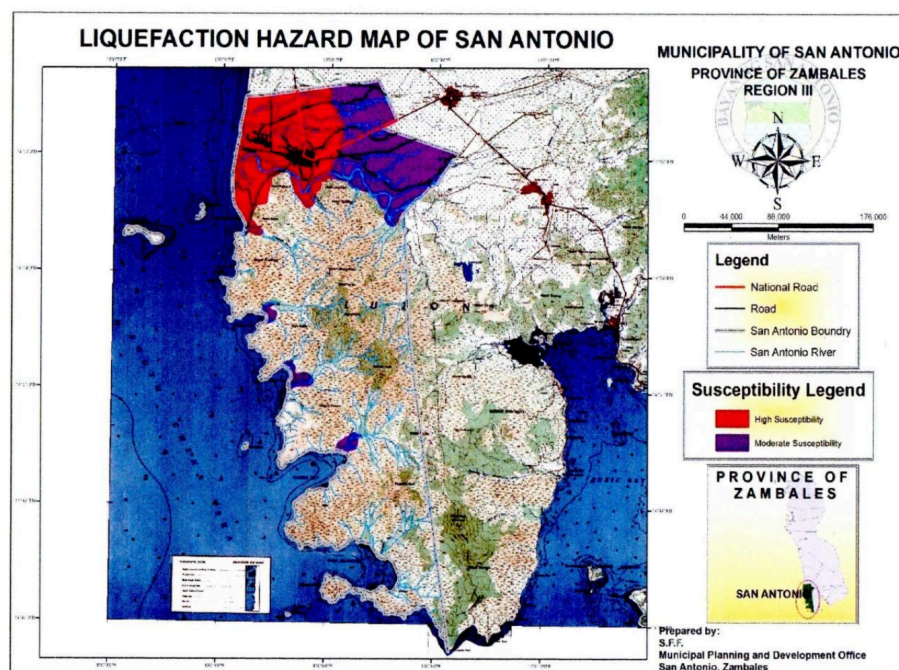
The site of boring is located in San Antonio, Texas classified as Site Class C based on 2012 International Building Code (IBC) and ASCE 7-10, which means that it is considered as a low seismic zone (Gorrondona and Associates, Inc., 2018). According to a map of seismic zones USGS and requirements of ASCE 7-16, the territory of Texas is categorized as a zone with low seismic activity and all calculations are based on Seismic Design Category A. There are no provided Vs30 measurements of shear wave for sites of boring, due to the fact that this parameter is not significant for low seismic activity zones. The main objective of this part will be analyzing weak clay soils (CH and CL), which are prone to swelling and sedimentation due to high plasticity, and ensuring stability of construction under static loads (Aviles Engineering Corporation, 2019).

4.1.5 Liquefaction

Liquefaction is a geotechnical process in which saturated sandy or silty soils lose shear strength and stiffness under the influence of excess pore water pressure during seismic vibrations. This can lead to significant settlement, lateral spreading, or foundation failure.

According to IBC 2018, Chapter 18 (Soils and Foundations), and ASCE 7-16, liquefaction assessments should be conducted in cases where both seismically hazardous and seismically susceptible soils are present. Clayey soils with deep limestone predominate in the subsurface profile at the San Antonio, Texas, project site, located in an area of very low seismic activity. Because cohesive clays and rocks do not have the same pore pressure response as unconsolidated sands, these soil types are not susceptible to liquefaction.

Therefore, the potential for liquefaction in this area is low and no special design measures to prevent liquefaction are required for the planned nine-story hotel with three basements.



Municipal Disaster Risk Reduction and Management Plan 2020-2022

Figure 4.1.5.1. Map of San Antonio

4.1.6 Slope stability

The city of San Antonio has a flat topography, as can be seen from the topographic map of the city, especially in the area around the construction site. The flatness can be seen from the figure below (Topographic-Map.com, *n.d*).

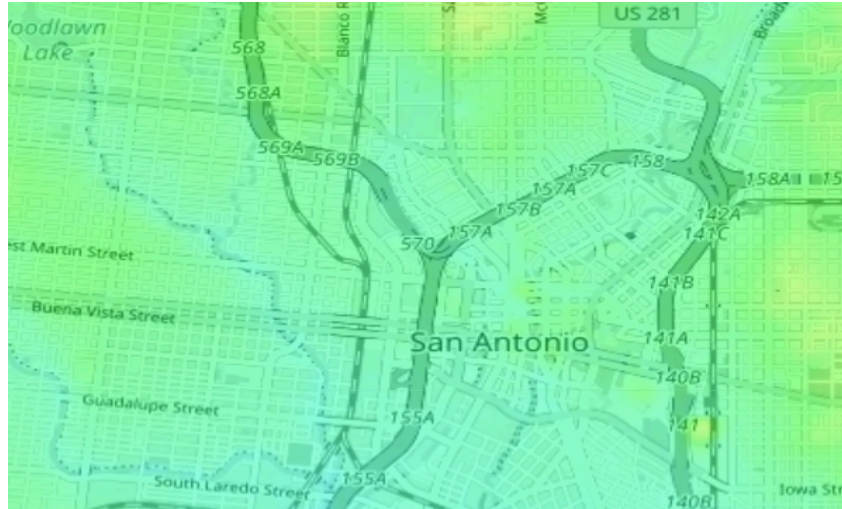


Figure 4.1.6.1. Elevation records of San Antonio

4.2 Foundation Design

4.2.1 Shallow foundations

4.2.1.1 Stronger Soil underlain by Weaker Soil

In our design, three-story basements have a depth of 9.75 m total. With the shallow foundation depth of 1.5, the foundation would be located in a 9.75 - 11.25 m depth, which is a stiff clay layer.

For this layer, the relevant soil parameters are:

unit weight, $\gamma_1 = 21.36 \text{ kN/m}^3$

undrained shear strength, $c_{u1} = 144 \text{ kPa}$

foundation embedment depth, $D_f = 1.5 \text{ m}$

Vertical effective stress is found using the following formula:

$$q = \gamma_1 \times D_f \quad (4.2.1.1.1)$$

where the effective unit weight is

$$\gamma_1' = \gamma_1 - \gamma_w = 21.36 - 9.81 = 11.53 \text{ kN/m}^3$$

Thus, the vertical effective stress are becoming:

$$q = 11.53 \times 2 = 23.1 \text{ kN/m}^2$$

However, this stiff layer is followed by a softer layer, as shown in Table 1, which is 1.53 m below the bottom of the foundation. The properties of this softer clay are:

unit weight, $\gamma_2 = 18.85 \text{ kN/m}^3$

undrained shear strength, $c_{u2} = 38.4 \text{ kPa}$

For the shallow foundation, it is assumed that $B = L$ as a quadratic form.

Hence, bearing capacity factors can be found from the data set for each friction angle (Das, 2019). For the soil at the foundation level, the corresponding bearing-capacity factors are:

$$N_c = 5.7 \quad N_q = 1 \quad N_y = 0$$

The shape factors for a square footing become:

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi = 1 \quad (4.2.1.1.2)$$

$$F_{ys} = 1 - 0.4\left(\frac{B}{L}\right) = 0.6 \quad (4.2.1.1.3)$$

Since the topography of the site is level, the depth factors are:

$$F_{qd} = F_{yd} = 1$$

According to Das (2019), bearing capacity for Stronger soil underlain by weaker soil can be found following the next formulas and steps.

$$q_u = \left(1 + 0.2\frac{B}{L}\right)5.14c_{u2} + \left(1 + \frac{B}{L}\right)\left(\frac{2c_a H}{B}\right) + q \quad (4.2.1.1.4)$$

where

$B = \text{width of the foundation}$

$c_a = \text{adhesive force}$

$H = \text{depth from bottom of the foundation}$

And this value should be equal to or less than;

$$q_t = \left(1 + 0.2\frac{B}{L}\right)5.14c_{u1} + q \quad (4.2.1.1.5)$$

Adhesive force c_a can be found using the correlations between c_a and c_{u1} from the figure,

with c_{a1} and c_{u1} known,

$$c_a = 0.93 \times 144 = 133.92 \text{ kPa}$$

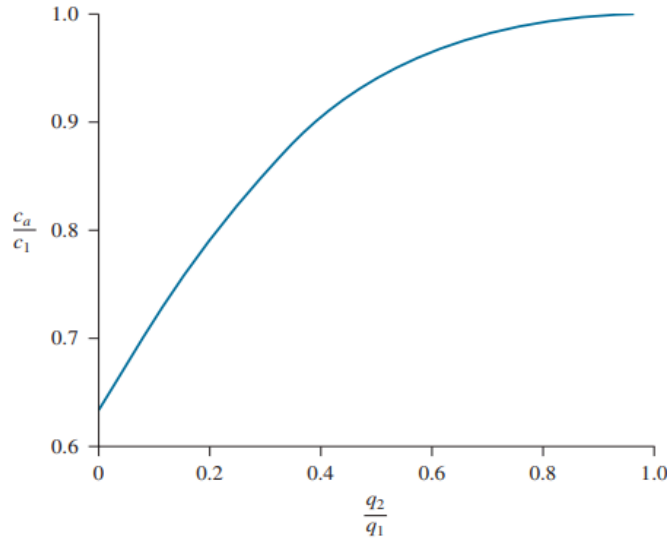


Figure 4.2.1.1 Correlation between c_a and c_{u1}

By using the upper equations, 3 and 4, we obtain:

$$q_u = (1 + 0.2) \times 5.14 \times 38.4 + (2) \left(\frac{2 \times 133.92 \times 1.53}{B} \right) + 23.1 = 259.95 + \frac{1071.36}{B}$$

$$q_t = (1 + 0.2) 5.14 \times 144 + 21.4 = 909.59 \text{ kN/m}^2$$

From equating the values, we can find $B = 1.65 \text{ m}$.

Therefore, we obtain this relation:

$$q_u = 259.95 + \frac{1071.36}{1.65} = 909.26 \text{ kN/m}^2$$

With the factor of safety of 2.5,

$$q_{all} = 909.26 / 2.5 = 363.71 \text{ kN/m}^2$$

4.2.1.2 Terzaghi's method

The classical method of estimating the ultimate bearing capacity of shallow foundations is known as Terzaghi bearing capacity method. Terzaghi treated the underlying failure mechanism under a strip footing and the ultimate bearing capacity was the assembly of cohesion, surcharge as well as unit-weight.

According to Terzaghi's general bearing capacity equation:

$$q_{ult} = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma \quad (4.2.1.2.1)$$

where:

$N_c N_q N_\gamma$ - bearing capacity factors,

c' – soil cohesion,

q – effective overburden pressure at the foundation base,

γ – unit weight of soil,

B – foundation width.

For clays ($\phi=0^\circ$), Terzaghi showed that:

$$N_c = 5.7$$

$$N_q = 1$$

$$N_\gamma = 0$$

since $N_\gamma = 0$, the last term disappears and the formula becomes:

$$q_{ult} = 5.7c + \gamma D_f \quad (4.2.1.2.2)$$

The resulting ultimate bearing capacities for different foundation sizes are summarized in the table below.

Table 4.2.1.2.1 Terzaghi Bearing Capacity Results

(B × L)	N _c	N _q	N _γ	q (kN/m ²)
2 × 3	5.7	1.0	0.0	852.84
3 × 4	5.7	1.0	0.0	852.84
4 × 5	5.7	1.0	0.0	852.84
40 × 60	5.7	1.0	0.0	852.84

4.2.1.3 Meyerhof's method

Meyerhof bearing capacity method is common when it comes to assessing the ultimate and the allowable bearing capacity of shallow foundations. Meyerhof gave the classical theory of Terzaghi a further extension, where he added the influence of the shape of the foundation, depth of the foundation, load inclination, and ground inclination. His formulation presents the ultimate bearing capacity as a product of three factors, namely cohesion, surcharge and unit weight. The components are multiplied by bearing capacity

factors and correction factors, which are used to consider the geometry and loading conditions.

The general form of the Meyerhof bearing capacity equation is:

$$q_{ult} = c'N_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (4.2.1.3.1)$$

where:

$N_c N_q N_\gamma$ - classical bearing capacity factors dependent on the soil friction angle ϕ ,

$F_{cs}, F_{qs}, F_{\gamma s}$ - shape of the foundation,

$F_{cd}, F_{qd}, F_{\gamma d}$ - depth effects,

$F_{ci}, F_{qi}, F_{\gamma i}$ - load inclination of the applied load.

To foundations on granular soil or on cohesive soils of low cohesion, the third term is often negligible, and the formula may simplify to:

$$q_{ult} = cN_c F_{cs} F_{cd} + qN_q F_{qs} F_{qd} \quad (4.2.1.3.2)$$

The correction factors F in this paper (shape, depth, and inclination factors) are taken based on the equations of Meyerhof:

Shape factors

$$F_{cs} = 1 + 0.2 \frac{B}{L} \quad (4.2.1.3.3)$$

$$F_{qs} = 1$$

$$F_{\gamma s} = 1 - 0.4 \frac{B}{L} \quad (4.2.1.3.4)$$

Depth factors

$$F_{cd} = 1 + 0.2 \frac{D_f}{B} \quad (4.2.1.3.5)$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1 + 0.1 \frac{D_f}{B} \quad (4.2.1.3.6)$$

Inclination factors

$$F_{ci} = F_{qi} = F_{yi} = 1 \quad (4.2.1.3.7)$$

These factors adjust the bearing capacity to account for foundation geometry and depth, which is essential for realistic evaluation in practical geotechnical design.

Table 4.2.1.3.1 Meyerhof Bearing Capacity Correction Factors

B × L (m)	F_{cs}	F_{qs}	F_{ys}	F_{cd}	F_{qd}	F_{yd}	q (kN/m²)
2 × 3	1.1169 59	1.000	0.733333	1.300	1.000	1.000	834.14
3 × 4	1.1315 79	1.000	0.700000	1.200	1.000	1.000	782.62
4 × 5	1.1403 51	1.000	0.680000	1.50	1.000	1.000	757.18
40 × 60	1.1079 62	1.000	0.753846	1.015	1.000	1.000	854.55

4.2.1.4 Hensen`s method

The method by Hansen (1970), builds on the classical Terzaghi and Meyerhof methodology, but includes a more sophisticated system of the factors of correction that consider the form of foundation, depth of embedment, inclination of load, and ground slope. These correction factors are applied alongside general bearing capacity equation to calculate final capacity of shallow foundations when applied to the real field conditions.

The general Hansen bearing capacity equation is:

$$q_{ult} = c'N_c S_c D_c i_c + \bar{q} N_q S_q D_q i_q + \frac{1}{2} \gamma B' N_\gamma S_\gamma D_\gamma i_\gamma \quad (4.2.1.4.1)$$

Since the soil at the site is cohesive clay with $\phi=0^\circ$, the bearing capacity terms simplify because:

$$N_q = 1$$

$$N_\gamma = 0$$

Therefore, the reduced form becomes:

$$q_{ult} = c' N_c S_c D_c i_c + \bar{q} N_q S_q D_q i_q \quad (4.2.1.4.2)$$

where all correction factors must be defined according to Hansen's original empirical relationships.

To evaluate the bearing capacity, Hansen introduced a set of empirical correction factors. The *shape factors* modify the capacity to reflect the plan geometry of the footing:

$$S_c = 1 + 0.2 \frac{B}{L} \quad (4.2.1.4.3)$$

$$S_q = 1 + 0.1 \frac{B}{L} \quad (4.2.1.4.4)$$

with the unit-weight shape factor

$$S_\gamma = 1 - 0.4 \frac{B}{L} \quad (4.2.1.4.5)$$

(Since $N_\gamma = 0$, S_γ does not affect the results.)

The depth factors adjust for embedment:

$$D_c = 1 + 0.2 \frac{D_f}{B} \quad (4.2.1.4.6)$$

$$D_q = 1 + 0.1 \frac{D_f}{B} \quad (4.2.1.4.7)$$

The factor D_γ is not used because $N_\gamma = 0$.)

The inclination factors become equal to 1 in this project because the applied load is vertical:

$$i_c = i_q = i_\gamma = 1$$

All these factors were evaluated for each footing dimension considered in the design.

The resulting Hansen coefficients and the corresponding bearing capacity values are summarised in the following table.

Table 4.2.1.4.1 Hensen factors

B × L (m)	Sc	Dc	Sq	Dq	Sy	k	q (kN/m²)

2 × 3	1.116959	1.30	1	1	0.733333	0.75	837.7993
3 × 4	1.131579	1.20	1	1	0.700000	0.50	786.1130
4 × 5	1.140351	1.05	1	1	0.680000	0.125	698.1732
40 × 60	1.107962	1.015	1	1	0.753846	0.0375	658.7128

4.2.1.5 Summary of Shallow and Mat foundation Bearing Capacity Results

To determine the bearing performance of the suggested shallow foundation, three commonly accepted methods of analysis, namely, Terzaghi, Meyerhof and Hansen, were used to obtain both ultimate and allowable bearing of all footing sizes used. The difference in these methods lies in the assumptions they make about the distribution of stress, failure mechanisms and the effect of the foundation geometry and depth of embedment.

Consequently, predictive capacities across the different formulations might differ such that this offers a good understanding of the design sensitivity to methodological selection. All the calculations were done using identical soil parameters, dimensions of the foundation, and safety factors, in order to allow an easy comparison of the approaches. The three methods yield the following values of $q_{ultimate}$ and $q_{allowable}$ as summarised in the table below.

To obtain the results we assume that factor of safety 2.5 and use these equations :

$$q_{all} = \frac{q_{ult}}{FS} \quad (4.2.1.5.1)$$

$$Q_{all} = q_{all} \times Area \quad (4.2.1.5.2)$$

Table 4.2.1.5.1 Summary of footing foundation outcomes

Method	Bearing capacity	2 × 3	3 × 4	4 × 5
Terzaghi	$q_{ultimate} (kN/m^2)$	852.84	852.84	852.84
	$Q_{allowable} (kN)$	2046.81	4093.63	6822.72

Meyerhof	$q_{ultimate} (kN/m^2)$	834.14	782.62	757.18
	$Q_{allowable} (kN/m^2)$	2001.93	3756.57	6057.44
Hansen	$q_{ultimate} (kN/m^2)$	837.7993	786.1130	698.1732
	$Q_{allowable} (kN/m^2)$	2010.71832	3773.3424	5585.3856

Table 4.2.1.5.2 Design load Comparison

Safest Footing Foundation	Maximum Allowable Bearing Capacity(kN)	Maximum load for one column(kN)
4 × 5	6822.72	10444.892

With the 36 columns in our design and total load of more than 300 000 kN, footing foundation is considered not safe due to low value of bearing capacity.

Table 4.2.1.5.3 Outcomes for mat foundation.

Method	$q_{ultimate} (kN/m^2)$	$Q_{allowable} (kN)$
Terzaghi	852.84	818726.41
Meyerhof	854.55	628753.03
Hansen	658.71	632364.29

Mat foundation is considered safe due to the bearing capacity that can endure the load applied. However, settlement should be considered.

4.2.1.6 Shallow foundation elastic settlement

Shallow foundations pass on structural loads to the supporting soils, and are applied where the rigid layers of load-bearing are found at moderate depths. In this project, the base is on stiff clay and this gives it enough shear strength. To achieve acceptable deformations, however, a settlement analysis is needed. The following equation can be used to estimate instant settlement:

$$S_e = A_1 A_2 \frac{q_0 B}{E_s} \quad (4.2.1.6.1)$$

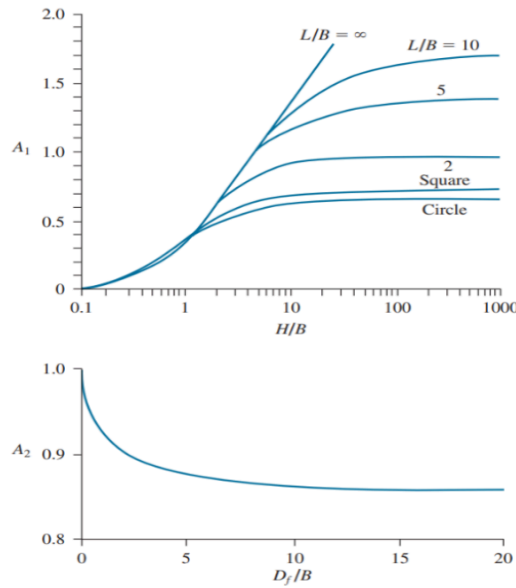


Figure 4.2.1.6.1. Variation of A_1 and A_2

In addition to the soil modulus, settlement calculations require influence factors A_1 and A_2 , which takes into account the thickness of the compressible layer and the foundation depth. These values are obtained from empirical diagrams (Figures 4.2.1 and 4.2.2).

From these charts, the factors were determined as $A_1 = 0.60$ and $A_2 = 0.97$. And $A_1 = 0.8$ and $A_2 = 1$ for mat foundation.

The average contact pressure was taken to be equal to the bearing's allowable pressure, which corresponds to a safety factor of 2.5.

$$q_0 = \frac{q_u}{2.5}$$

As the soil on site is composed of overconsolidated clays, the Overconsolidation Ratio (OCR) should be taken into account. OCR reveals the soil history of stress and has a

great effect on the stiffness and settlement properties. The larger the OCR the smaller the settlements. The compressibility and deformation behaviour of a clay deposit depend heavily on the stress history of the clay deposit. In order to explain this, the Overconsolidation Ratio (OCR) is presented.

OCR is a ratio between the highest previous effective vertical stress that has been experienced by the soil and the existing in-situ effective vertical stress at the depth of interest. It is defined as:

$$OCR = \frac{\sigma'_c}{\sigma'_{v0}} \quad (4.2.1.6.2)$$

where:

σ'_c - is the preconsolidation pressure

σ'_{v0} - is the initial effective vertical stress in the ground at mid-layer depth.

For the clay layer considered in this project, the preconsolidation pressure was interpreted from the consolidation test as:

$$\sigma'_c = 0.47N_{60}p_a \quad (4.2.1.6.3)$$

$$\sigma'_{v0} = \gamma_{sat} \times H \quad (4.2.1.6.4)$$

Substituting these values into the OCR expression gives:

Table 4.2.1.6.1 Preconsolidation and vertical effective pressure

Clay Layer	σ'_c (kPa)	σ'_{v0} (kPa)	OCR
1	846	130.28	6.49
2	658	115.712	5.68

This value is an indication that the clay is highly overconsolidated. These soils show relatively high degrees of stiffness and experience very little deformation to working loads. The applied foundation stress is not greater than the preconsolidation pressure and therefore the settlement behaviour of the soil stays in the entire recompression range and the recompression index should be applied in the process of calculating the primary consolidation settlement.

To calculate the soil modulus, empirical correlations are applied. The modulus is expressed through the relationship:

$$E_s = \beta c_u \quad (4.2.1.6.5)$$

where β is a coefficient dependent on both the OCR and the plasticity index (PI).

Table 4.2.1.6.2 . Range of β for Saturated Clay

<i>Plasticity Index</i>	<i>OCR = 1</i>	<i>OCR = 2</i>	<i>OCR = 3</i>	<i>OCR = 4</i>	<i>OCR = 5</i>
<i>< 30</i>	<i>1500–600</i>	<i>1380–500</i>	<i>1200–580</i>	<i>950–380</i>	<i>730–300</i>
<i>30 to 50</i>	<i>600–300</i>	<i>550–270</i>	<i>580–220</i>	<i>380–180</i>	<i>300–150</i>
<i>> 50</i>	<i>300–150</i>	<i>270–120</i>	<i>220–100</i>	<i>180–90</i>	<i>150–75</i>

Based on $PI = 29$ and $OCR = 6.49$, the corresponding range of β is approximately 300–700.

For design purposes, a conservative value of $\beta = 400$ was adopted and using equation 28;

$$E_s = 400 \times 144 = 57600 \text{ kPa}$$

Substituting the obtained values into the settlement equation:

Table 4.2.1.6.3. Elastic Settlement of Shallow foundation

Foundation	1.65 × 1.65	2 × 3	3 × 4	4 × 5	40 × 60
Elastic Settlement (m)	0.005	0.008	0.011	0.014	0.176

4.2.1.7 Primary consolidation settlement

Along with the short-term (elastic) settlement, with time, the clays are also prone to experience a time-dependent deformation as a result of consolidation. These deformations can be classified into primary and secondary consolidation, which is due to the loss of excess pore water pressure and the creep over time of the soil skeleton respectively. The two contributions should be taken together to come up with the total settlement of the foundation. With OCR of 5.48, the soil is overconsolidated.

The primary consolidation settlement is calculated using the following expression:

$$S_{c(p)} = \frac{C_c H_c}{1+e_0} \log \frac{\sigma'_0 + \Delta\sigma'_{av}}{\sigma'_0} \quad (4.2.1.7.1)$$

where

C_c is the compression index;

H_c is the thickness of the compressible layer;

e_0 is the initial void ratio;

σ'_0 is the initial effective vertical stress;

$\Delta\sigma'_{av}$ is the average increase in vertical stress due to foundation loading.

$$\Delta\sigma' = q_0 \frac{BL}{(B+z)(L+z)} \quad (4.2.1.7.2)$$

Table 4.2.1.7.1 Consolidation parameters of clay layers

Clay Layer Below the Basement	H_c	C_c	C_s	e_0	$\Delta\sigma'(\text{kPa})$
1	1.53	0.194	0.039	0.8	127
2	1.52	0.299	0.06	0.9	152

Since

$$\sigma'_0 < \sigma'_c < \sigma'_0 + \Delta\sigma'_{av}$$

We have to apply

$$S_{c(p)} = \frac{C_c H_c}{1+e_0} \log \frac{\sigma'_c}{\sigma'_0} + \frac{C_c H_c}{1+e_0} \log \frac{\sigma'_0 + \Delta\sigma'_{av}}{\sigma'_c} \quad (4.2.1.7.3)$$

Table 4.2.1.7.2. Primary Consolidation Settlement

Primary Consolidation Settlement(m)	1.65 × 1.65	2 × 3	3 × 4	4 × 5	40 × 60
1	0.1016	0.1154	0.127	0.1345	0.1663
2	0.0156	0.022	0.028	0.0323	0.1284

The secondary consolidation settlement is evaluated according to:

$$S_{c(s)} = C_{\sigma}^{\prime} H_c \log \frac{t_2}{t_1} \quad (4.2.1.7.4)$$

where

$C_{\sigma}^{\prime}$ is the secondary compression index,

H_c is the compressible layer thickness,

$\frac{t_2}{t_1}$ represent the time at the end of primary consolidation and the considered service period.

With the assumption of secondary compression index of $C_{\sigma 1}^{\prime}$ and $C_{\sigma 2}^{\prime}$ equal to 0.008 and 0.012 respectively, and design life of 50 years the calculated secondary settlement is 21 mm.

Table 4.2.1.7.3. Summary of Settlement Calculation Results

Settlement	1.65 × 1.65	2 × 3	3 × 4	4 × 5	40 × 60
S_e	0.005	0.008	0.011	0.014	0.176
$S_{(c)p}$	0.117	0.137	0.155	0.167	0.295
$S_{(c)s}$	0.021	0.021	0.021	0.021	0.021
S_t	0.143	0.166	0.187	0.202	0.492

Even though, mat foundation could endure the load applied due to the high value of bearing capacity, it has a high value of settlement and should not be designed for this project.

4.2.2 Deep foundations

A pile foundation is a deep foundation system made usually from steel, concrete or timber where depth much bigger than its width. It is suggested when shallow foundation are

insufficient where there is a big lateral and axial load are involved. Piles transfer the load to the deeper soil layers offering stability.

The subsoil profile consists of stiff to very stiff clays with an occasional sand layer located at depths of 22-23.5 m. Undrained shear strength of the clay varies between 48 and 144 kPa, where intermediate layers have strength equal to 57.6 kPa and 96 kPa. Also, unit weights for the soil are varies between 18.85 kN/m^3 and 20.98 kN/m^3 . Furthermore, parameters such as lateral pressure coefficient $K=1.5$, internal friction angle $\varphi = 35$, friction angle $\delta = 28$ were used in calculations. Meyerhof and Vesic with N_c , α -method by layer, β -method using K , φ , δ were used in methodology.

4.2.2.1 Single Pile Analysis

4.2.2.1.1 Meyerhof method for calculating point bearing capacity

This method is one of the classical approaches for defining the ultimate bearing capacity of piles in cohesive clay soils. It suggests that the resistance of piles forms by undrained shear strength and plastic failure mechanism.

$$Q_p = N_c \times c_u \times A_p \quad (4.2.2.1.1.1)$$

where,

Q_p = ultimate end bearing resistance (kN),

N_c = bearing capacity factor (≈ 9 for deep piles in clay),

c_u = undrained shear strength of clay at pile tip (kPa),

$A_p = \frac{\pi d^2}{4}$ = base area of the pile (m^2).

