DESIGN OF MULTI-STORY ACADEMIC BUILDING AT NAZARBAYEV UNIVERSITY, ASTANA, KAZAKHSTAN

(Capstone II Project)

Bachelor of Engineering

(Civil Engineering)

NAZARBAYEV UNIVERSITY

Aidana Agibayeva
Ardana Aldonggarova
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Zhanbolat Kulzhabek
Alisher Suleimen

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DECLARATION

We hereby declare that this report entitled “Design of multi-story academic building at Nazarbayev University” 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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11.04.2017

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Abstract

Nazarbayev University has experienced significant growth in the past few years and it is projected to increase in the future. Hence, the necessity of multi-story academic building at Nazarbayev University is high. This report includes the design of 9 story academic building with the total area of 16000 m^2. Based on the literature review performed within the capstone I, architectural, structural and geotechnical designs were developed. Moreover, feasibility study and project management including cost estimations were determined.

Architectural part of the project involves the detailed design based on the CNaR of the Republic of Kazakhstan. The site layout with its landscape and parking zone, technical drawings including 3D of the building, ground and typical floor plans and others were developed. In addition, selection of non-structural materials for floor, ceiling, exterior and partition are identified.

The geotechnical analysis is one of the main parts of every project as it includes the design of one of the most important details of the structure, namely foundation. This highrise building is located in on area with windy and snowy climate and where the groundwater table is located near the ground level. Therefore, the accurate analysis have to be performed in order to solve the challenges such as high axial and shear loads. This report covers all steps starting from the soil analysis to foundation reinforcement design. All necessary features of the project were considered during the design of the footings.It was found that deep pile foundations are one of the best solutions for structures that are prone to high lateral loads.

Last, but not the least important part of the report is project management where cost estimations, economic analysis, risk assessment, duration of the project were determined using different methods including the quantity take-off, WBS, Gantt chart and etc. Additionally, quality management, occupational safety and health and environmental impacts of the project were considered.

Finally, obtained structural and geotechnical detailed design of the project is appropriate to withstand all applied loads. During the design stage, architectural standards were considered and proven to be feasible project. Hence, it can be concluded that all objectives were fulfilled.

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GOST 30826-2001 Laminated glass

1 Introduction

1.1 Project Description

Multistory academic building at Nazarbayev University is an additional facility that will be missioned as an international educational center in Astana. Implementation of this project takes an essential role, since it will considerably contribute to the establishment of the benchmark for higher education institutions of the country. In order to make it real, the Milky Way Construction Company offers its service to design and implement the project. Since the client requirements are large conference halls, computer labs and provided land of 100 x 150 m, it is challenge for company members to provide with 10-story unique architectural and sustainable structural and geotechnical design.

1.2 Scope of the Project

The scope of the project is to carry out major civil engineering discipline designs including architectural, structural, geotechnical and construction management of the building. By analyzing different international and local standards, performing literature review, conceptual design has to be developed. In the meanwhile, theoretical approaches, procedures and conceptual estimations will be determined and calculated. For the Capstone II, more accurate calculations and thorough design analysis will be provided.

1.3 Members of the Team

Milky Way Construction Company team consists of 5 members and each of them have own responsibilities:

1. Zhanbolat Kulzhabek – Chief engineer and is responsible for structural analysis and technical drawings.
2. Alisher Suleimen – Structural engineer and is responsible for structural analysis.
3. Ardana Aldonggarova – Architectural engineer and is responsible for architectural design.
4. Aidar Kenzhebek – Geotechnical engineer and is responsible for geotechnical design.
5. Aidana Agibayeva – Project manager and is responsible for project management.

2 Architectural Design

2.1 Design Statement

2.1.1 Main Function of the Building

The building is planned to be the part of the Nazarbayev University campus (L1 Block) and serve as academic building. It consists of the main building and 10-storey tower, which are connected with each other on a ground floor. There are four lecture halls in the main building with the cumulative capacity of 1500 people that can be rent by local organizations to held conferences. Although the main block possesses the ground floor only, it has two storey height. Thus, the tower rising up from the main block counts its floors starting from the third floor. The first 5 floors of the tower are designed to allocate 4 computer labs and 15 classrooms of different sizes, while the rooms for staff with the overall capacity up to 200 people are located on the highest 3 floors. According to the CNaR RK 2.02-05-2009, the functional classification is F4.2 (buildings of educational institutions of higher education) building (Section 6).

2.1.2 Site Location

Two suitable locations were considered under the construction of 10 storey academic building in Nazarbayev University campus (see Figure 2.1.1). The given area of the site is 100m x 150m (15,000m^2). Both sites meet CN of RK 3.01-01-2013 having access to main highways of the city – Kabanbay Batyr and Turan avenues (Section 6.2). The territory also allows creating free access for emergency services, and easy installation of infrastructure networks. Although, both sites are suitable for construction, the majority of the academic buildings of NU is located closer to Kabanbay Batyr avenue, while dormitories are located closer to Turan avenue. Therefore, in order to ensure the consistency between NU buildings the site illustrated in Figure 2.1. (a) will be used in the project and in geotechnical analyses.

(a) (b)

Figure 2.1. Site on the (a) north-east part of NU campus, adjacent to Kabanbay Batyr ave. and (b) north-west of NU campus, adjacent to adjacent to Turan ave.

2.1.3 Preliminary Design

There are several designs that were considered during the brainstorming stage. All of them considered an availability to allocate lecture halls, to reach the height requirements set by client as well as to achieve aesthetically pleasing view. Three alternative designs are represented in Figure 2.2.

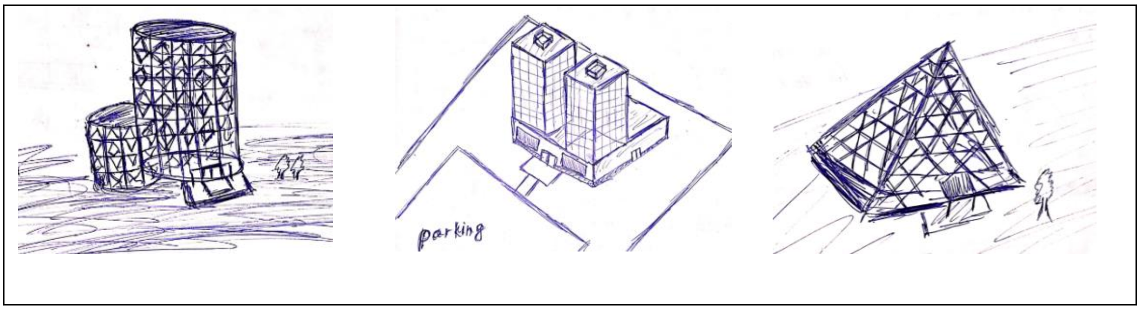


Figure 2.2. Preliminary design alternatives

It is widely known that the rectangular shape is easier and faster to construct than amorphous shapes that require detailed structural engineering and additional testing for their response to different natural forces. Moreover, Steadman (2006) insists that the rectangular shape of the building provides economically beneficial design. Therefore, it was decided to design both the main block and the tower in rectangular shape. The final preliminary design of the L1 Block is given in Figure 2.3 below.

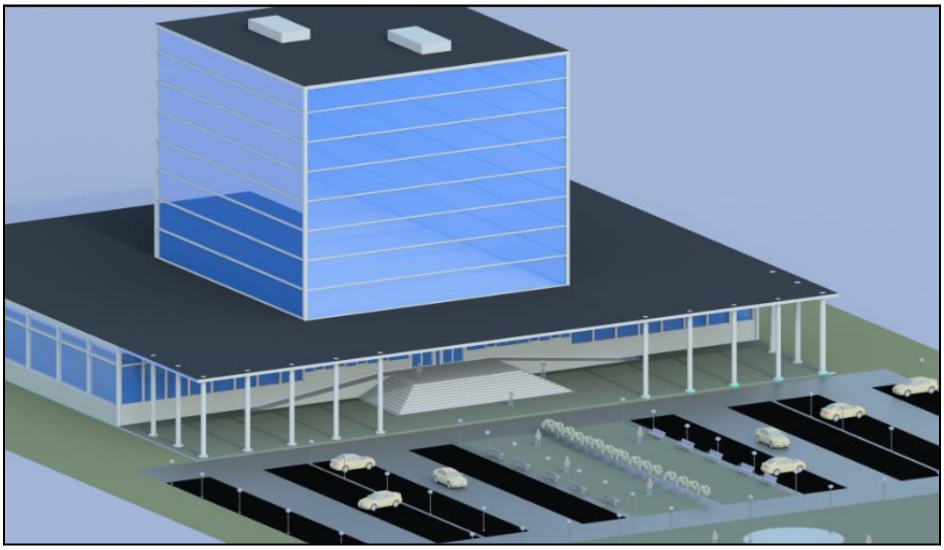


Figure 2.3. Final preliminary design of the building (L1 Block)

Preliminary design considered the concrete shear core as a primary structural element for the tower, since it provides relatively large column-free space. Consequently, it calls for the main building services (staircases and elevators) located inside the shear core. Moreover, the curtain wall was considered in order to ensure more access of the daylight. The preliminary layout of the tower can be seen from Figure 2.4.

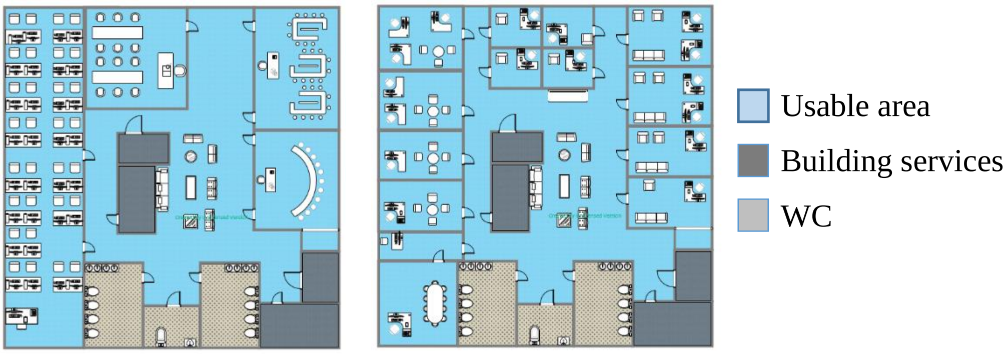


Figure 2.4. The preliminary layout of the tower (students and administrative floors respectively)

However, the group had to refuse the design of the concrete shear core because of the high cost of the construction technologies used and the increased construction duration. The curtain wall was also rejected due to the condensation related problems (Goncalves, 2007). As a result, the design of the building was revised using the square column-grid with a 6 m span and the building services located on the edges of the floorplate. Moreover, the dimensions of the tower were increased by 10 m in both direction (from 30 x 30m^2 to 40 x 40m^2) in order to subsequently increase the total capacity. The curtain wall was replaced by the façade glazing layer placed on the concrete wall.

2.2 Height and Areas

2.2.1 The Building Height and Areas

In order to reveal whether there are any limitations with area and height the degree of fire resistance of the building need to be calculated. According to CNaR RK 2.02-05-2009 buildings with bearing and enclosing structures of natural or artificial stone materials, concrete or reinforced concrete with sheet or plate non-combustible materials is classified to have 1st degree of fire resistance (Appendix 2). Thus, according to the CNaR RK 3.02-02-2009 the buildings with 1st degree of fire resistance have no limitation with height. On the other hand, floor area between fire walls in the 10-16 storey building should be not more than 2500 m^2, which was considered in the planning of the main block.

2.2.2 Gross, Usable and Projected Areas

According to CNaR RK 3.02-02-2009 the total area of a public building is defined as the sum of the areas of all floors that should be measured within the internal surfaces of the outer walls (Appendix 3). Therefore, the gross area for the main block is $71 \times 60 = 4260$ m^2, while the gross area of each floor of the tower is $40 \times 40 = 1600$ m^2. The usable area of the public building is defined as the sum of the areas of all placed in it premises except for stairwells (19 m^2/floor), elevator shafts and ramps. Moreover, the projected area of the public buildings requires subtraction of the areas of corridors and vestibules as well (ibid.). Thus, the cumulative usable areas for the main block and the tower are 4104 m^2 and 11859 m^2 respectively. The total projected areas for the main block and tower are 2222 m^2 and 6973 m^2 (and 5363 m^2 for the floors with faculty members' room) accordingly.

The height of each individual floor was dictated by the CNaR RK 3.02-02-2009 that is 3 m resulting in 6 m and 30 m in height for the main block and tower respectively (Section 4.2). Therefore, since the CNaR RK 3.02-02-2009 applies to design of public buildings up to 50 m high, the use of this state standard is justified (Section 1).

The total area of the given site is 150 x 100 m^2. 71 x 48 m^2 of the total site area is occupied by the building and parking area takes 65 x 35 m^2 while the rest of the site is intended for planting of greenery.

2.3 Site planning

According to the technical regulations in "General Requirement for Fire Safety" the distance from the edge of the roadway to the wall of the public buildings and structures shall be not more than 10 m for the buildings higher than 28 m. Moreover, the public buildings should have an access throughout its length on both sides if the width of the building is more than 18 m (Section 3.2.1). Thus, the building itself is located on the far left corner of the site plan in order to provide easy access to at least two sides of the building for fire engines. The location of the building on the site plan is given in Figure 2.5 with the fire engines' route highlighted.

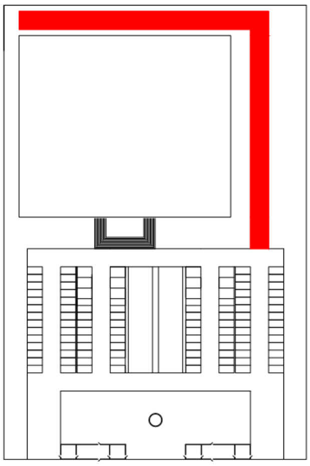


Figure 2.5. Fire engines' route

2.4 Main Facilities

2.4.1 Lecture Halls

All 4 lecture halls are located on the ground floor and on the corners of the main block in order to insure:

- Easy access to the full capacity number of people;
- Evacuation routes directly to the outside;
- Decreased number of elevators.

All lecture halls are equipped with the widescreen. Therefore, seats in the lecture halls are recommended to design within the visible area shown in Figure 2.6 (op.cit., Appendix 6). It is formed by the triangle that makes 45° with the normal to the screen. The optimal allocation of the seats with respect to the screen should be calculated as $L' = 0{,}36L$. Where, L - length of the hall through its axis from the screen to the backrest of the last row and L' - distance through axis of the lecture hall from the screen to the backrest of the first row (see Figure 2.4.1). It also demonstrates the screen dimensions where, W is the width of the working area of the screen and H is the height of the screen. While the ratio for the widescreen is H : W = 1 : 2,2. The screen width (W) depending on the length of the auditorium (L) is recommended to take W = 0,6L. Moreover, the distance from the screen to the backrest of the first row (L'), depending on the screen width (W) is recommended to take L' at least 0.6W (ibid).

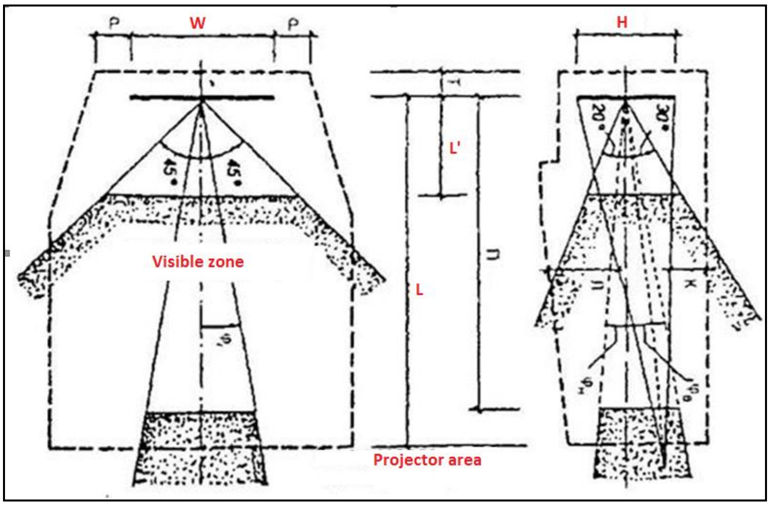


Figure 2.6. Lecture hall parameters

(op.cit., Appendix 6).

Moreover, according to the same CNaR RK 3.02-02-2009 and CNaR RK 2.02-05 2009 students' distribution from the seats to the outputs of the hall along the direction of their motion in emergency cases should be taken according to the scheme shown in Figure 2.7. (Appendix 7).

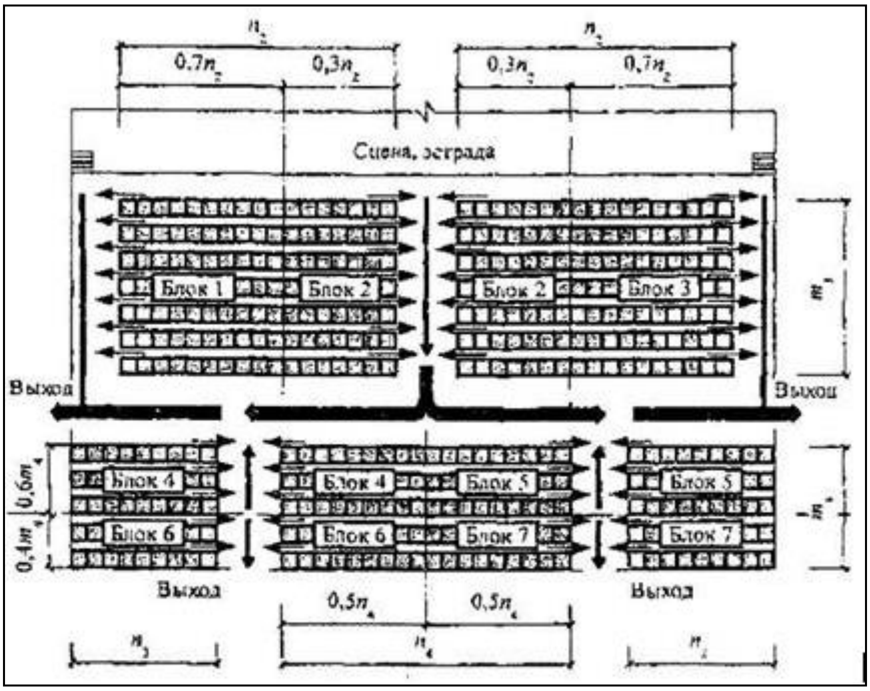


Figure 2.7. Evacuation routes

(CNaR RK 3.02-02-2009, Appendix 7.1).