Table 4.2.2.1.1 Point Bearing Capacity for Meyerhof method

D (m)	L (m)	C_u (kPa)	N_c	Q_p (kN)
0.4	12	48	9	54.26
0.4	9	96	9	108.51
0.4	15	144	9	192.77
0.3	12	48	9	30.52
0.3	9	96	9	61.05
0.3	15	144	9	91.56

0.5	12	48	9	84.78
0.5	9	96	9	169.56
0.5	15	144	9	254.34

4.2.2.1.2 Vesic method for calculating point bearing capacity.

This is an enhanced version of the theory of plasticity for clays, considering the effect of pile rigidity and base shape, which defines stress influence under the pile tip. In contrast to Meyerhof, the method of Vesic corrects bearing capacity by adding a rigidity shape factor (I_{rr}), which depends on the stiffness of the soil.

$$Q_p = N_c \times c_u \times A_p \quad (4.2.2.1.2.1)$$

The shape or rigidity factor for the calculation of bearing Vesic is defined as:

$$N_c^* = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1 \quad (4.2.2.1.2.2)$$

where

I_{rr} - rigidity or shape factor.

And can be calculated with the following expression, when there no volume change and friction angle is zero.

$$I_{rr} = I_r$$

$$I_r = 347 \left(\frac{c_u}{p_a} \right) - 33 \quad (4.2.2.1.2.3)$$

Table 4.2.2.1.2.1 Point Bearing Capacity for Vesic method

D (m)	L (m)	C_u (kPa)	p_a (kPa)	I_{rr}	N_c	Q_p (kN)
0.4	12	48	100	133.68	10.72	62.87
0.4	9	96	100	300.12	12.09	138.77
0.4	15	144	100	466.68	12.09	248.81
0.3	12	48	100	133.56	10.72	35.36
0.3	9	96	100	300.12	12.09	78.05
0.3	15	144	100	466.68	12.09	123.08
0.5	12	48	100	133.56	10.72	98.24
0.5	9	96	100	300.12	12.09	216.83

0.5	15	144	100	466.68	12.09	341.87
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4.2.2.1.3. α -method for calculating Frictional Resistance in Clay

Frictional resistance in clay arises due to many variables in clay that are not easy to quantify.

Several methods to determine the frictional resistance of clay are illustrated below.

α -method widely applies for calculating frictional resistance of piles in clay during undrained conditions. This method is based on suggestion where shear resistance along pile shaft is proportional to undrained shear strength.

$$f = \alpha \cdot c_u \quad (4.2.2.1.3.1)$$

where

f - frictional resistance (kN);

α - adhesion factor;

c_u - undrained shear strength for each layer.

Ultimate side resistance will be:

$$Q_s = \sum f p \Delta L = \sum \alpha c_u p \Delta L \quad (4.2.2.1.3.2)$$

where

p - perimeter of pile section;

ΔL - incremental pile length.

It is most appropriate for stratified soil profiles, due to considering each layer.

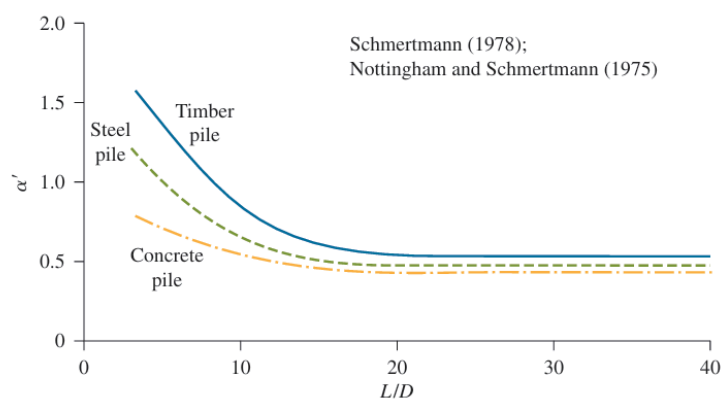


Figure 4.2.2.1.3.1 Alpha values

Table 4.2.2.1.3.1 α -method results

D (m)	L (m)	Q_s (kN)
-------	-------	------------

0.4	12	727.36
0.4	9	589.74
0.4	15	906.37
0.3	12	502.96
0.3	9	404.897
0.3	15	876.37
0.5	12	982.14
0.5	9	799.51
0.5	15	1185.22

4.2.2.1.4 λ -method for calculating Frictional Resistance in Clay

This method is proposed by Vijayvergiya and Focht (1972) to find total stress. It demonstrates effect of pile driving in passive lateral pressure on displacement of soil and unit skin resistance average as:

$$f_{av} = \lambda(\sigma'_o + 2c_u)$$

(4.2.2.1.4.1)

where

f_{av} - unit skin resistance average;

λ - empirical lambda coefficient;

σ'_o - effective stress (kPa);

c_u - shear strength.

The total skin resistance will be:

$$Q_s = plf_{av}$$

(4.2.2.1.4.2)

where

p - perimeter of pile section;

l - pile length.

Table 4.2.2.1.4.1 λ -values

L(m)	0	5	10	15	20	25	30	35
λ	0.5	0.336	0.245	0.2	0.173	0.15	0.136	0.132

λ for the designed values were ger interpolating the above values.

Table 4.2.2.1.4.2 λ -method results

D (m)	L (m)	Q_s (kN)
0.4	12	731.25
0.4	9	625.87
0.4	15	882.39
0.3	12	527.16
0.3	9	450.69
0.3	15	759.36
0.5	12	950.53
0.5	9	813.51
0.5	15	1110.37

4.2.2.1.5 Skin Resitance in sand

As there is a sand layer when 15 m pile is used at the depth of 21.94-23.46 m, skin resistance for this soil layer should be calculated separately.

$$f_s = K \cdot \sigma'_v \cdot \tan \delta$$

(4.2.2.1.5.1)

$$Q_{s,sand} = \Sigma(f_s \cdot A_s)$$

(4.2.2.1.5.2)

where

K - coefficient of lateral earth pressure (1.5),

σ'_v - effective vertical stress at layer mid-depth (kPa),

δ - pile–soil friction angle (28°).

Table 4.2.2.1.5.1 Skin friction for the sand layer

L(m)	D(m)	K	f_{angle} (°)	δ_{angle} (°)	Q_s (kN)
------	------	---	-----------------	----------------------	------------

15	0.4	1.5	35	28	369.42
15	0.3	1.5	35	28	254.56
15	0.5	1.5	35	28	424.27

4.2.2.1.6 Negative Skin Resistance in sand

Negative is a downward drag force exerted on the pile by surrounding soil. As the site has a granular soil fill over the clay layers Negative skin friction is calculated.

Granular Soil Fill over Clay

$$f_n = K'\sigma_0'tan\delta' \quad (4.2.2.1.6.1)$$

where

K' - earth pressure coefficient

σ_0' - vertical effective stress

δ' - soil-pile friction angle

$$Q_n = \sum_0^{L_1} p f_n dz = (pK'y_f'H_f tan\delta')L_1 + \frac{L_1^2 pK'y'tan\delta'}{2} \quad (4.2.2.1.6.2)$$

Table 4.2.2.1.6,1 Negative Skin friction Results

D (m)	L (m)	Q_n (kN)
0.4	12	957.58
0.4	9	636.89
0.4	15	1210.62
0.3	12	686.62
0.3	9	477.66
0.3	15	1025.57
0.5	12	1194.35
0.5	9	796.11
0.5	15	1312.61

Combining all the averaged values for the pile bearing capacities , ultimate bearing capacity is calculated to evaluate the pile design.

4.2.2.1.7 Summary of Single Pile Bearing Capacity Results

Combining all the averaged values for the pile bearing capacities , ultimate bearing capacity is calculated to evaluate the pile design.

Table 4.2.2.1.7.1 Single pile axial capacity

D (m)	L (m)	$Q_{p_{avg}}$ (kN)	$Q_{s\text{ clay}}$ (kN)	$Q_{s\text{ sand}}$ (kN)	Q_n (kN)	Q_u (kN)	Q_{all} (kN)
0.3	9	69.54	450.69	-	477.66	997.89	399.16
0.3	12	32.95	527.16	-	686.62	1246.73	498.69
0.3	15	107.32	759.36	254.56	1025.57	2146.81	858.72
0.4	9	123.65	625.87	-	636.89	1386.38	554.56
0.4	12	58.57	731.25	-	957.58	1747.4	698.96
0.4	15	220.79	858.43	369.41	1210.62	2559.25	1123.71
0.5	9	193.19	827.49	-	796.11	1816.79	726.71
0.5	12	91.52	918.93	-	1194.35	3262.5	881.92
0.5	15	298.11	1035.53	424.26	1312.61	3070.51	1228.21

4.2.2.1.8. Elastic Settlement of Single Piles

Vesic's method divides the total settlement of a pile into three components, elastic shortening of the pile, working load settlement and settlement along the pile shaft.

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \quad (4.2.2.1.8.1)$$

where,

Q_{wp} - working load at the pile point,

Q_{ws} - working load along the pile shaft,

E_p - cross-sectional area of the pile point,

A_p - cross-sectional area of the pile point.

$$Q_{wp} = \frac{Q_p}{FoS} \quad (4.2.2.1.8.2)$$

$$Q_{ws} = \frac{Q_s}{FoS} \quad (4.2.2.1.8.3)$$

$$S_{e(2)} = \frac{q_{wp} D}{E_s} (1 - \mu_s^2) I_{wp} \quad (4.2.2.1.8.4)$$

where,

q_{wp} - point load per unit area at the pile point Q_{wp}/A_p

E_s -modulus of elasticity of soil at or below the pile point

μ_s -Poisson's ratio of soil

I_{wp} -influence factor - 0.85

$$S_{e(3)} = \left(\frac{Q_{ws}}{pL}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \quad (4.2.2.1.8.5)$$

where,

I_{ws} - influence factor

$$I_{ws} = 2 + 0.35 \sqrt{\frac{L}{D}} \quad (4.2.2.1.8.6)$$

With $E_s = 30720$ and $E_p = 4 \times 10^7 \text{ kN/m}^2$

The following table shows the results for the elastic settlement

Table 4.2.2.1.8.1 Elastic settlement of the single pile

D(m)	L(m)	Q_{wp} (kN)	Q_{ws} (kN)	q_{wp} (kN/m ²)	I_{ws}	$S_{e(1)}$ (mm)	$S_{e(2)}$ (mm)	$S_{e(2)}$ (mm)	$S_{e(t)}$ (mm)
0.3	9	23.42	292.50	186.52	3.91	0.75	0.9	0.61	2.26
0.3	12	49.45	250.35	393.77	3.66	0.87	1.62	0.81	3.30
0.3	15	76.32	476.73	607.61	4.14	1.15	2.21	1.18	4.54
0.4	9	13.18	210.8	186.52	4.21	0.95	1.13	0.71	2.79

			6						
0.4	12	27.82	180.28	393.78	3.91	1.17	1.49	0.82	4.25
0.4	15	42.93	405.56	607.61	4.47	1.38	2.73	1.18	5.29
0.5	9	36.61	380.21	186.52	3.71	1.01	1.38	1.12	3.51
0.5	12	77.28	325.41	393.78	3.48	1.46	3.15	1.42	6.03
0.5	15	119.25	613.8	607.61	3.91	1.61	5.89	1.66	9.16

4.2.2.1.9. Lateral Bearing Capacity of Single Piles

The proposed foundation system is exposed to the lateral force caused by wind and seismic forces on piles. These are horizontal forces that result in bending the pile shaft and mobilize passive resistance of soils around the pile in the surrounding. Owing to the varying proportions of the total lateral force drawn by the columns, representative lateral loads were put on corner, exterior and interior group of piles depending on their structural location and anticipated stiffness input.

Corner columns are most likely to receive the greatest lateral forces as a result of drift and torsional effects. Intermediate loading is supported by exterior columns, and the weakest lateral requirement is supported by interior columns which are located in the proximity of the stiffness center of a building. In accordance with the global lateral response of the building, table below summarizes the forces at the base of each type of column.

Table 4.2.2.1.9.1 Lateral loads

Column Type	Load Combination	F _x (kN)	F _y (kN)	No. of Piles	Lateral Load (kN)
Corner	1.2D + 1L +	-1.12	498.60	4	124.65

	1E _y				
Exterior	1.2D + 1L + 1E _x	0.52	311.40	6	51.9
Interior	0.9D + 1L	0.34	502.70	9	55.85

The main supporting soil layers were analyzed on the basis of constant modulus of horizontal subgrade reaction n_h and subgrade modulus at depth z , k_z , with respect to the length of the piles. The pile in this project goes through multiple layers of stiff clay and a layer of silty sand and this combination gives the prevalent lateral resistance. The depths and the type of soil used as representative of the soil profile and n_h were selected and estimated using the correlations with SPT N-values of the stiff clays and silty sand.

Table 4.2.2.1.9.2 Modified lateral bearing capacity values

Depth (m)	Type of soil	n_h (kN/m ³)	k_z (kN/m ²)
6.8	Stiff clay (CH)	3 900	26 500
16.0	Stiff clay (CL)	6 600	105 600
20.6	Stiff clay (CL–ML)	9 000	185 000
22.1	Silty sand (SM)	11 000	243 000

The values demonstrate the gradual gradual increase in subgrade stiffness and depth and soil strength. Hardened clays ensure a lot of lateral support in the upper and the middles of the pile and the silty sand layer at the depth of about 22 m makes the pile even stiff towards the toe.

By using Brom's method we obtain characteristic length of a pile:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \quad (4.2.2.1.9.1)$$

where:

E_p - modulus of elasticity of pile material,

I_p - moment of inertia of the pile section,

n_h - constant modulus of horizontal subgrade reaction.

The concrete modulus elasticity is:

$$E_p = 27\,806 \times 10^3$$

The moment of inertia for a circular pile with diameter = 0.40 is:

$$I_p = \frac{\pi D^4}{64} = \frac{\pi (0.40)^4}{64} \approx 1.26 \times 10^{-3} \text{ m}^4$$

A representative value of the horizontal reaction of the soil foundation is selected from a layer of stiff clay at a depth of about 16 m:

$$n_h = 6600 \text{ kN/m}^3$$

By substituting all these values together, we obtain:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \approx \sqrt[5]{\frac{27\,806 \times 10^3 \times 1.26 \times 10^{-3}}{6.6 \times 10^3}} \approx 1.4 \text{ m}$$

Since the pile length is 12, the ratio becomes:

$$\frac{L}{T} = \frac{12}{1.40} \approx 8.6 > 5$$

Therefore, according to Brom's classification, the pile behaves like a long pile under the lateral loading.

Once the characteristic length T is determined, the next step is the analysis of the contribution made to the lateral resistance of the pile by the surrounding soil. Because bending of the pile is mainly caused by mobilization of passive pressures, similar angle of friction of $\varphi = 30$ was used to model the composite behavior of the layers of the silty sand and stiff clay at the site. The passive earth-pressure coefficient may be calculated using this value as:

$$K_p = \tan^2(45 + \frac{\varphi'}{2}) = \tan^2(60) = 3 \quad (4.2.2.1.9.2)$$

This coefficient is the reflection of the growth of horizontal stress before the pile, which appears when the pile deflects laterally, and is one of the main parameters in the formula of Broms of ultimate lateral capacity.

As the soil is only capable of offering resistance to a certain point to which the section of the pile to be supported is still structurally sufficient, the bending strength of the piece of piling itself must be verified as well. To this end, the section modulus of the circular reinforced-concrete pile is calculated as follows using:

$$S = \frac{I_p}{D/2} \quad (4.2.2.1.9.3)$$

that gives a direct estimate of the capacity of the section to withstand bending strains.

Replacing the pile diameter of 0.40 m and the moment of inertia calculated above will give:

$$S = \frac{0.00126}{0.20} = 0.00628 \text{ m}^3$$

It is possible to determine the yield moment using this value multiplied by the product of the section modulus and the product of the yield stress of the concrete:

$$M_y = S \times F_y \quad (4.2.2.1.9.4)$$

$$M_y = 0.00628 \times 35000 = 219.8 \text{ kN/m}$$

This point marks the point beyond which the pile would start bending.

With the determination of the soil passive resistance parameter as well as the structural bending limit of the pile, the ultimate capacity to take place in the lateral direction can be estimated now. The approach by Broms correlates the parameter of nondimensional bending $\frac{M_y}{D^4 \gamma' K_p}$ to a coefficient C, that can be later used to calculate the final resistance. An

effective unit weight of soil $\gamma' = 18 \text{ kN/m}^3$ will give a nondimensional moment ratio of:

$$\frac{M_y}{D^4 \gamma' K_p} = \frac{219.8}{0.40^4 \times 18 \times 3} \approx 159$$

According to the Brom's, this corresponds to a coefficient of approximately $C = 45$.

Then the ultimate lateral capacity is determined by the formula:

$$Q_{u(g)} = C \times K_p \gamma' D^3 \quad (4.2.2.1.9.5)$$

$$Q_{u(g)} = 45 \times 3 \times 162 \times 0.4^3 = 1399.68 \text{ kN}$$

After all that steps, the factor of safety 2.5 is applied:

$$Q_{all} = \frac{Q_{u(g)}}{FS} \quad (4.2.2.1.9.6)$$

The calculated lateral bearing capacity of the columns (interior, exterior, and corner) were calculated by multiplying the ultimate and allowable bearing lateral capacities of a single pile with the number of piles in support of a column. The values obtained are summarized in the table below with the applied lateral loads of each type of column

Table 4.2.2.1.9.3 Reinforced design for a single pile

Type of column	$Q_{u(g)}$	Q_{all}	$Q_{lateral}$

Interior	1399.68	466.56	63
Exterior	933.12	311.04	53.94
Corner	622.08	207.36	32.54

According to the findings in the table, the lateral loads applied to all the types of columns are quite low compared to the respective allowable lateral capacities. Thus, the sideways performance of the pile foundation system is also good, and all the piles are safe against the applied lateral pressure.

4.2.2.2 Lateral Deflection

The deflection of piles under lateral loads is an important consideration in foundation design, particularly of high-rise buildings. This report provides two methods for calculating the deflection of the piles in the foundation system - Brom's Method and the Elastic Method. These methods help to understand how piles will perform when subjected to lateral loads and ensure both the strength and deflection of the piles is satisfactory.

The Brom method, commonly used for calculating deflections in granular soils, such as sand, offers a straightforward solution. It determines the dimensionless lateral deflection (DLD) considering the pile and soil characteristics. Alternatively, the Elastic Method is a more advanced method which considers both the load and the moments from the structure. It is applicable for more complicated systems and offers an in-depth understanding of deflection.

Brom's method uses the concept of dimensionless lateral deflection. It requires specifying the dimensionless pile length η , which is given by the expression:

$$\eta = \sqrt[5]{\frac{n_h}{E_p I_p}} \quad (4.2.2.2.1)$$

where:

η - is the soil parameter (e.g. soil density),

E_p - is the modulus of elasticity of the pile,

I_p - is the pile flexural rigidity.

The value of DLD for the fixed pile in the sand in the report is taken to be 0.1. The value is used to calculate the deflection using this equation (if $z = 0$):

$$x_z = \frac{DLD \times Q_g \times L}{(n_h)^{\frac{2}{5}} \times (E_p I_p)^{\frac{3}{5}}} \quad (4.2.2.2.2)$$

where:

Q_g - is lateral load of the pile

L - is the length of the pile.

The equation will produce the deflection for the pile under the load and also this approach can be used to calculate the deflection for simpler pile-soil systems with homogeneous soils, and a good method for estimating deflection in simple soil-pile interactions where the soil is considered to be uniform.

The Elastic Method is an advanced approach that takes into account the lateral load acting on the pile, as well as the moment and forces in the system. The deflection at a certain depth is given by:

$$x_z(z) = A_x \frac{Q_g \times T^3}{E_p I_p} + B_x \frac{M T^2}{E_p I_p} \quad (4.2.2.2.3)$$

where:

A_x and B_x - are functions of the depth z

T - is the length scale

M - is the moment due to the lateral load.

The length T of the pile-soil system is found using:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \quad (4.2.2.2.4)$$

The Elastic Method is especially applicable in the case of more complex systems, in which the interaction between the soil and the pile should be further detailed. This type of analysis is very informative in the way the piles will behave when subjected to various loading conditions when both the deflection and internal forces are considered.

The results of the calculations of the deflections of the various groups of piles under the applied lateral loads using both methods are below:

Table 4.2.2.2.1 Deflection calculation results

Pile Group	Axis	Lateral Load Q_g (kN)	Moment (M) (kN·m)	Deflection (x_z) (mm)
Group 1	x	86	835.6	1.34
Group 1	y	-73	563.1	2.44

Group 2	x	85	892.4	2.3
Group 2	y	110	658.3	0.32
Group 3	x	50	734.6	2.3
Group 3	y	92	454.2	2.6

The findings show that the deflections are not excessive as they do not surpass the allowable values of the design. Such calculations make sure that the foundation system will work as anticipated when it is subjected to lateral loads.

Subgrade Reaction Coefficient

Coefficient of subgrade reaction is one of the important parameters in the soil-pile interaction. It is determined by various techniques:

Linear Analysis by Brom and Francis (1968):

$$k_h = 0.65 \times \left(\frac{s}{1-v^2} \right) \times \frac{E_s}{E_p} \quad (4.2.2.2.5)$$

Davisson and Reese method:

$$k_h = n_h \times z \quad (4.2.2.2.6)$$

SPT correlation:

$$k_h = \frac{N}{5} \quad (4.2.2.2.7)$$

The findings show that the deflections are not higher than the acceptable limits and within the acceptable values of the design. These computations will make sure that the foundation system will behave as it is supposed to in terms of lateral loads.

These techniques are applied to find out how the soil will react to the movement of the pile so that the foundation of the system is optimized to meet the anticipated lateral loads.

Both the Method of Brom and the Elastic Method gave the same results as the lateral deflections of the piles. The deflections made are under the acceptable range which assures the safety and integrity of the foundation design. The subgrade reaction coefficients also help determine stability of the system so that the soil-pile interaction is duly considered during the design.

This detailed analysis establishes that the design is appropriate in relation to the necessary structural requirements, and the foundation system can support the anticipated lateral forces.

4.2.2.3. Group Piles

Designed loads of the columns were obtained from Structural Analysis of the project

Table 4.2.2.3.1 Design capacity of columns

Column Type	Interior	Exterior (5m span)	Exterior (20m span)	Corner
Q_{design} (kN)	10444.892	7769.544	10152.79	5350.357

Group pile design was analysed by comparing number of piles calculated by dividing design load to allowable bearing capacity. Then the closest n1 and n1 values were selected.

$$Number\ of\ piles = \frac{Q_{design}}{Q_{all}} \quad (4.2.2.3.1)$$

Table 4.2.2.3.2 Calculation of number of piles

D(m)	L(m)	Interior	Exterior (5m span)	Exterior (20m span)	Corner	Internal corner	Internal Middle
0.3	9	6x7	5x5	5x6	4x5	6x7	6x6
0.3	12	5x6	4x4	5x5	3x4	5x6	5x5
0.3	15	4x4	3x3	3x4	3x3	4x4	4x4
0.4	9	5x5	4x4	4x5	3x4	5x5	5x5
0.4	12	4x5	3x4	5x5	3x3	4x5	4x5
0.4	15	4x4	3x3	3x4	2x3	3x4	3x4
0.5	9	4x5	3x4	4x4	3x3	4x5	4x4
0.5	12	4x4	3x3	3x4	3x3	4x4	4x4
0.5	15	3x4	3x3	3x3	2x3	3x4	3x4

4.2.2.3.1 Group Piles Efficiency. Converse–Labarre Group Efficiency Method

The overlap of the stress zones of piles when they are used in a group decreases the efficiency of the system as a whole. The Converse Labarre method is employed in order to consider this interaction effect. It offers a low order of calculating the effect of the arrangement of piles in the performance of the group.

In the adjusted form of analysis used in this project, the efficiency is determined as

$$\eta = (1 - \frac{(n_1-1)n_2 + (n_2-1)n_1}{90n_1n_2}) \times \theta \quad (4.2.2.3.1.1)$$

and the first of the expression represents the interaction between the piles which are adjacent, and θ is a spacing-dependent term that varies the efficiency by the actual ratio of the piles.

The value of θ is obtained directly from the spacing relationship

$$\theta = 6\left(\frac{d}{D}\right)^2 - 5\left(\frac{d}{D}\right) \quad (4.2.2.3.1.2)$$

which provides the spacing influence for the given ratio $\frac{d}{D}$

For the pile arrangement used in this project, the spacing ratio is $\frac{d}{D} = 3$. Substituting this value into the expression gives the final coefficient:

$$\theta = 18.44$$

The efficiency value is then used in the equation of efficiency above to give the ultimate group efficiency of each pile configuration. The results obtained are computed as shown in the table below.

Table 4.2.2.3.1.1. Converse–Labarre Group Efficiency Results

D (m)	d (m)	L (m)	η (Interior)	η (Exterior 5)	η (Exterior 20)	η (Corner)
0.4	1.2	9	0.692	0.726	0.709	0.76
		12	0.692	0.726	0.709	0.76
		15	0.709	0.76	0.726	0.761
0.3	0.9	9	0.682	0.709	0.692	0.726
		12	0.682	0.709	0.692	0.726
		15	0.692	0.726	0.709	0.76
0.5	1.5	9	0.709	0.760963	0.726	0.7951

		12	0.709	0.76	0.726	0.795
		15	0.726	0.76	0.76	0.795

4.2.2.3.2 Group Piles Efficiency. Angeles Group Action Method

Los Angeles Group Action technique is an empirical method that was constructed to assess the decrease in load-carrying capacity which is experienced when the piles work as a group. As piles are built in close patterned arrangements, the stress areas surrounding each pile intersect and this causes the general group efficiency to reduce. The approach measures this interaction in a geometric relation which takes into consideration piles spacing, piles diameter, and the number of piles in each main direction.

The Los Angeles method of determining the efficiency of a pile group has the following equation:

$$\eta = 1 - \frac{D}{\pi d n_1 n_2} [n_1(n_2 - 1) + n_2(n_2 - 1) + \sqrt{2}(n_1 - 1)(n_2 - 1)]$$

(4.2.2.3.2.1)

where

D - denotes the pile diameter,

d - the center-to-center spacing between adjacent piles,

n_1, n_2 - represent the number of piles along the width and length of the group

respectively,

η - the resulting efficiency factor.

The three interaction terms within the brackets are the major ways of piling interaction in a rectangular group: piles in one row interacting with each other, piles in one row interacting with those in the next row, and the piles in the remaining rows interacting diagonally. The diagonal component is weighted by 2 which indicates the longer path of influence through the group geometry. The whole phrase is normalised by the product which modulates the interaction his/her size by the size of the pile group.

Such an approach gives a conservative but realistic estimate of the efficiency of groups and is commonly applied in engineering practice to give a preliminary analysis of the behaviour of a pile-group. Efficiency values obtained were determined through use of the above expression on each pile arrangement given consideration in this study.

Table 4.2.2.3.2.1 Los Angeles Group Action Efficiency

D (m)	d (m)	L (m)	η (Interior)	η (Exterior 5)	η (Exterior 20)	η (Corner)
0.4	1.2	9	0.756	0.791	0.774	0.826
		12	0.756	0.791	0.774	0.826
		15	0.774	0.826	0.791	0.856
0.3	0.9	9	0.745	0.774	0.756	0.791
		12	0.745	0.774	0.756	0.791
		15	0.756	0.826	0.7743	0.826
0.5	1.5	9	0.774	0.774	0.774	0.826
		12	0.774	0.774	0.774	0.856
		15	0.791	0.826	0.791	0.856

Increasing the pile length from 9 m to 15 m enhances shaft resistance due to more interaction with clay and sand layers, therefore lengthening is an efficient way to increase load bearing capacity on this profile.

After analysing cost effectiveness it was decided to use the piles with a diameter of 0.4 m and length of 15 m which also allows use of less number of piles for the same total load. Moreover, 0.4 m diameter piles offer effective pile cap arrangement meaning less group pile settlement.

The efficiency of pile group also clarifies 2 options are best suitable. The above tables for calculating the efficiency ratio illustrate the 0.4 m diameter and 15 m length pile is one of the most efficient options.

4.2.2.3.3.Determination of Equivalent Foundation Dimensions for the Pile Group

The group is considered to be an equivalent block foundation to carry out the bearing capacity and settlement analysis of the pile group. Thus, the width (B) and length (L) of the pile group should be specified. These dimensions are affected by the diameter of the piles

(D), the distance that exists between the piles at the center (d), and the number of piles per direction (n_1 and n_2).