Therefore, in order to satisfy both construction and fire safety norms, the seats in lecture halls are allocated within the visible area, but still keeping the evacuation routes available between seat blocks.

All lecture halls are designed to have sloped ceiling and equipped by self tipping lecture theatre seats as shown in Figure 2.8. In a room with a sloped ceiling the average height of the room must meet the height requirements of 3 m. Moreover, the room height in any part of it should be at least 2.5 m (op.cit., Section 4.5). Seat way, that is a minimum clear space between rows, has a minimum recommended value of 300 mm (The Black Book, 2016). Gangway, that is the access space internally on a tiered seating block, provides the means of distributing people on a seating area and for evacuation from the area and has a width of 2.3 m. Lateral gangway, the access space across the seating block, joints two or more vertical gangways and has a width of 1.8 m. The row depth and rise is 280 mm and 170 mm accordingly (ibid).

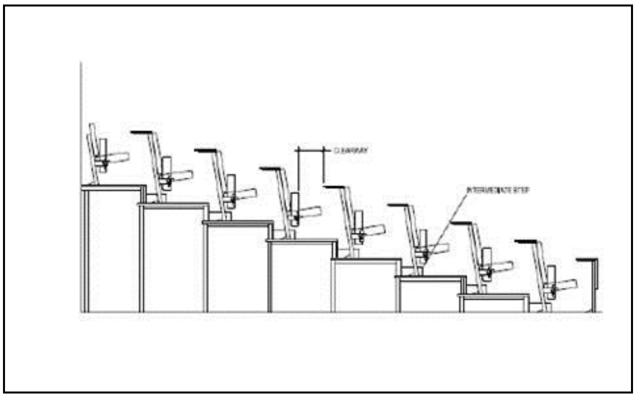


Figure 2.8. Theatre seating detail

(ibid.)

The maximum number of seats per row – 20. Maximum number of rows per seating block – 10. The final layout of the big lecture hall is shown in Figure 2.9. While the layout of the small hall is given in technical drawings of this document.

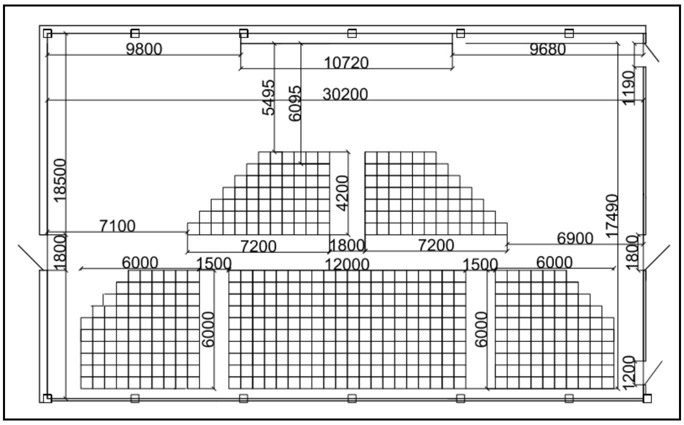


Figure 2.9. Lecture halls

To sum up, the parameters of the lecture halls are:

Table 2.1. Lecture hall parameters

Parameters	Big Lecture Hall	Small Lecture Hall
Length of the auditorium (L)	17.5 m	15.3 m
Distance from the screen to the backrest of the first row (L')	5.5 m	5.1 m
Width of the screen (W)	10.8 m	9.5 m
Height of the screen (H)	4.9 m	4.3 m
Capacity	506 people	255 people
Area	562.4 m^2	282.5 m^2
Volume	3374.4 m^3	1695 m^3

Area of the lecture halls (capacity over 150 people) should be calculated to provide at least 1.1 m^2 per person (CNaR RK 3.02-02-2009, Section 5.19). While the volume of the lecture halls should be calculated to provide at least 4-5m^3 (CNaR RK 3.02-02-2009, Section 5.25). Thus, it can be seen that the area and volume requirements are met.

2.4.2 Computer Labs

There are 4 computer labs located at the lowest 4 floors of the high rise tower. According to CNaR RK 3.02-02-2009 the computer labs should be designed to provide 6.0 m^2 per one place in front of the display (Section 5.8). Since, the computer laboratory classes already located in SEng of NU possess an approximately 35 computers each, the area of the computer labs should be not less than $6\times35=210$ m^2. The individual area of computer labs is 217 m^2.

2.4.3 Floors for Staff

In order to avoid unnecessary flow of the people between the floors, it is feasible to allocate the floors according to the number of indwelling people on each floor (in descending order). Therefore, the highest 3 floors are intended for allocation of faculty members, administration and operating personnel.

Faculty Rooms

Before the start of the faculty room design, the approximate number of faculty members should be calculated. Undoubtedly, the number of faculty members depends on the number of students studying. According to the report of the committee on statistics of Ministry of National Economy of the Republic of Kazakhstan, there are 51235 students and 4745 faculty members in Astana for 2015/2016 academic year resulting in proportion of $\frac{4745}{51235} = 0.093$. Moreover, since the area of the tower is approximately twice as much bigger than the area of SEng of NU, it was assumed that there are twice more students graduating each year from a new academic building (i.e. ≈ 400 students). Therefore, it is expected to have:

$$0.093\frac{faculty\ member}{student} \times 400\ students\ \times 4\ years\ of\ bachelor\ \approx 149\ faculty\ members$$

.

The design of faculty room should be calculated in order to provide at least 6.0 m^2 per person + office equipment (op.cit., Section 5.8). Therefore, it is required to provide at least 149 x 6 = 894 m^2 of the office area for the faculty members.

Administration

The number of the administration members should be roughly estimated to calculate the total area required. Therefore, the Government Regulation (№77, 2008) on the typical staffs in organizations of higher education was used. Then, the total area required is calculated using the recommended design space standards found in sanitary requirements №1431 (2011). Table 2.2 represents the extracted information and for administration members only.

Table 2.2. Administration members and required areas in organizations of higher education

Position	Number of staffs depending on number of students (for 400 students)	Area per employee, m^2	Total area, m^2
Office director	1	9.0	9.0
Deputy director	4	7.5	30
HR department director	1	9.0	9.0
HR inspector	1	6.5	6.5
Secretary	2	5.0	10
		Total Area:	64.5 m^2

Operational personnel

The total number of operational personnel is calculated from the typical cross-industry standards for the number of employees engaged in service for administrative and public buildings (ibid). First of all, the percentage of the area occupied by the equipment and furniture should be found. For organizations of higher education, about 50% of the total area is usually occupied by the equipment and/or furniture. The Table 2.3 represents the calculation of the operational personnel.

Table 2.3. Calculation of the employees engaged in service

1 cleaner per area	
Premises	400 m^2
Conference and lecture halls	770 m^2
Halls and corridors	960 m^2
Stairwells	730 m^2

Thus, there are

$$\left(\frac{A_{p,sf}}{400}+\frac{A_{p,af}}{400}+\frac{\sum A_h}{770}+\frac{\sum A_c}{960}+\frac{A_s}{730}\right)=\left(\frac{6973}{400}+\frac{5363}{400}+\frac{562.4\times 2+282.5\times 2}{770}+\frac{611\times 5+812\times 3}{960}+\frac{19\times 10\ floors}{730}\right)\approx 39$$

cleaners.

Where, $A_{p,\,sf}$ – Projected area of the students' floor

$A_{p,\,af}$ – Projected area of the administrative floor

A_h – Area of the big/small halls

A_c – Area of the corridors

A_s – Area of the stairwell

Moreover, 1 elevator operator, 4 workers engaged in maintenance and repair of the engineering equipment of the building and 4 employees engaged in repair of structural elements of the building (MLSP RK № 401-o, Section 3). As a result, there are 48 of operational personnel. According to the Sanitary Requirement №1431, there should be 2 m^2 per operational personnel: $48 \times 2 = 96$ m^2.

To sum up, the cumulative usable area for the administrative floor is 1471 m^2. Previous required area calculation of the faculty rooms, operational personnel and administration members ensures that there is enough space for their allocation on the highest 3 floors: 894 + 64.5 + 96 = 1055 m^2 (< 1471 m^2). Moreover, it insures enough space in case of the students' number increase and consequent increase of the faculty members' number.

2.5 Other Facilities

2.5.1 Wardrobe

An outerwear wardrobe capacity at the lobby of the main building is calculated to provide hooks for 80% of the total capacity of the halls and 15% of the total student's number. Each place in a wardrobe requires area of 0.1 m^2 (Sanitary Requirement №1431, 2011). That results in $(1500 \times 80\% + 400 \times 15\%) \times 0.1 = 126$ m^2. Besides, there are 3 cloakroom attendants required (MLSP RK № 401-o, Section 3).

2.5.2 Cleaning Equipment Room

The area of the cleaning equipment room is calculated as 0.8 m^2 per 100 m^2 of the floor area. That is $A_{CER} = \frac{A_u}{100} \times 0.8$, where A_U = usable area per floor. Therefore, there should be $\frac{4104.25}{100} \times 0.8 = 32.8$ m^2 for the main building and $\frac{1471}{100} \times 0.8 = 12$ m^2 for the tower (per floor) intended for cleaning equipment room.

2.5.3 Water Closets

Sanitary facilities should be provided separately for the staff, faculty and students (visitors). Since, the floors have been already divided according to the functional responsibilities (student or faculty member), there are no need to design separate water closets. To calculate the number sanitary appliances, the ratio of men and women should be indicated in the technological part of the design assignment. According to the statistics provided by NU, there are 155 females versus 175 male students admitted to the bachelor program for 2012-2016 (8). Thus, the proportion is 0.88. Therefore, it was assumed that there will be 185 females and 215 male students studying in the new academic building.

Calculated load on one sanitary unit is assigned depending on the type of public building. For the academic building: males - one toilet per 20 – 50 people (the floor capacity of 150 – 200 people), 1 pissoir per 18 people (the floor capacity up to 150 people), 1 washbasin per 4 toilet (but not less than 1 per WC); females - one toilet per 15 – 30 people (the floor capacity of 75 – 100 people), 1 washbasin per 2 toil (but not less than 1 per WC) (CNaR RK 3.02-02-2009, Section 5.32). Thus, using the approximate students' number the number of sanitary units is calculated.

Table 2.4. The calculated number of sanitary units for the main building

	Male	Female
Toilet	16	24
Washbasin	6	12
Pissoir (urinals)	20	-

2.6 Building Services

2.6.1 Stairwells

The main objective of the stairwells design stage is the choose of economical and comfortable slope of the stairwell, that also fits local construction codes and norms. The CNaR RK 3.02-02-2009 is used in design of the stairwell. According to above mentioned code, the number of stairs in one march on the half-pace stairwells should not exceed 16 stairs, while the slope of the stairwell intended for people evacuation should not take more that 1:2 (Sections 4.91, 4.95). The width of the stairs in public places should not be less than the exit width to the staircase, but not less than 1.35 m for the buildings with maximum indwelling people more than 200 people (CNaR RK 3.02-02-2009, Sections 4.91, 4.95 and 4.97). Moreover, the width of the steps should be at least 300 mm, while its height should not exceed 150 mm (CNaR RK 3.02-02-2009, Section 7.10). As it can be seen from the Figure 2.10, the chosen steps' sizes correspond to the 26°40' that lies within the region of comfortable angle. Therefore, for the new academic building it is needed to have $\frac{3\ m}{0.15\ m} = 20$ steps. Thus, it is designed to have two half-pace stairwells each having 10 steps.

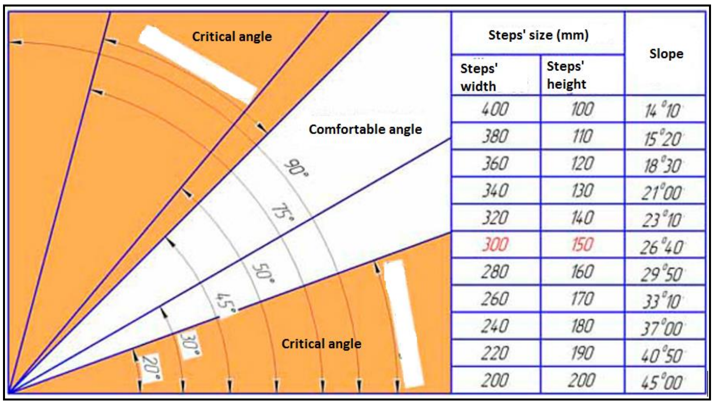


Figure 2.10. Steps' sizes and slope calculation

(CNaR RK 3.02-02-2009, Appendix 3.2)

One of the two staircases should be smoke tight of N1 type. And the distance between two staircases should not be less than 2.5 m (CNaR RK 3.02-02-2009, Section 4.138). Therefore, the designed staircases are to be placed on two corners of the tower floorplate that also provide enough daylight to the staircases.

2.6.2 Elevators

Miller (2012) claims that one elevator should be provided for every 4180.6 m^2 of net usable area. Although, this results in minimum 3 elevators for 11859 m^2 of net usable area of the tower, 4 elevators are going to be placed in order to be conservative. Moreover, KONE Elevator Traffic Calculation tool gives that the actual number of required elevators is 4. The assumption that the building type is office having single tenant with fixed working hours fairly fits the academic building type. See Figure 2.11.

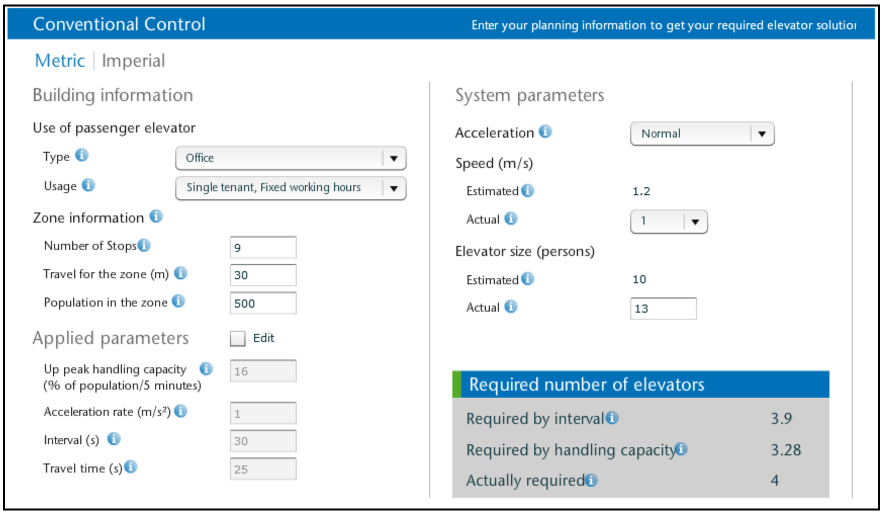


Figure 2.11. Calculations on number of elevators

(Kone, 2016)

The number of passenger elevators should be set by calculation, but is usually not less than two. It is allowed to replace the second elevator by the service lift if the transportation of people is permitted there. One of the elevators in the building must have a minimum depth of the cabin 2100 mm in order to transport a human on a stretcher. (CNaR RK 3.02-02-2009, Section 4.141).

Moreover, at least one elevator should be equipped with fire service mode for the buildings with 10 – 16 floors. The elevators should be placed so that the distance from the door of the most remote room to the door of the nearest passenger lift does not exceed 60 m. Besides, access to the elevators should be provided through lift lobby. The width of the lift lobby should be not less than 1.3 of the shortest depth of the elevator's cabin, but at least 2.5 m (CNaR RK 3.02-02-2009, Sections 4.143 – 4.145). Therefore, two hoist ways are planned to be on the two corners of the tower floorplate near with the staircases. One service and one passenger lifts will be located in each hoist way.

2.6.3 Garbage Disposal and Dust Cleaning

In public buildings the garbage disposal and dust cleaning system, as well as the temporary (within sanitary norms) waste storage and the possibility of its removal should be considered. In order to avoid unpleasant smell of the garbage, the use of refuse chutes in academic building was declined. Instead, the chamber collecting area with a paved surface is provided outside the building (CNaR RK 3.02-02-2009, Section 4.149).

Due to the high visitor capacity of the building the content of fiber fragments from clothing and footwear, as well as particles of soot and fumes that come from the street on the soles of the footwear is very high. This kind of dust can eat any organic material and cannot be removed without professional cleaner. According to CNaR RK 3.02-02-2009, the public buildings equipped with certain facilities including lecture halls should be provided by the centralized or combined vacuum dust cleaning system (Section 4.153). Therefore, the vacuum duct cleaning system is provided in a main building, while the duct cleaning in the tower is done manually. The system consists of pipelines laid in the walls, inlets, resembling electrical outlets, and the power unit (pump) with a filtration system placed in a separate room. To vacuum, a hose with the length of 6 meters should be plugged in wall inlet. The power unit (pump) is automatically switched on. The dust entered the power unit through the brush, hose, wall inlet and pipe is cleaned and removed outside the building (Blizzard Lufttechnik, 2016). While designing the combined vacuum dust cleaning system the service radius of one valve should not exceed 50 m (Section 4.145). As a result, Blizzard Lufttechnik centralized dust cleaning system will be installed in a main building having 4 valves in each lecture halls.

2.7 Lightening

The level of natural and artificial lighting facilities in public buildings must meet the requirements of CNaR RK 2.04-05-2002. According to the above mentioned norms, the natural light access should be provided to all spaces intended for human occupancy

(Section 5.2.1). However, the lecture halls as well as all water closets can be designed with only artificial light (CNaR RK 3.02-02-2009, Section 3.41).

Artificial lighting is subdivided into operating, emergency and security lightning. Operational lighting should be provided for all areas of buildings, as well as areas of open space, designed for work and people passage. The fluorescent lamps are going to be used in all parts of the building providing the lowest illumination of horizontal surface of 100 lux (CNaR RK 2.04-05-2002, Sections 6.2.1 and 6.2.5).

Emergency lighting will be provided in the event of power failure of the operational light and is connected to an independent power supply. Evacuation lightening of the large areas (anti-panic lighting) is provided in lecture halls, and is intended to prevent panic and to ensure conditions for a safe approach to the escape routes. The illumination is 0.5 lux on the whole free floor area. The duration of the evacuation lighting is 1 hour. Moreover, 50% of the rated illumination is provided within 5 seconds after the failure of the operational lighting, and 100% within 10 seconds (CNaR RK 2.04-05-2002, Section 7.5.5).

Security lightening will be provided along the boundaries of the site at night. The illumination is 0.5 lux at ground level using LED light sources (CNaR RK 2.04-05-2002, Section 7.6.1).