The standard geometric relationships are used to determine the equivalent foundation footprint

$$B = (n_1 - 1)d + D \quad (4.2.2.3.3.1)$$

$$L = (n_2 - 1)d + D \quad (4.2.2.3.3.2)$$

where

D - pile diameter,

d - center-to-center spacing between piles,

n_1 - number of piles along the width direction,

n_2 - number of piles along the length direction.

These formulas are the total distance between the center of the first pile to the center of the last pile all with an addition of one diameter of a pile. When all the piles are applied to each pile configuration, the values of B and L will be as presented in the table below.

Table 4.2.2.3.3.1 Equivalent Foundation Dimensions for the Pile Group and the group Bearing Capacity

Diameter (m)	Centre to centre spacing(m)	B (m)	L (m)	Group pile $n_1 \times n_2$	Efficiency η	$\eta n_1 n_2 Q_{all}$ (kN)
0.4	1.2	4,4	4,4	4x4	0,774	13907,23
0.4	1.2	4	2.8	3x4	0,826	11131,18
0.4	1.2	2.8	2.8	3x3	0,774	7822,82
0.4	1.2	2.8	1.6	2x3	0,774	5745,22

The value acquired with the efficiency of the group pile is higher than the design value, since acceptable.

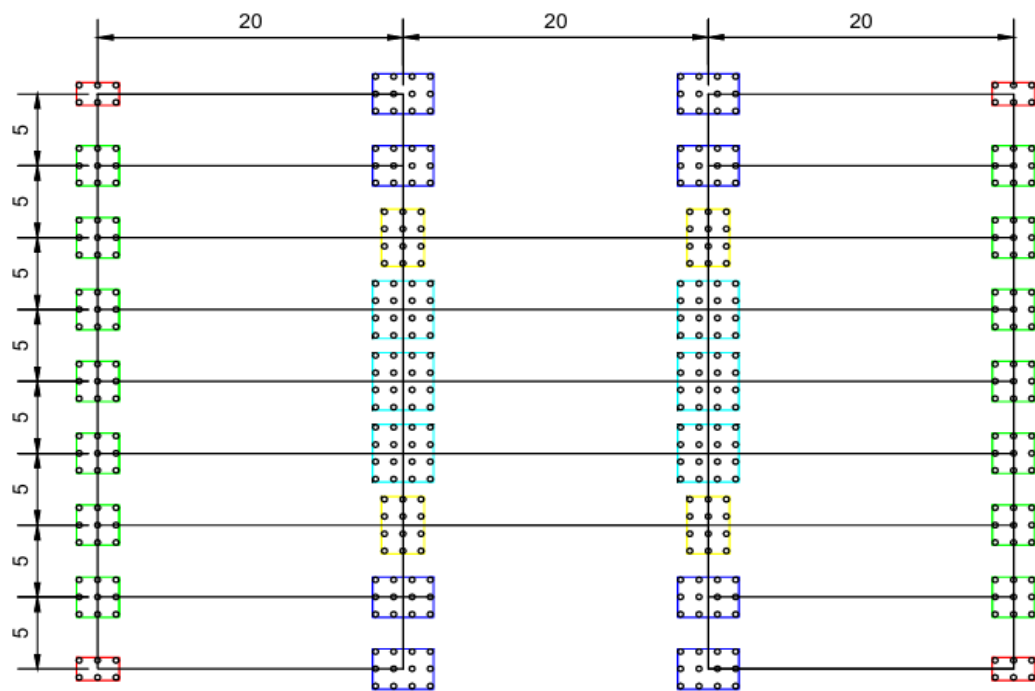


Figure 4.2.2.3.3.1.1. Group Pile Illustration

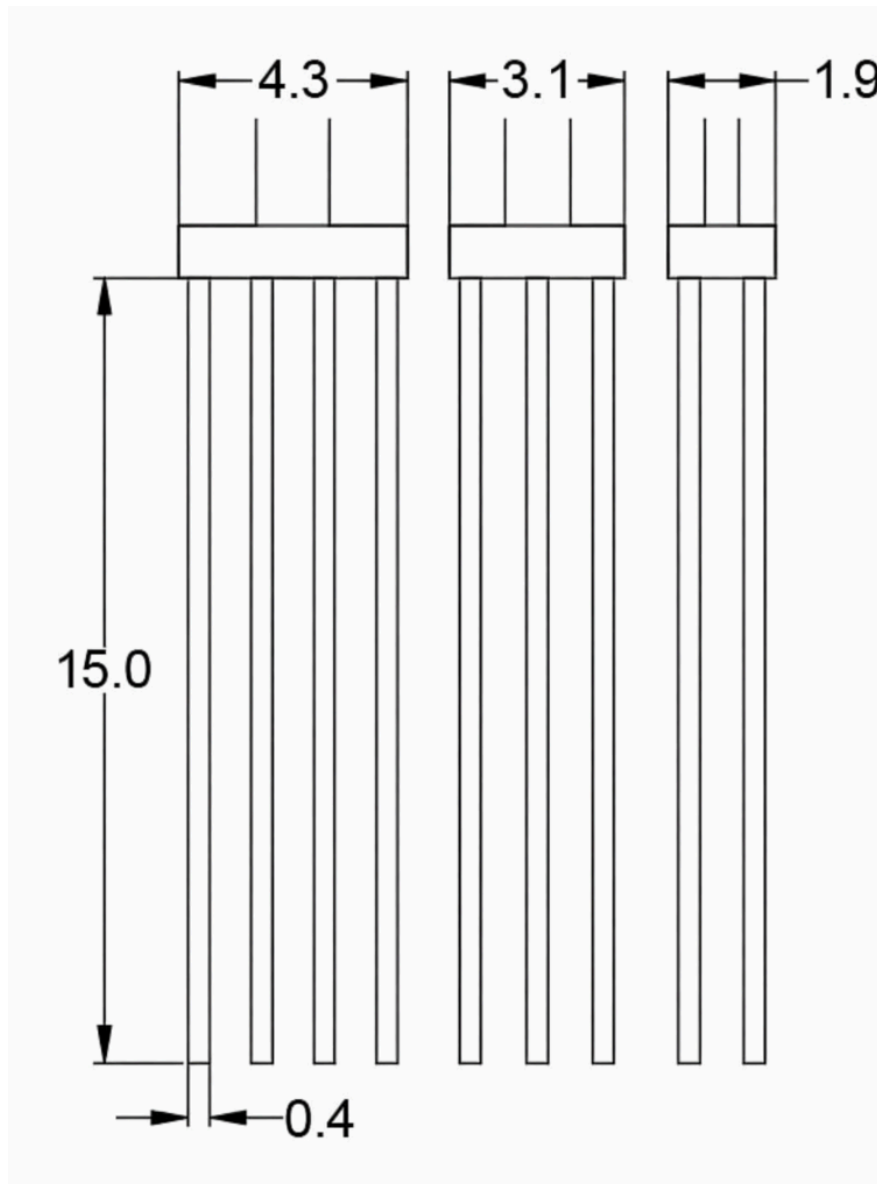


Figure 4.2.2.3.3.1.2 Illustration of Column to Pile Connection

4.2.2.3.4 Elastic Settlement of Group Pile

Elastic settlement of the pile group was determined by modeling the pile group as being an inflexible block on the supporting soil. Owing to the fact that the piles end in hard clay, the deformation process is mainly elastic in nature and the settlement can be modeled as a function of the individual-pile deformation as well as the geometric size of the group. The more the width of the pile block, the more mass of the soil is mobilised and this leads to a greater elastic deformation than with a single pile. In order to accommodate this interaction, the elastic settlement of the single pile is scaled by a factor that is dependent on the ratio of the equivalent group width and the pile length. This results in the relation below:

$$S_{e(g)} = \sqrt{\frac{B_g}{L}} S_e \quad (4.2.2.3.4.1)$$

where:

$S_{e(g)}$ - the elastic settlement of the pile group.

s_e - the elastic settlement of one pile derived out of component analysis.

B_g - the pile group's width.

L - the pile length governing the depth over which the stresses propagate.

The elastic settlement of the single pile, which is summed up in the elastic shortening of the shaft, the deformation of the base, and the interaction of the shaft with the soil, is:

$$S_{e(total)} = S_{e1} + S_{e2} + S_{e3} \quad (4.2.2.3.4.2)$$

By substituting our values by this equation, we obtain:

$$S_{e(total)} = 0.00138 + 0.00273 + 0.00118 = 0.005 \text{ m}$$

The settlement of the pile group calculated using the group-interactions expression is calculated by replacing the relevant equivalent widths in each configuration.

For the group width $B_g = 4 \text{ m}$:

$$S_{e(g)} = \sqrt{\frac{4}{15}} \times 0.00529$$

$$S_{e(g)} = 0.0167 \text{ m}$$

For the group width $B_g = 2.8 \text{ m}$:

$$S_{e(g)} = \sqrt{\frac{2.8}{15}} \times 0.00529$$

$$S_{e(g)} = 0.0142 \text{ m}$$

For the group width $B_g = 1.6 \text{ m}$:

$$S_{e(g)} = \sqrt{\frac{1.6}{15}} \times 0.00529$$

$$S_{e(g)} = 0.0106 \text{ m}$$

These values are the elastic deformation of the pile group in working loads and the increase of settlement with the expanding effective block width.

Table 4.2.2.3.4.1. Elastic Settlement of the Pile Group

Single-Pile Elastic Settlement S_e (m)	Equivalent Group Width B_g (m)	Group Elastic Settlement $S_{e(g)}$ (m)
0.00529	4.0	0.0167
	2.8	0.0142
	1.6	0.0106

4.2.2.3.5 Consolidation Settlement of Group Pile

The compression settlement of the pile group was also considered based on compressible layers of clay that were found under the pile cap. Though the piles load the stiff clay at the pile ends, the occurrence of soft clay above the end causes time-dependent settlement as a result of dissipation of additional pore pressure. The behaviour is controlled by compressibility properties of individual layers of clay and the increase in effective stress caused by the pile group.

The settlement was estimated by calculating the increment of effective stress in the middle depth of every compressible layer, which was done by the stress-distribution method. After getting the increment of stress, the primary consolidation settlement of each layer was determined, using the classical one-dimensional relationship of consolidation. The aggregate consolidation settlement of the pile group is then obtained as a sum of the contributions of the individual clay layers.

The settlement of a layer will be calculated as below:

$$S_{c(i)} = \frac{C_c}{1+e_0} H_i \log\left(\frac{\sigma'_0 + \Delta\sigma'_i}{\sigma'_0}\right) \quad (4.2.2.3.5.1)$$

where

$S_{c(i)}$ - the major consolidation settlement of layer,

C_c - the compression index of the clay is,

e_0 - the initial void ratio,

H_i - the thickness of the layer,

σ'_0 - the first effective vertical stress at the middle of the layer,

$\Delta\sigma'_i$ - the increase in the effective vertical stress due to the pile group load.

Replacement of the soil parameters which were determined in the site investigation and replacement of the calculated stress increments per clay layer gives the main settlement in consolidation. A comparison of the two compressible layers expression gives the summation of the consolidation settlement of the group per block width. The same happens with the response of the different configurations analyzed since the group of piles affects the overlying soft layers in a similar way irrespective of the width of the group.

The process of the first clay layer is depicted below to explain how the final settlement values were retrieved.

In this layer, the thickness is H_1 1.53 m, compression index C_c is 0.194, initial void ratio e_0 is 0.8 and the increment of stress caused by the pile group $\Delta\sigma'_i$ is 127 kPa. The replacement of these values gives:

$$S_{c(1)} = \frac{0.194}{1+0.8} 1.53 \times \log\left(\frac{\sigma'_0 + 152}{\sigma'_0}\right)$$

The second layer of clay is done in the same manner.

$H_2 = 1.52$ m; $C_c = 0.299$; $e_0 = 0.9$ m and $\Delta\sigma'_2 = 152$ kPa.

$$S_{c(2)} = \frac{0.299}{1+0.9} 1.53 \times \log\left(\frac{\sigma'_0 + 152}{\sigma'_0}\right)$$

The initial effective stresses at the mid-depths of the layers were obtained out of the overburden calculations that were made in the preceding section. On comparing both expressions, the settlements of the two layers are added:

$$S_{c(g)} = S_{c(1)} + S_{c(2)} \quad (4.2.2.3.5.2)$$

The calculations carried out result in a total consolidation settlement of around are shown below in the table.

Table 4.2.2.3.5.1 Consolidation Settlement of the Pile Group

Equivalent Group Width (B_g) (m)	Total Consolidation Settlement (S_{c(g)}) (m)	Group Total Settlement S_(g) (m)
4.0 (4x4)	0.0067	0.017065
2.8 (4x3)	0.0059	0.014277
2.8 (3x3)	0.0059	0.010113

1.6 (2x3)	0.0059	0.010793
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The values of these reflect the influence of the properties of the upper soft clay to control dominantly the consolidation element, and is thus only weakly dependent on the block width of the pile group.

4.2.2.3.6 Reinforcement Analysis

4.2.2.3.6.1 Pile Reinforcement

Structural design of individual bored concrete piles was performed in accordance with the requirements of ACI 318 Building Code requirements. The main aim was to make sure that the pile has enough bearing capacity for safe transfer of factored loads from pile cap to supporting soil stratum.

Based on the prescribed geotechnical sizing selection, for all of the individual piles in the foundation system were used following parameters:

Table 4.2.2.3.6.1.1 Pile parameters

Compressive strength	30 MPa
Yield strength	420 MPa
Cross sectional area	125664 mm^2
Pile diameter	400 mm

Critical loading for individual piles happens in the case of 6 pile cap group in corner column. Factored load (P_u) affecting this group is equal to 5350 kN. Considering uniform distribution of load, maximum load on individual pile is:

$$- P_u \text{ for each pile} = \frac{5350}{6} = 891.67 \text{ kN}$$

This value illustrates maximum compressive force which will experience each individual pile with diameter of 400 mm.

For elements of deep foundation affected by axial compression with minimal bending, standards of ACI requiring minimum of longitudinal reinforcement ratio (ρ_{min}) in the value

of 1% from the cross-sectional area. This is required to prevent the case of accidental shrinkage or eccentricity.

- Required steel area (A_{sc}): $A_{sc} = 0.01 * A_g = 0.01 * 125664 \text{ mm}^2 = 1257 \text{ mm}^2$

To satisfy this requirement ensuring symmetrical circular cage which is guarantee of uniform bending resistance in all directions, T20 bars (20 mm) were chosen.

- Area of one bar = 314 mm^2
- Chosen reinforcement: 6 of T20 bars
- Area of the provided steel: $6 * 314 \text{ mm}^2 = 1884 \text{ mm}^2 > 1257 \text{ mm}^2 \rightarrow \text{OK}$

Axial compressive strength is calculated with the following formula:

$$\phi P_n = 0.65 * 0.8 * (0.85f'_c(A_g - A_s) + f_y A_s) \quad (4.2.2.3.6.1.1)$$

$$\phi P_n = 0.65 * 0.8 * (0.85 * 30(125664 - 1884) + 420 * 1884) = 2053 \text{ kN}$$

Table 4.2.2.2.6.1.2 Reinforcement design for Piles

Column Type	Total Ultimate Load (P_u)	No. of Piles	Load per Pile (P_u)	Required Steel (A_{sc})	Provided Reinforcement	Pile Capacity (ϕP_n)	Status
Corner Column (6-pile)	5350.00 kN	6	891.67 kN	1257 mm^2	6-T20 (1884 mm^2)	2,053 kN	Safe
Inner Column (16-pile)	10444.89 kN	16	652.81 kN	1257 mm^2	6-T20 (1884 mm^2)	2,053 kN	Safe
Inner Column (12-pile)	10444.89 kN	12	870.41 kN	1257 mm^2	6-T20 (1884 mm^2)	2,053 kN	Safe
Outer Column (12-pile)	7769.54 kN	12	647.46 kN	1257 mm^2	6-T20 (1884 mm^2)	2,053 kN	Safe
Outer Column (9-pile)	7769.54 kN	9	863.28 kN	1257 mm^2	6-T20 (1884 mm^2)	2,053 kN	Safe

Calculated capacity with the most load is 2053 kN. This value significantly more than factored load, which means design is safe. For all of the piles reinforcement will be T20 bars

uniformly distributed in the circle and circular ties T10 with spacing of 150 mm along the pile to ensure shear resistance.

4.2.2.3.6.2 Pile Cap Reinforcement

Pile cap is purposed for safe transformation of concentrated loads and moments of columns for each pile. Designing of pile cap for flexural design performed by ACI 318 Code requirements, considering that pile caps as rigid slab.

For designing of all pile caps in foundation were used following parameters:

Table 4.2.2.3.6.2.1 Parameters for pile cap design calculations

Column dimensions	750 x 750 mm
Pile Cap thickness	1 m
Clear cover	75 mm
Effective depth	905 mm
Pile spacing	1.2 m
Edge distance	0.35 m
Reinforcement	T25
Pile Diameter	0.4 m

Flexural reinforcement was found through calculation of maximum bending moment in critical sections. Lever arm for each pile measured from center of pile to column edge. Ultimate moment then was calculated by summing of moments, which are created by upward factored reactions of piles located outside of critical section.

Required steel area calculated using equations of flexural resistance in sections. Furthermore, area of steel checked by ACI requirements such as temperature reinforcement and minimum shrinkage.

$$A_{s,min} = 0.0018 * B * h \quad (4.2.2.3.6.2.1)$$

Calculations performed for five different group pile configurations and load conditions. Longitudinal direction related to reinforcement which is parallel to longest pile cap and transverse direction to parallel smaller size. For square pile caps they are same in both sides.

Table 4.2.2.3.6.2.2 Pile Cap Design

Group Configuration	Column type	Ultimate Load (P_u)	Cap Dimensions (Width × Length)	Reinforcement in y - direction	Reinforcement in x - direction
6-Pile Group	corner	5350 kN	1.9 m × 3.1 m	10 - T25	12 - T25
9-Pile Group	exterior	7769.54 kN	3.1 m × 3.1 m	14 - T25	14 - T25
12-Pile Group (Light)	exterior	7769.54 kN	3.1 m × 4.3 m	20 - T25	16 - T25
12-Pile Group (Heavy)	interior	10444.89 kN	3.1 m × 4.3 m	27 - T25	18 - T25
16-Pile Group	interior	10444.89 kN	4.3 m × 4.3 m	27 - T25	27 - T25

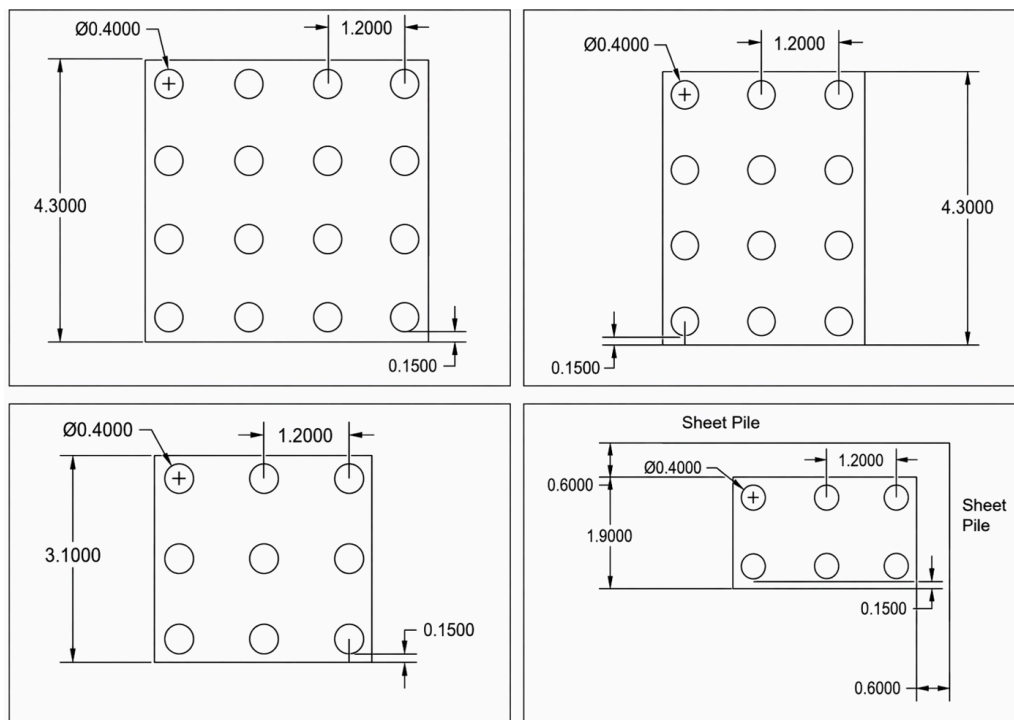


Figure 4.2.2.3.6.2.1 Pile Cap Design

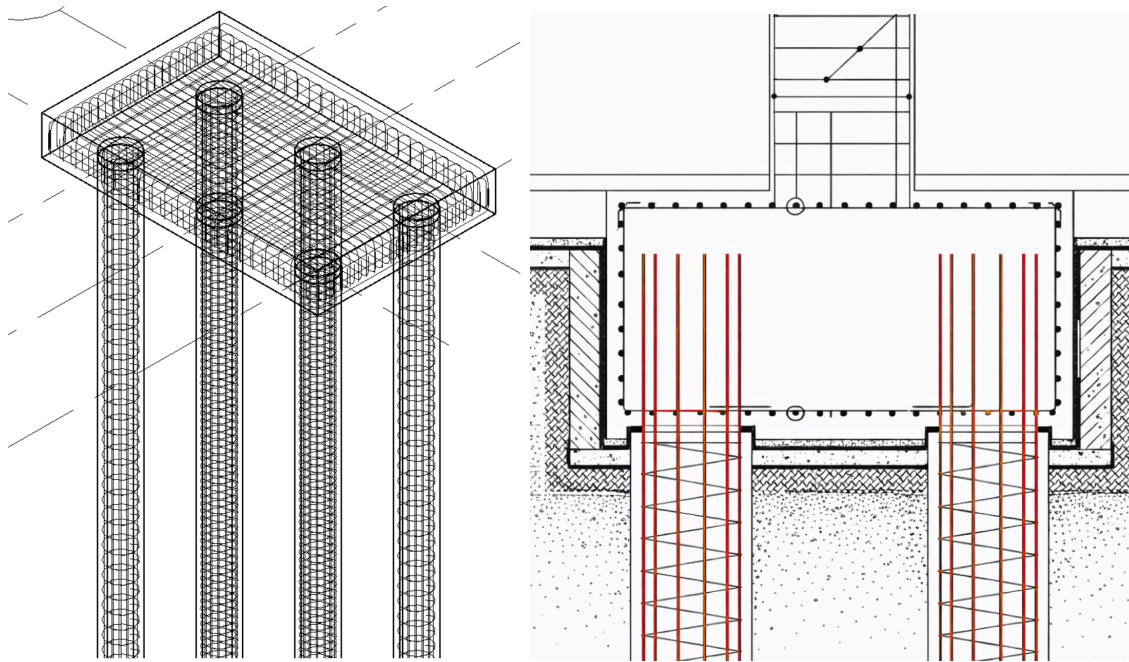


Figure 4.2.2.3.6.2.2 Pile Cap and Pile reinforcement 3D Design Structure

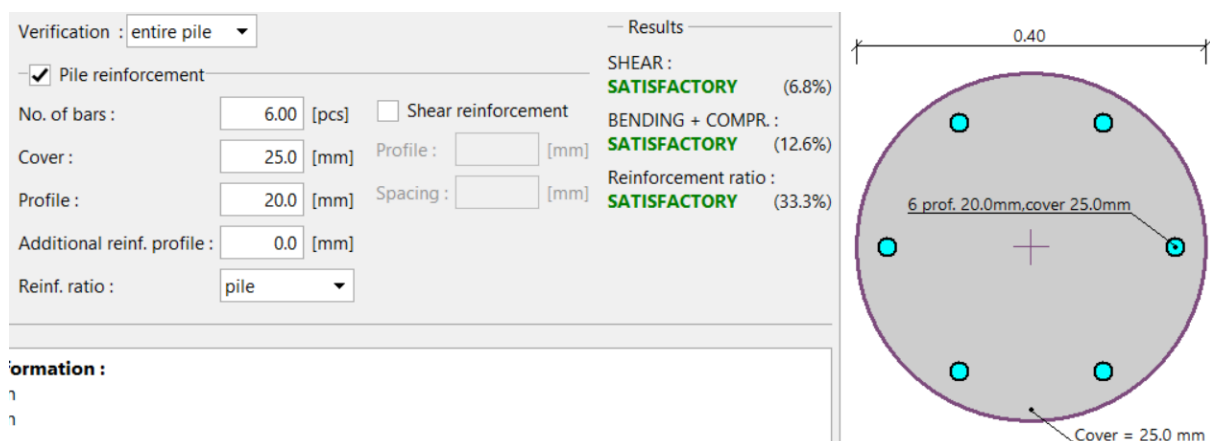


Figure 4.2.2.3.6.2.3 Pile Cap Check in GEO5

Pile to pile distance was used as 3 times the diameter of the pile according to the regulations. In the result of Pile Cap check shear, bending, reinforcement ratio factors were satisfactory.

4.2.2.3.7 Pile head deflection

Though calculations of Ultimate Limit State asserts that 12.15 m contiguous pile wall has enough bearing capacity to resist maximum bending moments from excavation depth in 9.75 m, assessment of Serviceability Limit State is necessary to evaluate lateral deformations. Too much later deflection of retaining wall can lead to ground subsidence, which has a negative effect on neighbour structures and underground utilities of hotel.

For assessment of horizontal pile head deflection performed calculations using Elastic and Broms method. In analytical purposes wall modelled as a pile with the width of 1 meter, embedment depth of 2.4 m in stiff clay, which is subjected to lateral deflection and bending moment in dredge line level from retained soil.

Table 4.2.2.3.7.1 Pile head initial parameters

Lateral shear force	175 kN/m
Bending moment	207.5 kN/m
Embedment depth	2.4 m
Wall thickness	0.6 m
Flexural rigidity	$5.34 * 10^5 \text{ kN} * \text{m}^2/\text{m}$

Elastic method

The method of Elastic Subgrade reaction modelling embedment soil as a linear elastic springs. Initially relative stiffness factor was calculated:

$$R = \sqrt[4]{\frac{E_p I_p}{k_s B}} = 2.27 \text{ m}$$

Considering that embedment depth ratio consists approximately 1.05, recessing part of wall acts as short, rigid block, but not as flexible pile. Pile head deflection in the depth of 9.75 m calculated as:

$$y_{top} = y_d + (\theta_d * H_{exc}) + y_{bend} = 0.012 + 0.176 = 0.188 \text{ m}$$

To assess also was used Broms method for cohesive soils under working loads. Using parameter of stiffness for cohesive soils, embedment length consisted 0.746, which confirms wall as short and rigid. Result total deflection in the pile head is:

$$y_{top} = y_d + (\theta * 9.75) = 0.219 \text{ m}$$

Results of two method represent large lateral deflections. In urban excavation projects allowable lateral wall displacements limited with 25-40 mm to prevent destruction of surrounding infrastructure.

4.2.2.4. Software Analysis of a deep foundation

4.2.2.4.1 Single Pile Software Analysis

With all the soil properties inserted into Geo 5 software, the bearing capacity of a single pile of 0.4m diameter and 15m length was analysed.

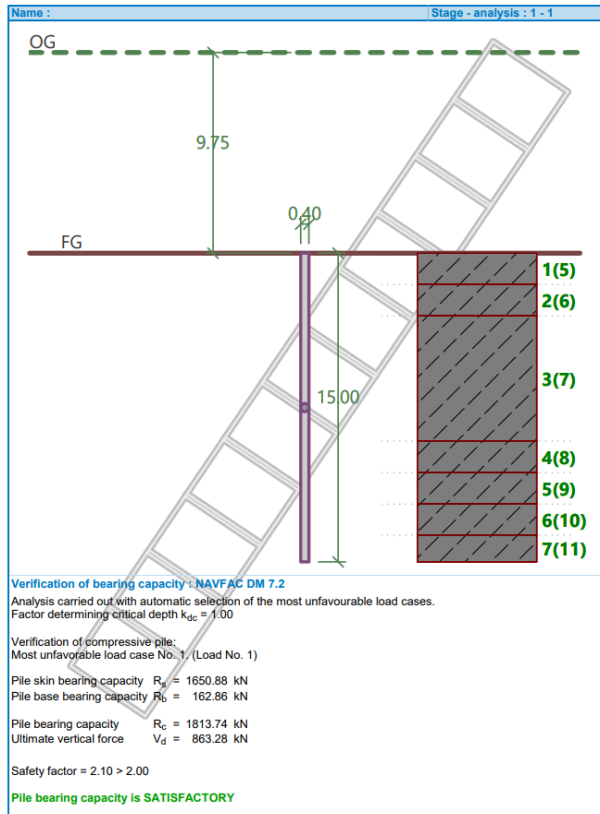


Figure 4.2.2.4.1.1 Single pile analysis on Geo 5.

As can be seen from figure 4.2.2.3.1.1, designed single pile experiences skin bearing capacity of $R_s = 1650.88 \text{ kN}$, and base bearing capacity of $R_b = 162.86 \text{ kN}$, which was later applied into Plaxis 3D for further analysis. With the maximum load possible on a single pile (evenly distributed in a group pile) of $V_d = 863.28 \text{ kN}$ and total bearing capacity of $R_c = 1813.74 \text{ kN}$, pile got the factor of safety equal to 2.1 making sure on the safety of the foundation.

Table 4.2.2.4.1.1. Comparison of Hand calculations and Software analysis of a single pile

Method	D(m)	L(m)	Qu(kN)	Qd(kN)	FoS
Hand Calculations	0.4	15	2559.25	863.28	2.96
Software analysis	0.4	15	1813.74	863.28	2.1

4.2.2.4.2 Software Analysis of a Lateral bearing capacity

Further, we have calculated the lateral bearing capacity in GEO5 of four groups (4x4 interior, 4x3 exterior, 3x3 exterior, 3x2 corner). This is also because we switched the calculation to the Broms method, which enabled us to get these results.

Analysis of deflection :

Value taken from chart = 8.28

Deflection of the pile at ground level. $y = 2.3 \text{ mm}$

Maximum horizontal force $Q_{\max} = 1.69 \text{ kN}$

Reduced resistance of pile $Q_u = 47.18 \text{ kN}$

Horizontal pile resistance is SATISFACTORY

Figure 4.2.2.4.2.1. Lateral bearing capacity for interior 4x4

For example, the lateral bearing capacity of the 4x4 interior group in the GEO5 software is 47.18, as shown in the figure above. Our hand calculation for this case was 63. The following table shows the data for each group, from hand calculations and from GEO5.

Table 4.2.4.2.2.1. Calculation Results

Method	4x4	4x3	3x3	3x2
Hand Calculation	63	45	53.94	32.54
Software Calculation	47.18	47.24	47.21	47.20

We can observe from the table above that the data from the hand calculations and software are different. But, as we can see, the gap between the hand calculation and the software is not large and is acceptable, so it tells us that the calculations are correct.

This slight difference could be caused by human error, lack of attention or rounding or decimal point errors. Furthermore, the discrepancy could be due to a different interpretation of some of the parameters, rough calculations, or different data used.

In summary, these small discrepancies are common in engineering calculations and do not have any impact on the validity of the results.

Furthermore, it is possible to use GEO5 verification data for the lateral bearing capacity, which also contains the factor of safety. This enables us to verify the accuracy of the results.

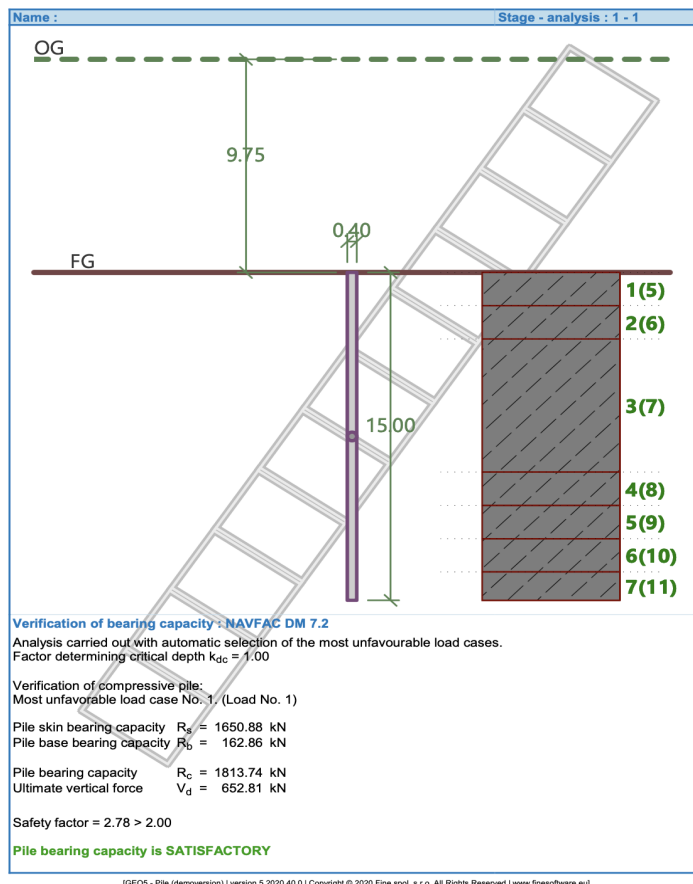


Figure 4.2.2.4.2.2. Factor of safety for interior 4x4

The factor of safety was determined for the four groups and it was confirmed that it is greater than 2, which is the minimum value for safe structure design. This ensures that the lateral bearing capacity values are not only correct, but also meet the safety requirements, confirming that the design is safe.

In summary, the results from the GEO5 software and manual calculations are in good agreement, and the factor of safety indicates that the design is safe and reliable. The small differences are within engineering tolerance and do not affect the design.

4.2.2.4.3 Software Analysis of a Lateral Deflection

Likewise, we got the data for lateral deflection from the PLAXIS software and GEO5 software that gave the relevant data. The previous modeling for the 4x4 interior group is shown in the figure above where more data can also be observed (such as the soil profile in GEO5).

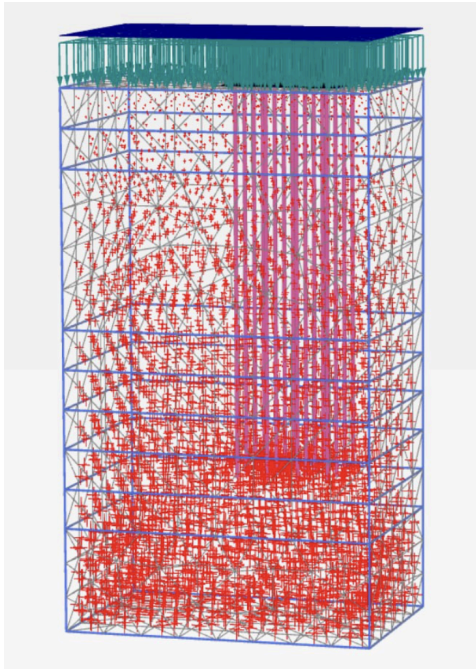


Figure 4.2.2.4.3.1. PLAXIS 3D modelling

The table below presents the lateral deflection using three different calculation methods: GEO5, PLAXIS and manual calculations. It can be seen that the results from all three methods are different, which is due to the presence of small rounding errors and/or small differences in how the calculations are performed.

Table 4.2.2.4.3.1. Calculation of Lateral deflection

Group piles	Number of piles	GEO5	PLAXIS 3D	Elastic method	Brom's method
Interior	4	4.45	1.34	2.4	1.34
Exterior	6	3.25	0.42	1.65	2.3
Corner	9	7.42	0.98	1.99	2.6

While the results obtained are not exactly the same, we can conclude that the calculations are all correct. But we must investigate to find the reason for the difference in one of the values.

4.2.2.4.4 Software Analysis of Group Pile

Geo 5 was further used to analyse the group pile bearing axial bearing capacities. The results are provided below.

Table 4.2.2.3.2.1. Bearing capacity comparison of software analysis and hand calculations of group piles

File Group	4x4	4x3	3x3	3x2
Hand Calculations (kN)	27814.432	22262.36	17645.64	12497.444
Software Analysis (Geo 5) (kN)	22291.211	17970.456	15629.358	10911.155
Design Load (kN)	10444.892	8869.444	7769.544	5350.357
FoS (HC)	2.66	2.51	2.27	2.33
FoS (SA)	2.14	2.03	2.01	2.04

As a result of both hand calculation and software analysis, bearing capacity of group piles are considered to be safe, (FoS higher than 2 in all cases), hence concluded as safe to design, Further, settlements of pile groups were analysed in Plaxis 3D software. Soil Profile was inserted using Mohr-Coulomb model, while piles as embedded beams and pile caps as plates.

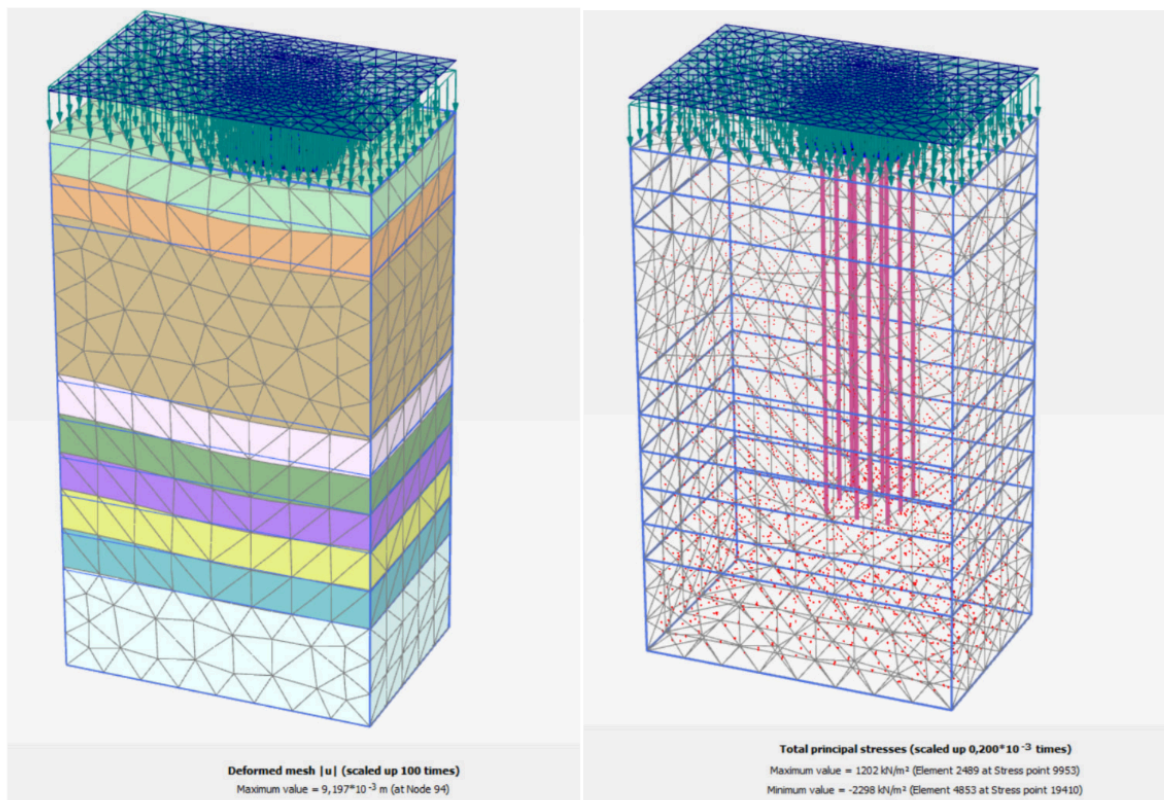


Figure 4.2.2.3.2.1. Deflection of 3x3 group pile on Plaxis 3D

Table 4.2.2.3.2.2. Comparison of hand calculation and software analysis deflection of group piles

Group Pile	4x4	4x3	3x3	3x2
Hand Calculation (m)	0.017065	0.010113	0.014277	0.010793
Software Analysis (Plaxis 3D) (m)	0.015212	0.006316	0.009917	0.008705

Table above shows the total deflections of group piles with the design load applied. Both hand calculations and software analysis shows the satisfied values of deformation not exceeding the allowable limit.

4.3 Retaining Wall Design

4.3.1 Cantilever Wall Design

For the retaining wall, the cantilever design was analyzed as an option. Parameters of the wall were assumed in accordance with the excavation depth

Table 4.3.1.1 Cantilever Wall parameters

H(m)	B(m)	b_t (m)	t_s (m)	b_h (m)	t_b (m)
12.8	7.5	2.5	0.8	4.2	1

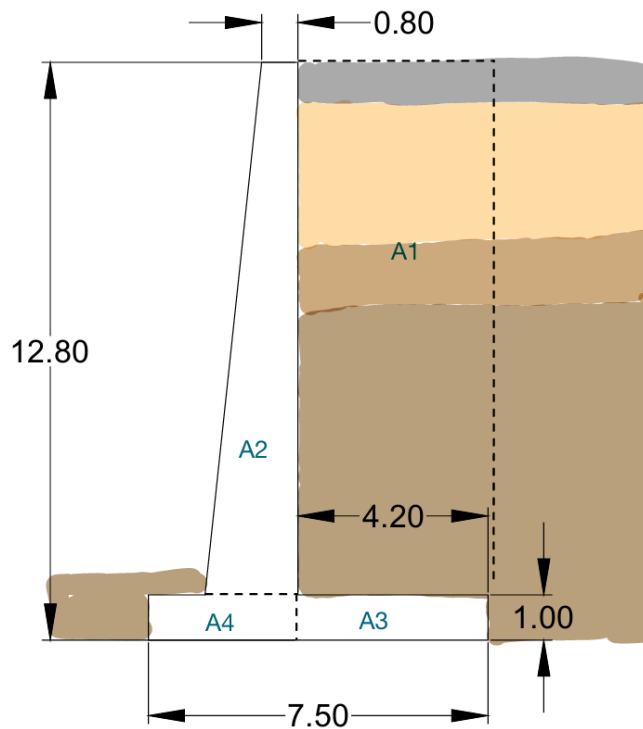


Figure 4.3.1.1. Cantilever Wall

The cantilever wall is mainly designed for up to 7 meters and should be analyzed thoroughly for its safety.

Table 4.3.1.1 Moment Distributuion of Cantilvar Wall

Section	Description	Area, m^2	Unit weight (kN/m^3)	Weight (k/m)	Moment Arm From toe	Moment kN/m/m
1	Soil above heel	53.76	18,85	967,68	5,4	5222,47
2	Stem	10.24	24	245,76	2,9	712,7
3	Heel slab	4.2	24	100,8	5,4	544,32
4	Toe slab	2.5	42	60	1,25	75
Σ	----	-----	----	1374,24	-----	6557,5

There are three main stability checks for retaining walls,

1.Overturning about the toe.

$$FS_{\text{overturning}} = \frac{\Sigma M_R}{\Sigma M_o} \quad (4.3.1.1)$$

where

ΣM_R -sum of the moments of forces tending to overturn about point C

ΣM_o - sum of the moments of forces tending to resist overturning about point C

$$M_o = P_a \frac{H}{3} \quad (4.3.1.2)$$

2.Sliding along the base.

$$FS_{\text{sliding}} = \frac{\Sigma F_R}{\Sigma F_d}$$

where

ΣF_R - sum of the horizontal resisting forces,

ΣF_d - sum of the horizontal driving forces.

$$FS_{\text{sliding}} = \frac{(\Sigma V)\tan(k_1\phi) + Bk_1c_2 + P_p}{P_a \cos\alpha} \quad (4.3.1.3)$$

where

c_2 – average cohesion of the soil

3.Loss of bearing capacity of the supporting soil.

$$FS_{\text{bearing capacity}} = \frac{q_u}{q_{\text{toe(max)}}} \quad (4.3.1.4)$$

Average horizontal location of the resultant from the toe

$$X = \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} \quad (4.3.1.5)$$

$$X = \frac{6557.5 - 2517.3}{1374.24} = 2.94 \text{ m}$$

Eccentricity of the wall

$$e = \frac{B}{2} - X \quad (4.3.1.6)$$

$$e = 0.81 \text{ m}$$

Compression

$$q_{\max(\text{toe})} = \frac{\Sigma V}{B} \left(1 + \frac{6e}{B}\right) \quad (4.3.1.7)$$

$$q_{\max(\text{toe})} = 302 \text{ kPA}$$

q_u can be found with the general bearing capacity equation with the reduction factors evaluated below

$$\varphi = \tan^{-1} \left(\frac{P_a}{\Sigma V} \right) \quad (4.3.1.8)$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\varphi}{90}\right)^2 \quad (4.3.1.9)$$

$$F_{cd} = F_{qd} = 1$$

$$c_{\text{below the wall}} = 96 \text{ kN/m}^2$$

Table 4.3.3.2 Moment Distributuion of Cantilever Wall

Check For	Factor of Safety
Overturning	1.86
Sliding along the base	4.16
Bearing Capacity Failure	0.6

From the table above we can see that Cantilever Wall is not suitable for this design as the Factors of Safety are too low too consider. It might result it excessive settlement and increased shear failure.

4.3.2 Sheet Pile

The basement of a nine-story hotel under construction in San Antonio, Texas, will have three levels, which need to be dug out to around 10 meters. Consequently, the retaining walls should have the capability of counterbalancing the sideways forces exerted by the soils that surround it. For this reason, sheet pile of around 1.5 times excavation depth should be used as a retaining wall.

4.3.2.1 Sheet Pile design

Lateral stresses are seen to be on the underlying sand and clay layers. The earth pressure theory put forward by Rankine was used to calculate these pressures. Using this technique, it is possible to estimate both active (K_a) and passive (K_p) earth pressure coefficients using the obtained friction angle (ϕ) of soil samples in the laboratory.

The coefficients are defined as:

$$K_a = \tan^2(45 - \frac{\phi'}{2}) \quad (4.3.2.1.1)$$

$$K_p = \tan^2(45 + \frac{\phi'}{2}) \quad (4.3.2.1.2)$$

where

K_a - Active earth pressure coefficient,

K_p - Passive earth pressure coefficient.

The corresponding Rankine pressure equations are:

$$\sigma_a' = \gamma H K_a - 2c'\sqrt{K_a} \quad (4.3.2.1.3)$$

$$\sigma_p' = \gamma H K_p + 2c'\sqrt{K_p} \quad (4.3.2.1.4)$$

where:

σ_a' - Active earth pressure,

σ_p' - Passive earth pressure,

γ - Effective unit weight of soil,

H - Depth below ground surface,

c' - Effective cohesion.

The total lateral force per unit length of wall is calculated as:

$$P_a = \frac{1}{2} \gamma H^2 K_a - 2c'H\sqrt{K_a} \quad (4.3.2.1.5)$$

$$P_p = \frac{1}{2} \gamma H^2 K_p + 2c'H\sqrt{K_p} \quad (4.3.2.1.6)$$

where

P_a - active lateral force per unit wall length,

P_p - passive lateral force per unit wall length.

Since the sandy layers are non-cohesive soils ($c' = 0$), the cohesion term is neglected. Thus, the lateral pressures vary linearly with depth.

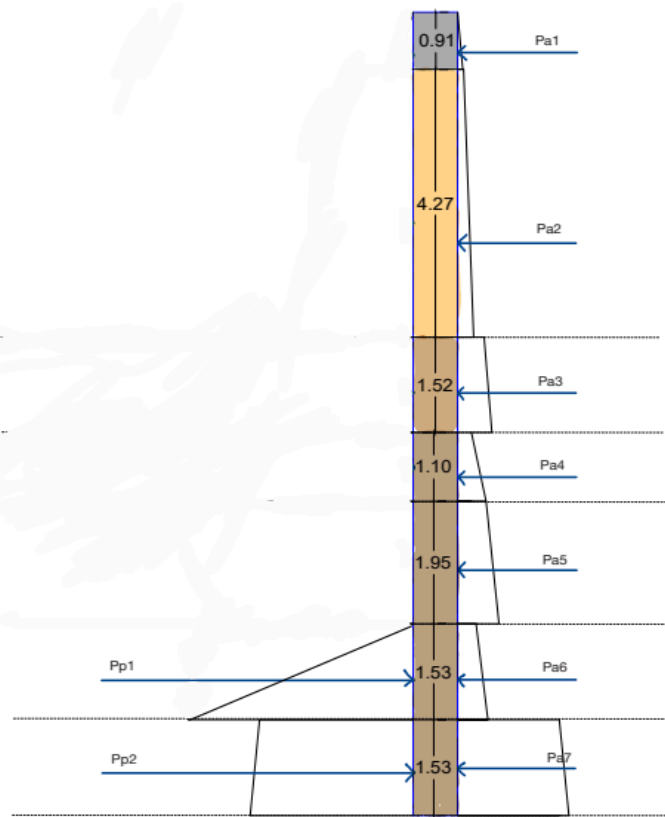


Figure 4.3.2.1.1 Lateral Pressure

Table below presents the calculated active and passive earth pressures for each soil layer considered in the design.

Table 4.3.2.1.1 Lateral soil pressure

Layer	Depth	σ'_0 (kN/m ²)	K_a	σ'_a (kN/m ²)	P_a (kN/m)	K_p	σ'_p (kN/m ²)	P_p (kN/m)
1	0.91+	17.15	1.0	7.55	3.43	-	-	-
	0.91-		1.0	9.15		-	-	
2	5.18+	100.29	1.0	23.69	70.22	-	-	-
	5.18-		0.39	39.16		-	-	
3	6.7+	130.61	0.39	50.99	84.71	-	-	-
	6.7-		0.39	20.61		-	-	
4	7.8+	152.20	1.0	42.20	77.75	-	-	-
	7.8-		1.0	42.20		-	-	
5	9.75+	171.36	1.0	61.35	100.96	-	-	-

	9.75–		1.0	27.35		-	-	
6	11.28+	189.03	1.0	45.03	187.15	1.0	333.03	770.86
	11.28–		1.0	150.62		1.0	227.43	
7	12.8+	202.76	1.0	164.36	290.47	1.0	241.17	920.14
	12.8–		1.0	106.86		1.0	298.85	
8	16.75+	242.18	1.0	146.18	715.46	1.0	338.18	1740.21
	Total	-	-	-	1530.15	-	-	3431.21

Hand calculation for the sheet pile consists of two parts. The first part is the calculation of the total lateral force above the dredge line and its centroid. For this, Rankine's active earth pressure formulas are used, as given in Equation 1 for cohesionless soil and Equation 2 for soils with cohesion. The general equation for active earth pressure is shown in Equation 3.

$$K_{A1} = \frac{1 - \sin\phi}{1 + \sin\phi} \quad (4.3.2.1.1)$$

$$P_v = \gamma H_1 + \gamma' H_2 + q \quad (4.3.2.1.2)$$

$$P_A = K_{A1} P_v \quad \# \quad P_A = K_{A2} P_v - 2c\sqrt{K_{A2}} + \gamma_w z \quad (4.3.2.1.3)$$

$$\gamma' = \gamma_{sat} - \gamma_w \approx 9.81 \text{ kN/m}^3 \quad (4.3.2.1.4)$$

The excavation depth is 9.75 m, and the water table is located at 7.8 m below ground level. The calculations were carried out for each soil layer and are presented in the table below.

Table: Active lateral pressure calculations

As can be seen, some of the calculated active earth pressures are negative. All negative active earth pressure values were taken as zero. Based on the results, the only layer with a non-negative active earth pressure is the compacted silt layer. From this, the total lateral force and its centroid were calculated. The total force is equal to $R_0 = 68.44 \text{ kN/m}$, vertical force on dredge line $q_0 = 171.36 \text{ kN/m}^2$ and the centroid is located $y_0 = 3.77 \text{ m}$ above the dredge line.

For the analysis below the dredge line, the method proposed by DeNatale and Ibarra-Encinas (1992) was used. This source presents a total-stress analysis for cantilever sheet piling in layered clay and is suitable for cases where the soil below dredge line is predominantly cohesive. Since the soil profile in this case is approximately clay, this method was considered appropriate for the calculation.

Using the equation $\sigma_i = 4c_i - q_0$, the values for the layers were calculated. As shown in the calculation below, the second layer gives a negative value, $\sigma_2 = -17.76 \text{ kPa}$. This indicates that the thin soft layer is too weak relative to the vertical stress at dredge line and cannot be relied on to provide sufficient resistance. Because of this, it was assumed that the effective resistance starts in Layer 3. Therefore, the approximate embedment depth was taken as $D = h_1 + h_2 + D_3$, where D_3 is the additional embedment required inside the deeper clay layer with cohesion $c = 96 \text{ kPa}$. To calculate D_3 , the homogeneous clay equations from the same source were used, shown below.

$$0 = AD^2 + BD + C$$

$$A = \frac{\sigma(\sigma-6c)}{12c} = \frac{212.64(212.64-6 \cdot 96)}{12 \cdot 96} = -67.07$$

$$B = \frac{R_0(6c-\sigma)}{6c} = \frac{68.44(6 \cdot 96-212.64)}{6 \cdot 96} = 43.17$$

$$C = \frac{R_0(12cy_0+R_0)}{12c} = \frac{68.44(12 \cdot 96 \cdot 3.777+68.44)}{12 \cdot 96} = 262.56$$

So the quadratic becomes:

$$0 = -67.07D_3^2 + 43.17D_3 + 262.56$$

From the equation, $D_3 = 2.33 \text{ m}$. Hence, the approximate embedment depth below the dredge line is $3.05 + 2.18 = 5.38 \text{ m}$. In practice, the depth should be increased by 30% for safety.

$$D_{actual} = 1.3D_{theory} = 6.99 \text{ m}$$

This means that the total sheet pile length from the ground surface is $9.75 + 6.99 = 16.74 \text{ m}$

With the known maximum bending moment and shear force, a steel sheet pile section can be selected. One of the most common steel grades in the United States is (50 ksi / 345 MPa). For the section type, a hot-rolled Z-type section will be used for the permanent retaining work, particularly where lower wall permeability and better stiffness are important. The code also notes that hot-rolled Z-sections have higher interlock efficiency and are well suited to cases such as this, where reduced permeability is required. The following figure shows the most used sections for sheet piles and their dimensions.

Section	Width+	Height+	Web Thick-ness+	Flange Thick-ness+	Weight		Moment of Inertia		Section Modulus		Nominal Coating Area*
	in.	in.	in.	in.	lb / ft	lb / ft ²	in ⁴	in ⁴ / wft	in ³	in ³ / wft	ft ² / ft
PZC 13	27.88	12.56	0.375	0.375	50.4	21.7	353.0	152.0	56.2	24.2	5.60
	708	319	9.5	9.5	75.1	106.0	14,690	20,760	920	1,300	1.71
PZC 14	27.88	12.60	0.420	0.420	55.0	23.7	381.6	164.3	60.5	26.0	5.60
	708	320	10.7	10.7	81.8	115.5	15,890	22,440	990	1,400	1.71
PZC 18	25.00	15.25	0.375	0.375	50.4	24.2	532.2	255.5	69.8	33.5	5.60
	635	387	9.5	9.5	75.1	118.2	22,150	34,890	1,145	1,800	1.71
PZC 19	25.00	15.30	0.420	0.420	55.0	26.4	576.3	276.6	75.3	36.1	5.60
	635	388	10.7	10.7	81.8	128.8	23,990	37,780	1,235	1,945	1.71
PZC 25	27.88	17.66	0.485	0.560	69.4	29.9	938.7	404.1	106.3	45.7	6.15
	708	449	12.3	14.2	103.3	145.9	39,070	55,190	1,740	2,455	1.87
PZC 26	27.88	17.70	0.525	0.600	73.9	31.8	994.3	428.1	112.4	48.4	6.15
	708	450	13.3	15.2	110.0	155.4	41,390	58,460	1,840	2,600	1.87
PZC 28	27.88	17.75	0.570	0.645	79.0	34.0	1,057	455.1	119.1	51.3	6.15
	708	451	14.5	16.4	117.6	166.1	44,000	62,150	1,950	2,755	1.87
Sections in development — inquire for availability.											
PZC 37	22.50	21.02	0.488	0.563	69.6	37.1	1,349	719.6	128.4	68.5	6.15
	572	534	12.4	14.3	103.6	181.2	56,160	98,270	2,100	3,680	1.87
PZC 39	22.50	21.05	0.525	0.600	74.0	39.5	1,429	762.1	135.6	72.3	6.15
	572	535	13.3	15.2	110.2	192.8	59,480	104,100	2,220	3,890	1.87
PZC 41	22.50	21.09	0.561	0.636	78.4	41.8	1,507	803.6	142.7	76.1	6.15
	572	536	14.2	16.2	116.6	204.1	62,720	109,700	2,340	4,090	1.87

Available Grades: ASTM A572 Gr. 50 and 60, A588 and A690
+Values stated are nominal
*Both sides of sheet; excludes socket interior and ball interlock

PZC™ is a trademark of Gerdau

Figure 4.3.2.1.2 Hot rolled Z-section sheet piles.