Moreover, exterior architectural lighting will be provided in order to enhance the evening visibility and expressiveness of the most important features of the building and to improve the comfort of the light environment of the city (CNaR RK 2.04-05-2002, Section 7.4.1).

2.8 HVAC System

Heating, ventilation and air conditioning systems of the building should meet the requirements of security, reliability, serviceability, taking into account aspects of efficiency and durability, to avoid incurring unacceptable risk of harm to the environment and human health and life.

2.9 Parking

In order to calculate the total number of parking places required Norms on calculation the number of parking spaces for different types of public buildings №444 is used (2007). The number of provided parking places for every 100 faculty members and

100 students are 20 and 5 respectively. Thus, for assumed number of faculty members and students (149 and 1600 accordingly) there are $149 \times 0.2 + 1600 \times 0.05 = 109.8 \approx 110$ parking places. Besides, 5 parking places and additional 3% of the total number of parking places, results in $5 + 0.03 \times 110 = 8.3 \approx 9$ parking places for disabled people. Thus, the parking area includes 100 parking places + 10 parking places for disabled people. ICN 2.02-05-2000 claims that the parking place dimensions should be 2.5 m x 5 m and 3.5 m x 5 m for disabled people parking place. The final preliminary layout of the parking zone is given in Figure 2.12 below.

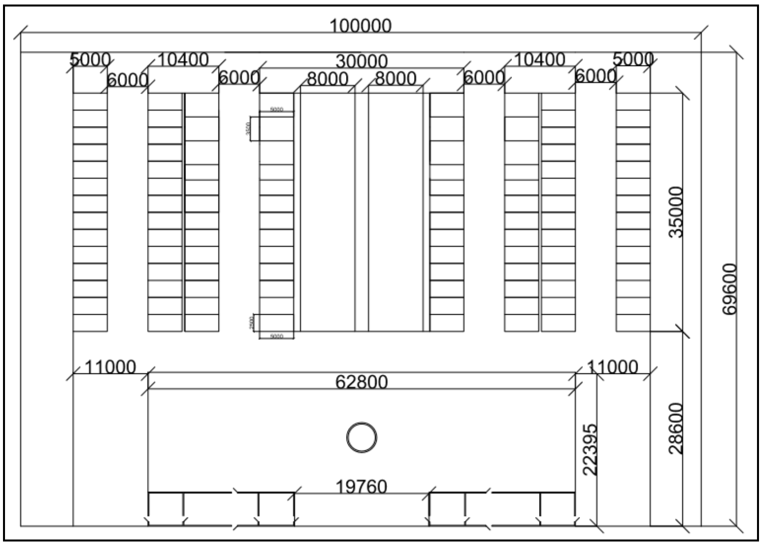


Figure 2.12. Parking zone

2.10 Non-Structural Materials

2.10.1 Flooring Design

The design of the flooring is accomplished in full compliance with CNaR RK 3.02-03-2003. There are several layers of the flooring. The name and their functions are included in Appendix B.

Selection of the flooring's constructive solutions is carried out on the basis of technical and economic feasibility of the decision to ensure the reliability and durability of the adopted structure as well as an economical use of cement, metal, wood and other building materials. Moreover, the influence of harmful particles in the materials applied in construction floors and the optimum hygienic conditions are important factors in flooring design.

Covering

According to CNaR RK 3.02-03-2003 flooring of the auditoriums, classrooms, laboratories, teaching rooms and lecture halls should be covered by parquet, laminate, linoleum or carpet covering. Parquet is an environmentally friendly, progressive facing material for the floor and other surfaces that require high wear resistance and good sound and heat insulation. It has also high water resistance and meets the strength, heat, acoustic and other performance requirements of floorings (Section 2). Thus, parquet was chosen for the floor covering.

Interlayer

According to the CNaR RK 3.02-03-2003 flooring interlayer material type and thickness is selected according to the functional class of the building. Since the academic building is of class F4.2, the cement-sand mortar interlayer of 10-15 mm thickness was selected (Section 3).

Screed

Screed is needed to level the underlying layer, hide pipelines and to distribute the loads on insulation layers. 40 mm thick concrete screed will be placed (CNaR RK 3.02-03-2003, Section 4.2).

Thermal and Sound Isolation

For the floor covering made of parquet, thermal and sound isolation should be considered. Rubble and sand from expanded perlite is used as the thermal and sound isolation of 40 mm in thickness (CNaR RK 3.02-03-2003, Section 6).

The final flooring design with the material selected, their thickness adjusted and the volumetric masses can be seen from Table 2.5. (CNaR RK 3.02-03-2003).

Table 2.5. Flooring

#	Layer	Material	Thickness, mm	Volumetric Mass, kg/m^3
1	Covering	Parquet	25	880
2	Interlayer	Cement-sand mortar interlayer	10-15	1800
3	Screed	Lightweight concrete	40	800
4	Isolation	Rubble and sand from expanded perlite	40	200
5	Slab	Reinforced concrete	150	2500

2.10.2 Ceiling

The ceiling material is selected according to the functional classification of the building using CNaR RK 2.04-10-2004. Thus, for F4.2 class building is recommended to install Armstrong (acoustic) ceilings. Armstrong ceiling consists of the special metal profiles and mineral plates mounted on the suspension system. They have several advantages among others on the market:

- Easy for installation and removal for a quick contamination wash off;
- Space to hide ventilation and air conditioning systems;
- Acoustic isolation;
- Safe and eco-friendly.

The ceiling thickness is 2.5 cm having a volumetric weight of 200 kg/m^3.

2.10.3 Exterior

Laminated glass for building is going to be used to cover the façade of the building. The glass should be in compliance with international standard and meet technical specifications (2003). The laminated glass can be bulletproof, flame retardant, explosion-proof, noise protected and frost resistant. Considering the climatic conditions of Astana, the frost resistant laminated glass of 20 mm thickness with the volumetric weight 2225 kg/m^3.

2.10.4 Partition

Since the lower self-weight of the materials used is desired in order to decrease overall dead load applied on the structures, the gypsum plasterboards connected with aluminum studs are used. The gypsum plasterboards of thickness 20 mm are used with volumetric weight of 8 kg/m^3 (CNaR RK II-22-81).

2.11 Roof

To perform functions of the roof several layers are provided. They are protection layer that protects the waterproofing layer from mechanical damage and external influences (solar radiation, wind), waterproofing layer – protects the roof structure from atmospheric moisture, ventilation space (ventilated gap, ventilated attic) – provides the appropriate temperature and humidity conditions for work of the thermal insulation and thermal insulation that provides an appropriate thermal mode of the building.

To remove rainwater and meltwater from the roof external drainage system organized to meet the requirements of CN RK 3.02-37-2013. According to the mentioned standard the angle of the roof depends not only on its design and features of the building façade and roofing material, but also on other factors. It is necessary to take into account the climatic conditions of the Astana. Since strong winds dominate in Astana, it is advised to construct parapets along he perimeters of the building . Most often, the range of inclination in this case – 9 - 20 degrees (CN RK 3.02-37-2013).

2.12 CAD standards

The CAD drawing standards mentioned in the A/E/C CAD standard provides presentation graphics, level/layer assignments, electronic file naming, and standard symbology. It was done to develop a CAD standard that is that is general enough to function under various CAD software applications (Autodesk's AutoCAD) and work with existing industry standards when possible.

2.12.1 File accuracy

CAD software provides real world dimensions which allow to the engineer to deal with it. Meters and millimeters are one of the international SI units which are used in the CAD system.

2.12.2 Origin

Every electronic file has its origin, from where all drawings area started. The origin of a sketch document is important the reason is that it serves as the point of orientation from which all other elements are positioned. Origins are usually defined in a drawing file by the Cartesian coordinate system of x, y, and z.

2.12.3 Model Files and Sheet Files

CAD has two types of files, which are model files and sheet files. Model files presents physical elements of the construction; for example, walls, window and etc. Model files are represented at full scale and mainly provides plans, elevation and etc. a sheet file is a drawing within a border sheet. In addition, it contains information as text, dimensions and symbols. In other words, sheet file is ready to plot CAD document.

3 Structural Design

3.1 Design load calculation

Before proceeding with the design of structural members, the design loads are needed to be calculated first. Design loads are the most significant and challenging part of any construction. The following part of the report discusses the calculation procedures of all forces acting on the building following Eurocode 2. This includes dead, live, wind and snow loads.

3.1.1 Dead load

The dead load, also called the structural load, includes loads that are relatively constant over time, including the heaviness of the structure itself and immovable fixtures such as walls, plaster board, carpet, floors, stairways, fixed service equipment and elevators.

Table 3.1. Summary of dead load calculations

Name of Element	**Quantity**	**Area (m^2)**	**Volume (m^3)**	**Unit Weight (kN/m^3)**	**Weight of 1 item**	**Weight on each floor (kN)**
Columns (Floors 7-9)	64	0.09	0.27	25	6.75	432
Columns (Floors 4-6)	64	0.2025	0.6075	25	15.19	972.16
Columns (Floors 2-3)	64	0.3025	0.9075	25	22.69	1452.16
Columns (1st floor)	130	0.3025	1.815	25	45.38	5899.4
Columns (hoistway & stairway)	20	0.08	0.24	25	6	120
Stairway	2	-	2.52	23	57.96	115.92
Slabs	1	1534	230	25	5750	5750
Beams (Floors 5-9)	64 48	0.15	0.825 0.9	25	20.625 22.5	1320 1080
Beams (4th floor)	32 32 48	0.15 0.18 0.15	0.825 0.99 0.9	25	20.625 24.75 22.5	660 792 1080
Beams (Floors 1-3)	64 48	0.18	0.99 1.08	25	24.75 27	1584 1296
Beams (Atrium)	34	0.18	0.99	25	24.75	841.5
Beams (lecture halls)	27 14	0.4	7.628 6.034	25	190.7 150.85	5148.9 2111
Minor	28	0.08	0.44	25	11	308

beams	21		0.48		12	252
Glass (Floors 2-9)	1	480		0.012	5.76	5.76
Glass (1st floor)	1	784.5		0.012	9.414	9.414
Total Weight of Typical Floor (7-9)						**9383.68**
Total Weight of Typical Floor (5-6)						**9923.84**
Total Weight of Typical Floor (4)						**10055.84**
Total Weight of Typical Floor (2-3)						**10883.84**
Total Weight of First Floor						**23436.134**

3.1.2 Live load

Live load, or imposed load, is temporary, of short period, or a moving weight. These active loads consider impact, momentum, vibration, dynamics of fluids and material fatigue. Table 3.2 below provide information how dose different areas of use effect on the structure. Live load does not include environmental loads as wind load and snow loads.

Table 3.2 Minimum uniform distributed live loads

Occupancy or use	Normative values of load (kPa)
Premises of administrative personnel and the faculty; classrooms; utility rooms (dressing rooms, showers, washrooms and toilets)	2
Halls	4
Lobbies on the first floor	4
Open space	1.5
Corridors above the first floor	3
Ladders	3

3.1.3 Wind Load

The wind load estimation is one of the main parts for the project, as Astana city has windy climate because of which, buildings are prone to horizontal loadings. For the calculation of the wind loadings the Eurocode was used as a basis in this report.

First of all, the basic wind velocity for the local area has to be calculated with the following formula:

$$v_b = c_{dir} * c_{season} * v_{b,0} \tag{3.1}$$

Where, v_b – Basic wind velocity

$v_{b,0}$ – fundamental value of basic wind velocity

c_{season} – seasonal factor

c_{dir} – directional factor

According to the Eurocode, the recommended value for both, directional factor and seasonal factor, is 1. The fundamental value for the basic wind velocity, $v_{b,0}$, is the velocity of wind for the local area with the return period of 50 years. However, in the technical report for the Astana city, there are only wind velocities with return periods of 5, 10 and 100 years (Ospanova, 2015). Therefore, by using the data from table 3.3, the value for the $v_{b,0}$ was interpolated with the trend line in Excel program.

Table 3.3 Wind velocities with return periods

Return period (years)	Wind velocity (m/s)
5	31
10	33
100	40

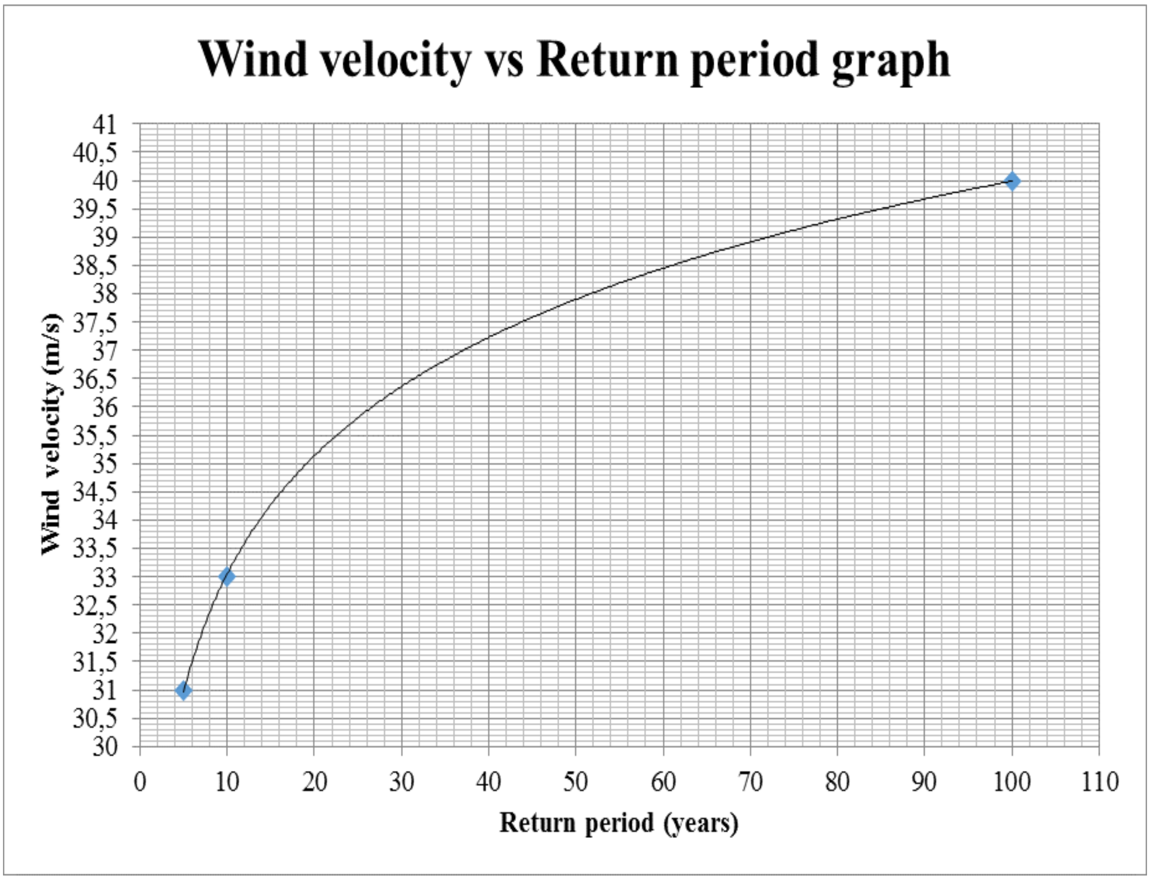


Figure 3.1 Wind velocity versus return period graph for interpolation

From figure 3.1 it can be seen that for return period of 50 years, the value of wind velocity is approximately 37.8 m/s. The basic wind velocity will have the same value in this case.

The next step is the calculation of the mean wind velocity with the help of the following formula:

$$v_m(z) = c_r(z) * c_o(z) * v_b \tag{3.2}$$

Where, v_m – Mean wind velocity

z – Height above the ground

c_r – Roughness factor

c_o – Orography factor

Here, v_b is taken from equation (3.1) and $c_o(z)$ is recommended to be 1 according to Eurocode. The terrain roughness, $c_r(z)$ is obtained from following formulas:

$$c_r(z) = k_r * \ln\left(\frac{z}{z_o}\right) \quad \text{for} \quad z_{min} \leq z \leq z_{max} \tag{3.3}$$

$$c_r(z) = c_r(z_{min}) \qquad \text{for} \qquad z \leq z_{min} \tag{3.4}$$

Where, k_r – Terrain factor

z_0 – Roughness length

z_{max} – Maximum height

z_{min} – Minimum height

Terrain factor, k_r, is calculated using

$$k_r = 0.19 * \left(\frac{z}{z_{o,II}}\right)^{0.07} \tag{3.5}$$

According to Eurocode z_{max} = 200m, $z_{o,II}$ = 0.05 and values for z_o and z_{min} are obtained from table 3.4.

Table 3.4 Roughness lengths and minimum heights for each terrain category.

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

Our construction site can be relegated to the second (II) category, where z_o = 0.05m and z_{min} = 2m. So by putting these numbers and the elevation value the mean wind velocity can be obtained for some specific height.

After that, the turbulence intensity of the wind has to be calculated with this formula:

$$I_v(Z) = \frac{\sigma_v}{V_m(Z)} = \frac{k_I}{C_o(Z) * \ln(Z/Z_o)} \quad \text{for} \quad z_{min} \leq z \leq z_{max} \tag{3.6}$$

$$I_v(Z) = I_v(z_{min}) \quad \text{for} \quad z \leq z_{min} \tag{3.7}$$

Where, I_v – Turbulence intensity,

σ_v is obtained by using

$$\sigma_v = k_r * V_b * k_I \tag{3.8}$$

The recommended value for k_I is 1 according to the Eurocode. After finding the turbulence intensity, the peak velocity pressure by the wind can be calculated with the formula given below:

$$q_p(z) = [1 + 7 * I_v(z)] * 0.5 * \rho * v_m^2(z) \tag{3.9}$$

The air density value (ρ) is recommended to be 1.25 kg/m^3.

After finding the velocity pressure of the wind, the external pressure coefficients have to be found, in order to calculate the load distribution by the wind. The external pressure coefficients are given in the table 3.5.

Table 3.5 External pressure coefficients for the building.

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

External pressure coefficients are different for each zone. In our situation, we are interested in windward and leeward side of the building, which, according to Eurocode, correspond to zones D and E respectively. As the loaded area of the structure is larger than 10 m^2, $c_{pe,10}$ coefficients will be used in this project.