For a steel sheet pile section, USACE uses:

$$S_{min} = \frac{M_{max}}{f_b} \quad (4.3.2.1.4)$$

$$A_{v,min} = \frac{V_{max}}{f_v} \quad (4.3.2.1.5)$$

Where:

$$f_b = 0.5f_y = 172.5 \text{ MPa}$$

$$f_v = 0.33f_y = 113.85 \text{ MPa}$$

Thus:

$$S = 12.764 * 10^6 / 172.5 = 7.40 \times 10^4 \text{ mm}^3/m$$

$$A_{v,min} = \frac{V_{max}}{f_v} = \frac{68.44 \times 10^3}{113.85} = 601 \text{ mm}^2$$

$$A_v = \frac{t_w h}{b} = \frac{9.5(319 - 2 \times 9.5)}{708} = 4025 \text{ mm}^2$$

From the table, the PZ-37 passes both moment and shear requirements.

Table 4.3.2.1.2 PZ-37 section dimensions

PZ-37				
b	h	tw	tf	S
572 mm	534 mm	12.4 mm	14.3 mm	$3.68 \times 10^6 \text{ mm}^3/m$

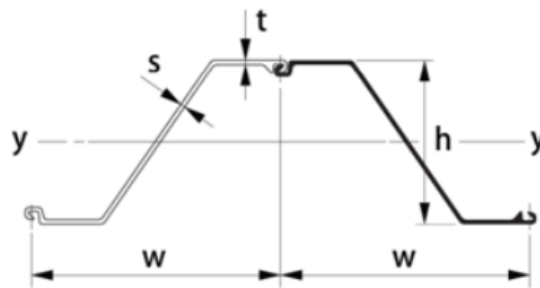


Figure 4.3.2.1.3 Hot rolled Z section

From the table above, safety checks could be done. The table below in contrast, shows the factor of safety against sliding (FoS=2.24), which is the dividend of passive forces to active forces and safety against overturning (FoS=3.14), dividend of moments. Moment arm was found geometrically.

Table 4.3.2.2.1 Safety checks against sliding and overturning

	Values	Moment Arm	Moment (kNm)
Passive Forces (kN)	3431.21	3.72	12764.111
Active Forces (kN)	1530.15	2.65	4054.89
FoS	2.24	-	3.14

4.3.2.2 Software analysis

To verify the safety, sheet pile was analysed using Geo 5 software. Following figures are screenshots taken from the software analysis, where safety passes all checks.

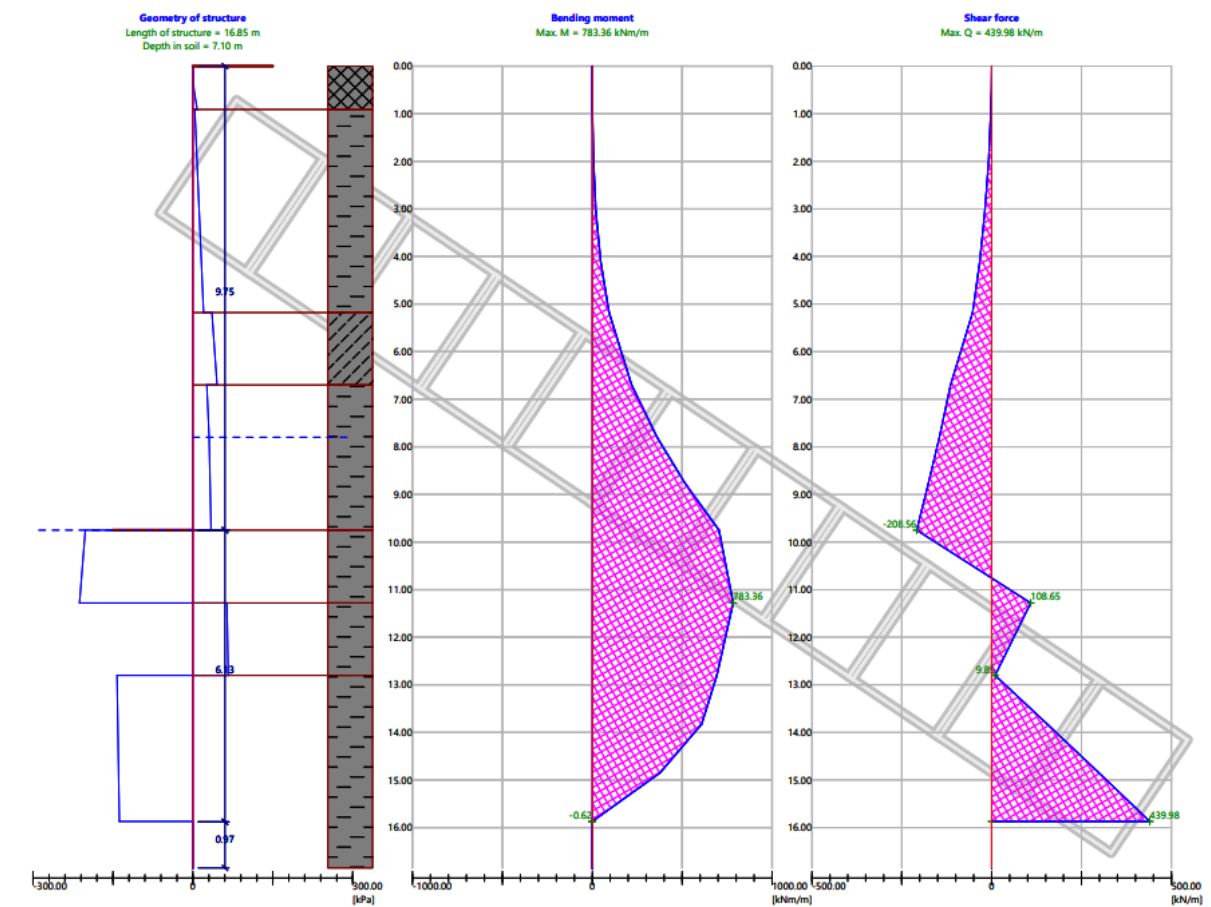


Figure 4.3.2.2.1 Structure geometry, bending, and shear force diagram

Design of non-anchored wall

Coeff. of reduction of passive pressure = 1.00

Max. value of shear force	= 439.98 kN/m
Max. value of moment	= 783.36 kNm/m
Required depth of structure in soil	= 7.10 m
Overall length of structure	= 16.85 m

Figure 4.3.2.2.2 Structure design

Verification of steel section according to safety factors

All construction stages are taken into the analysis.
Reduct. coefficient of bearing capacity = 1.00

Internal forces per 1 m of wall

$$\begin{aligned} M_{\max} &= 783.36 \text{ kNm/m}; & Q &= 108.65 \text{ kN/m} \\ Q_{\max} &= 439.98 \text{ kN/m}; & M &= 0.62 \text{ kNm/m} \end{aligned}$$

Verification of max. moment $M_{\max} + Q$:

Verification of bending:

Normal stress at the edge of cross-section $\sigma = 211.83 \text{ MPa}$
 $f_y/\sigma = 1.68 \geq 1.50$ **Is satisfactory**

Verification of shear:

Shear stress $\tau = 10.30 \text{ MPa}$
 $f_y/\sqrt{3}/\tau = 19.90 \geq 1.50$ **Is satisfactory**

Verification of plane state of stress:

Normal stress $\sigma_x = 200.48 \text{ MPa}$
Shear stress $\tau = 6.59 \text{ MPa}$

Verification: $f_y/\sqrt{(\sigma_x^2 + 3\tau^2)} = 1.77 \geq 1.50$ **Is satisfactory**

Verification of max. shear force $Q_{\max} + M$:

Verification of bending:

Normal stress at the edge of cross-section $\sigma = 0.17 \text{ MPa}$
 $f_y/\sigma = 2122.79 \geq 1.50$ **Is satisfactory**

Verification of shear:

Shear stress $\tau = 41.71 \text{ MPa}$
 $f_y/\sqrt{3}/\tau = 4.91 \geq 1.50$ **Is satisfactory**

Verification of plane state of stress:

Normal stress $\sigma_x = 0.16 \text{ MPa}$
Shear stress $\tau = 26.68 \text{ MPa}$

Verification: $f_y/\sqrt{(\sigma_x^2 + 3\tau^2)} = 7.68 \geq 1.50$ **Is satisfactory**

Cross section is SATISFACTORY

Figure 4.3.2.2.3 Shear and bending analysis check

Slope stability verification (Bishop)

Sum of active forces : $F_a = 2395.62 \text{ kN/m}$

Sum of passive forces : $F_p = 4796.59 \text{ kN/m}$

Sliding moment : $M_a = 70407.13 \text{ kNm/m}$

Resisting moment : $M_p = 140971.87 \text{ kNm/m}$

Factor of safety = $2.00 > 1.50$

Slope stability ACCEPTABLE

Figure 4.3.2.2.3 Structure stability check

Sheet pile design is safe and will be used in our construction site.

4.4 Whole Foundation Analysis

With all the safety checks made, whole foundation was analysed in Plaxis 3D. All of the group piles were inserted according to the allocations with the loads applied from the columns. As construction process suggests, sheet pile was installed beforehand and 0.6 meters of spacing between edge pile caps and sheet pile was there to ensure enough space for work.

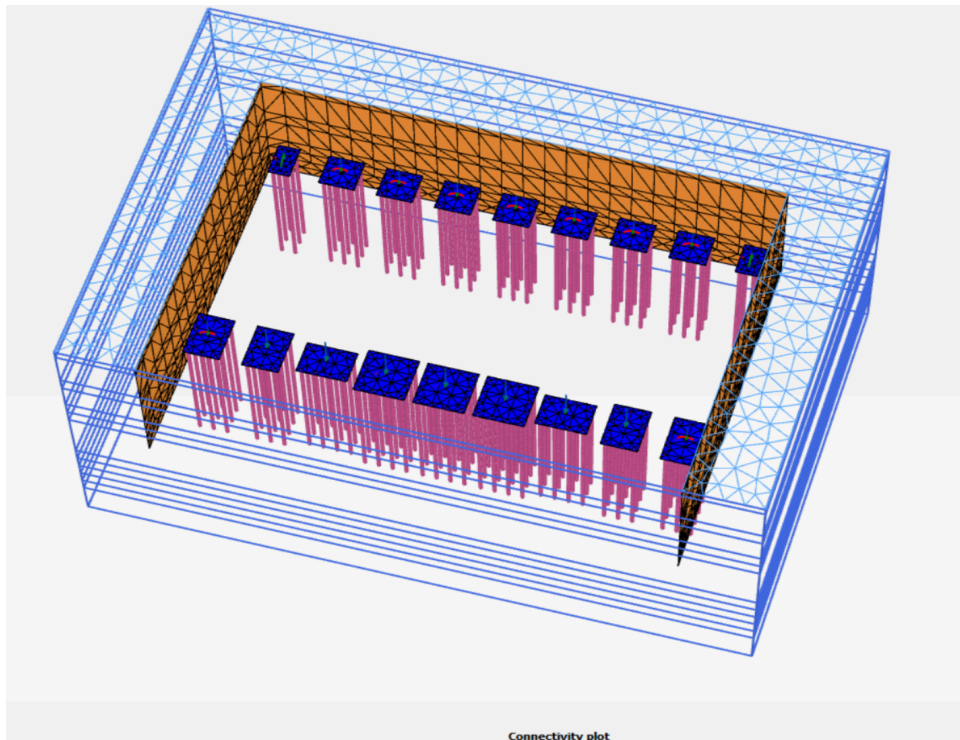


Figure 4.4.1 Connectivity plot of group piles and sheet pile.

Phases in Plaxis 3D were successfully completed and the calculations were made. Phases included, Initial phase (inserting soil properties), Sheet Pile(installing sheet pile), Excavation (removing 9.75 meters of soil as an excavation), Surcharge (applying 10 kPa of surcharge load on the soil nearby), Stability(checking the stability of the soil), Deep Foundation(Installing groups piles and pile caps) and Loads(applying design loads).

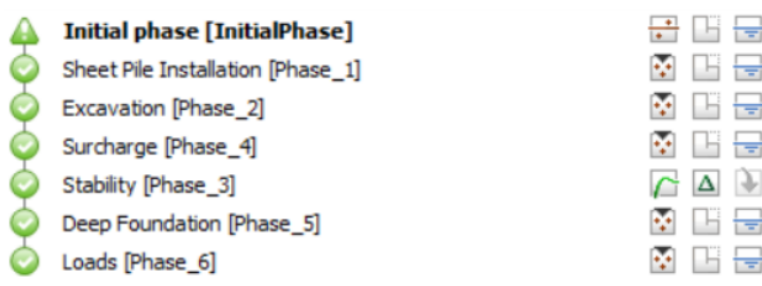


Figure 4.4.2. Plaxis 3D phases

In the stability phase, reached safety factor was 2.931 meaning the behaviour of the soil is safe. Furthermore, total deflection of the entire foundation was 15.46 mm.

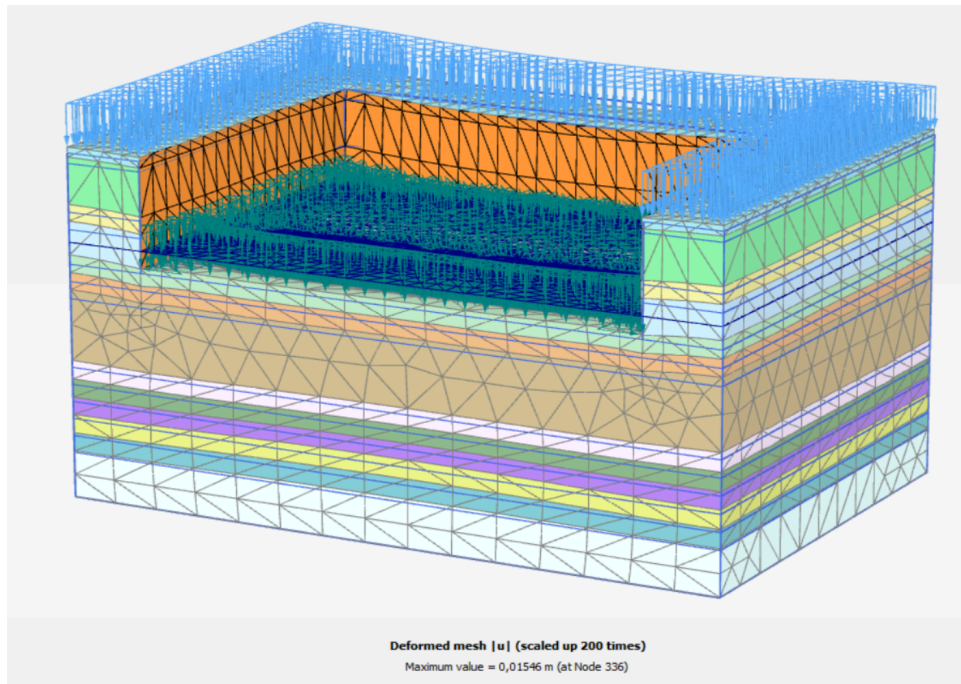


Figure 4.4.3. Plaxis 3D deflection and cross section of soil/foundation.

4.5 Construction Procedure

In the case of the Altyn Fiesta multi-story hotel development in San Antonio, the groundwater and the presence of soft clay strata are the factors, which led to the decision to use large-diameter bored cast-in-place piles as the main system of deep foundations. Installing sequence is closely monitored to get stability of the shafts and structural integrity as well as extended performance.

The stages in which the foundation works incorporate the following:

- **Site Preparation and Pile Layout** - involve the preparation of the site and piles. The first step is to have a stable working platform on which heavy rotary drilling equipments are supported. Total station surveying is used to transfer the pile grid of structural drawings. The location of each pile is checked to ensure that there would be no misalignment of a pile group system that may cause eccentric bearing.
- **Borehole Stabilization Strategy** - it is a strategy employed to manage boreholes that have been compromised because of insufficient engineering or technical expertise in managing the bores or irregular fractures in the formation of the underground rock. Considering the presence of the cohesive layers of clay and the impact of the groundwater, the drilling supported by slurry is chosen. Bentonite slurry is added during the drilling to give the boreholes hydrostatic pressure against the walls of the

boreholes. To prevent collapse of the walls and intrusion of the soil the slurry level is kept above the level of the groundwater. The properties of slurry like its density and viscosity are checked periodically to check its stability with regard to excavation.

- Borehole Cleaning and drilling - rotary drilling then opts the borehole to the preset embedment depth. Verticality is kept track of during the activity of drilling. When the final depth is reached, the bottom is washed off to get rid of the sediment that is deposited there, to provide the bottom with end-bearing resistance, and to prevent a decrease in the load carrying capacity.
- Cage Placement - this involves the use of a cage to reinforce the dig to stay in a particular spot. Reinforcement cages are ready-made according to the structural detailing specifications. Centralizers are put to control homogenous concrete cover. The cage is gently lowered into the borehole that has been stabilized and then attached to the position before concreting.
- Placement of concrete by pouring through tremie - before discharge, concrete is put in place by pouring through a tremie pipe which is inserted at the bottom of the shaft. Drilling slurry is used to ensure continuous pouring so as to avoid segregation and contamination. The tremie outlet is a hole left in fresh concrete throughout the procedure. To permit the removal of any upper concrete that may be contaminated by curing, concrete is poured over the level of the design cut-off to enable it to be removed.
- Casing Extraction and Quality Control - in temporary casing, a temporary casing is provided around the top which is slowly removed but with adequate concrete head to avoid necking or decreasing the diameter of the piles. Checks on quality are done in the depth of piles, slurry properties, checking on reinforcements, and checking on the concrete strength.

Excavation Part

The excavation is significant especially because there are three basement levels. The process is carried out in phases to maintain the stability of the soil as well as to regulate the lateral earth pressure. The excavation activity includes the following activities that are controlled:

- Site Clearing - the first issue is site clearing and boundary definition. Surface materials and inappropriate soils are eroded. Limits at excavations are drawn as per structural drawings and equipment access routes are created.

- Staged Excavation - the excavation is executed in lifts instead of being removed completely. This staged methodology ensures that stress is redistributed less in the clayey soil mass and that there is less danger of the massive movement at lateral stages
- Temporal Measures of Stability - this one should be introduced as measures to mitigate the impact of increasing volatility during the coming year. Temporary retaining systems are adopted, or controlled side slopes, depending on the excavation depth and the behavior of the soil. Constant monitoring of sidewall stability is carried out in order to avoid local failures.
- Groundwater Management - in the cases where seepage is experienced, localized dewatering facilities are implemented to ensure that there is dry working environment and exposure to softening of the layers of exposed clay is avoided.
- Subgrade Inspection and Preparation - the geotechnical engineer checks the exposed formation when it has reached the design elevation. Any loose areas are eliminated and filled up with compacted granular material where required. The bottom surface is cut off and made ready to commence construction of a foundation.

5. Environmental Engineering

5.1. Waste management options

The first step in developing a successful Integrated Solid Waste Management (ISWM) strategy for Altyn Fiesta is to accurately quantify waste streams produced from the various functional areas of the building, such as guest rooms, restaurant kitchen and office space. The overarching goal of this project is waste avoidance and reduction, as outlined in the internationally adopted Waste Management Hierarchy, contributing to the building's goal of LEED Platinum certification. This approach includes the adoption of strict policies to eliminate single-use plastics and prolong the life of durable products used by staff and guests. But due to the nature of a mixed-use building, it is impossible to eliminate waste. Thus, an established hierarchy of mitigation strategies should be used to handle the residual waste while reducing environmental impacts and increasing resource reuse.

In this approach, organic waste - which represents a substantial fraction of the daily mass - is treated with an industrial composting system to avoid production of methane, which results from anaerobic degradation in landfill. For inorganic waste streams (paper, cardboard, metals and plastics), recycling is preferred. This pathway is made more efficient with the use of an on-site industrial baler to compact and densify the recyclables for reuse as secondary materials. For the remaining streams that cannot be recovered or recycled, a company specialized in residual waste will deal with it. The combination of baling technology with a multiple contractor approach that includes partners such as WM and Republic Services is expected to achieve over 65% diversion, comply with Austin's Universal Recycling Ordinance (URO) and ensure that the plan remains financially sustainable and viable in the long run.

5.1.1. Recycling potential

The recycling potential of the Altyn Fiesta hotel is determined by the very high-precision waste diversion program that meets the high environmental standards and the utmost possible recovery of resources. The analysis of the waste profile shows that the total quantity of solid waste produced on the property is 775.0kg per day, equating to 283 metric tons per year.

To come up with an achievable recovery profile, the daily 775 kg waste stream stream was separated into definite, high-value fractions based on the location of their origin (restaurant, office, and hotel areas). The data provided below will show the daily production of all major categories of recyclables:

- Paper: 201.5 kg/day (74 tons/year) equal to the highest recyclable percentage of wastes at 26.0%.
- Cardboard: 72.6kg/day (26tons/year) 9.4% of the total daily output.
- Plastics: 119.8 kg/day (44 tons/year) 15.5% of the stream.
- Glass: 75.5 kg/day (28 tons/year) 9.7% of the waste.
- Metals: 44.4kg/day (16tons/year) 5.7% of its daily production.

The waste management system does not expect 100% recovery according to industry standards and the particular metrics as presented in the Recyclable Materials Market Value analysis. Rather, it uses the fixed rates of recycling to determine the mass recycled annually:

Table 5.1.1.1. Annual recycling potential

Recyclable Stream	Generation (Tons/Year)	Applied Recycling Rate	Expected Recycled Mass (Tons/Year)
Paper	74.0	66%	48.84
Cardboard	26.0	73%	18.98
Plastics (PET/HDPE)	44.0	28.5%	12.54
Metals (Alum/Steel)	16.0	51.5%	8.24
Glass	28.0	25.0%	7.00

5.1.2. Composting potential

The organic waste management strategy of Altyn Fiesta is a critical part of its sustainability strategy. With the current waste profile, the property produces 170.6 kg of organic waste daily, equating to 62.3 metric tonnes of waste every year, which sums up to 22% of total waste annually.

To achieve a realistic evaluation, the total organic volume is subdivided into three separate sources of generation, and each of them has the definite capture rate depending on the extent of the environmental control and the possibility to be contaminated:

- Kitchen and Restaurant (110.7kg /day): This is the stream that can be recovered most. In this case the capture rate used is 75% since the back-of-house is very controlled. The employees can be taught to use special containers on vegetable trimmings, coffee grounds, and plate scrapings at the place of their origin.
- Hotel Guest-Related (38.9 kg/day): This stream is a much harder one to recover. Food scraps are often mixed up in the bin with toiletries and packages. Thus, a more cautious capture rate of 55% is taken with the specific efforts in mind, like labeled bins and the secondary sorting by housekeeping or waste managing staff.
- Office Organics (21.0kg/day): Like guest-related streams, office organics (principally food leftovers during staff breaks) are hard to comply with. These operational constraints are represented by a hotel's capture rate of 55%.

Table 5.1.2.1. Daily and Annual composting potential

Generation Source	Daily Generation (kg/day)	Applied Capture Rate (%)	Realistically Compostable Mass(kg/day)	Annual Composting (Tonnes/Year)
Restaurant / Kitchen	110.7	75%	83.0	30.3
Hotel Guest Rooms	38.9	55%	21.4	7.8
Office Areas	21.0	55%	11.6	4.2
TOTAL	170.6	68%	116.0	42.3

The resulting integrated capture rate in all streams is about 68%, which can be compared to leading programs in the hospitality industry that are aimed at the high education of the staff and proper signage. Altyn Fiesta will also use off-site composting services and exploit the existing facilities in the city to reduce the capital cost but still achieve full adherence to the

organic waste diversion regulations on the first day of operation. Using this method, 15% of the total 775kg of daily waste stream is dealt with composting only.

5.1.3. Incineration potential

Waste hierarchy lays more emphasis on the reduction, reuse, recycling and composting as the leading options and then on the recovery of energy by burning the remaining waste. For Altyn Fiesta, the burnable waste stream is measured under two different operating conditions, which are a residual stream after on-site sorting, and an alternative scheme of 100% of the waste into direct mass burn. Both options promote the idea of a Lowering the Landfill mass policy as they make sure that all the non-diverted waste could be used to generate useful energy. Energy content was calculated from Lower Heating Values (LHV) (Basu, 2010):

$$\text{Energy (MJ)} = \text{Waste Mass (kg)} \times \text{LHV (MJ/kg)}$$

Scenario A:

In the base plan, the stream of incineration will include the leftover material after the establishment of composting and recycling diversion measures.

Numerous methods and models exist to compute the total residual stream, especially when it is utilized in conjunction with other streams of the energy supply. There are many ways and models to calculate the total residual stream, particularly where it has been used alongside other streams of the energy supply.

Total Residual Stream Calculation:

- Total Waste Generated: 775.0 kg/day (282.9 tonnes/year).
- Less: Composting Diversion: -116.0 kg/day (at 68% capture).
- Less: Recycling Diversion: -163.0 kg/day (at 21% recovery).
- Total Residual Waste (to Incineration): 496.0 kg/day (181.0 tonnes/year).

Combustible Composition

Though all 496.0 kg/day is sent to the incineration plant, non-recycled glass and metals are inert and do not contribute to energy recovery. Thus, the mass of residual matter that is being burnt is about 441.2kg/day.

Table 5.1.3.1. Composition of combustible

Parameter	Value
Total Combustible Residual	441.2 kg/day
Average LHV	17.7 MJ/kg
Total Daily Thermal Potential	7,809.2 MJ/day
Electrical Conversion Efficiency	25%
Hourly Electrical Energy	22.60 kWh/hour
Daily Electrical Energy	542.3 kWh/day
Annual Electrical Energy	197,940 kWh/year
Operational Impact	Powers 18–25 guest rooms continuously

Scenario B:

Altyn Fiesta also explores a strategy which avoids on-site sorting to improve the simplicity of its operation. Although, due to a larger amount of organic (wet) waste, the total energy output is lower.

Table 5.1.3.2. Scenario B Composition of combustible

Operational Metric	Value
Total Daily Waste Generation	775.0 kg/day
Inert Materials (Glass & Metals)	119.9 kg/day
Net Combustible Mass	655.1 kg/day
Average LHV	10.0 MJ/kg
Total Daily Thermal Potential	6,551.0 MJ/day

Electrical Conversion Efficiency	25%
Hourly Electrical Energy	455.0 kWh/day
Daily Electrical Energy	18.96 kWh/hour
Annual Electrical Energy	166,075 kWh/year
Operational Impact	Powers 15–22 guest rooms

5.2. Waste management plan

Having a total daily waste production of 775 kg, the facility will require an expert solution to the sorting and logistics matters to achieve diversion out of landfills giving an opportunity to recover energy and money.

In order to cope with such high production rates, Altyn Fiesta uses a multi-stream system of collection. The streams are delivered by nominated hauling partners and their frequencies designed to ensure hygiene and minimise on-site storage volume:

Table 5.2.1. Waste management plan

Waste Stream	Daily Mass (kg)	Annual Mass (Tons)	Hauling Company	Pickup Frequency
Organics	170.6	62.3	Burrtec	Daily (7 days/week)
Recyclables	308.2	112.5	Waste Management (WM)	3 times per week
Residual (WtE)	296.2	108.1	Republic Services	3 times per week
TOTAL	775.0	282.9		

The WM Materials Recovery Facility (MRF) recycles recyclables, with a major composition of 274.1 kg/day being paper and cardboard.

All the waste that remains is sent to Waste-to-Energy (WtE) facilities run by Republic Services where the waste is turned into electricity.

A special team is hired to maintain the integrity of the waste streams and monitor recycling goals are achieved.

The Waste Management Team is comprised of:

1 Head Waste Steward (Supervisor):

- Coordinates with external haulers to ensure the timely pickups and registers the weights of waste on a daily basis.
- Makes quality-control checks to all streams to reduce contamination costs.
- Make certain that safety measures have been observed.

3 Waste Stewards (Technicians):

- Conduct secondary sorting on the waste of guest rooms to retrieve the high-value paper, plastic PET, and aluminum.
- Weekly use the industrial cardboard baler to minimize the amount of 72.6kg of cardboard.
- Keep the bins clean.

Daily Operational Schedule

The schedule below will make sure that the 775kg of daily waste is handled without losing the luxury guest experience in the hotel:

07:00 AM - 08:30 AM: First Collection and Docking.

08:30 AM - 11:30 AM: Peak Sorting Phase

11:30 AM - 01:00 PM: Organic Waste Transfer

01:00 PM - 02:30 PM: Compaction and Baling

02:30 PM - 03:30 PM: Final Quality Control and Sanitization.