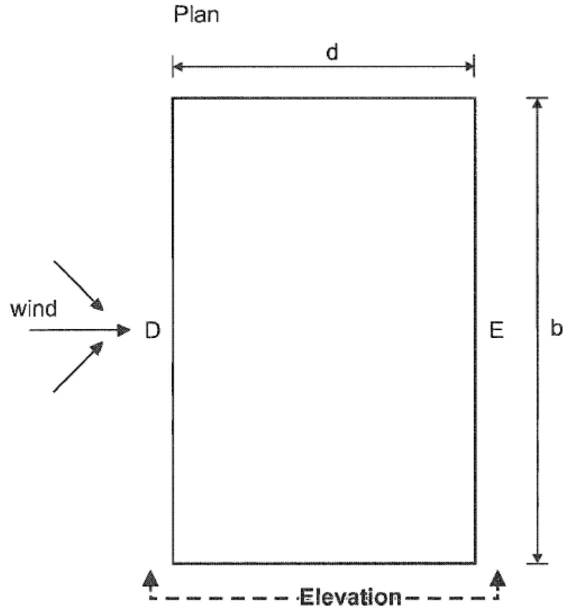


Figure 3.2 Plan view of the structure exposed to the wind.

According to Eurocode in cases where it is not considered justified, to estimate the internal pressure, the more onerous value of -0.3 or +0.2 is taken as C_{pi}. In our case, C_{pi}=0.2.

With the help of the pressure coefficients, the wind pressure on the building can be obtained with the following formula:

$$w = q_p(z) * c_{p,net} \quad (3.10)$$

Where $c_{p,net} = c_{pe}$-c_{pi}, and by putting the necessary values we can find the wind pressures acting on different sides of the building.

In order to obtain the load distribution of the wind, the wind pressure has to be multiplied to the value of the spacing between the columns. For this project, the critical values of spacing will be used in order to ensure that the safety of the building will be increased by considering most severe cases of loadings. The largest spacing between columns for this building is 6m.

By making the relevant calculations the loadings at each level of the building were found as shown in table 3.6.

Table 3.6 Wind loads at each stage of the building

Cr(z)	**Vm(z)**	**Iv(z)**	**Qp(z)**	**Floor**	**Altitude**	**h/d**	**Pressure coefficient**	**Wind pressure (kN/m2)**	**Load distr (kN/m)**	**Load (kN)**
1.048	39.626	0.181	2.227	Tower floor 2	12.45	0.31	0.708	1.577	9.460	32.448
1.093	41.315	0.174	2.365	Tower floor 3	15.75	0.39	0.719	1.701	10.205	34.763
1.129	42.681	0.168	2.480	Tower floor 4	19.05	0.48	0.730	1.811	10.863	36.839
1.159	43.829	0.164	2.578	Tower floor 5	22.35	0.56	0.741	1.911	11.463	38.749
1.186	44.818	0.160	2.664	Tower floor 6	25.65	0.64	0.752	2.003	12.021	40.537
1.209	45.687	0.157	2.740	Tower floor 7	28.95	0.72	0.763	2.091	12.547	42.232
1.229	46.462	0.155	2.809	Tower floor 8	32.25	0.81	0.774	2.175	13.048	43.854
1.248	47.162	0.152	2.872	Tower	35.55	0.89	0.785	2.255	13.530	22.325

				floor 9						
0.990	37.415	0.192	2.051	Ground floor	9.15	0.1	0.700	1.435	8.612	41.319

Apart from the horizontal loadings, wind also causes the vertical loads on the roofs of tower and the ground floor. Both of the roofs are considered to be flat as the inclination angle is less than 5°. Eurocode divides the roofs into zones, which will have different values for loadings. The zones can be seen in figure 3.3.

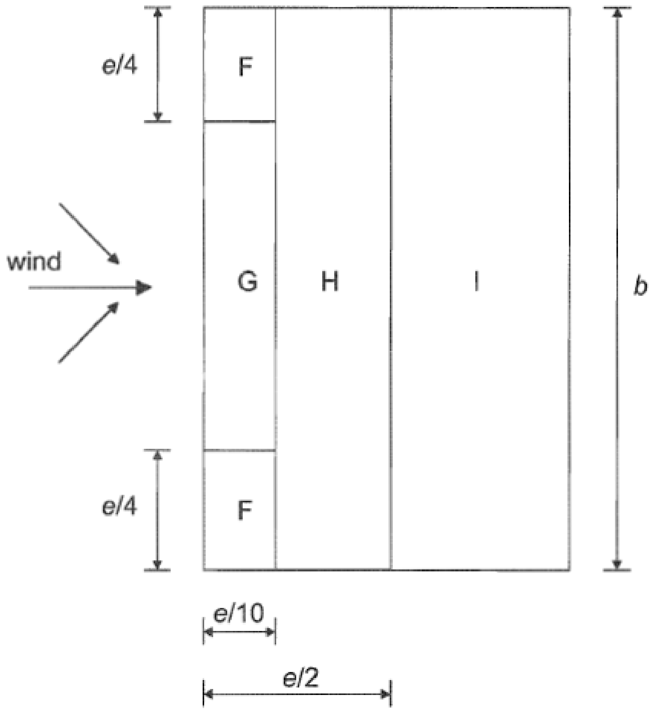


Figure 3.3 Wind pressure zoning of the flat roof

Where e is the smaller value between b or 2h, and in this case it equals 40m. The pressure coefficients are provided in table 3.7.

Table 3.7 Pressure coefficients for flat roof

Roof type		Zone							
		F		G		H		I	
		$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
Sharp eaves		-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2 -0,2	
With Parapets	h_p/h=0,025	-1,6	-2,2	-1,1	-1,8	-0,7	-1,2	+0,2 -0,2	
	h_p/h=0,05	-1,4	-2,0	-0,9	-1,6	-0,7	-1,2	+0,2 -0,2	
	h_p/h=0,10	-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	+0,2 -0,2	
Curved Eaves	r/h = 0,05	-1,0	-1,5	-1,2	-1,8	-0,4		+0,2 -0,2	
	r/h = 0,10	-0,7	-1,2	-0,8	-1,4	-0,3		+0,2 -0,2	
	r/h = 0,20	-0,5	-0,8	-0,5	-0,8	-0,3		+0,2 -0,2	
Mansard Eaves	α = 30°	-1,0	-1,5	-1,0	-1,5	-0,3		+0,2 -0,2	
	α = 45°	-1,2	-1,8	-1,3	-1,9	-0,4		+0,2 -0,2	
	α = 60°	-1,3	-1,9	-1,3	-1,9	-0,5		+0,2 -0,2	

The load distribution for the tower roof is given in table TT, whereas for the ground floor roof it is in table 3.8. Table 3.8 Loads at tower roof and ground floor roof

Tower roof		Ground floor roof	
Roof zone	Load (kN/m)	Roof zone	Load (kN/m)
Zone F	-27,572	Zone F	-24,606
Zone G	-18,955	Zone G	-17,224
Zone H	-15,509	Zone H	-11,073

Zone I	-6,893	Zone I	-4,921

3.1.4 Snow Load

The effect of snow loadings is significant during the winter times in Astana. For that reason, it is necessary to estimate the loadings caused by the snow and ensure that building will have good performance in terms of safety. In this report, the Eurocodes will be used as a basis of our estimations.

The snow loads on the roofs are calculated with the following formula:

$$s = \mu_i * C_e * C_t * s_k \qquad (3.11)$$

C_e – Exposure coefficient

C_t – Thermal coefficient

μ – Snow load shape coefficient

s_k – Characteristic value of snow on the ground at the relevant site [kN/m^2]

Due to the fact that the climate of Astana is windy and there are not many obstructions near the construction site, the exposure coefficient value is taken as 0.8. Thermal coefficient is recommended to be 1 according to the Eurocode.

In the Eurocode, the characteristic value of snow load on the ground is usually estimated with appropriate formulas for each country or region. Each region has its own formula based on the data on snow loads obtained from local meteorological stations (Sanpaolesi, 1997). However, due to the fact that Kazakhstan is not considered in the mentioned countries or regions, it is not possible to calculate the s_k value. Therefore, the relevant information was obtained from SNaR RK 2.01.07-85*. According to the document, Astana city, located in region III by snow covering (Ospanova, 2015), has 1.8 kPa for characteristic value of snow load on the ground.

Snow load shape coefficient will vary due to the fact that snow is drifted due to the wind in Astana. The shape coefficient for uniform snow loading will be 0.8 as the pitchs of the both, tower roof and ground floor roof, are less than 30 degrees. For the tower, the snow load shape coefficient will increase from 0.8 and reach the value $h*\gamma/s_k$ near parapet. In this case h is the height of the parapet and it is equal to 1.5m. Thus, the maximum load shape coefficient will be equal to 1.67 and the length of snow drifting will be equal to five times the h, namely 7.5m. By using the equation (3.11), we find that snow load will vary from 1.152 kN/m^2 to 2.4 kN/m^2.

In the case of the ground floor, the roof is close to the taller construction, the tower. Because of this the snow will be drifted in this case too. The length of the snow drift is found with equation

$$I_s = 2 * h \qquad (3.12)$$

Where, h – Height of construction work [m]

I_s – Length of snow drift or snow loaded area [m]

Here, h is the height of the tower and it equals to 26.4m. The snow drift value is 52.8, which does not satisfy the limits set by Eurocode, 5 ≤ Is ≤ 15 m. Moreover, as the drifting length exceeds the ground floor roof length, the snow load projection will be similar shape as in figure 3.4.

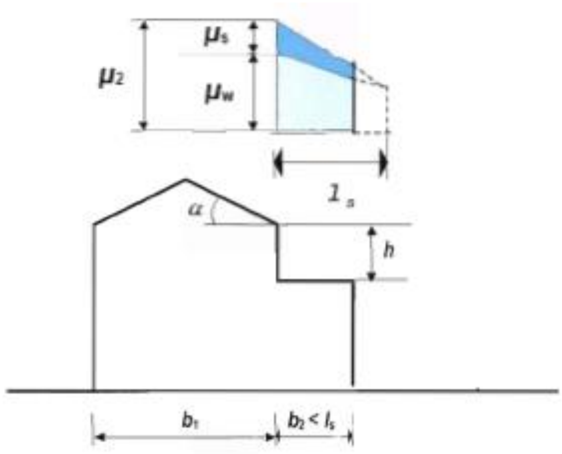


Figure 3.4 Snow load projection on roofs close to taller construction works

Where, A – Site altitude above sea level [m]

b – Width of construction work [m]

d – Depth of the snow layer [m]

α – Pitch of rood, measured from horizontal [°]

γ – Weight density of snow [kN/m3]

The shape coefficient of snow load in this case will be the sum of snow load shape coefficient due to sliding of snow from the upper roof, μ_s, and snow load shape coefficient due to wind, μ_w. As the pitch of the roof is less than 15 degrees the value of μ_s will be equal to 0. Whereas, the snow load shape coefficient due to wind is found with the following formula:

$$\mu_w = (b_1 + b_2)/2h \leq \gamma h/s_k \quad (3.13)$$

Where, μ – Snow load shape coefficient

s – Snow load on the roof [kN/m2]

By putting the necessary values, we find that μ_w equals 1.136 and is less than its maximum limit, 29.3, which is fine. The snow load coefficient at the edge of the roof was found with interpolation method and its value is equal to 1.012. By using the (3.13) formula, it can be estimated that on the roof of the ground floor the snow load varies from 1.457kN/m^2 to 1.636kN/m^2.

3.1.5 Load estimation

Eurocode provides load combination which are provided below.

Ultimate limit state (ULS):

$$\sum \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Serviceability Limit States (SLS):

- Characteristic combination (irreversible)

$$\sum G_{k,j} + P + Q_{k,1} + \sum \psi_{0,i} Q_{k,i}$$

- Frequent combination (reversible)

$$\sum G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum \psi_{1,i} Q_{k,i}$$

- Quasi-permanent combination

$$\sum G_{k,j} + P + \sum \psi_{2,i} Q_{k,i}$$

The following equations are used in combination with a Table 3.9 which provides the coefficient for action loads of different variables. These load combination are used to determine most critical loading applied on the structure in order to analyze building frames and design section frames.

Table 3.9. Design values of actions

Failure State	Permanent action		Leading variable action		Leading variable action	
	Unfovaroble	favorable	Unfovaroble	favorable	Unfovaroble	favorable
EQU	1.1 Gk	0.9 Gk	1.5 Qk	0.0	1.5ψQk	0.0
STR	1.35 Gk	0.9 Gk	1.5 Qk	0.0	1.5ψQk	0.0

EQU - Loss of equilibrium of the structure.

STR - Internal failure or excessive deformation of the structure or structural member

SAP2000 itself contains different codes, these structure was designed by using Eurocode. As a result, the program has such combinations to analyze and check/design options.

1. 1.35DL
2. 1.35DL+1.5LL
3. 1.35DL+1.35LL+1.35WL
4. 1.35DL+1.35LL-1.45WL
5. 1.35DL+1.5WL
6. 1.35DL-1.5WL
7. 1DL+1.5WL
8. Dl-1.5WL

3.2 Building Frame Analysis

3.2.1 SAP2000 results of Frame Analysis

This section analyzes the load applied with the help of SAP2000. All loads have been changed into point loads, and wind load is applied as a joint force. These steps were made in order to easily put values and compare the program result with hand calculations. Figure below shows how these loads are applied.

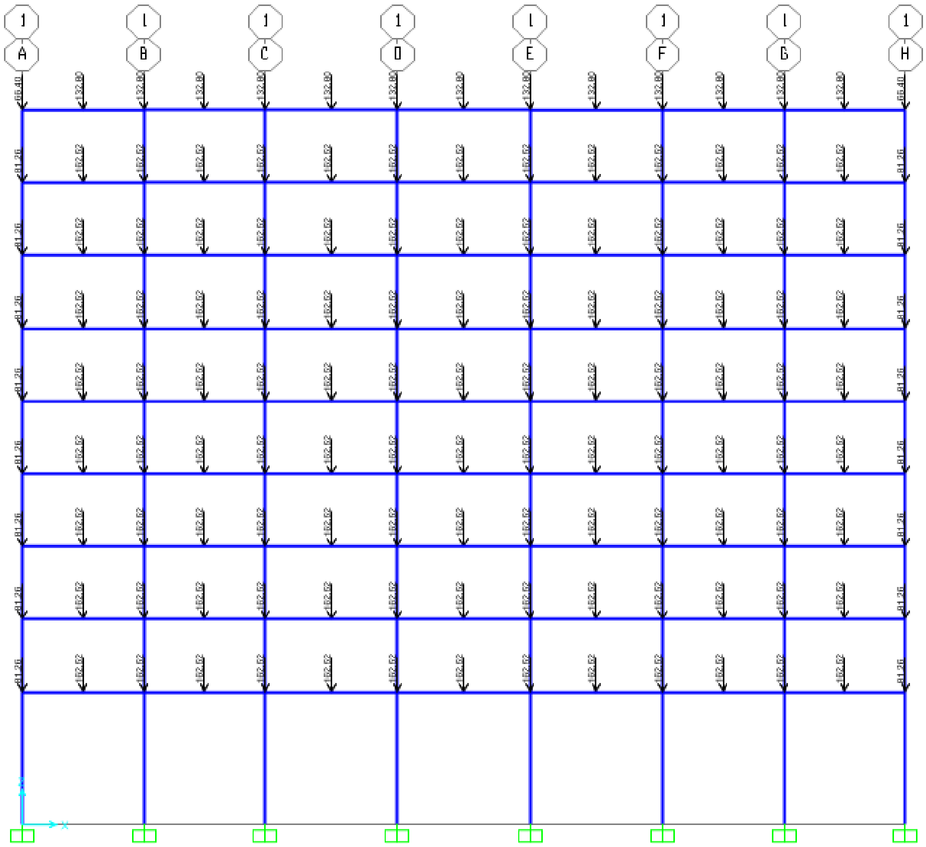


Figure 3.5. Dead loads applied as a point load in 2d frame in Z-X direction

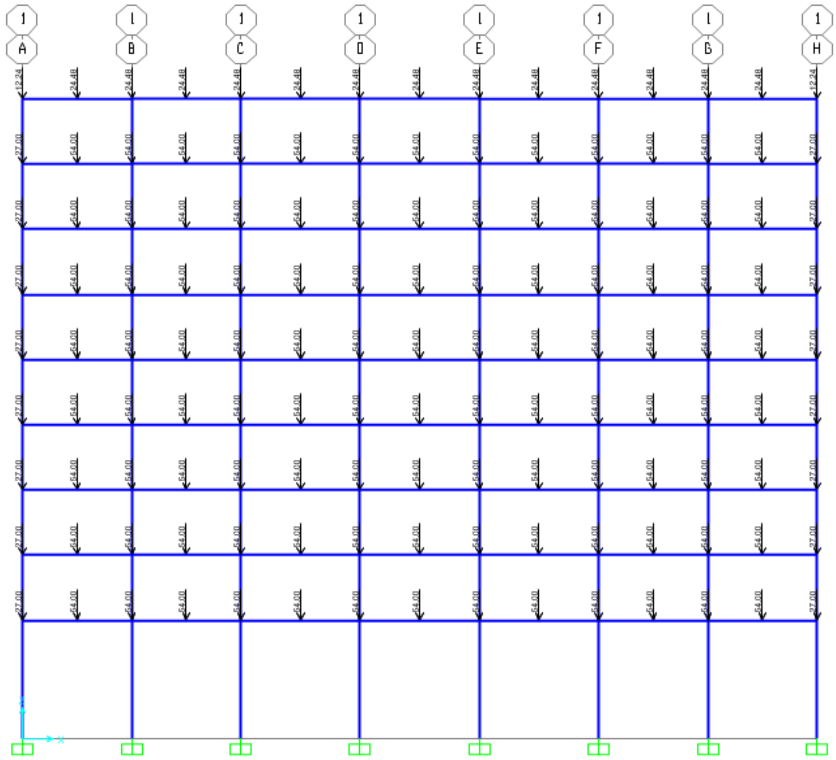


Figure 3.6. Live loads applied as a point load in 2d frame in Z-X direction

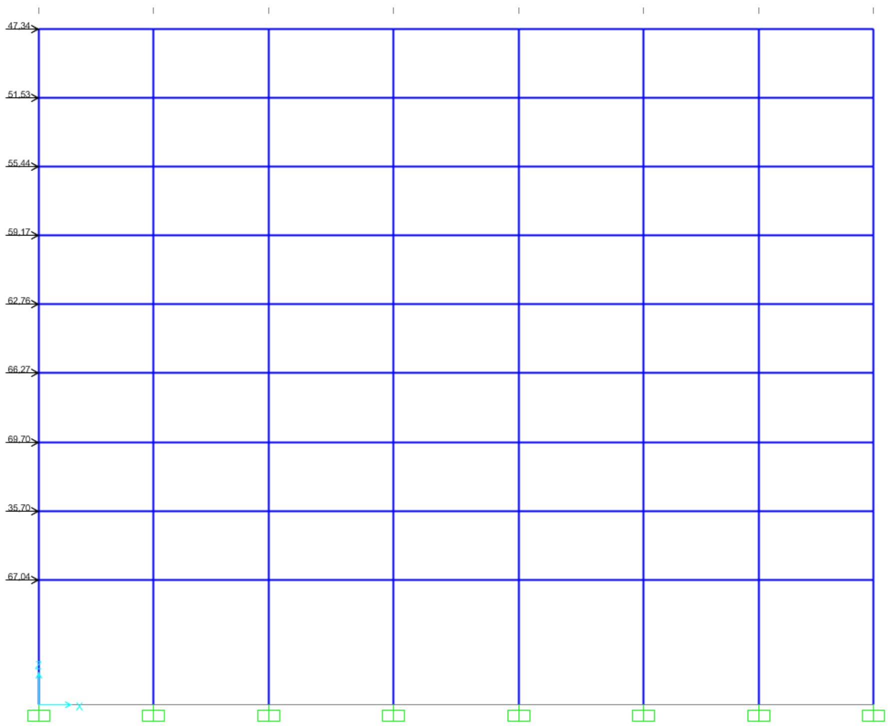


Figure 3.7. Wind loads applied as a point load in 2d frame in X direction

3.2.2 Analysis under Wind Load

Since the building may be influenced by lateral (wind or seismic) loads during its lifetime, it is essential that it has to be designed to withstand these loads. In this case, as Astana is not located in earthquake zone, only wind load will be considered. In order to analyze the internal forces, the portal method was used. It is an approximate analysis of statically indeterminate building frames. Since it is required to reduce the structure to the statically determinate, the following assumptions have to be made:

- Points of zero moment occurs at the mid height of each column and beam;
- The horizontal shear is divided among all the columns in such way that each interior column takes twice as much as exterior column.

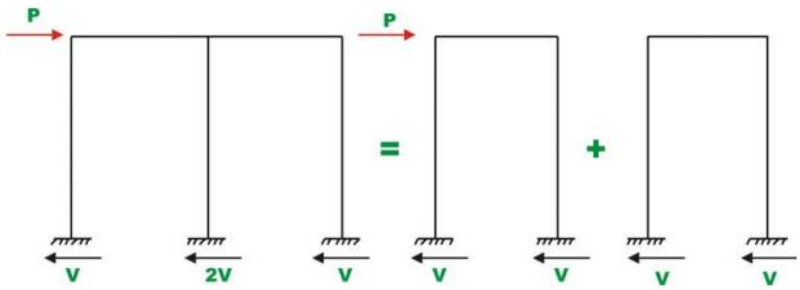


Figure 3.8. Shear distribution of Portal method for the building frame

The solution procedure of this method is presented in the following hand calculation figures. For now, in order to determine all the forces, Excel spreadsheet was created for exterior beams and columns of each storeys and tabulated in Appendix with SAP2000 results.

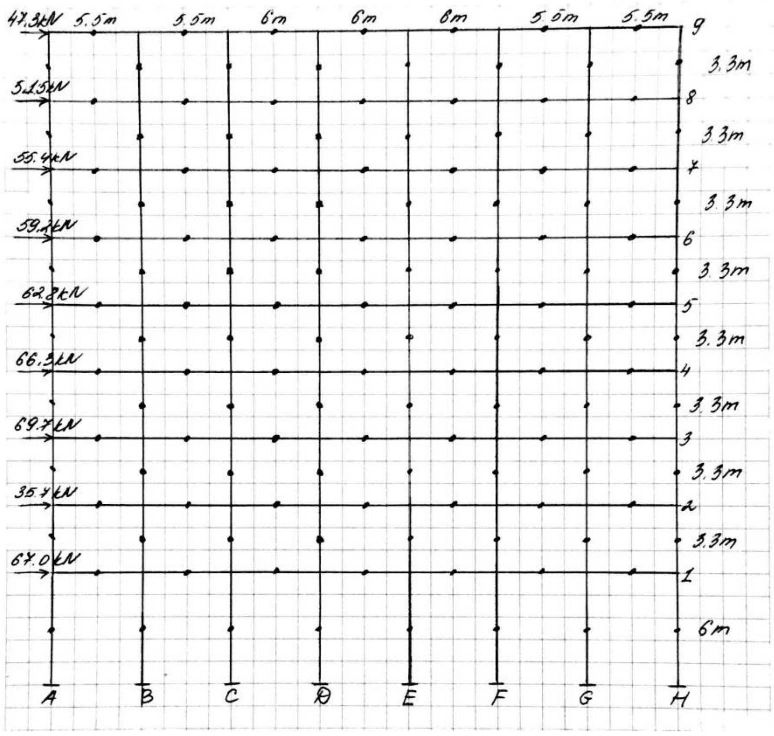


Figure 3.9. Frame with applied wind loads

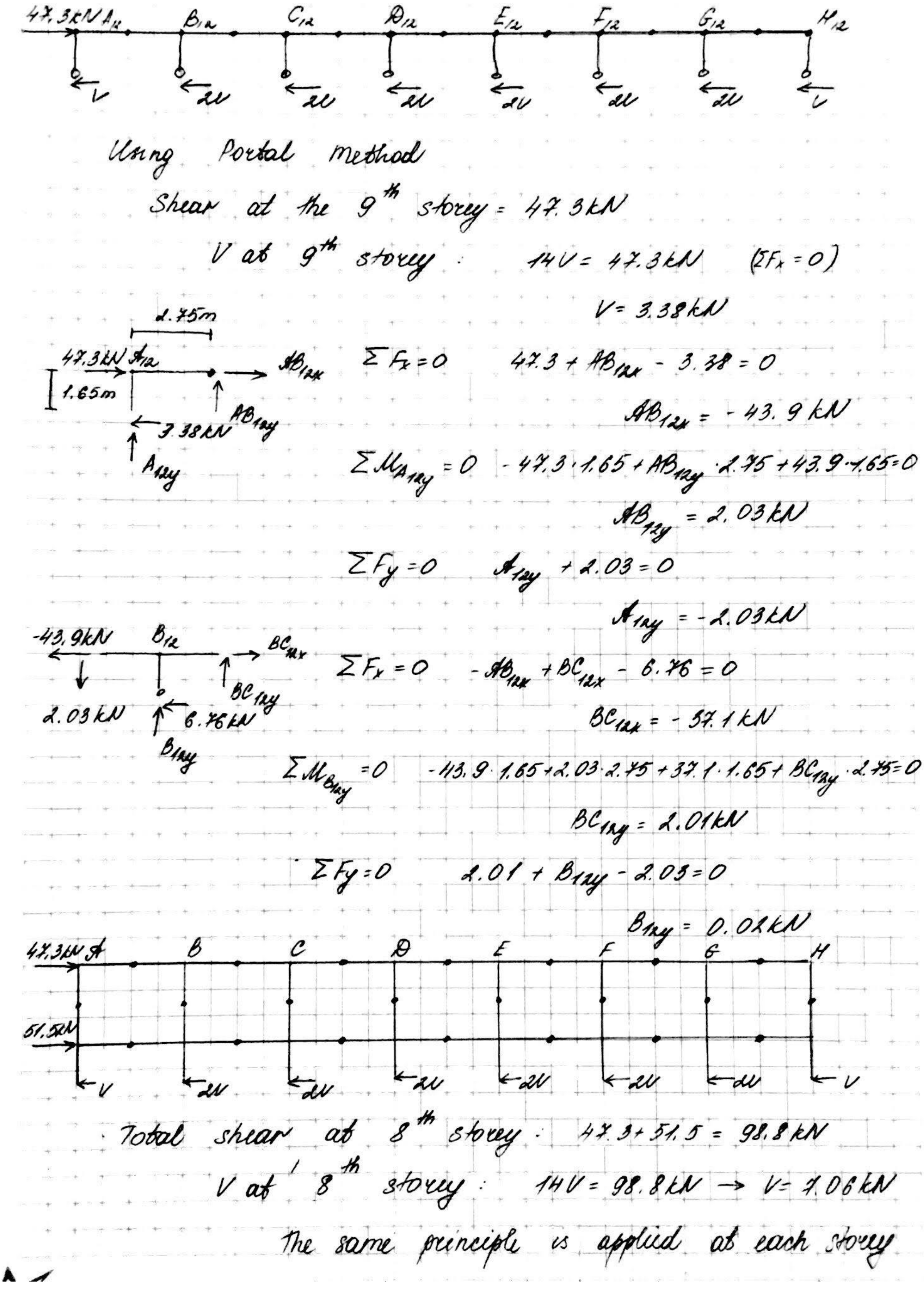


Figure 3.10. Hand calculation of wind load by portal method

3.2.3 Analysis under Dead Load

Before design step in SAP2000 it is important to compare results from SAP2000 with hand calculations. As a result, dead load pattern was taken in SAP2000 as a separate value to compare moment diagrams. In this part of calculations it can be seen that beam connections to the column are not simply supported or pin connected. They are connected in combination of both types; figure below illustrates the connection type.

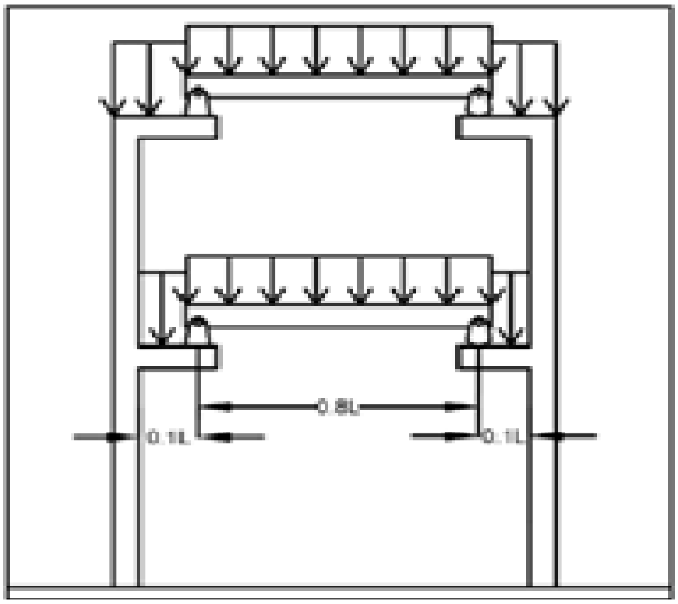


Figure 3.11. Beam loading in building

In this type of connection to define the maximum moment it is necessary to calculate zero moment point, which is about 0.1L from the support according to approximate method; on the other hand, this value is different for interior beam. As the calculation of these zero moment points is complicated, they have been taken from SAP2000 to compare the moment diagrams. Calculations are provided below.

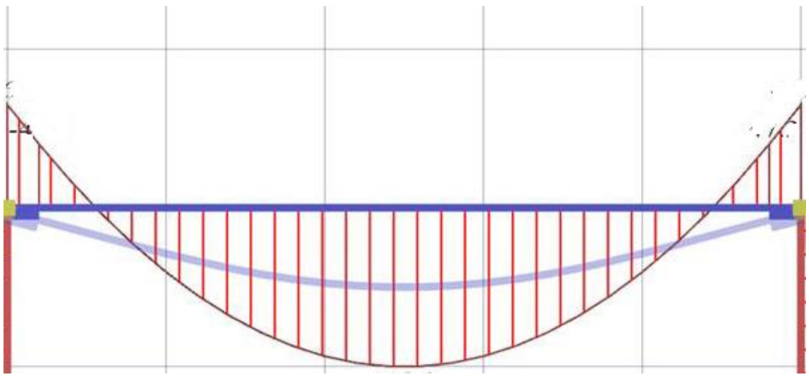


Figure 3.12. Bending moment in Frame

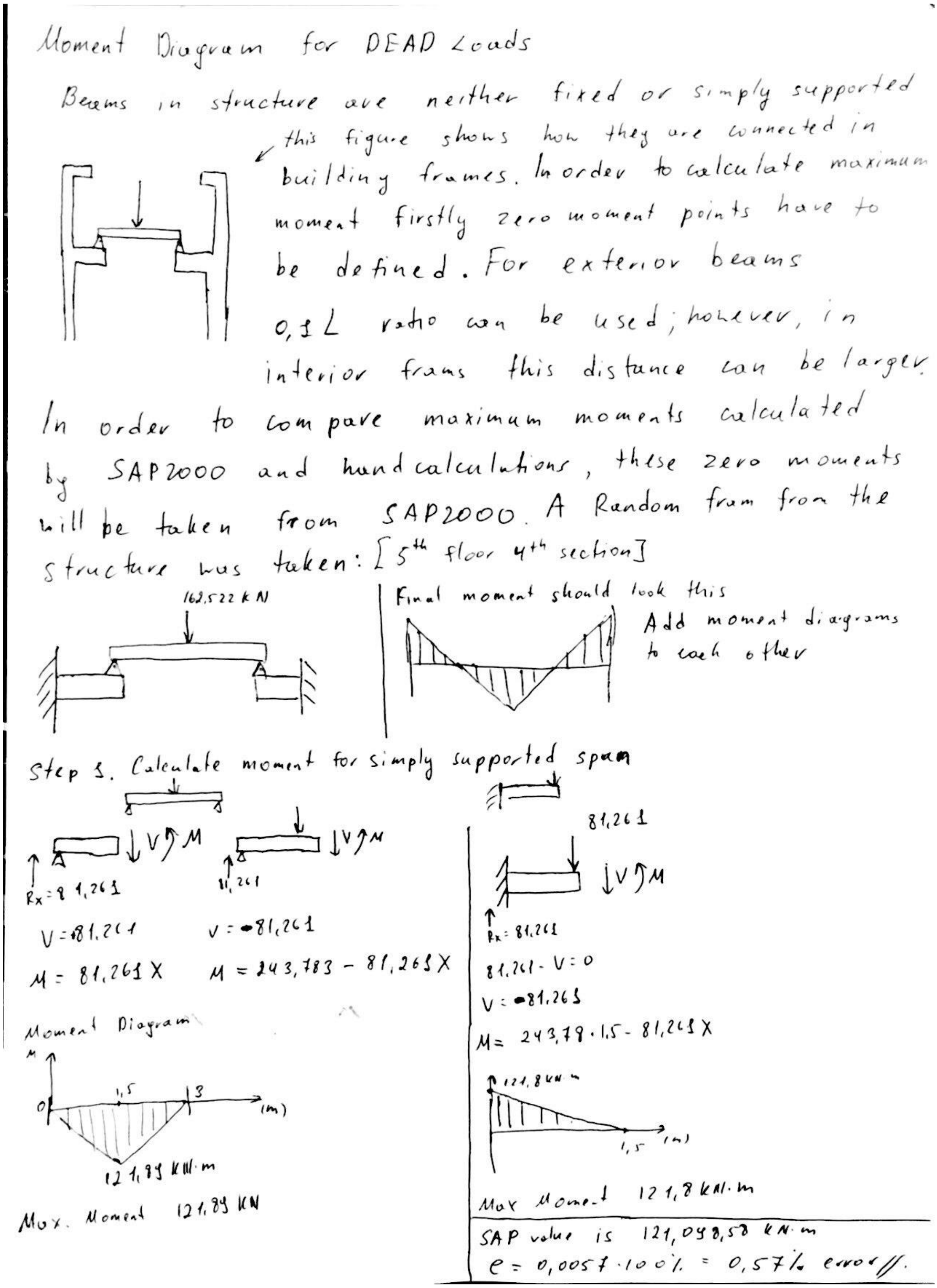


Figure 3.13. Approximate method calculations

According to the results obtained from hand calculations and SAP2000, it can be seen that the values are approximately similar for axial forces, while for the shear and moment forces have distinguishing differences. The reason of such outputs can be assumptions made during calculations. In general, it is beneficial approximate method of analyzing the building frame.

3.2.4 SAP2000 Design and Check

After finishing the analysis part the design was considered. Default values for beams and columns have been used in order to check and design. Result of these analyses can be seen below. Orange and grey colors indicate that set values are okay to use; on the other hand, red colors indicate that sections have to been changed. For example, red beam sections have an error "Reinforcing required exceeds maximum allowed" which means that provided by SAP2000 reinforcement exceeds the code requirements (1-4%). Table below provides section sizes used in this attempt. Smaller values have been used than actual calculated ones in order to let the program provide the most optimal design.

Table 3.10. Initial section sizes

	Interior beam (mm)	Exterior Beam (mm)	Interior Column (mm)	Exterior Column (mm)
1st Floor	500x300	450x300	550x550	550x550
2nd Floor	500x300	450x300	550x550	550x550
3rd Floor	500x300	450x300	450x450	450x450
4th Floor	500x300	450x300	450x450	450x450
5th Floor	500x300	450x300	450x450	450x450
6th Floor	500x300	450x300	450x450	300x300
7th Floor	500x300	450x300	450x450	300x300
8th Floor	500x300	450x300	450x450	300x300
9th Floor	500x300	450x300	450x450	300x300

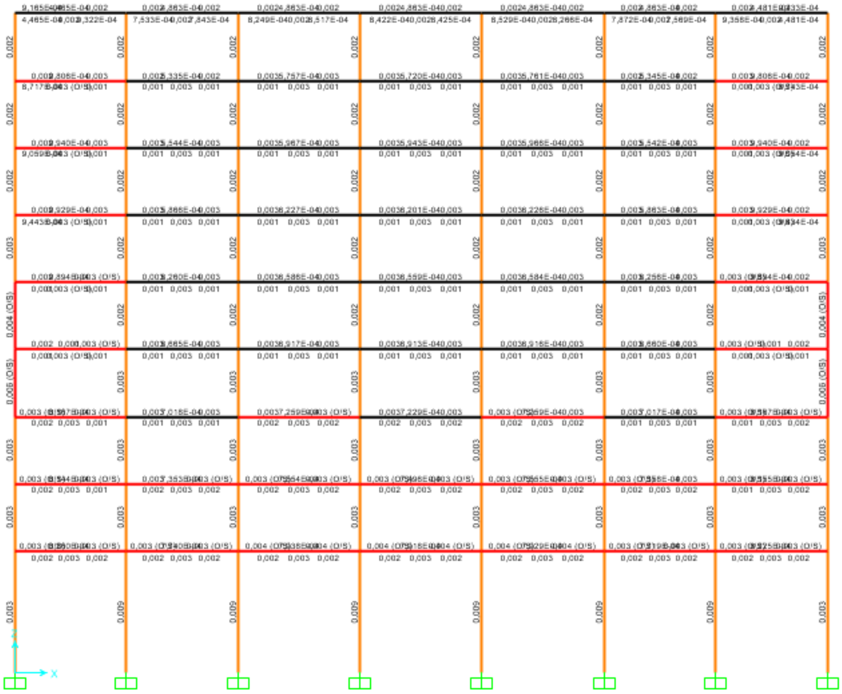


Figure 3.14. SAP2000 initial result of check/design

According to this first result of SAP2000, the red colored sections have to be changed to larger value. In exterior frames, beams of upper 5 floors have to be changed to 500x300 mm^2, while beams of the first 4 floors have to be changed to 600x300 mm^2 beams, and columns on the 3rd and 6th floors have to be changed to 550x550 mm^2 and 450x450 mm^2 respectively. As a result of SAP2000 check/design mode is provided in figure 3.15 for Z-X frame.