5.3. Waste collection plan

The waste collection plan defines the quantity, type, model, size and location of all on-site containers and equipment; collection frequency and pull rate for each waste stream; and staffing needs for daily waste management. This plan is developed from weekly volume calculations and sized to the space and operational requirements of a full-service hotel.

Table 5.3.1. Weekly Waste Volume Computations

No	Waste Category	kg/day	kg/week	Bulk Density (kg/m ³)	Volume (m ³ /week)	Volume (cu.yd./week)	Source
1	Paper + Cardboard	274.1	1,919	85	22.6	29.5	Abeynayaka & Werellagama (2007)
2	Plastics (PET/HDPE)	119.8	839	85	9.9	12.9	Abeynayaka & Werellagama (2007)
3	Glass	75.5	529	320	1.6	2.1	Abeynayaka & Werellagama (2007)
4	Metals	44.4	311	490	0.6	0.8	Abeynayaka & Werellagama (2007)
5	Food (kitchen)	110.7	775	305	2.5	3.3	NZWC (2023)
6	Organics	59.9	419	270	1.6	2.1	Abeynayaka & Werellagama (2007)

7	Miscellaneous	46.8	328	270	1.2	1.6	Abeynayaka & Werellagama (2007)
8	Special or HHW	43.8	307	165	1.9	2.5	Sadeghi (2020)
	TOTAL	775.0	5,427		41.9	54.8	

The total uncompacted weekly waste volume is 54.8 cubic yards. Critical insights driving volume container choices and frequency are:

- Paper/Cardboard/Plastics/Glass/Metals: 45.3 cu.yd./week uncompacted. Baling with the Bramidan B4 reduces the effective volume - conservatively to 40-50% of the uncompacted volume (18-23 cu.yd./week). 1x4 cu.yd. bin weekly (1x4 cu.yd. throughput) suggests that more than 1 bin and/or a larger one may be required - but the poster confirms that a weekly 4-yd bin is rented at 52 pulls/year, so that baling provides the volume reduction required.
- Organic/food waste (kitchen): 3.3 cu.yd./week. One 4-yd bin collected daily (7×per week) yields 28 cu.yd. of throughput per week, far greater than generation, confirming that capacity for the daily pickups is driven by hygiene.
- Non-recyclables: 6.0 cu.yd./week (after diversion). Two 4-yd bins (8 cu.yd. total) collected 3×per week provide 24 cu.yd. of throughput - 4×312 pulls/year at \$47/pull.

All vessels and equipment are specified from products widely available as per the poster presentation. The Bramidan B4 Small Footprint Industrial Baler was chosen for its small size, suitability for the hotel back-of-house, and ability to handle the 72.6 kg/day of cardboard and plastic film. The high-pressure washer is necessary for hygiene given the food waste daily quantity and outdoor location of the bin. The complete container and equipment specification is listed in the table below.

Table 5.3.2. Container and Equipment Specifications

Container / Equipment	Target Waste Stream	Capacity/Specs	Qty
3-Stream Sorting Station	Recyclables, organics, trash	3×15 gal	12

(Public/Office areas)			
In-room Dual-Stream Bin	Recyclables and trash	32 gal	150
Sealed Food-Waste Bin (Kitchens)	Organic waste only	15–20 L	20
Bramidan B4 Small Footprint Industrial Baler	Cardboard and plastic compaction	Vertical press with 72 kg/cycle capacity	1
High-Pressure Washer	Hygiene	2,000 PSI	1
32-gal Carts Floor Level (Colour-coded lids)	Sorting of recyclables	32 gal	150
Sealed HHW Cart	Hazardous and special waste	32 gal	2
Total On-site Units			336

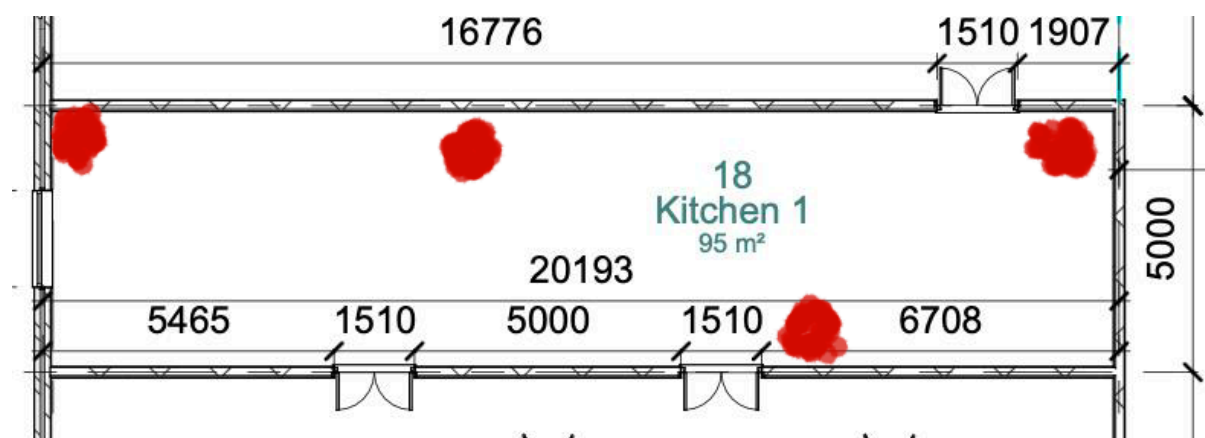


Figure 5.3.1. Kitchens' bin placement scheme

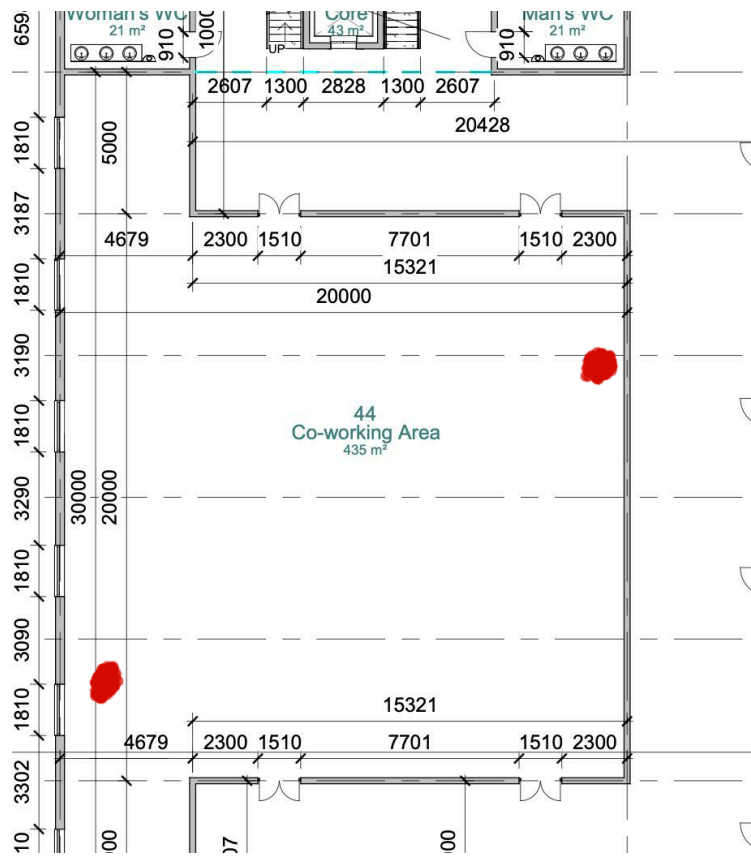


Figure 5.3.2. Offices' bin placement scheme

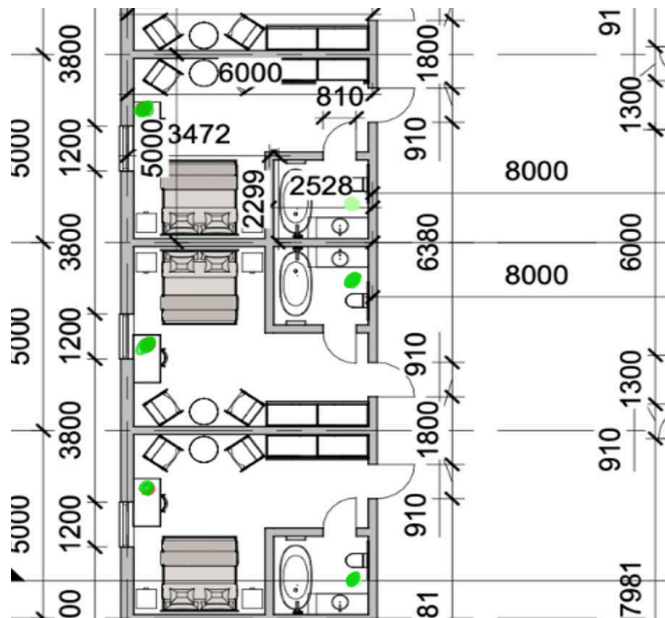


Figure 5.3.3. Hotel rooms' bin placement scheme

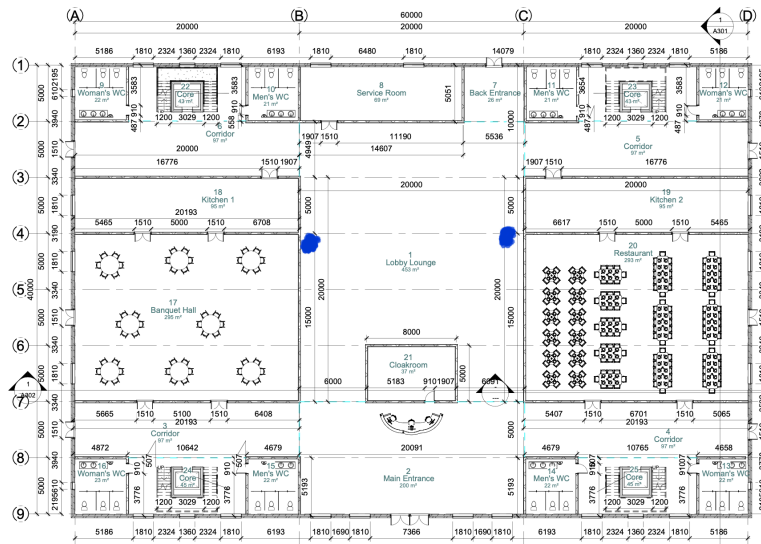


Figure 5.3.4. Lobby's three-stream sorting bin placement scheme

There are 150 dual-stream in-room bins, one per guest room (green dots); a high-end sorting unit for recyclables and trash at the source. The 12 three-stream public sorting bins (blue dots) are strategically located in high foot traffic areas (lobby, restaurant, conference room, and poolside) with recyclables, organics, and trash outlets in a single unit with colour-coded bins. And 150 additional 32-gallon rolling carts are placed on service and back-of-house corridors for daily pick-up by the Waste Stewards.

Receptacles are colour-coded according to City of Los Angeles and Austin URO waste sorting standards (LADPW, 2022 & City of Austin, 2012) as blue for recyclables, green for organics and grey for residual trash. This uniformity limits guest and employee waste sorting errors and contamination of the recyclables stream to maximise the programme's LEED diversion targets.

The success of waste separation infrastructure is dependent on occupant education and point-of-generation signage (Bernstad, 2014 & WRAP, 2013). Altyn Fiesta adopts a two-pronged training and signage strategy:

- **Staff Training Video System:** In the staff room, a commercial display with media player is installed to broadcast a looped 10-minute tutorial in English, Spanish, Filipino and Korean on: identifying streams, using bins, operating the baler, and documenting LEED points. All new employees watch the tutorial as part of their training; the video is shown every six months (Bernstad, 2014).
- **Multilingual Sorting Signage:** Thirty laminated A3 colour-coded sorting signs in four languages, with icon-based visual depiction of recyclable, organic and residual waste

streams are placed at all waste collection points on the property. Lamination with UV protection means a lifespan of at least five years. The icon-based approach is independent of literacy, as per best practice in the hospitality sector (WRAP, 2013).

This programme therefore overcomes the main operational factor affecting recyclables stream contamination in the hotel environment and meets LEED, which requires collection facilities for all waste streams to be labelled, accessible and communicated (USGBC, 2019).

The frequency schedule is based on the volume analysis, hygiene, and the pull rate pricing of the poster.

Table 5.3.3. Waste Collection Frequency

Waste Stream	Container	Frequency	Contractor	Annual Pickups
Food waste (kitchen)	Sealed HDPE bin > Organic 4-yd container	Daily (7×/week)	Burrtec/New Earth	365
Organic	1×4-yd bin	Daily (included above)	Burrtec/New Earth	364
Recyclables	1×4-yd bin (post-baling)	Weekly (1×/week)	Waste Management (WM)	52
Residual	2×4-yd bins	3× per week	Republic Services	312
HHW	Sealed 32-gal carts	Monthly drop-off	S.A.F.E. Centre	12
Total Annual Collections				1,105

Food waste is the only stream collected daily, due to the decay rate of organic waste from the kitchen and hygiene requirements for a hotel food service. Staff details are in the table below.

Table 5.3.4. On-site Waste Management Personnel

Role	FTE	Schedule / Rate
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Head Waste Steward	1	\$30/hr, full-time 40 hrs/wk×52 wks = \$62,400/yr
Waste Steward / Janitor (Sorting)	3	\$20/hr, rotating shifts 7 days/week = \$124,800/yr

5.4. Integrated solid waste management plan

The Integrated Solid Waste Management Plan (ISWMP) for Altyn Fiesta, brings all collection, segregation, transportation and disposal operations together to create an integrated operational system in line with the ISWM objective stated in the poster: to maximize landfill diversion with a high efficiency three-stream system, while meeting LEED Platinum and full URO compliance. The plan's main goals are to divert at least 65% of waste from the landfill in the first year and comply with URO and LEED document standards.

The material flow is structured in three streams as per the flowchart in the poster. Waste is collected from the hotel zones, sorted onsite at sorting stations and floor carts, and separated in the main collection zone on the ground floor by three 8-hour shift Waste Stewards. Each stream is placed in its container and picked up by the appropriate contractor from the collection zone:

- Recyclables (113 tons/year): Collected at source (room and 3-stream sorting stations). Cardboard and plastic film are baled daily using the Bramidan B4 baler to reduce the outgoing volume to a level that can be handled by weekly 4-yard bin collection. Collected weekly by Waste Management (WM) and taken to the WM Materials Recovery Facility (MRF) for processing and sale to commodity markets, generating \$6,200/year in revenue.

- Organics (62 tons/year): Food waste from the kitchen is collected in sealed HDPE containers at the preparation stations and transported daily to the 4-yard organic bin at the kitchen service entrance and collected daily by Burrtec / New Earth for off-site composting via WM's composting network. This represents 22% of waste by weight and is 100% diverted from landfill.

- Residual (108 tons/year): Residual trash, post-sorting, is placed into two 4-yard bins and collected 3× per week by Republic Services. If a licensed WtE facility is present in the service area (preferred), residual waste is thermally recovered according to Scenario A (542.3 kWh/day; 197,940 kWh/year). This, along with recycling and composting, results in a diversion rate of 87.5% (assuming WtE is considered diversion from landfill).

- HHW and Special and E-waste: Low-volume hazardous waste is captured in 32-gallon sealed carts at two locations in the maintenance area and hauled monthly to the nearest S.A.F.E. Centre (in compliance with RCRA standards).

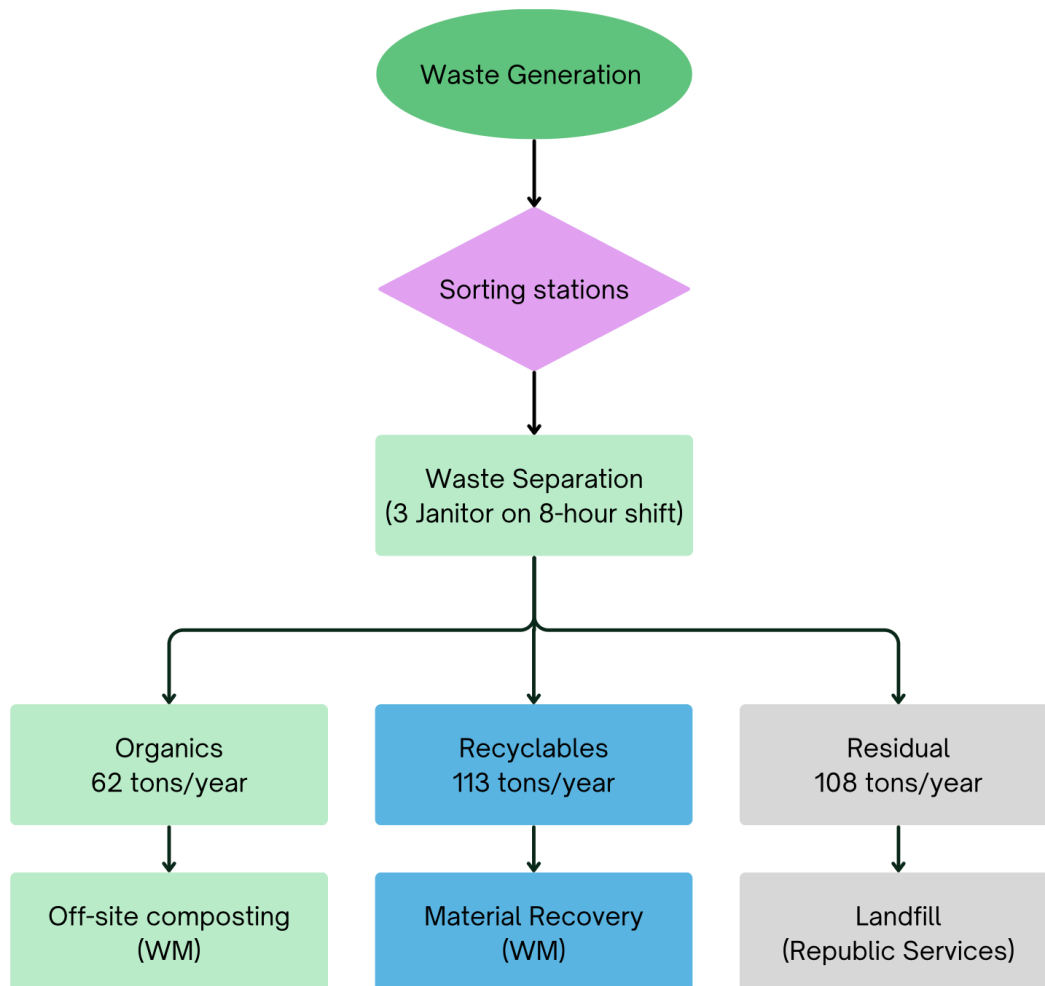


Figure 5.4.1. Waste Management Plan Flow Chart

5.5. Operational finances of waste management plan

The financial viability of the waste management strategy for Altyn Fiesta is assessed by balancing the initial infrastructure investment against recurring logistics costs and potential revenue streams from material recovery. The following analysis breaks down the financial components into Capital Expenditures (CapEx), Operational Expenditures (OpEx), and projected revenues.

5.5.1. Capital Expenditures (CapEx)

Capital investment is required to establish a robust source-separation infrastructure and on-site processing capabilities. These one-time costs ensure the hotel meets LEED Platinum requirements for waste diversion.

Table 5.5.1.1. CapEx Year 0

Item	Description	Unit Cost (USD)	Qty	Total (USD)
Waste Separation Infrastructure				
In-room Sorting Bins	Dual-stream units	\$25	150	\$3,750
Recycling Stations	3-stream stations for public/office areas	\$250	12	\$3,000
Bramidan B4 Industrial Baler	Vertical cardboard/plastic compactor	\$11,000	1	\$11,000
Cleaning Equipment	High-pressure washers	\$340	1	\$340
Subtotal				\$18,090
Staff Training Programme				
Staff Training Video System	TV display + media player for break-room onboarding loop	\$800	1	\$800
Multilingual Tutorial Production	10-min video (EN/ES/FIL/KOR)	\$1,200	1	\$1,200
Subtotal				\$2,000

Multilingual Sorting Signage				
Sorting Signage Package	Laminated A3 guides, multilingual (EN/ES/FIL/KO)	\$15	30	\$450
Signage Installation	Installation	\$200	1	\$200
Subtotal				\$650
Total CapEx				\$20,740

5.5.2. Operational Expenditures (OpEx)

Operational costs are primarily driven by hauling fees from external contractors (Waste Management, Republic Services, and etc). Costs are calculated based on the collection frequencies established in Section 3.2.

Table 5.5.2.1. Annual OpEx

Category	Item	Description / Calculation	Annual Cost (USD)
Waste Logistics & Container Rental			
Waste Logistics & Rental	Residual Waste	Rental of 2×4-yd bins+3×weekly pickup (\$100 rental/mo×12)+(312 pulls×\$47)	\$15,840
	Organic Waste	Rental of 1×4-yd bin+daily pickup (7×/week)(\$50 rental/mo×12)+(364 pulls×\$76)	\$28,200
	Recyclables	Rental of 1×4-yd bin+weekly pickup (1×/week)(\$50rental/mo×12)+(52 pulls×\$35)	\$2,400
Subtotal — Logistics & Rental			\$46,440
Labour Salaries			
Labor Salaries	Head Waste Steward (\$30/hr)	40 hrs/wk×52 wks×\$30/hr	\$62,400

	Waste Stewards (\$20/hr)	3×40 hrs/wk×52 wks×\$20/hr	\$124,800
Subtotal — Labour			\$187,200
Maintenance & Consumables			
Maintenance & Misc.	Amortisation	10-year depreciation on Bramidan B4 baler and sorting stations	\$1,800
	Consumables	Bags + 10% annual bin replacement fund	\$1,200
Subtotal — Maintenance			\$3,000
Gross Annual Operational Cost		Total annual operational cost	\$236,640
Recyclable Materials Revenue			
Revenue	Paper & Cardboard	100 metric tons/year×\$50/ton (ISTC, 2024)	\$5,000
	Plastics (PET) & Metals	Clean separated streams market rates (EPA, 2023)	\$1,200
	Glass	7 metric tons/year×\$57/ton (Glass Packaging Institute, 2023)	\$400
	Total Annual Revenue		(\$6,600)
Net Annual Waste Management Cost		\$236,640 - \$6,600	\$230,040

6. CONSTRUCTION MANAGEMENT

6.1 Project Charter

Project Name		Project Manager	Project Sponsor
8- story Hotel Development “Altyn Fiesta”		Bauyrzhan Tyleukhanov	Nazarbayev University
Email	Phone	Organizational Unit (s)	
bauyrzhan.tyleukhanov@altyn.fiesta.com	+7 777 667 67 67	Full Scope Construction Engineering	
Project	Altyn Fiesta project is aimed to build 8-story joint residential and commercial property to accommodate more people.		
Purpose of Project	The construction of an 8-story mixed-use building in San Antonio will address the city's acute shortage of multi-use developments for tourism and business while simultaneously facilitating economic development. It will provide 140 apartments and commercial space on the first three floors.		
Business Case	Total cost: \$138.82M Payback: 10 years Revenue: \$27.4M/year (average) EBITDA: ~\$8.3M/year (average) min. ADR: \$790/night (luxury suites \$1,580–1,975) 400+ jobs created		
Metrics	140 rooms 7,200 m² commercial space 68% occupancy LEED certified 30–35% energy savings with PCM		

	Budget control: $\pm 10\%$
Expected Deliverables	Completed 8-story hotel with 140 rooms, commercial areas, 7,200 m ² parking, restaurant, gym, and energy-efficient systems

Project Scope	• 8-story mixed-use hotel with 140 rooms and 3 floors of commercial space • 7,200 m² underground parking and recreational areas (restaurant, gym) • LEED certification and PCM-based energy efficiency system • Compliance with Texas codes, permits, and environmental standards • Full site management from start to finish • Hotel operation, management plan, and marketing strategy
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Key Dates	Start	Finish
Permits and Preconstructions	05/01/2026	29/06/2026
Excavation and Soil Stabilization	23/06/2026	18/12/2026
Foundation Works	07/12/2026	31/05/2027

Building Construction	04/05/2027	17/04/2028
MEP Rough-in	14/12/2027	07/12/2028
Facade Installation	02/11/2027	28/02/2028
Interior Fit-out	23/06/2028	03/12/2029
Commissioning and Handover	13/11/2029	25/02/2030

Project Team	Dariga Satybaldiyeva - Structural Engineer Aibibi Abuova - Structural Engineer Abralin Adilzhan - Environmental Engineer Bauyrzhan Tyleukhanov - Construction Management Kuzar Miras - Geotechnical Engineer Abay Auyelkhan - Geotechnical Engineer Dutbayev Nurtay - Geotechnical Engineer
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6.2 Feasibility study

A site-wide feasibility study occurred prior to the commencement of construction of this 8-storey residential building. This covered a review of the viability of the project based on numerous parameters, its compliance with the requirements of location, essence of sustainability and financial targets.

Site Analysis: The analysis site is downtown San Antonio central location, which is easily reachable and has modern facilities. The vicinity offered by its proximity to parks, transportation access, and other essential facilities renders this location especially friendly as a residential and commercial enterprise. The space used in constructing buildings, storing materials and creating parking zones, and recreational facilities will be large at the site. Moreover, operations in connexion with grading and foundation work are not hard since the terrain is flat and simplifies the construction.

Design and Development Plan: The design of the building follows San Antonio and Texas LEED building codes regarding power conservation and sustainability. Others such as ample natural daylight, extremely efficient insulation offer extended environmental and monetary advantages. The natural light exposure in the building and minimising of energy was done through the building design and use of sun protection. The state of accessibility and usability should equally be considered as part of the development plan depending on the scenario, availability of emergency exits, parking, and business spaces.

Environmental Impact Assessment: The Texas Environmental Quality Act provided that the environmental impacts of construction would be properly assessed. The design applied measures to prevent risks that are associated. Environmentally friendliness and non-toxicity of the materials used will minimise the environmental impact on the construction and the wastes incorporated that will be under waste management systems will offer sustainability on waste management throughout construction and lifecycle use of the construction.

Legal Analysis: Legally, the project meets all the licencing and building code requirements, which simplify the approval process. Environmental compliance, safety standards as well as zoning permits have been acquired. The construction team is also mindful regarding worker rights and the safety of its workforce, which complies with Texas labour legislations and safety measures to maintain safe working environment.

Economic Impact Analysis: It is projected that a great economic impact will be created by the use of construction workers, engineers, and technical staff in the project. The commercial facilities would support high-quality services to the 140-plus guests in the hotel whereas the commercial facility would generate extra revenue (estimated nightly rates are between 400 and 1000, based on the size of the unit but the proposed commercial lease costs are 780 per square foot (per year)).

The payback interval of the project would be in the initial decade of working.

Project Schedule and Timeline: To offset cost of time the detailed work plan was elaborated in terms of snow fall, holidays or any other possibility of delays. Starting from January of 2026, the project has a clear start and a series of milestones which will be made in major phases, starting with survey works, through to completion. The completion date is expected to be at the beginning of 2030.

In general, the feasibility study confirms an 8-storey commercial project in the Downtown area of San Antonio is possible. It has everything required to be successful because it is strategically positioning with the required environmental and regulatory standards to succeed in its distribution and provision of quality residential and related opportunities in matters of retail.

6.3 Cost/Benefit analysis

6.3.1 RSMeans calculation

One of the essential activities in project management has always been to estimate the cost of construction projects. In many cases, estimates are never accurate at the construction stage, which causes financial imbalances and wastage of resources in the project. To meet these needs, RSMeans (RSM) software has become a vital addition in construction projects, which offers professionals with an extensive cost account on different regions.

\$57,669,607.11 Building Cost	SAN ANTONIO, TX Location	8 Stories (Ea.)	Yes Basement
\$239.00 Cost per S.F.	241,300 Floor Area	10.00 Story Height	\$0.00 Additive Cost

Figure 6.1. RSMeans output of cost estimation

This paper estimated the construction cost of a 8-story hotel with a basement and RSMeans was used to estimate the projected construction cost, which was estimated at \$57, 669, 607 with a rate of \$239/sqft. Nevertheless, to determine this cost, RSMeans has considered only the floor area and the number of stories, yet other information like the availability of a three-story underground parking garage was considered by the program. Thus, the calculated price is not accurate and needs further manual estimations.