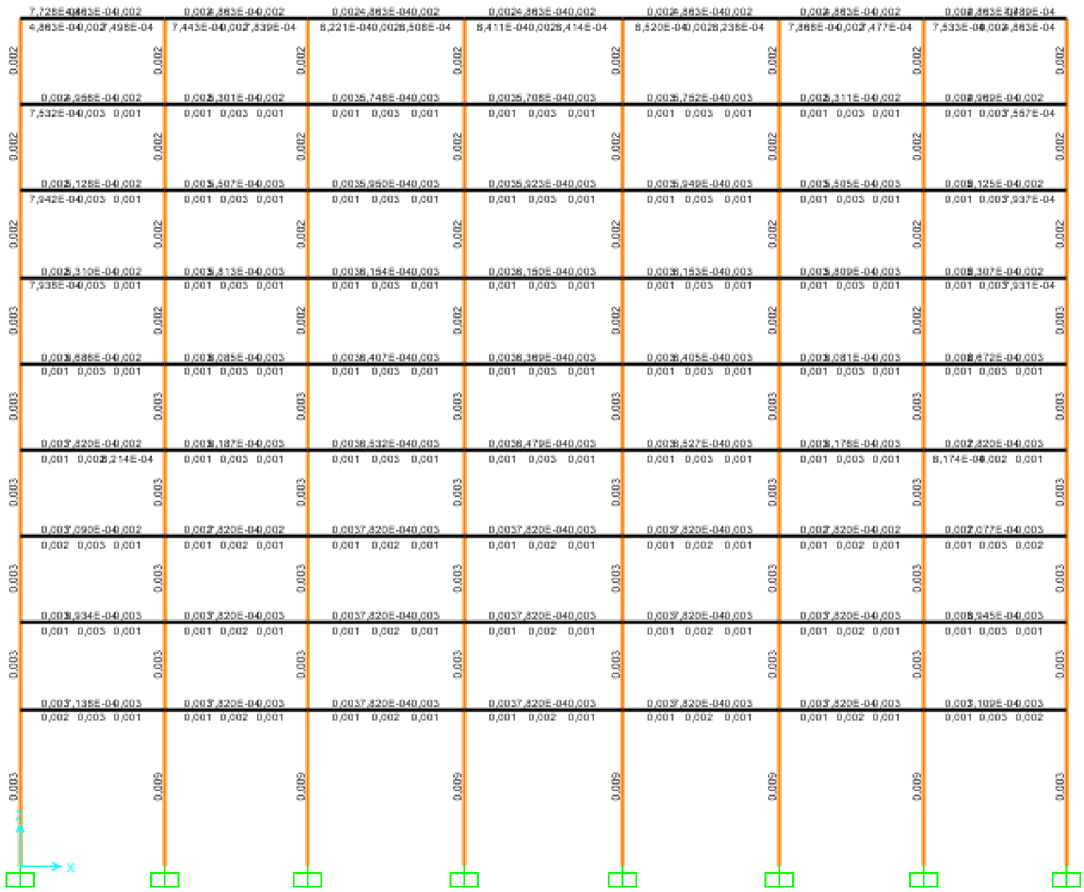


Figure 3.15. SAP2000 2nd attempt results

In order to provide more accurate design for construction, the second frame of building was analyzed in Z-Y directions. After two frame analysis, it was decided to design beams according Z-X frame and Z-Y direction for columns. Point loads in these two frames have differences, which affect on the design and check process in SAP2000; consequently, critical values have been used in order to provide accurate solution.

After 2 attempts design was completed and SAP2000 clarified it by providing such information figure shown below, which mean that all sections are in code requirement range.

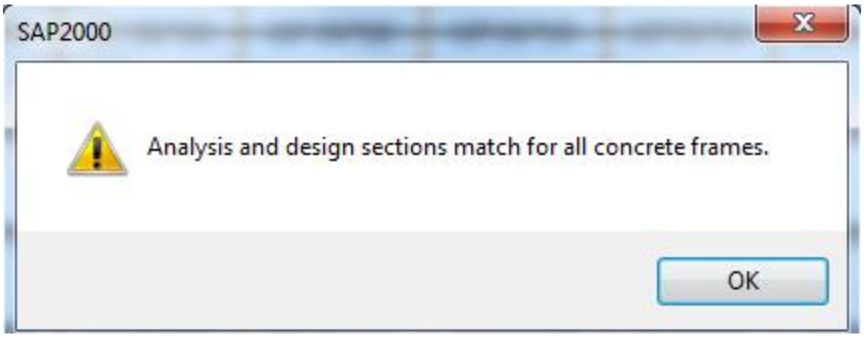
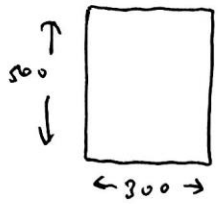


Figure 3.16. Design conformation

3.2.5 Hand calculations

Beam and column hand calculations were made according to moments taken from SAP2000 and column was estimated according axial forces from the program

Reinforcement Design for Beam located in 6th floor of section B.



$f_{cu} = 25$ kN/m³

C25/30

$f_y = 500$ Gr500

$F_{ctm} = 2,6$ MPa

SAP2000 results

$M_u = 210,65$ kN·m

$A_{s,req} = 1228,495$ mm²

$$d_{min} = \sqrt{\frac{M_u}{k \cdot b \cdot f_{cu}}} = 384,32 \text{ mm} < 450 \text{ mm} \quad \underline{ok}$$

$$K = \frac{M_u}{b d^2 \cdot f_{cu}} = \frac{210,65 \cdot 10^6}{300 \cdot 450^2 \cdot 25} = 0,139 < 0,167$$

$$\frac{z}{d} = 0,5\left[1 + \left(1 - \frac{2k}{1}\right)^{0,5}\right] = 0,882 \Rightarrow z = 396,9 \text{ mm}$$

$$A_s = \frac{\gamma_s \cdot M_u}{h \cdot z} = \frac{1,15 \cdot 210.65 \cdot 10^6}{500 \cdot 396,9} = 1220,6 \text{ mm}^2 \text{ required}$$

$A_{s,min}$ check

$$A_{s,min} \begin{cases} 0,26 \, f_{ctm} \cdot b_t \cdot d / f_y = 182,52 \text{ mm}^2 \\ 0,0013 \, b d = 175,5 \text{ mm}^2 \end{cases} < 1220,6 \text{ mm}^2 \quad \underline{ok}$$

$$A_{s,max} = 0,04 A_c = 6000 \text{ mm}^2 > A_s \quad \underline{ok}$$

Error]

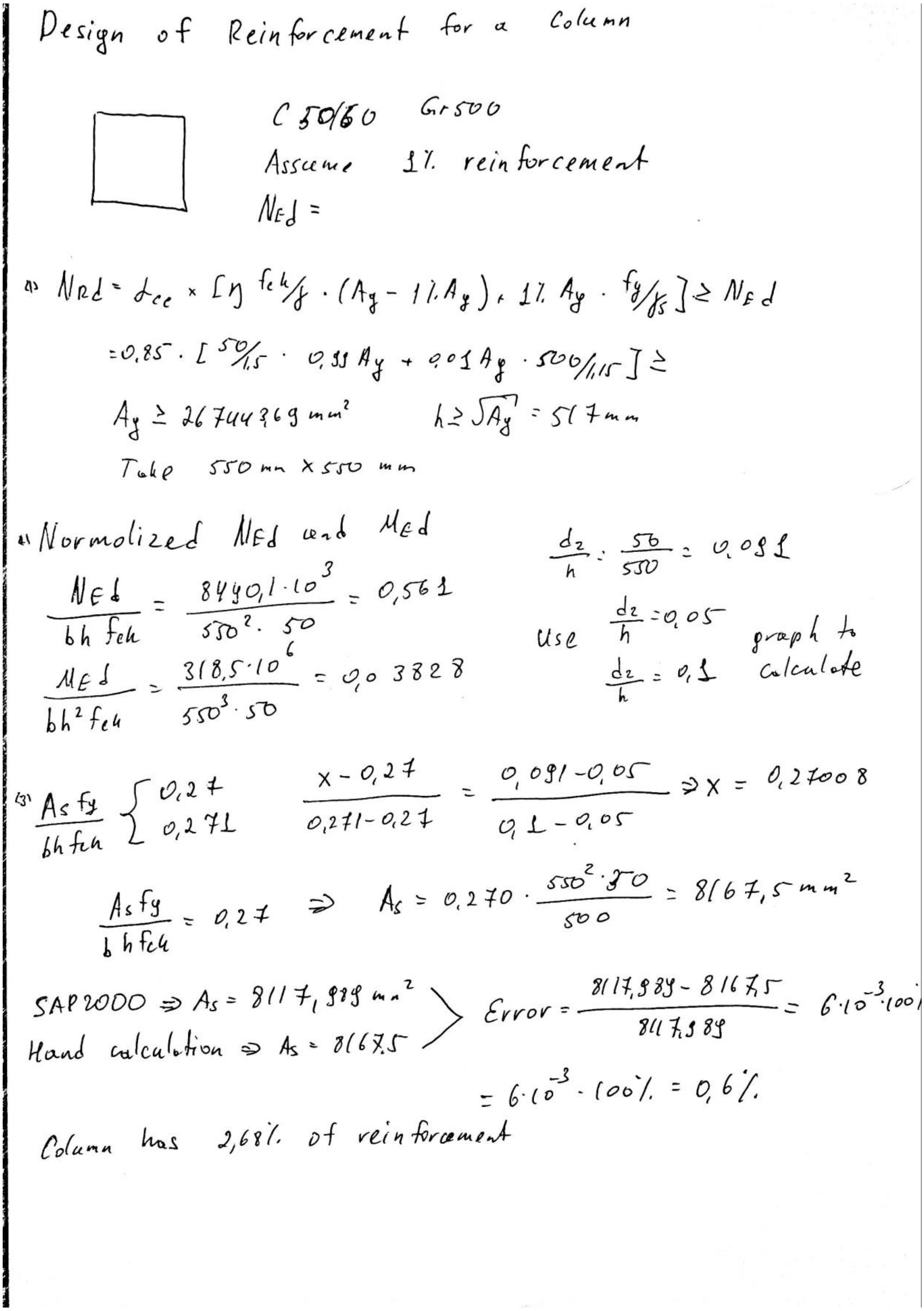
SAP2000 – 1228,49 mm²

Hand calculation – 1220,6 mm²

$$e = \frac{1228,49 - 1220,6}{1228,49} = 0,0064 \cdot 100\%$$

$$= 0,64\%$$

Figure 3.17 Hand calculations of beam



Design of Reinforcement for a Column

C 50/60 Gr500

Assume 1% reinforcement

$N_{Ed} =$

(1) $N_{Rd} = \alpha_{cc} \times [\eta\, f_{ck}/\gamma \cdot (A_g - 1\% A_g) + 1\% A_g \cdot f_y/\gamma_s] \geq N_{Ed}$

$= 0,85 \cdot [50/1,5 \cdot 0,99 A_g + 0,01 A_g \cdot 500/1,15] \geq$

$A_g \geq 267443,69\ mm^2 \qquad h \geq \sqrt{A_g} = 517\ mm$

Take 550 mm x 550 mm

(2) Normolized NEd and MEd

$\frac{N_{Ed}}{b h f_{ck}} = \frac{8440,1 \cdot 10^3}{550^2 \cdot 50} = 0,561$

$\frac{M_{Ed}}{b h^2 f_{ck}} = \frac{318,5 \cdot 10^6}{550^3 \cdot 50} = 0,03828$

$\frac{d_2}{h} : \frac{50}{550} = 0,091$

Use $\frac{d_2}{h} = 0,05$ and $\frac{d_2}{h} = 0,1$ graph to calculate

(3) $\frac{A_s f_y}{b h f_{ck}} \begin{cases} 0,27 \\ 0,271 \end{cases} \quad \frac{x - 0,27}{0,271 - 0,27} = \frac{0,091 - 0,05}{0,1 - 0,05} \Rightarrow x = 0,27008$

$\frac{A_s f_y}{b h f_{ck}} = 0,27 \Rightarrow A_s = 0,270 \cdot \frac{550^2 \cdot 50}{500} = 8167,5\ mm^2$

SAP2000 $\Rightarrow A_s = 8117,989\ mm^2$

Hand calculation $\Rightarrow A_s = 8167,5$

$Error = \frac{8117,989 - 8167,5}{8117,989} = 6 \cdot 10^{-3} \cdot 100\%$

$= 6 \cdot 10^{-3} \cdot 100\% = 0,6\%$

Column has 2,68% of reinforcement

Figure 3.18 Hand calculation for column design

(4) $A_{s,min}$ check

$$A_{s,min} \begin{cases} 0,1 \cdot \dfrac{N_{Ed}}{f_y/\gamma_s} = \dfrac{0,1 \cdot 8430.1 \cdot 10^3}{500/1,15} = 1852,7\ mm^2 \\ 0,002 A_g = 605\ mm^2 \end{cases}$$

less than A_s ok

$A_{s,max} = 0,04 A_g = 12100\ mm^2 > A_s$ ok

(5) Design links

$$d_t \geq \begin{cases} 5\ mm \\ \frac{1}{4} d_t = \frac{40}{4} = 10\ mm \checkmark \end{cases}$$

max spacing

$$S_{max} \leq \begin{cases} 20\, d_t = 800\ mm \checkmark \\ h = 350\ mm \\ 400\ mm \end{cases}$$

choose 10 @ 400

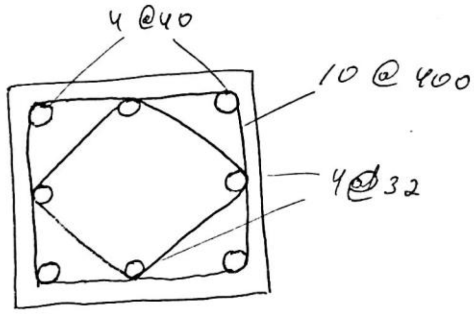


Figure 3.19 Hand calculations of column (cont.)

From the hand calculation it can be seen that are results are similar to SAP2000 result. Thus, it can surely said that SAP2000 provides adequate beam and column detailing that are used in a design.

3.3 Structural detailing

To this point the required amount of reinforcement, including main and shear reinforcements for the beam and columns, is obtained from SAP2000. Next step of the structural design includes structural detailing such as slab design and serviceability limit states check.

3.3.1 Slab design

The design of slab reinforcement that is outlined in section below was carried out following Eurocode 2. The slab reinforcement is designed parallel to the alphabetic grid, since the floor system is a one way slab. This means that the maximum moment occurs in the middle of the span and at the supports.

There are two span lengths in the building (2.75m and 3m) and the following calculation is of the interior slab of 3m. Calculations for interior and end spans of 2.75m follows the same procedure, therefore, the obtained results are tabulated in Appendix B-3.

The first step in slab design was to assign the slab depth of 150mm, which is based on the minimum slab thickness according to Eurocode: span/24=3000/24=125mm. Thereafter, the dead load is calculated by summing up the self-weight of the slab and floor finishing and ceiling, which are obtained before in this report. Moreover, it should be noted that all interior walls and partitions lie on beams; therefore, their load can be neglected in slab design. The live load is also given in previous sections of this report. The total factored load is given as:

$$w = 1.35 \times w_D + 1.5 \times w_L = (1.35 \times 6.146) + (1.5 \times 3) = 12.7971\ kN/m^2$$

The design moments for the one way slab can be obtained using the moment coefficients provided in Eurocode 2. In order to be more conservative the end supports are assumed to be fixed, although end spans are assumed to be pinned. The results are tabulated below.

Table 3.11 Bending moment for slab.

Location		Moment	Shear
End support/span	End span	$0.086Fl = 0.086 \times (12.80 \times 3) \times 3 = 9.91\ kN \cdot m/m$	$0.46F = 0.46 \times 12.80 \times 3 = 17.66\ kN/m$
	End support	$-0.04Fl = -0.04 \times (12.80 \times 3) \times 3 = 4.61\ kN \cdot m/m$	–
Interior support/span	First interior supports	$-0.086Fl = -0.086 \times (12.80 \times 3) \times 3 = -9.91\ kN \cdot m/m$	$0.6F = 0.6 \times (12.80 \times 3) = 23.04\ kN/m$
	Interior spans	$0.063Fl = 0.063 \times (12.80 \times 3) \times 3 = 7.26\ kN \cdot m/m$	–
	Interior supports	$-0.063Fl = -0.063 \times (12.80 \times 3) \times 3 = -7.26\ kN \cdot m/m$	$0.5F = 0.5 \times (12.80 \times 3)$ $= 19.2 kN/m$

It is assumed that the diameter of the reinforcement is 8mm. The minimum axis dimension (the distance from the edge of the slab to the center of the bar) is dictated by Eurocode to be 20mm. Therefore, the clear cover is set to be 20 mm resulting in effective depth of the slab to be 126mm. The ultimate bending moment can be calculated now.

$$M_u = \frac{0.168 f_{ck} d^2}{1000} = 66.68\ kNm.$$

Next step is to determine the value of K from $K = \frac{M}{f_{ck} d^2}$ and compare it with the K_{max} value that is equal to 0.206 for span moments and 0.166 for support moments (assuming 15% redistribution). $K = \frac{M}{f_{ck} d^2} = \frac{7.26}{25 \times 126^2} = 0.018 < K_{max} = 0.206.$ Therefore, there is no compression reinforcement required.

The lever arm is calculated using the following formula:

$$z = 0.5\left(1 + \left(1 - \frac{3k}{\eta}\right)^{\frac{1}{2}}\right) \times d = 124.25$$

And the required reinforcement:

$$A_{s,req} = \frac{M}{0.87 f_{yk} z} = \frac{7.26 \times 10^6}{0.87 \times 500 \times 124.25} = 134.25 mm^2$$

Check minimum reinforcement requirement:

$$A_{s,min} = \frac{0.26 f_{ctm} b_t d}{f_{yk}} = \frac{0.26 \times 2.6 \times 1000 \times 126}{500} = 170.35\ mm^2$$

$$A_{s,min} = 0.0013 b_t d = 0.0013 \times 1000 \times 126 = 163.8\ mm^2$$

Since, $A_{s,req} < A_{s,min}$, the required reinforcement amount is 170.35 mm^2. Therefore, it is provided 8mm diameter bar at 275 mm spacing, which results in 183 mm^2.

Check maximum reinforcement requirement:

$$A_{s,max} = 0.04 A_c = 0.04 \times 1000 \times 150 = 6000\ mm^2$$

Slab deflection check

According to Eurocode 2 clause 7.4.2 deflection check may be omitted if the span to effective depth ratio lie within the limits given in the following formula:

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}} \cdot \frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}(\frac{\rho_o}{\rho} - 1)^{3/2}\right]\ for\ \rho < \rho_o$$

Where, K for end spans and interior spans are 1.3 and 1.5 respectively. The values of $\rho = \frac{A_{s,req}}{bd}$ and $\rho_o = \frac{\sqrt{f_{ck}}}{1000}$. The deflection check results are tabulated in Appendix.

After the main reinforcement is designed, distribution reinforcement along the long direction, the distribution reinforcement is designed. The required area of distribution reinforcement is dictated by Eurocode 2 minimum area reinforcement requirement. All slab details including the bar spacing and its diameter are given in Appendix B-3.

3.3.2 Serviceability Limit States Check

The Serviceability Limit State: the structure should not fail due to excessive deflection, cracking or vibration.

- Cracking - should be kept within reasonable limits by correct detailing.
- Deflection - the deformation of the structure should not adversely affect its efficiency or appearance.

- Drift – interstorey drift have to be in range of acceptable displacement which is given in code.
- Development Length - the reinforcement should be anchored properly to fully develop its strength – bond stress.

Cracking

Rather than directly calculate the crack width, code requires a maximum spacing for the reinforcement for crack control.

$$w = 0.076 \times 10^{-3} \beta_n f_s \sqrt[s]{t_b A}$$

Where, B_n – h_2/h_1 where $h_1$is a distance from the centroid of tension bars to the N.A, h_2 is distance from the extreme tension fiber to the N.A.

t_b – distance from extreme tension fiber to the center of the closest bar

A – average effective area of concrete in tension around each bar reinforcing.

f_s – steel stress under service load in ksi.

$$d_b \leq s \leq \frac{70000\beta_b}{f_y} \leq 300$$

$$d_b \leq s \leq \frac{47000\beta_b}{f_s} \leq 300$$

$$f_s = \frac{2f_y A_{s\ req}}{3A_{s\ prov}} \times \frac{1}{\beta_b}$$

Where:

S – clear spacing

B_b is the ratio:

$$\frac{\text{moment at the section after redistribution}}{\text{moment at the section after redistribution}}$$

For simply supported B_b is equal to 1 and for multi span slab is 0.8

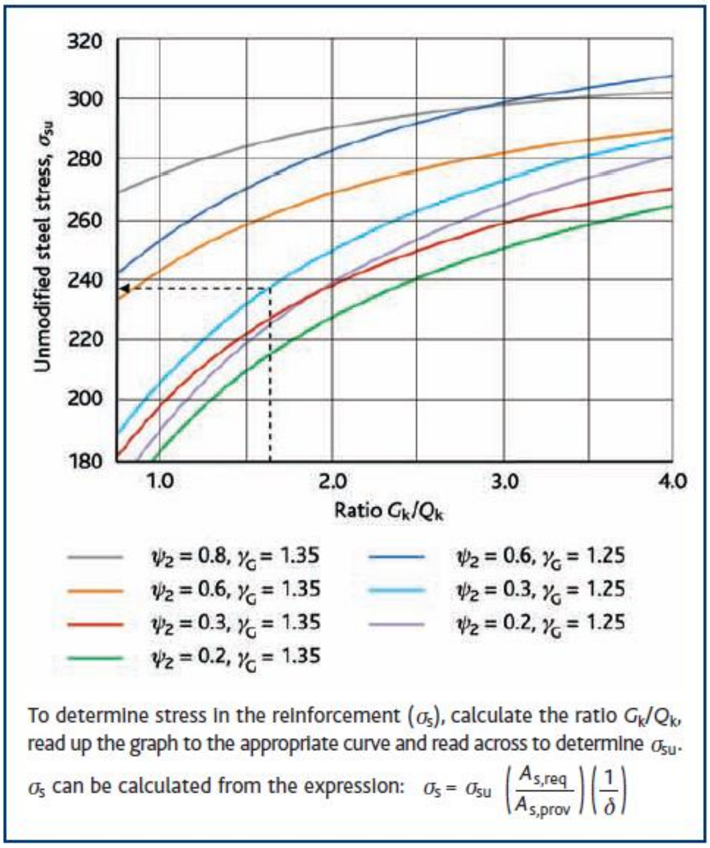


Figure 3.20. Determination of steel stress for crack width control

In order to limit the crack width Eurocode provide a table in which the maximum bar sizes are provided.

Table 3.12. Maximum bar size and spacing to limit crack width

Steel stress (бs) Mpa	Wmax=0.4			Wmax=0.3		
	maximum bar sizes (mm)		maximum bar sizes (mm)	maximum bar sizes (mm)		maximum bar sizes (mm)
160	40		300	32		300
200	32	OR	300	25	OR	250
240	20		250	26		200
280	16		200	23		150
320	12		150	10		200
360	10		100	8		50

$$\sigma_s = \frac{f_{yk} m A_{s,req}}{\gamma_{ms} A_{s,porv} n \sigma}$$

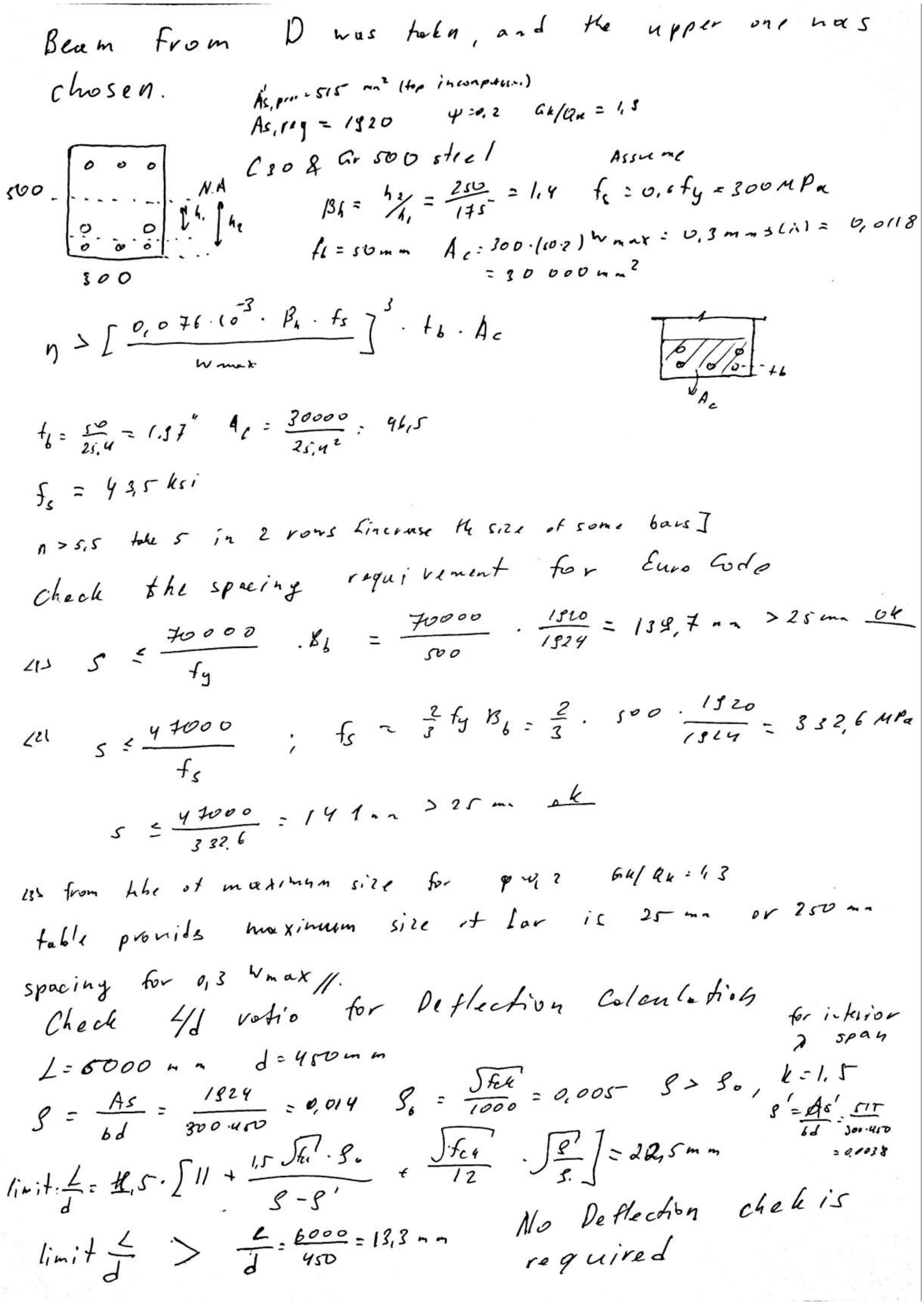
Where, F_{yk} – characteristic reinforcement yield stress

y_{ms} – partial factor for reinforcing steel

m – total load from quai-permenent combination

n – total load from ULS combination

Ϭ – ratio of redistributed moment to elastic moment



Beam from D was taken, and the upper one was chosen.

$A'_{s,prov} = 515\ mm^2$ (top in compression)

$A_{s,req} = 1920$ $\quad \psi = 0.2 \quad G_k/Q_k = 1.3$

C30 & Gr 500 steel

Assume

$\beta_h = \frac{h_2}{h_1} = \frac{250}{175} = 1.4 \quad f_s = 0.6 f_y = 300\ MPa$

$t_b = 50\ mm \quad A_c = 300 \cdot (50 \cdot 2) \quad w_{max} = 0.3\ mm = 0.0118$

$= 30\ 000\ mm^2$

$$n > \left[\frac{0.076 \cdot 10^{-3} \cdot \beta_h \cdot f_s}{w_{max}} \right]^3 \cdot t_b \cdot A_c$$

$t_b = \frac{50}{25.4} = 1.97''$ $\quad A_c = \frac{30000}{25.4^2} = 46.5$

$f_s = 43.5\ ksi$

n > 5.5 take 5 in 2 rows [increase the size of some bars]

Check the spacing requirement for Euro Code

(1) $s \le \frac{70000}{f_y} \cdot \beta_b = \frac{70000}{500} \cdot \frac{1920}{1924} = 139.7\ mm > 25\ mm$ ok

(2) $s \le \frac{47000}{f_s}$; $f_s = \frac{2}{3} f_y \beta_b = \frac{2}{3} \cdot 500 \cdot \frac{1920}{1924} = 332.6\ MPa$

$s \le \frac{47000}{332.6} = 141\ mm > 25\ mm$ ok

(3) from table of maximum size for $\psi = 0.2$ $\quad G_k/Q_k = 1.3$

table provides maximum size of bar is 25 mm or 250 mm spacing for 0.3 w_{max} //.

Check L/d ratio for Deflection Calculations

for interior span

$L = 6000\ mm \quad d = 450\ mm$ $\quad k = 1.5$

$\rho = \frac{A_s}{bd} = \frac{1924}{300 \cdot 450} = 0.014 \quad \rho_0 = \frac{\sqrt{f_{ck}}}{1000} = 0.005 \quad \rho > \rho_0,$

$\rho' = \frac{A_s'}{bd} = \frac{515}{300 \cdot 450} = 0.0038$

$$\text{limit } \frac{L}{d} = 1.5 \cdot \left[11 + \frac{1.5\sqrt{f_{ck}} \cdot \rho_0}{\rho - \rho'} + \frac{\sqrt{f_{ck}}}{12} \cdot \sqrt{\frac{\rho'}{\rho_0}} \right] = 22.5\ mm$$

$\text{limit } \frac{L}{d} > \frac{L}{d} = \frac{6000}{450} = 13.3\ mm$ $\quad$ No Deflection check is required

Figure 3.21. Hand calculation of cracking and deflection

Deflection in beams

According the Eurocode if L/250 is less than limit or L/d is larger than limit then deflection calculation is not needed. In order to estimate does the section need the deflection check the following equations can be used.

$$\frac{L}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}p_o}{p} + 3.2\sqrt{f_{ck}}\left(\frac{p_o}{p} - 1\right)^{1.5}\right]$$

where $p<p_o$

$$\frac{L}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}p_o}{p} + \frac{\sqrt{f_{ck}}}{12}\sqrt{\frac{p'}{p_o}}\right]$$

where $p>p_o$

These equations have used in order to determine which beams need deflections check. Consequently, it has been found that beams do not need deflection calculations.

Limitations of interstorey drift

In order to start design of structural elements it is important to analyze drift limitations. According to Eurocode, buildings containing non-structural elements should satisfy following conditions:

a) For buildings having non-structural elements of brittle materials attached to the structure:
   dr v<0,005 h
b) For buildings having ductile non-structural elements:
   dr v<0,0075 h
c) For buildings having non-structural elements fixed in a way so as not to interfere with
   structural deformations, or without non-structural elements:
   dr v<0,010 h

where, dr – design interstorey drift;

h – the storey height;

v – the reduction factor which takes into account the lower period of the seismic (wind) action associated with the damage limitation requirement. It is depends on the type category of the structure. For academic building category v is equal to 0.5.

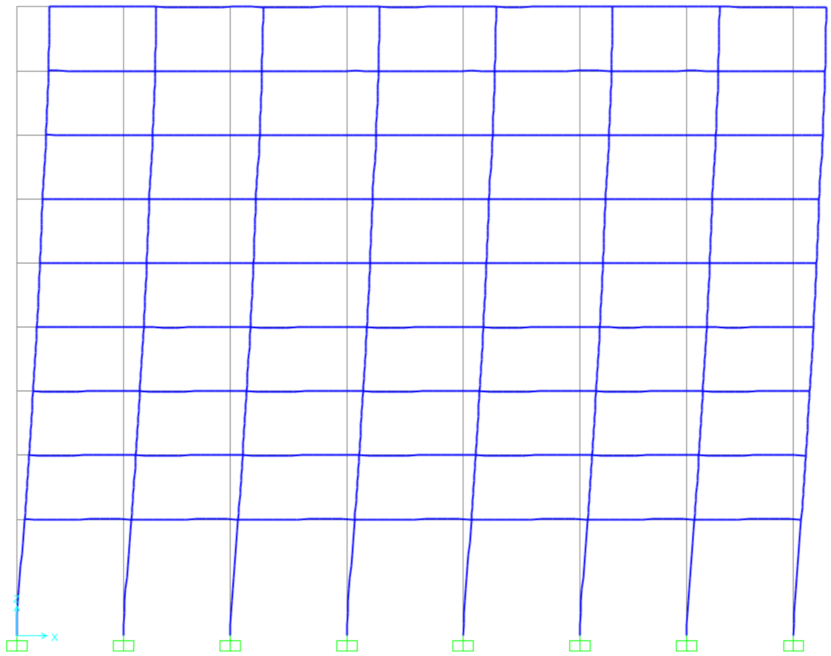


Figure 3.22. Lateral drift result of wind load from SAP2000

Table 3.11 provides the result of SAP2000 under wind load. Astana is located on non-seismic zone; consequently, only wind load was considered in order to estimate lateral drift. According code and results it can be seen that all conditions are satisfied for any type of non-structural elements.

Table 3.13. Limitation of interstorey drift

Story	U1 displacement (m)	interstorey drift	v*dr (m)
9	0.034	0.001	0.0005
8	0.033	0.003	0.0015
7	0.030	0.003	0.0015
6	0.027	0.003	0.0015
5	0.024	0.003	0.0015
4	0.021	0.004	0.002
3	0.017	0.004	0.002
2	0.013	0.005	0.0025
1	0.008	0.008	0.004
0	0.000	0	0

Anchorage and lap lengths

According the Eurocode 2 there are six factors (a1 to a6) have to be considered in order to estimate appropriate anchorage and lap lengths. By using these factor the ultimate bond failure stress can be estimated according following equations:

$$f_{bd}=2.25*\eta_1*\eta_2 f_{ctd}$$

$$l_{b,rqd}=(d/4)(\sigma_{sd}/f_{bd})$$

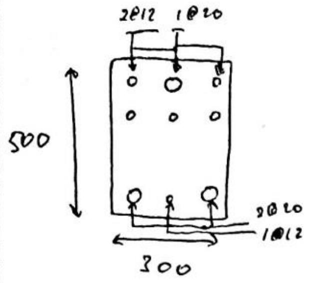
where, η_1 – is a coefficient related to the quality of the bond condition and the position of the bar during concreting.

η_2 – is related to the bar diameter, equal to 1.0 when diameter is less than 32mm and η_2=(132-d)/100 for d>32 mm.

$$l_{bd}=\alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd>}\ l_{b,min}$$

where, $l_{b,min}$ – the minimum anchorage length if no other limitations applied.

For anchorage in tension $l_{b,min}$=max(0.3 $l_{b,rqd}$; 10d; 100 mm), for anchorage in compression $l_{b,min}$ >max(0.6 $l_{b,rqd}$;10d; 100mm). According to the above equations hand calculation for anchorage is given in figure 3.22 by using factors from tables in Appendix B-2.