Table 6.1. Building Cost Estimate via RSMeans

Code	Description	Unit	Quantity	Material Cost (\$)	Labor Cost (\$)	Total Cost (\$)
A	SUBSTRUCTURE					\$3,446,554
A1010	Standard Foundations (Strip & Column Footings)	L.F.	1,150	\$155,000	\$113,719	\$268,719
A1020	Special Foundations (Driven Steel H-Piles)	Ea.	145	\$1,620,000	\$926,050	\$2,546,050
A2010	Basement Excavation	C.Y.	18,500	\$35,000	\$156,322	\$191,322

	ion (3 Levels)					
A2020	Basement Walls (Reinforced Concrete 14")	S.F.	22,000	\$245,000	\$195,463	\$440,463
B	SHELL					\$13,561,008
B10	Superstructure (Frame & Floors)			\$3,706,760	\$3,018,440	\$6,725,200
B1010	Ready-Mix Concrete (4000 psi)	m³	4,822	\$1,109,060	\$530,420	\$1,639,480
B1010	Reinforcing Steel	Ton	650	\$1,300,000	\$780,000	\$2,080,000

	(Rebar Grade 60)					
B1010	PT Strands (7-wire ASTM A416, Gr 270)	kg	39,560	\$205,70 0	\$138,02 0	\$343,72 0
B1010	Formw ork, Shoring , and PT Tension ing	S.F.	185,000	\$1,092,0 00	\$1,570,0 00	\$2,662,0 00
B20	Exterio r Enclosu re					\$6,540,4 01
B2010	Exterio r Walls (Mason ry & Type IB	S.F.	55,000	\$2,426,4 50	\$1,805,0 00	\$4,231,4 50

	Insul.)					
B2020	Exterior Windows (Aluminum Frame/ Low-E)	S.F.	14,500	\$1,610,020	\$698,931	\$2,308,951
B3010	Roof Coverings (EPDM Membrane & Insulation)	S.F.	12,000	\$155,407	\$140,000	\$295,407
C	INTERIORS					\$6,812,431
C1010	Interior Partitions (Sound-rated	S.F.	95,000	\$945,610	\$900,000	\$1,845,610

	GWB)					
C1020	Interior Doors (Hotel Suite & Common)	Ea.	320	\$1,020,321	\$500,000	\$1,520,321
C3010	Wall Finishes (Paint, Vinyl, Tile)	S.F.	145,000	\$880,450	\$800,000	\$1,680,450
C3020	Floor Finishes (Carpet, LVT, Porcelain)	S.F.	115,000	\$953,810	\$812,240	\$1,766,050
D	SERVICES					\$16,395,571
D1010	Elevators (4 Units: 3	Ea.	4	\$950,000	\$500,000	\$1,450,000

	Pass. + 1 Service)					
D2010	Plumbi ng Fixture s (140 Rooms & Kitchen)	Ea.	140	\$2,420,0 00	\$1,700,0 00	\$4,120,0 00
D3010	HVAC (VRF System & Central Ventilat ion)	S.F.	115,000	\$2,500,0 00	\$1,350,0 00	\$3,850,0 00
D4010	Fire Protecti on (Wet Pipe Sprinkl ers)	S.F.	115,000	\$680,00 0	\$400,00 0	\$1,080,0 00

D5010	Electric al (Service , Lightin g, Branch)	S.F.	115,000	\$2,600,0 00	\$1,350,0 00	\$3,950,0 00
D5090	Securit y, IT, & Commu nication Systems	S.F.	115,000	\$750,00 0	\$370,57 1	\$1,120,5 71
E/F	EQUIP MENT & SPECI AL					\$114,20 0
E/F	Special Constr uction (PCM Integrat ion)	Sum	1	\$110,00 0	\$4,200	\$114,20 0
G	TOTAL DIREC					\$40,329,

	T COSTS					764
H	OVER HEAD & FEES					\$11,292, 334
H10	General Condi tions & Site Manage ment	5%				\$2,016,4 88
H20	Contrac tor Fee (Profit & Overhe ad)	15%				\$6,049,4 65
H30	Archite ctural & Enginee ring Fees	8%				\$3,226,3 81

I	TOTAL BUILD ING COST					\$51,622, 098
J	Land Acquisi tion Cost					\$7,500,0 00
K	TOTAL PROJE CT BUDG ET					\$59,122, 098

Table 6.2. Cost Estimation

Cost Estimation	Value (\$)
Land Cost	\$ 7,500,000.00
Materials	\$ 59,122,098.00
Labour	\$ 12,826,099.93

Services	\$ 1,728,799.41
Taxes	\$ 8,242,217.41
Total Cost	\$ 89,419,214.75

Land and Labor Costs

The cost of the land was calculated by putting the size of the site area at 5, 000 square meters and the estimated market value per square meter at \$1, 500,000, which gave a total of a land acquisition cost of 7, 500,000. The valuation indicates the high commercial real estate rates in San Antonio, TX, as at the year 2025.

The regional labor cost range was taken as the upper limit in the case of the labor expenditure, which is 550 per sq. ft. Such premium cost is obligatory to take into consideration the professional expertise needed to carry out the post-tensioned concrete work which is specialized and installation of PCM (Phase Change Materials) to achieve the sustainability requirements.

Adoptions of taxation and inflation (Baseline 2018 to 2025)

The combined sales tax rate in San Antonio, TX is charged at 8.25 percent in 2025 fiscal year.

All material costs initially were taken out of RSMeans database (baseline 2018). A cumulative inflation rate of 35 per cent was used to bring these figures to the economic reality by the year 2025. Moreover, there was the inclusion of a certain 15% reserve on the total project budget, to allow the countering of risks caused by unexpected technical or market factors at construction time.

Table 6.3. Finalisation of Cost

Total	
Cost + Inflation Rate	120,715,939.91

Final Cost	138,823,330.9
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The cost of the project, as per the estimation, is 138.82 million. The hotel price per night can be calculated based on a 10-year payback period. A 140-room hotel would be costed at:

$$\text{Target EBITDA/Year} = \text{Total Investment/Payback Period.}$$

$$\text{EBITDA} = 138.82\text{million} / 10 = 13.88 \text{ million USD/year}$$

$$\text{Total Revenue (average operation expenses)}$$

$$\text{EBITDA} = (1-0.65) \times \text{Total Revenue} = 0.35 \times \text{Total Revenue.}$$

$$\text{Minimum Revenue} = 13.88/0.35 = 39.66 \text{ million USD/year.}$$

$$\text{Rooms Revenue (assuming F\&B and other} = 45\% \text{ of Rooms Revenue)}$$

$$\text{Total Revenue} = 1.45 \times \text{Rooms Revenue.}$$

$$\text{Rooms Revenue} = 39.66/1.45 = 27.35 \text{ million USD/year}$$

$$\text{Average Daily Rate (ADR)}$$

$$\text{Rooms Revenue} = \text{ADR} \times \text{No. of Rooms} \times 365 \times \text{Occupancy Rate.}$$

$$\text{Available room nights} = 140 \times 365 \times 0.68 = 34,750$$

$$\text{ADR} = 27,350,000 / 34,750 = 790 \text{ USD per night}$$

$$\text{Standard room ADR (required)} = 790 \text{ USD per night.}$$

$$\text{Luxury rate (2-2.5} \times \text{ADR)} = 1580\text{-}1975 \text{ USD per night.}$$

Table 6.4. Payback period estimation

Year	Rooms Revenue (USD million)	F&B & Other (USD million)	Total Revenue (USD million)	Operating Expenses (USD million)	EBITDA (USD million)	Cumulative EBITDA (USD million)	Cumulative Net Cash (USD million)
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1	27,35	12,31	39,66	25,78	13,88	13,88	-124,94
2	28,03	12,62	40,65	26,42	14,23	28,11	-110,71
3	28,73	12,93	41,67	27,08	14,58	42,69	-96,13
4	29,45	13,25	42,71	27,76	14,95	57,64	-81,18
5	30,19	13,59	43,77	28,45	15,32	72,96	-65,86
6	30,94	13,92	44,87	29,16	15,7	88,66	-50,16
7	31,72	14,27	45,99	29,89	16,1	104,76	-34,06
8	32,51	14,63	47,14	30,64	16,5	121,26	-17,56
9	33,32	15	48,32	31,41	16,91	138,17	-0,65
10	34,16	15,37	49,53	32,19	17,33	155,5	16,68

Table 7.3.4 is a calculation of the payback period. The hotel comprises 140 rooms and has 28 rooms in each floor. The rooms can have 2 individuals. All told, the total occupancy will be 280 guests. We will assume room rate at \$400 per night. This is the mean price in the hotels around our selected region. The occupancy rate in San Antonio is 67.7 on average in 2026. We will make the assumption that the rate of increase in the room rates will be 10% per annum, which would begin in the second year. Rent on the 1st 2 nd floors is about 200 thousands of dollars. Its average payback period is approximated to be 9 years.

6.3.2. PCM Cost and Energy Savings

Even though Phase Change Materials (PCM) are perceived as a new technology in the United States, the process of utilising PCM to regulate thermal environment in massive span hotel buildings would easily help lessen cooling loads. San Antonio, Texas lies within the hot climate zone (ASHRAE Climate Zone 2A -Hot-Humid) in which yearly cooling demand is extremely high. This offers PCM a potentially useful passive energy-saving strategy.

In this project, PCM wall boards will be applied on the east and west outside walls which are exposed to the greatest amount of sunlight during morning and afternoons. The information is provided by American suppliers and recent research shows that PCM wallboards that have a thickness of 10mm are sold on average of 35 USD hear/ft 3.25 USD/ft and are priced at 35 USD/m² (32.5 USD/ft). Distribution expenses are low as there are PCM panel U.S. producers and distributors.

The architectural drawings of Altyn Fiesta indicate that the overall east and west wall $40\text{ m} + 40\text{ m} = 80\text{ m}$, and the height of 39 m. The wall area that is to be occupied by PCM panels is thus 3,120 m². Table 6.3.1 depicts the detailed cost estimation.

Table 6.5. PCM cost estimation data

East and west wall length (m)	80
Height of the walls (m)	39
Area of the PCM covered (m ²)	3,120
Cost of PCM per m ² (USD)	35
Cost of PCM for required area (USD)	109,200
Delivery and installation cost (USD)	5,000
Total PCM cost (USD)	114,200

The potential of saving energy can be approximated through electricity saving in the cooling process. Experiments on hot -humid climates indicate that PCM use is likely to save overall cooling energy costs by 30-35 percent per year. According to the programme of the Energy Star Portfolio Manager (U. S), the average hotel energy use is 275 kWh/m²*year.

This means that it would save an annual amount of energy in the covered wall area:

Energy saving = 0.32 multiplied by 275 kilowatt hours per m² multiplied by 3,120 metre square = 274,560 kilowatt hours per year.

In San Antonio (CPS Energy, 2025), the commercial electricity tariff stands at 0.12 USD/kWh. The energy saved cost is hence:

Energy saving cost = $0.12 \times 274,560 = 32,948$ USD/year

With the introduction of PCM, the question that should be assessed is whether integrating PCM can be economically and technically viable to make a decision. The calculation of the static payback period (SPP) is the most widespread procedure of the analysis of feasibility as

it shows how quickly the investment is going to be recovered. The calculation of SPP can be performed in the following manner:

$$SPP = C_PCM / S \quad (6.1)$$

Where CPCM equals the cost incurred to attain purchase and instal PCM material and S represents the annual income getting saved based on saving energy.

In the case of Altyn Fiesta, the price of PCM material and installation was estimated to be 114,200 USD (see Table 6.3.1). As it has been previously established, we can hypothesise that there are other construction and integration expenses that are estimated to be about thirty-five percent of the material cost. Hence, the effective cost is:

$$C_PCM = 1.3 \times 114,200 = 148,500 \text{ USD}$$

PCM application saved 32,948 USD/year as annual energy savings (see Section 6.3).

Replacing these values in Equation (6.1):

$$SPP = 148,500 / 32,948 = 4.51 \text{ years}$$

Therefore, the break-even timeframe of PCM material in Altyn Fiesta is about 4.5 years and it can be described as reasonable to construct energy efficiency standards in warm climates. This implies that PCM combination is technically advantageous and economically viable, to promote sustainable structure functioning in San Antonio.

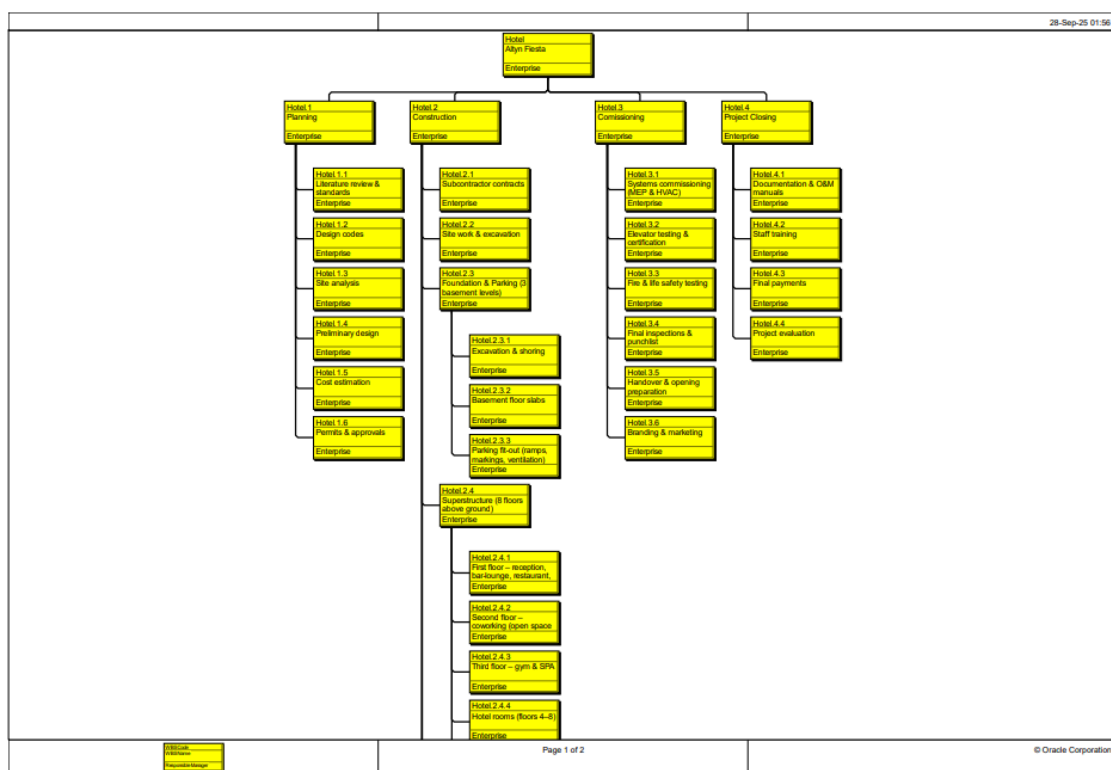
6.4 Work breakdown structure

With appropriate project scope management, the completion of the project is initiated on time and budget. Work Breakdown Structure (WBS) is one of the essential project scope management tools because it entails a hierarchical division of the total scope of work to be undertaken by the members of the project team. The resulting breakdown will not only explicitly define deliverables and the effort needed to get them, it also enhances process of planning and control mechanisms (Project Management Institute, 2006).

The recommendations by the PMBOK Guide have been used to prepare the WBS of the given project to ensure that it was a well-structured and exhaustive process illustration. The project was broken down into major phases like planning, execution, construction, monitoring and control, and completion. These stages were further split into respective

deliverables and related tasks and activities, which helped plan each project phase carefully and monitor the project life cycle efficiently (Project Management Institute, 2021).

The WBS was visualised and constructed in the Primavera p6 professional. This software was useful to formulate transparent and more detailed diagrams that ease to the active collaboration of the stakeholders and can be used as a reference point to trace the changes and the progress. The application of such tools allows the staff to have a regular picture of the scope and structure of the project, which, in its turn, will guarantee the success of the latter.



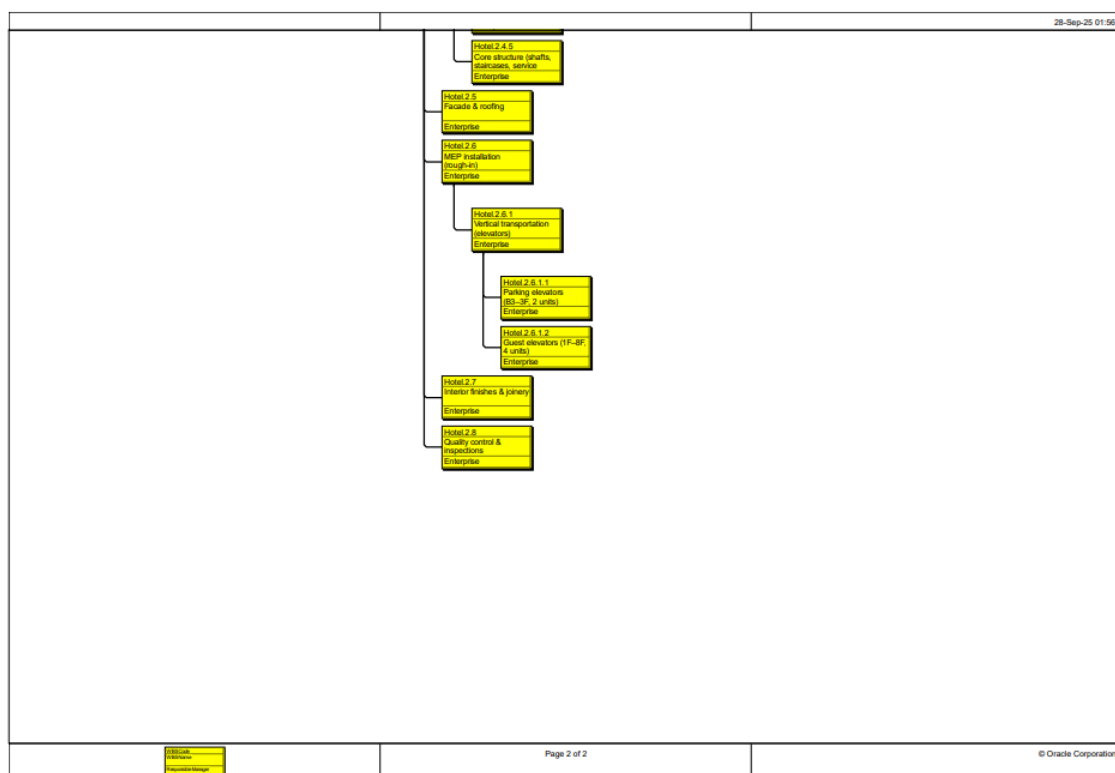


Figure 6.2. Work breakdown structure of project

The Work Breakdown Structure (WBS) was supported by a WBS Dictionary. The dictionary includes a description of the work package, its deliverables, owner, estimated time and the estimated cost. It is the authority on scope, serves as the baseline to schedule and estimate budget, and minimises scope creep during the execution phase.

Table 6.6. WBS Dictionary

WB S Cod e	Work Package	Description	Deliverabl e	Respons ible	Dur atio n (da ys)	Est. Cost (USD)
1.0	Project Initiation	Project charter, stakeholder register and initial scope baseline	Signed project charter	Project Manager	30	75,000
2.0	Permitting &	Obtain all building, zoning, electrical,	Permitting approved	GC + Architect	90	180,00 0

	Preconstruction	plumbing and fire permits from City of San Antonio DSD				
3.1	Site Excavation	Three-level basement excavation with shoring & dewatering (18,500 C.Y.)	Excavation for foundation	Earthwork Subcontractor	110	1,400,000
3.2	H-Pile Foundation	Drive and installation of 145 driven steel H-piles and pile caps	Tested pile foundation	Foundation Subcontractor	60	2,546,050
3.3	Basement Walls & Slabs	Placed-in-place reinforced concrete basement walls (22,000 S.F.) and slabs-on-grade	Watertight basement structure	Concrete Contractor	90	440,463
4.1	Superstructure Frame	8-storey post-tensioned concrete frame (4,822 m ³ ready-mix; 650 ton rebar; 39,560 kg PT strands)	Topped-out structural frame	Concrete Contractor	240	6,725,200
4.2	Exterior Enclosure	Masonry walls with Type IB insulation and aluminum Low-E windows (14,500 S.F.)	Weather-tight building envelope	Facade Subcontractor	120	6,540,401
4.3	Roof Coverings	EPDM roofing membrane and insulation (12,000 S.F.)	Leak-tested roof assembly	Roofing Subcontractor	30	295,407

5.1	Interior Partitions & Doors	Acoustical rated GWB partitions (95,000 S.F.) and 320 internal doors (hotel suite and common)	Framed and fitted interiors	Interior Contractor	150	3,365,931
5.2	Interior Finishes	Wall finishes (paint, vinyl, tile — 145,000 S.F.) and floor finishes (carpet, LVT, porcelain — 115,000 S.F.)	Finished interior surfaces	Finishing Subcontractor	120	3,446,500
6.1	MEP - Plumbing	Plumbing fixtures for 140 rooms and kitchen	Pressure-tested plumbing system	Plumbing Subcontractor	150	4,120,000
6.2	MEP - HVAC	Variable Refrigerant Flow system and central ventilation for 115,000 S.F.	Commissioned HVAC system	MEP Subcontractor	180	3,850,000
6.3	MEP - Electrical	Electrical service, branch wiring and lighting for 115,000 S.F.	Electrical system electrically energised and tested	Electrical Subcontractor	180	3,950,000
6.4	Fire Protection	115,000 S.F. wet-pipe sprinkler system	Code-compliant fire protection	Fire Protection Subcontractor	90	1,080,000

6.5	Vertical Transport	4 elevators (3 passenger + 1 service)	Commissioned elevators	Elevator Vendor	120	1,450,000
7.0	Sustainable Systems	East/west wall Phase Change Material (PCM) installation (3,120 m ²)	PCM panels installed and tested	Specialty Subcontractor	45	114,200
8.0	Commissioning & Handover	Building functionality, LEED documentation, punch-list and Owner training	Owner-accepted building	Project Manager + GC	105	500,000

The WBS Dictionary in Table 7.6 below shows the main work packages; the complete dictionary (more than 60 packages) is stored in the Primavera P6 project file, to be used by the scheduler to define duration of activities and by the cost engineer to provide detailed cost estimates.

6.5 Scheduling

Scheduling, the foundation for construction project management, is what will ensure not just completion of tasks on time but also efficient cost management. A well organised schedule will help in getting off to a good start and the team takes on the various stages of the project. It is also beneficial in pinpointing the potential bottlenecks and to take proactive measures to avoid wastage of time for better efficiency of the overall project. In this project, the Microsoft Project tool was used to get a detailed schedule of the project. This helped in the development of a Gantt chart as this chart is useful in identifying the stages of a project and the average time required for the stages. Non-work days, holidays and local constraints could be imposed to develop a schedule. In this way, identification of a critical path was illustrated and activities that affected the time span of the project were defined.

The project weeks were initially calculated to a period of 1,028 days and the Gantt chart will clearly show the critical activities. These planning tools not only simplify the planning process but also engage stakeholders because the project progress has the same representation and is in the form of a picture. This method is in line with best practices in construction management, which states it is vital to plan all elements and organise multiple events and resources of the project (ProjectManager, 2024).

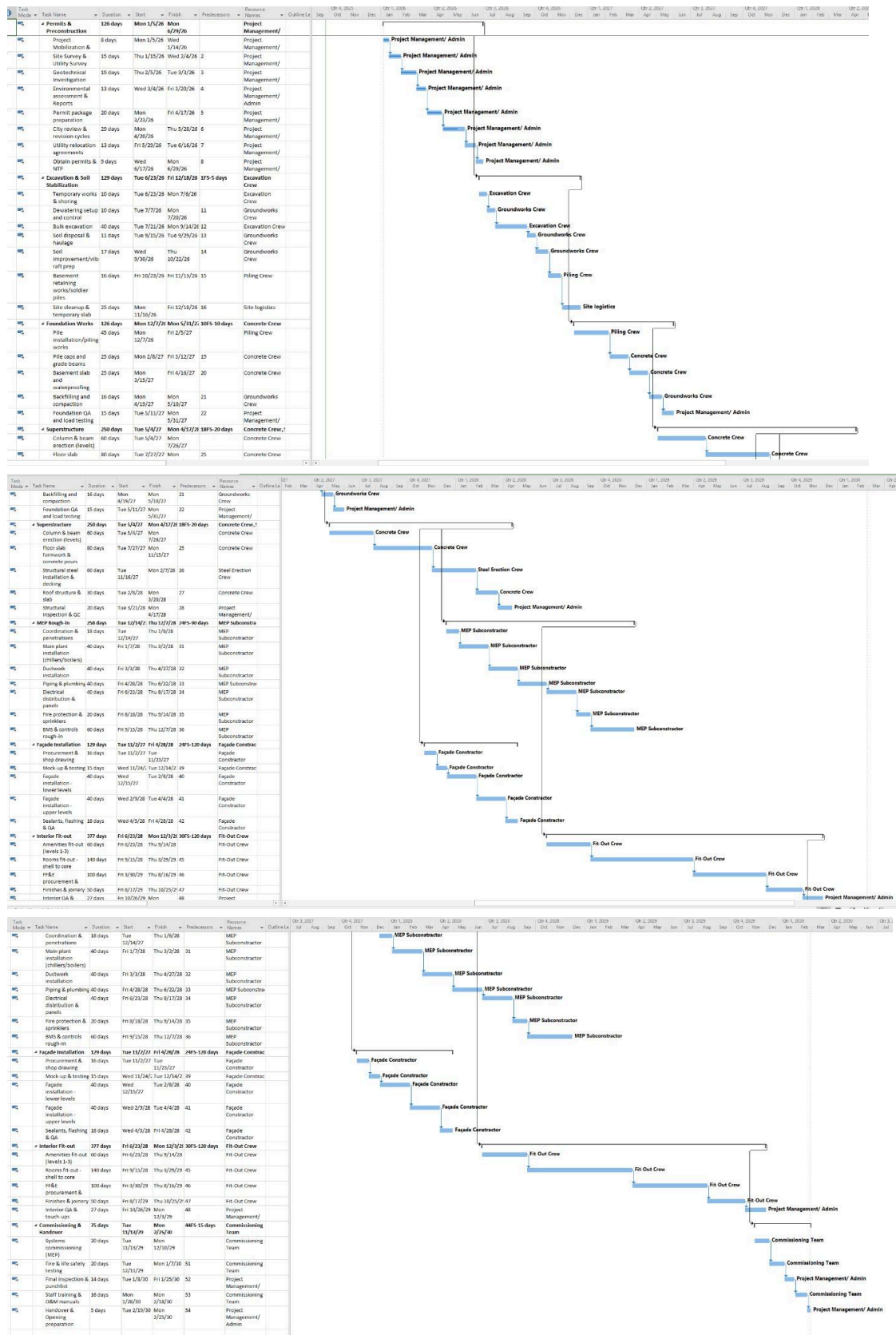


Figure 6.3. Gantt Chart of the project (Microsoft Project)

From the WBS Dictionary and the Gantt Chart, the resource schedule was developed to predict the labour, equipment, and material needs in each stage of the project. The duration

estimates were calculated from the final design information (work volumes from the structural and MEP drawings) and productivity rates for San Antonio, Texas. The table below presents a peak load for each phase.

Table 6.7. Resource Schedule Summary by Phase

Phase	Duration (months)	Peak Labour (workers)	Key Equipment	Key Materials
Permitting & Preconstruction	3	5	Surveying equipment	Permit documents
Excavation & Soil Stabilization	6	35	2 excavators, 8 dump trucks, 1 dewatering pump	Sheet piling, shoring bracing
Foundation Works	6	60	Pile driver, crane, concrete pumps	145 H-piles, 22,000 S.F. basement walls, rebar, ready-mix
Superstructure	11	90	2 tower cranes, concrete pumps, PT tensioning rigs	4,822 m ³ concrete, 650 t rebar, 39,560 kg PT strands, formwork
MEP Rough-in	12	70	Lifts, specialised MEP tooling	VRF units, piping, wiring, conduit, fixtures
Facade Installation	4	40	Mast climbers, crane support	Masonry units, Low-E windows, insulation
Interior Fit-out	17	85	Scissor lifts, finishing tools	GWB, doors, paint, LVT, carpet, tile
Commissioning & Handover	3.5	15	Test instruments	O&M manuals, LEED documentation

Through the superstructure phase, peak combined labour demand is approximately 90 workers which is used to determine the welfare space and parking on the site plan (Section 6.10). Plant operation is aligned with the two cranes at the north-west and south-east corners.

6.6 Risk management

The Modified Risk Matrix (MRM) assessment approach by Guo et al. (2021) was applied to building a 8-storey hotel in San Antonio. Identification of risk level, which is employed in the current method, has a formula:

$$R_{ik} = L_{ik} \times S_{ik}$$

Where, R - risk level;

i - unsafe behaviour;

k - accident type;

L - level of probability of unsafe behaviour;

S - severity level of the consequence.

The following is as Risk Assessment Table:

Table 6.8. Risk Assessment

Category		Risk	Likelihood (1-5)	Severity (1-5)	Risk Level	Mitigation Strategy
Site and Environmental	E1	Soil Instability / Liquefaction	2	4	8	Full geotechnical investigation prior to excavation; soil stabilization means; geotechnical engineer approval of foundation before it is poured.
	E2	Weather Delays	4	3	12	Planning weather sensitive tasks when the weather is favourable; use

						rain covers, water resistant materials.
	E3	Flooding and Stormwater	3	4	12	Develop a good storm water management system; install drainage systems, e.g. permeable paving.
	E4	Seismic Activity	1	3	3	Application process It can be initiated earlier, at least, permit application; provide information to the city regulators.
Legal and Regulatory	L1	Permit Delays	3	3	9	It is possible to initiate permit application early, and inform the city regulators regularly.
	L2	Changes in Building Codes/Regulations	2	3	6	Monitor Texas Building Code is revised quarterly, legal liaison to confirm that the code is followed properly, 5%

						schedule buffer to allow the regulatory code to be altered.
	L3	Environmental Compliance Penalties	2	4	8	Carry out environmental survey on monthly basis; keep the waste disposal and emission records; make sure all the subcontractors are informed about the TCEQ compliance.
Financial	F1	Cost Overruns	3	5	15	Enter into fixed price agreements with the suppliers. Working capital: have a buffer fund to meet price fluctuations.
	F2	Funding Shortfalls	2	5	10	Get variety in source of funds; in case of alternative source of funds have backup power ready in form of contingency plans.

	F3	Changes in labor and material prices	4	3	12	Sign fixed-price contracts on 70% of key materials; keep escalation clauses of supplier; stay on BLS construction index every quarter.
Project Management	M1	Labour Shortages	3	4	12	Arrange labor supply deal with 2 or more subcontractors; have on-demand labor pool; use retention bonuses in high season periods.
	M2	Scheduling Delays	3	4	12	Detailed project scheduled in the MS project, weekly project tracking, early identification of critical tasks, and addition of time buffers in case of delay.
	M3	Contractor Performance Issues	2	4	8	Prequalification/performance score; give contractual

						KPIs; use penalty provisions regarding delay above 5%.
Safety and Security	S1	Worker Injuries/ on-Site Accidents	3	5	15	Safety training of all of the workers, the use of PPE, daily inspection of the site, and emergency response plan and first aid team on the site.
	S2	Fire Hazard	2	4	8	Organize a fire protection scheme, appoint a fire officer, place fire extinguishers and fire alarms where required, regulate welding and electrical activities and provide frequent fire drills to employees.
Design and Quality	D1	Design Flaws / Revisions	3	4	12	Review the design with stakeholders on a regular basis, and permit changes.

	D2	Structural Integrity Issues	2	5	10	Establish 3rd party structural audit; load testing;
	D3	Poor Quality Materials	3	3	9	Validated suppliers; check on suppliers on delivery.
Supply Chain	SC1	Material Delivery Delays	3	4	12	Make sure that the suppliers are diverse, order early enough and make arrangements when it comes to storing inventory levels of the important materials.
	SC2	Shortage of Specialized Equipment	2	4	8	Early rent out specialized equipment; build contacts with various equipment renting companies.
Operational	O1	Unexpected Utility Disruptions	2	3	6	Arrange utility companies; prepare the positioning utility plan, with backup

						generators.
	O2	Community Complains	3	2	6	Carry out noisy work at the time when the noise is justifiable; and where feasible, adopt means of sound insulation.

The Risk Matrix is designed after determining the level of risk of the various forms of potential risks.

L_{ik}	S_{ik}				
	1 (Negligible)	2 (Minor)	3 (Moderate)	4 (Serious)	5 (Critical)
1 (Remote)	I	I	I	I	II
2 (Unlikely)	I	I	II	II	III
3 (Likely)	I	II	II	III	III
4 (Highly likely)	I	II	III	III	IV
5 (Near certainty)	II	III	III	IV	IV

Figure 6.4. Modified Risk Matrix (MRM) method (Guo et al., 2021)

A Risk Matrix of the hotel building project is illustrated as below:

		Severity Level				
		1	2	3	4	5
		(Negligible)	(Minor)	(Moderate)	(Serious)	(Critical)
Level of Probability	1 (Remote)			E4		
	2 (Unlikely)			L2 O1	E1 L3 M3 S2 SC2	F2 D2
	3 (Likely)		O2	L1 D3	E3 M1 M2 D1 SC1	F1 S1
	4 (Highly Likely)			E2 F3		
	5 (Near Certainty)					

Figure 6.5. Modified Risk Matrix (MRM) for the Altyn Fiesta project

There are several categories of risk that have been identified in the eight story hotel building in San Antonio. The highest risks are F1 and S1 as shown in Figure 6.5. These risks are related to the cost overruns and injury / accident on site to workers respectively and they are the most significant risks particularly in high wind areas. These risks are high and the risk mitigation strategies shall be followed. The lowering of the risk does not mean it is not important. It is necessary to follow the requirements and regulations in order to prevent risks, as well as to meet the effectiveness of the construction project assignment and prevent delays.

6.6.1. Risk Monitoring and Controlling

Risk monitoring and controlling is an ongoing process that involves monitoring of identified risks, re-evaluation of the remaining risks, identification of new risks and assessment of mitigation measures taken as the project evolves (PMI, 2021). For Altyn Fiesta project, the following monitoring is foreseen:

- Weekly risk review meetings (Project Manager chairs meetings with GC and key subcontractors): new risks, mitigation actions and trigger events.
- Monthly risk register update in Primavera P6; the residual risk rating is updated after verifications of mitigation actions.
- Progress measures (Earned Value Management): Schedule Performance Index (SPI) and Cost Performance Index (CPI) are reviewed weekly, a CPI or SPI less than 0.90 triggers action.

- Safety indicators: Total Recordable Incident Rate (TRIR) and near misses are logged weekly; a TRIR greater than 2.0 warrants a safety stand-down and re-training.
- Reserve management: 15% contingency reserve is monitored monthly; if used more than 30% in the first year a management review is triggered.
- Change management: all scope, schedule, cost changes are recorded, risk scored and approved by the Project Sponsor before being implemented.

Table 6.9. Risk Monitoring KPIs

KPI	Target	Tolerance / Trigger	Frequency	Responsible
Schedule Performance Index (SPI)	≥ 0.95	< 0.90 triggers corrective action	Weekly	Project Manager
Cost Performance Index (CPI)	≥ 0.95	< 0.90 triggers corrective action	Weekly	Cost Engineer
Total Recordable Incident Rate (TRIR)	< 1.5	> 2.0 triggers stand-down	Weekly	Safety Officer
NCR count (open)	≤ 5	> 10 triggers QA audit	Weekly	QA/QC Manager
Contingency reserve consumption	$\leq 30\%$ per year	$> 30\%$ triggers management review	Monthly	Project Sponsor
Change request backlog	≤ 5 open > 14 days	Breach triggers PMO escalation	Weekly	Project Manager

Any risks that reach their trigger point are escalated to the Project Sponsor within 48 hours and a corrective action plan is recorded in the risk register. The lessons learned from completed risks are documented monthly and used to close the continuous improvement loop in Section 6.7.

6.7 Quality Management

The entire building of the 8-story hotel Altyn Fiesta in San Antonio will utilize a set quality management plan at all levels throughout the process. In order to ensure that there is

conformance to the industry standards, the requirements of LEED certification and the Texas regulatory requirements, the Quality Management Plan sets some measures, checklists and metrics. The overall objective is to deliver a high quality mixed use facility with the least amount of flaws and the long term safety of the 140 rooms and the 7200 m² of the commercial space. This plan is based on the ISO 9001: 2015, ASTM and ANSI standards and was fully compliant with the San Antonio Building Codes and the needs of Texas Department of licensing and regulations (TDLR).

6.7.1. Quality Control Measures

Checking and Testing of Materials.

The ultimate objective of the inspection and testing is to ensure that all the materials used in the Altyn Fiesta project are in line with the specifications of the project and the general industry standards. Upon delivery of all structural and finishing materials to the site, they are expected to be in agreement with the requirements of ASTM (American Society for Testing and Materials) and ANSI (American National Standards Institute). An essential aspect of the process was the laboratory test of structural materials like concrete in the frame of 8 stories and steel reinforcements, to guarantee the design requirements of strength and durability of mixed use high occupancy buildings. Moreover, attention will be paid especially to the quality certificates of Phase Change Materials (PCM) and energy efficient systems which must be certified by LEED. The use of poor quality of materials will be entirely eliminated because structural or efficiency problems can be avoided.

Workmanship Assessment

Inspections of workmanship must show that the construction works are harmonizing with the architectural design, and the local rules that may be implemented locally in San Antonio, Texas. The performance is benchmarked against the standards of International Building Code (IBC) and with the personal sustainability goals of the specific project. Any irregularity is noted to take corrective measures. Since the 8-story building is complex, there will be a need to hire outside engineers to conduct special tests on the precincts, seismic reinforcement and integration of commercial grade HVAC systems on the lower three levels of the building.

Non-Conformance Reporting (NCR)

Non-Conformance Reporting (NCR) is a report which will be formally issued when failure to meet the established standards of quality in process or material is experienced. These reports will necessitate remedial actions within a very short time and a subsequent inspection would be carried out after which the measures would be adhered to determine whether they were achieved. Some of the approaches that will be used by the project are; a continuous improvement loop, under which, lessons learned during NCRs are documented to prevent the occurrence of the same issues when building other floors or commercial areas.

Subcontractor Quality Management

As part of the integrity of the Altyn Fiesta development, all the subcontractors are to show their rigid adherence to the general Quality Management Plan of the development. The subcontractors are expected to provide their individual Quality Control Plans (QCP) which must be approved before the work commences. This will be quantified through quality audits and site walk-throughs respectively every month and week respectively. The non-meeting of specifications of the specialized systems of the hotel (e.g., parking garage ventilation or gym facilities) by sub contractors would be rebuked as part of action plans or termination of the contract.

Document Control

Correct and open documentation is essential to the Altyn Fiesta project more so when it comes to the future audit on LEED and tracking payback period. A digital document management system will be employed in order to store accreditation of all material test reports, inspection checklists, supplier certifications, and NCR logs at a central point. This system follows ISO 9001:2015 model in such a way that no quality records will be modified, and all quality documents will be readily available to regulatory checks that will be carried out by the City of San Antonio or final delivery of the project to the sponsors of Nazarbayev University.

6.7.2. Quality Metrics



Figure 6.6. Quality Metrics

Table 6.10. Quality Metrics Description

Metric	Target / Requirement	Measurement Method
Material Quality Compliance	100% of structural materials must meet ASTM & ANSI standards.	Review of mill test reports and on-site laboratory testing of concrete samples.
Structural Integrity & Safety	Zero tolerance for structural deviations from San Antonio Building Codes.	Third-party inspections of foundation and 8-story steel/concrete frame.
Task Completion Rate	> 95% of scheduled activities completed	Weekly progress reports and updated Gantt

	according to the CPM schedule.	charts.
Sustainability & Energy Performance	30–35% energy savings compared to baseline (via PCM integration).	LEED energy modeling and post-installation thermal efficiency tests.
Budget Adherence	Final construction cost within $\pm$ 10% of the \$71.1M budget.	Monthly financial audits and Earned Value Management (EVM).
Rework Percentage	Less than 3% of total construction man-hours spent on rework.	Non-Conformance Report (NCR) tracking and site supervisor logs.

6.7.3. Quality Control Checklist

The checklist was developed based on the measures of quality control to determine that no issues exist with the construction of the hotel Altyn Fiesta. In Figure 6.9, the checklist can be seen. The checklist will include 4 phases which suggest crucial steps of the construction process, such as foundation up to the introduction of energy-saving systems.



Hotel Construction Quality Control Checklist

PROJECT INFORMATION

Project Name: "Altyn Fiesta" Multi-Story Hotel

Location: 611 N Flores St, San Antonio, TX 78205, USA

Date of Inspection: 25.03.2026

Project Manager: Bauyrzhan Tyleukhanov

Inspection Team Members:

- Dariga Satybaldiyeva
- Dutbayev Nurtay
- Aibibi Abuova

Phase 1: Pre-Construction Checklist

Site Preparation:

- ☐ Verify site clearing and removal of debris and vegetation
- ☐ Confirm grading and leveling according to site plans
- ☐ Check erosion and sediment control measures in place
- ☐ Ensure access routes for construction vehicles are established

Foundation Works:

- ☐ Confirm excavation depths and alignments are according to the drawings
- ☐ Verify soil conditions match geotechnical recommendations
- ☐ Inspect formwork and reinforcement placement
- ☐ Check that the concrete mix and curing procedures meet specifications

Phase 2: Structural Construction	Framing and Structural Elements ☐ Verify that structural steel and concrete installations follow design plans ☐ Inspect welding and bolting connections for compliance ☐ Confirm vertical and horizontal alignments Exterior and Roofing: ☐ Check the installation of exterior walls, cladding, and waterproofing ☐ Inspect roofing materials and drainage system installation
Phase 3: Interior and Finishing	Interior Installations: ☐ Inspect framing, drywall, and partitions for accuracy ☐ Verify mechanical, electrical, and plumbing (MEP) rough-ins ☐ Confirm fire-stopping and sound-proofing installations Finishing Works: ☐ Check painting, flooring, ceiling, and fixtures for quality standards ☐ Inspect doors, windows, and hardware installations
Phase 4: Final Inspections	☐ Verify system testing (fire alarm, HVAC, plumbing, electrical) ☐ Confirm all punch list items are completed ☐ Ensure the building is cleaned and ready for handover ☐ Obtain all necessary permits and certifications for occupancy

Figure 6.7. Quality Control Checklist for Altyn Fiesta Project

6.8. Procurement planning

6.8.1. Stakeholders Analysis

A stakeholder analysis is carried out to chart the network and hierarchy of all stakeholders involved in a construction project. Its main aim is to maintain a smooth flow of the project and eliminate the possible conflicts and avoid delays. With the help of such tools as Primavera P6, this analysis can create a clear hierarchy of authority and communication, starting with the project owner, and going down to the individual subcontractors, as seen in the hierarchy below:

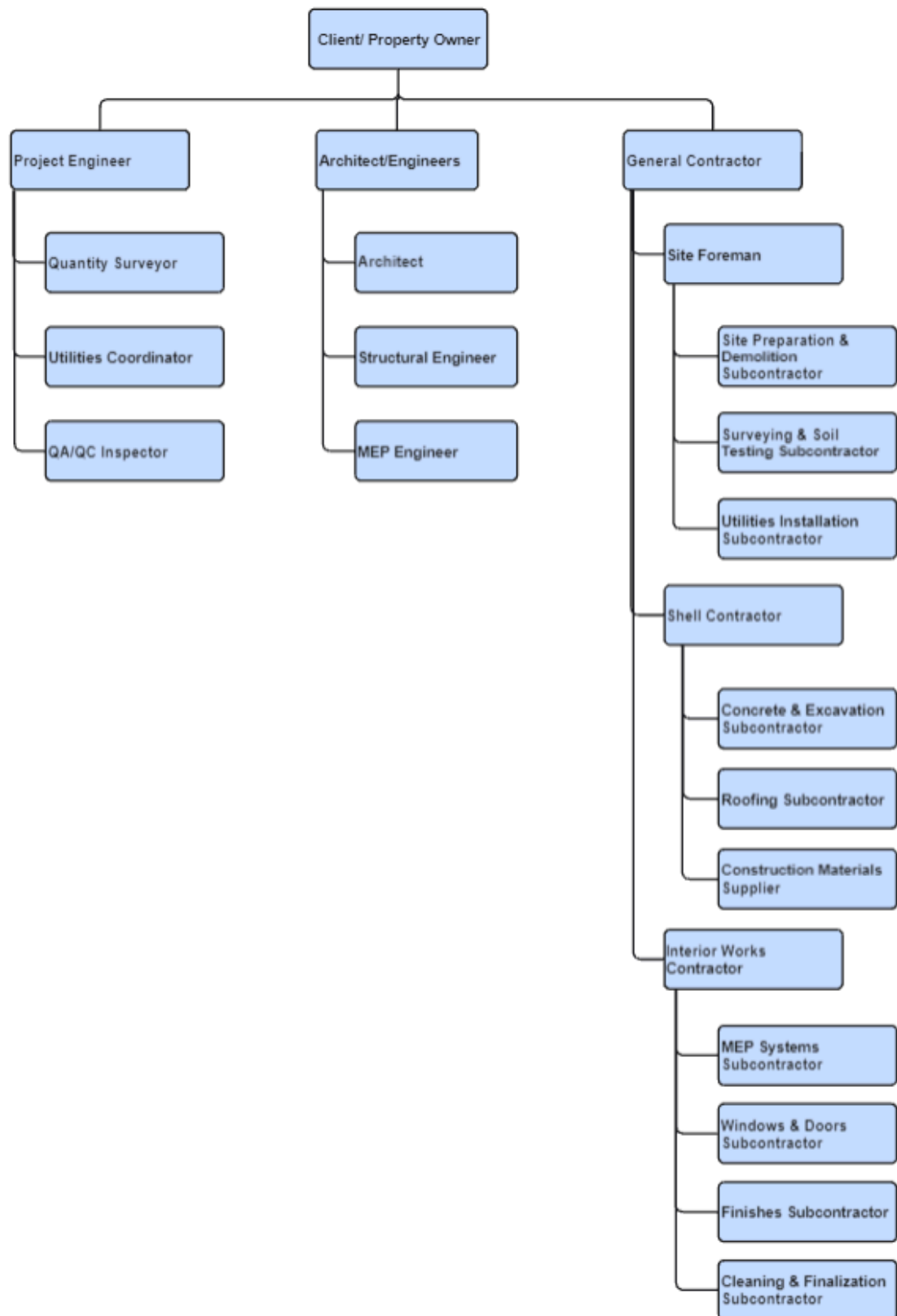


Figure 6.8. Stakeholders Hierarchy

The Project Sponsor will be the key stakeholder since he/she will have final say in terms of the financial support and strategic decision making to guarantee continuity in progress. The Lead Architects, Structural Engineers and the General Contractor are the secondary level of the project hierarchy, as they handle the technical implementation and control the delegation of duties to tertiary parties.

The Stakeholder Engagement Matrix is built on this organizational structure, where each member is assessed on the basis of two important indicators: Interest, or how personally or professionally they are invested in the final delivery, and Influence, or how much they can impact the project direction and outcome. These metrics are directly related to accountability: the more interest and influence they have, the more responsibility in the project lifecycle the stakeholder gets.

The detailed analysis of stakeholder is as shown in the following table:

Table 6.11. Stakeholders Analysis

Stakeholder Category	Core Project Responsibility	Level of Interest	Level of Influence
Client/ Property Owner	Strategic direction, financing and eventual approval of project.	High	High
Architectural & Design Team	Building design and adherence to Texas building code.	High	Moderate
Construction Manager (GC)	Coverage of the entire site, safety and implementation.	High	High
City of San Antonio (DSD)	Issuing permits, zoning, compulsory inspections.	Moderate	High
Specialized Subcontractors	Implementation of certain trades (MEP, framing and masonry).	Moderate	Moderate

Quality & Tech Supervision	Checking and inspection of material by the third party.	High	High
Supply Chain Partners	Logistics and supply of raw construction materials.	Moderate	Low
Utility Entities (CPS/SAWS)	Integration of infrastructure (power, water, and sewage).	Low	Moderate
Regulatory Safety (OSHA)	Supervision of health and safety at the work place.	Low	High
Field Labor / Technicians	Immediate physical constructions and technical installations.	Moderate	Low

6.8.2. Procurement Process

The Altyn Fiesta project has a procurement process based on four phases, which is recommended by PMBOK:

1. Plan Procurements - the Project Manager and Cost Engineer determine what is to be procured, its source, and the type of contract. The result of this step is the procurement management plan and the list of packages.
2. Carry Conduct Procurements - solicitation documents are sent to qualified bidders, bids received, appraised against the source selection criteria (Section 6.8.5) and contracts awarded.
3. Control Procurements - contracts are managed and the performance of vendors is controlled by way of inspections, progress reports and the quality measures specified in Section 6.7.2.
4. Close Procurements - when completed, verify deliverables, close punch-list, release final payments, document lessons-learned.

6.8.3. Procurement Documents

On this project, four primary types of procurement documents are in place which are chosen based on the scope and risk profile of each package.

Table 6.12. Procurement Document Types Used in the Project

Document Type	When Used	Example Packages on Altyn Fiesta.
Request for Information (RFI)	Market research at the initial stage when the scope is not defined yet.	Initial PCM supplier investigation; market scan geotechnical contractor.
Invitation to Bid (ITB) / Request for Quotation (RFQ).	Clear, commodity-based scope wherein price is the major factor.	Ready-mix concrete, rebar, formwork, standard interior finishes
Request for Proposal (RFP).	Multifaceted project where technical competence is as important as cost.	VRF HVAC system, post-tensioning, PCM integration, facade design-build
Master Service Agreement + Statement of Work (SOW).	Services that last a long period or several work orders on the same umbrella contract.	Architect of record, structural engineer, third-party QA/QC

6.8.4. Vendor List

An initial market research within San Antonio region generated a short-list of qualified suppliers to each of the key procurement categories. The vendors were chosen based on licensing in Texas, completion of at least five projects of the same magnitude, sound safety record ($EMR \leq 1.0$) and availability of current bonding capacity.

Table 6.13. Short-listed Vendors by Category

Category	Type	Vendor 1	Vendor 2	Vendor 3
Ready-mix concrete	Material	Alamo Concrete Products (CEMEX)	Martin Marietta Materials	Texas Materials (Campbell Concrete)
Reinforcing steel	Material	CMC Steel South Texas.	Nucor Rebar Fabrication	Harris Rebar

Structural steel / H-piles	Material	Triple-S Steel Holdings	Nucor Corporation	Steel Dynamics
Excavation & earthworks	Subcontractor	W.W. Webber	SpawGlass Civil Construction	Clark Construction of Texas
Structural concrete	Subcontractor	Bartlett Cocke	Capital Excavation	Balfour Beatty Construction Texas
Facade & glazing	Subcontractor	Trainor Glass	MG McGrath	Karas & Karas Glass
VRF HVAC	Specialist	Daikin Comfort Technologies (Carrollton, TX)	Mitsubishi Electric Trane HVAC US	LG Electronics USA HVAC
Elevators	Specialist	Otis Elevator Company	KONE Inc.	Schindler Elevator Corporation
PCM wallboard	Specialist	Phase Change Energy Solutions (BioPCM)	Insolcorp LLC (INFINITE R)	Microtek Laboratories
Quality & testing	Services	Terracon Consultants (Austin/San Antonio)	Raba Kistner	Arias Geoprofessionals

6.8.5. Source Selection Criteria

A scoring is based on a weighted score on the bids. The evaluation committee (Project Manager, Cost Engineer, Quality Manager and Safety Officer) rates each criterion on a 1-5 scale, multiplies it by its weight, and adds it up. The highest weighted total score is awarded to the bidder, as long as the price falls within a range of +/-10% of the median bid.

Table 6.14. Source Selection Criteria

Criterion	Weight	Description / Threshold
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Price	30%	Bid amount with taxes, which is totaled, and expressed as a ratio of the lowest bid.
Technical capability	20%	Experience on 3 or more similar projects (8 or higher stories, mixed use, LEED) documented.
Schedule	15%	Capability to achieve the project milestone dates with no schedule buffer erosion.
Financial stability	10%	Good credit (D&B rating), minimum bonding capacity equal to 125% of bid value.
Safety record	10%	Experience Modification Rate (EMR) 1.0 or less and no fatalities during the past 3 years.
Quality & references	10%	Two references that can be verified by similar projects done within the past 5 years.
Local presence / sustainability	5%	Practices that supply chain supports LEED and proximity to the San Antonio site.

Bids are rejected when one or more of the three pass/fail requirements are not satisfied: valid Texas license, bonding capacity (at least 125 percent of the bid value) and EMR (not exceeding 1.0). This makes sure that only technically and financially responsible bidders are allowed to be part of the weighted evaluation.

6.9. Construction Safety

Construction safety forms a crucial aspect of the project, to protect the workers and reduce the occurrence of accidents on the site throughout the construction process. The complexity of the building (8 stories and underground) will necessitate the identification and management of various hazards by ensuring that safety planning as well as adherence to regulations is properly implemented.

The table below shows the key construction safety hazards, the level of risk associated with them, and the control measures.

Table 6.15. Safety Risk Register

Hazard type	Likelihood (1-5)	Severity (1-5)	Risk Level	Control Measures
Working at Heights	4	5	20	Fall protection measures; safety training; inspection of scaffold.
Heavy Equipment Operation	3	5	15	Operators Certified; Equipment inspection; controlled zones.
Falling Objects	4	4	16	PPE (helmets); netting off; restricted access areas.
Electrical Hazards	3	5	15	Grounding; lockout/tagout; frequent inspection.
Fire Hazard	2	4	8	Fire extinguishers; hot work permit; fire drill.
Excavation Collapse	3	5	15	Shoring systems; inspections; controlled access.
Material Handling Injuries	4	3	12	Lifting equipment; supervision; training.
Slips, Trips, Falls	5	3	15	Maintenance; signs; walkways.
Noise and Dust Exposure.	4	3	12	PPE; dust control; monitoring.

Confined Space Hazards	2	5	10	Allowances; air conditioning; gas check.
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Safety measures, frequent checkups and training of the workers will be taken to minimize risks and adherence to safety standards during the project.

6.10. Construction Site Planning

Kingsbury St (west), L St (north) and Camaron St (south) surround the construction site, allowing easy access from several directions. The master building is situated in the middle of the site, facilitating vehicle movement around the site through an internal access road. The building is serviced by two tower cranes located on the north-west and south-east corners respectively to completely cover the working area. The eastern area is set up for material storage, with stockpiles for aggregates, cement and drywall, and reinforcement bars, allowing efficient material management and supply. The motor and equipment park is positioned in the north-east corner for convenience but not in the way of critical construction operations. Workers' facilities are nearby but isolated from the main workings to ensure safety. In summary, the design separates operations, allows for efficient movement of material and people, and provides a safe working environment for both workers and machinery.

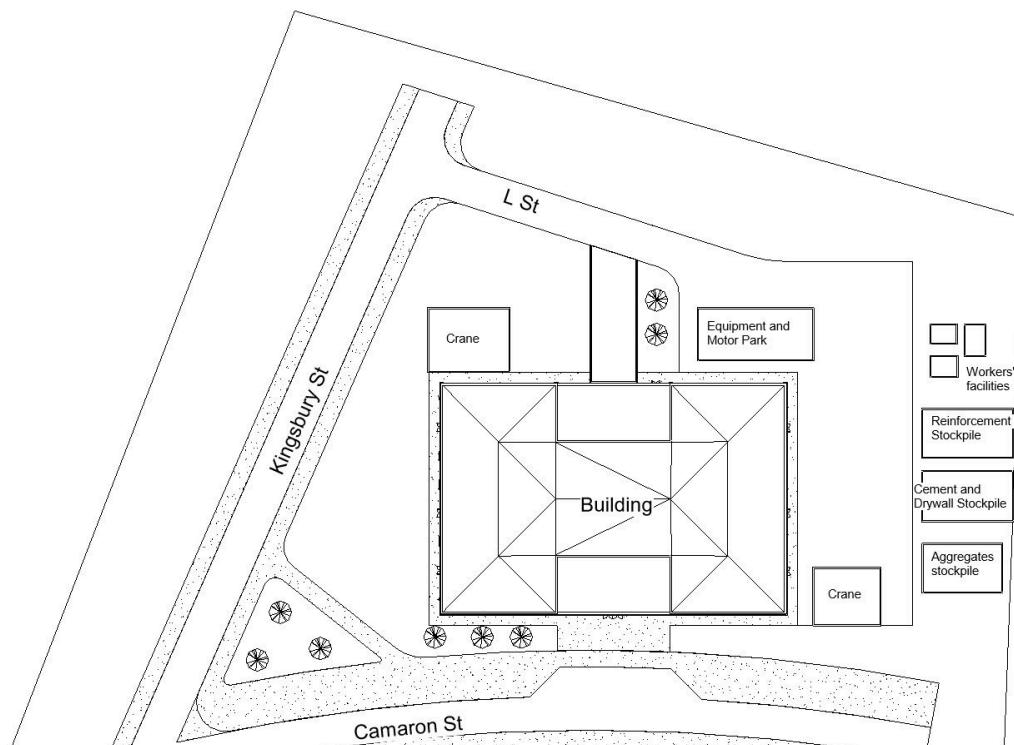


Figure 6.9. Site plan

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