C 25/30

(1) Bond strength $f_{bd} = 2{,}25\, \eta_1 \eta_2 \cdot f_{ctd}$

$\eta_1 = 0{,}7$ [$\eta_1 = 1$ for good conditions, $\eta = 0{,}7$ for all cases]

$\eta_2 = 1{,}0$ for bar diameter $\varphi < 32$ mm [20 mm]

$$\frac{f_{ctm}}{\gamma_c} = f_{ctd} = \frac{2{,}6}{1{,}5} = 1{,}73 \text{ MPa}$$

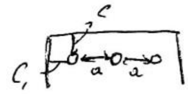
$f_{bd} = 2{,}25 \cdot 0{,}7 \cdot 1 \cdot 1{,}73 = 2{,}73$ MPa

(2) $l_{b,rqd} = (\varphi/4)(\sigma_{sd}/f_{bd}) = \frac{20}{4} \cdot \frac{500}{2{,}73} = 916$ mm

σ_{sd}: stress in steel (f_y)

(3) $l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5\, l_{b,rqd} \geq l_{b,min}$

* For straight anchorage



$\alpha_1 = 1$

$c_1 = c = 50 - \frac{20}{2} = 40$ mm

$$a = \frac{300 - 50 \cdot 2 - 2 \cdot 20 - 12}{2} = 0{,}84$$

$\sum A_{st} = 2 \cdot 8^2 \cdot 3{,}14/4 \cdot \frac{916}{200} = 207 \text{ mm}^2$

$\sum A_{st,min} = 0{,}25 \cdot 20^2 \cdot 3{,}14/4 = 78{,}5$

$A_s = 20^2 \cdot 3{,}14/4 = 314 \text{ mm}^2$

$$\lambda = \frac{\sum A_{st} - \sum A_{st,min}}{A_s} = 0{,}40$$

$K = 0{,}05$ for middle rebar

$\alpha_3 = 1 - K\lambda = 1 - 0{,}05 \cdot 0{,}4 = 0{,}98$

$\alpha_4 = \alpha_5 = 1$

$l_{bd} = 1 \cdot 0{,}84 \cdot 0{,}98 \cdot 1 \cdot 1 \cdot 916 = 764$ mm provide 800 mm $= l_{bd}$

(4) for 90° hook

$c_d = 40 < 3\varphi = 60$ mm $\alpha_1 = 1$

$\alpha_2 = 1 - 0{,}15\,(c_d - 3\varphi)/\varphi = 1{,}15 > 1$, use 1

$\alpha_3 = 0{,}98$ $\alpha_4, \alpha_5 = 1$

$l_{bd} = 1 \cdot 1 \cdot 0{,}98 \cdot 916 = 897$ mm provide 900 mm $= l_{bd}$

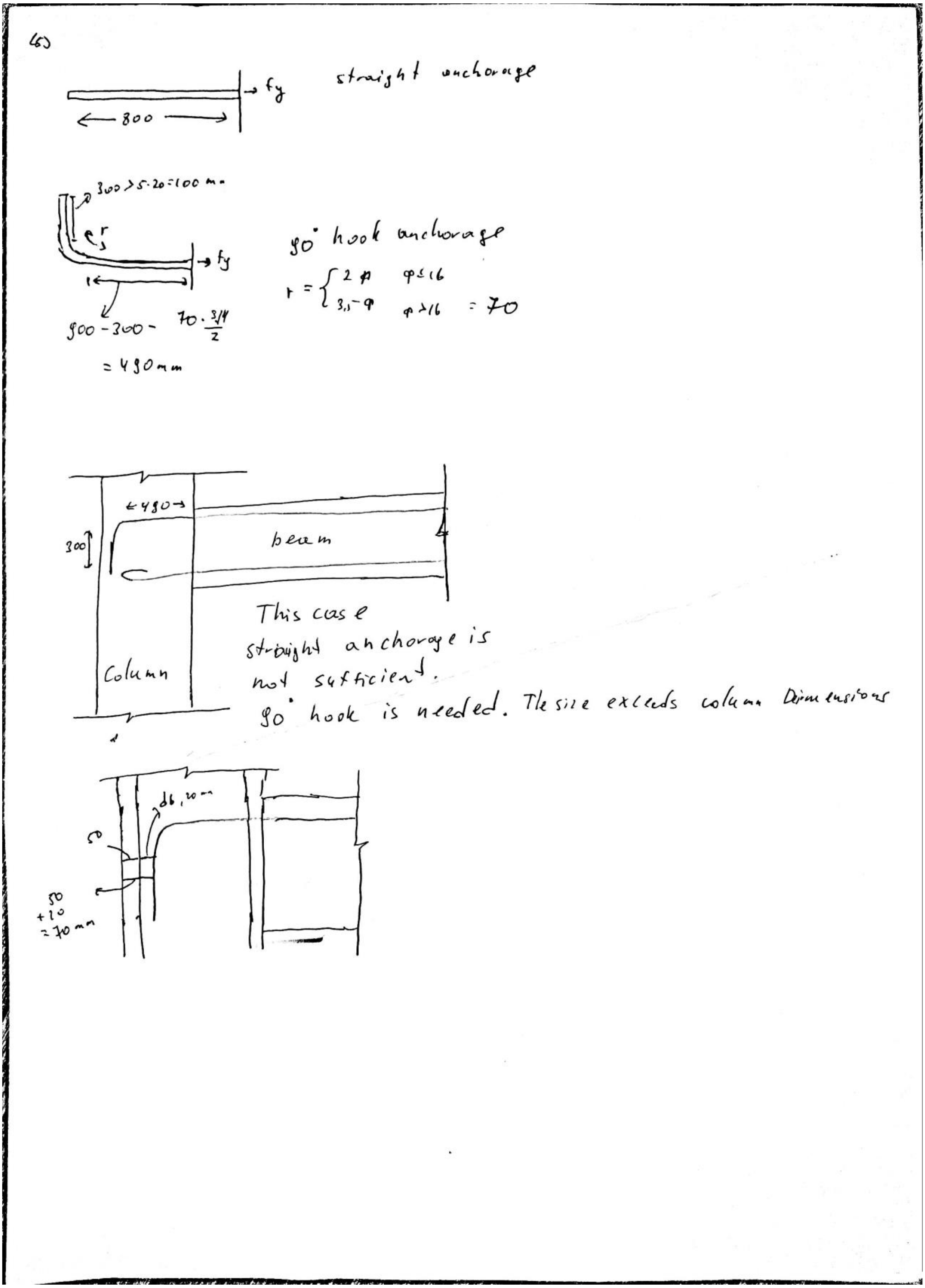


Figure 3.23. Hand calculation of anchorage lap length

Laps in members should be located in low stress applied, close to zero moment. The organization of lapped bars should look like in figure 3.23 and it can been seen that distance between lapped bars should not be greater than 4d or 50 mm, if these requirements are not met, the lap length have to be increased. The vertical distance between two laps should be larger than 0.3 of lap length, otherwise, it should considered as a lap in one section. In case of close lap of different bars the distance should not be less than 2d or 20 mm.

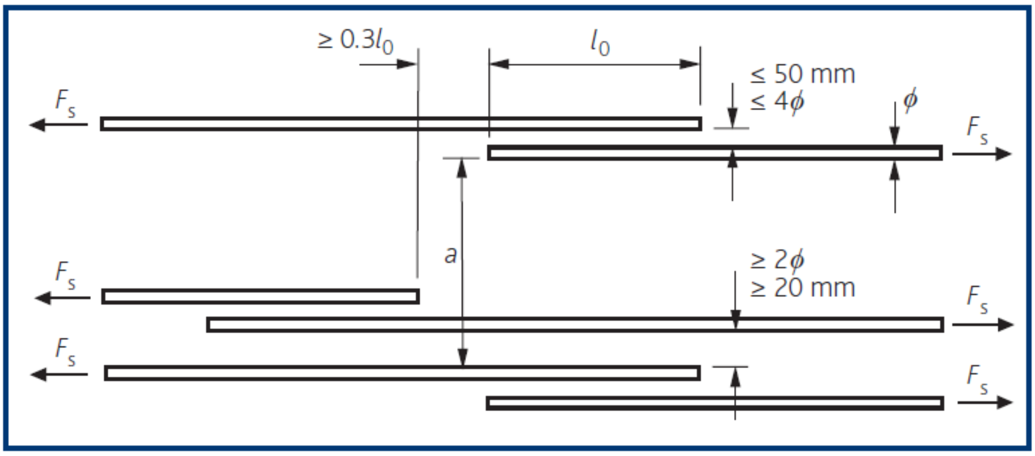


Figure 3.24. Arranging adjacent lapping bars

In case of 1 and 3 one layer contains all bars and has permissible 100% of lapped bars in tension. Consequently, according table in Appendix B-2 it should be counted as 100% lapped in one direction with a6=1.15 and increase the lap length. Figure 3.24 demostrates how should links connecttion look like.

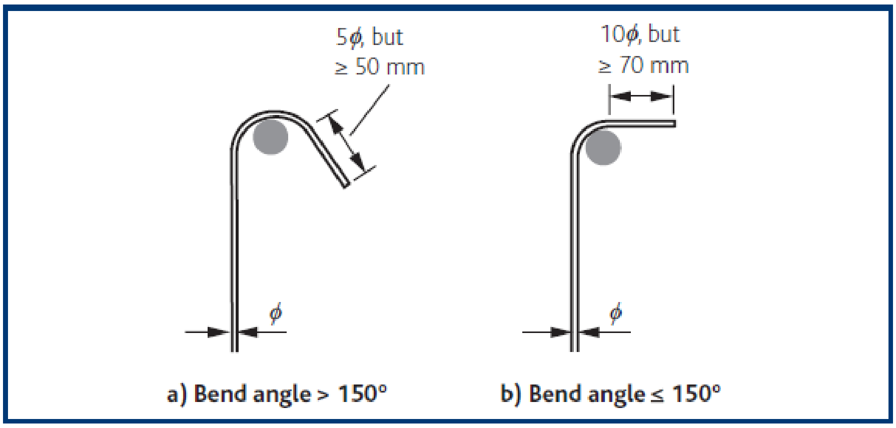


Figure 3.25. Anchorage of links

Transverse reinforcement

Bars in tension

Figure 3.25 demostrates how does transverse reinforcement for lapped splices is applied. It is stated that transverse tress occurs in the end points of the lap length, if the diameter of a bar is less than 20 mm or the percentage of lapped bars do not exceed 25% in any section. Then any transverse reinforcement needed by other criteria of Eurocode is enough to consider. In case of diameter of bar is greater than 20 mm, the transverse reinforcement should satisfy this condition $\Sigma Ast \geq 1.0As$. The transverse reinforcement should be located 90 degree to the direction of lapped reinforcement. If one location contains more than 50% of lapped bars and the distance between them is less than 10d; then transverse reinforcement should be shaped by U-bars or use link anchored to the body of the section. Moreover, one bar of transverse reinforcement has to be located outside of each end of the lap length of bars in compression and within 4d of the ends of the l_{bd} (l_o). Minimum area for transverse bars is provided in Table 3.12

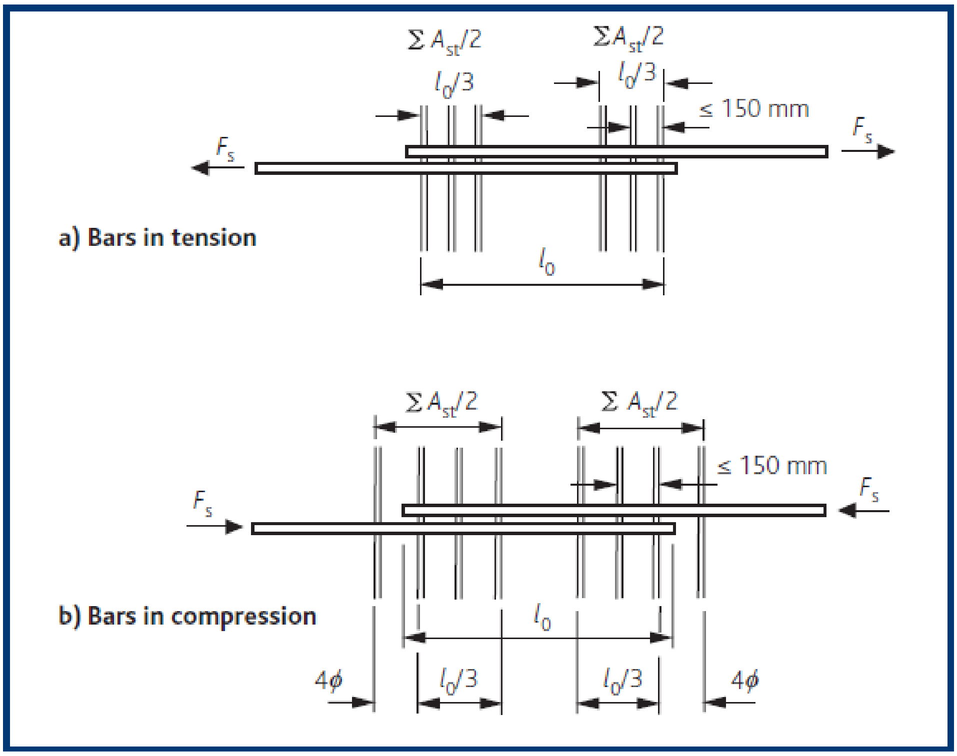


Figure 3.26. Transverse reinforcement for lapped splices

Table 3.14. Bar sizes for transverse reinforcement

lap length (mm), for transverse bars at 150 mm centres (a)	Number of bars at each lap	Bar size (mm)			
		20	25	32	40
		As=314	As=491	As=804	As=1260
<450	2	10	16	16	25
451-900	3	10	12	16	20
901-1350	4	8	10	12	16
1351-1800	5	8	8	12	16
1801-2250	6	8	8	10	12
2251-2700	7	N/A	8	10	12
(a)	For transverse bars at less than 150 mm centres use the following expreassion to calculate the required number of bars and hence required transverse bar diameter				
	Number of bars required =1+lo/(3s) where s=spacing of the transverse bars.				

The structural detailing of all beams, columns and slabs are given in Appendix B-3 with the drawings provided.

4 Geotechnical part

4.1 Soil conditions analysis

According to the requirements, our project has to be located at the territory of Nazarbayev University. Although, it was not possible to conduct any geological analysis on site, our team managed to obtain the report with local soil analysis from Mr. Chinwi Mgbere. This report was written by Ospanova G. geological engineer of "Karaganda GIIZ I K*" LTD, and it contains the data about local soil conditions such as the soil profile, groundwater table level, cone penetration test results and other relevant information about the ground at the site.

According to the report, the soil profile at the territory of Nazarbayev University is as shown in Table 4.1.

Table 4.1. Soil profile of the site

#	Soil type	Depth range (m)	Thickness range (m)	Description
1	Backfill	-	0.3-3	Loose, low density
2	Loam	0.3-3	3-5.5	Black, brown, from hard to loose, low density at average
3	Medium sand	4.6-5.5	0.8-2.4	Brown, has medium density, saturated,
4	Coarse sand	5.3-6.3	1.1-3.2	Brown and grey, has medium density, saturated
5	Sand and gravel	5.3-8.5	1.8-7.1	Brown and grey, has medium density, saturated, with interlayers of loam
6	Gravel	10.3-11.3	0.4-1.9	Greyish-brown, saturated
7	Loam	11.1-14.1	1.3-5.9	reddish, yellowish, with interlayers of clay and with insignificant presence of ballast

Here, it has to be mentioned that the depths and thicknesses of the soil layers are obtained from several locations. Thus, the values are not given as exact numbers, but as ranges. This table was further simplified as shown in Table 4.2.

Table 4.2. Simplified soil profile of the site

Soil type	Depth range (m)	Thickness range (m)	Average depth (m)	Thickness calculated (m)
Backfill	-	0.3-3	0	1.65
Loam	0.3-3	3-5.5	1.65	3.4
Medium sand	4.6-5.5	0.8-2.4	5	0.8
Coarse sand	5.3-6.3	1.1-3.2	5.8	1.1
Sand and gravel	5.3-8.5	1.8-7.1	6.9	3.9
Gravel	10.3-11.3	0.4-1.9	10.8	1.8
Loam	11.1-14.1	1.3-5.9	12.6	3.6

In Table 4.2, the depths mainly were taken as the average values, whereas thickness values are obtained from the differences between average depths. However, the thickness value of last layer was obtained from the average value of the thickness ranges. As it can be seen, the values in simplified table satisfy the ranges given in original table.

At the site, the groundwater table lies at minimum depth of 1.5 m and at maximum depth of 2.3 m. In further calculations, the average value of 1.9m will be considered as a groundwater table depth. As a result, starting from the third layer, which is medium sand, the soil layers are saturated. The physic-mechanical properties of the soil layers can be seen in Table 4.3.

Table 4.3. Physic-mechanical properties of soil layers (Ospanova 2015)

#	Soil type	Density (g/cm3)	Unit weight (kN/m3)	Cohesion (kPa)	Friction angle (ϕ)	Modulus of elasticity (mPa)
1	Backfill	1.87	18.34	-	-	-
2	Loam	1.97	19.32	18.22	22.23	-
3	Medium sand	1.92	18.83	2	35	17
4	Coarse sand	2	19.61	1	38	21
5	Sand and gravel	2	19.61	1	38	21
6	Gravel	2.05	20.1	-	-	23
7	Loam	1.93	18.93	33.78	32.01	-

The results of the electric cone penetration tests can be observed in Table 4.4.

Table 4.4. The results of electric cone penetration test (Ospanova, 2015)

#	**Soil type**	**Cone penetration resistance, q_c (mPa)**	**Frictional resistance, f_c (kPa)**
1	Backfill	2.8	122
2	Loam	1.5	38
3	Medium sand	11.8	87
4	Coarse sand	18.5	140
5	Sand and gravel	19.1	85

It has to be mentioned that Astana city is located in seismically inactive area. For this reason, the soil liquefaction effect, that may cause damage to the structure, will not be considered in this report. However, because of the cold winters of Astana city, the ground freezing occurs in the soils and reaches the average depth of 250cm. This may cause the frost heave (soil swell) of the upper layers of the ground (Rempel, et.al, 2004).

Moreover, it appears that steel is exposed to high level corrosion from soil and medium level corrosion from ground water. Whereas, reinforced concrete structures are exposed to low-medium level of aggression from soil and groundwater (Ospanova, 2015).

4.2 Foundation types

There are two general types of foundations, namely shallow and deep foundations. In order to select the appropriate type of the foundation, the factors such as soil conditions at the site, economic feasibility of foundations, ease of construction and performance under structural loads have to be considered.

4.2.1 Shallow foundations

In shallow foundations the structural load is transmitted to the surface layers of the soil. Mainly there are two types of shallow foundations, which are spread footings and mat foundations (Coduto, 2001).

Spread footing is a large concrete block that spreads the structural loads of column or bearing wall to the large area of soil. Spread footings are relatively cheap and easy to construct. Mostly, spread footings are suitable for small - medium buildings with average soil conditions or for large structures with good soil conditions (ibid).

The disadvantages of spread foundations are the high risk of failure on soils with weak surface layers, inability to support heavy loads and that it is prone to horizontal movements induced by lateral loads (ibid).

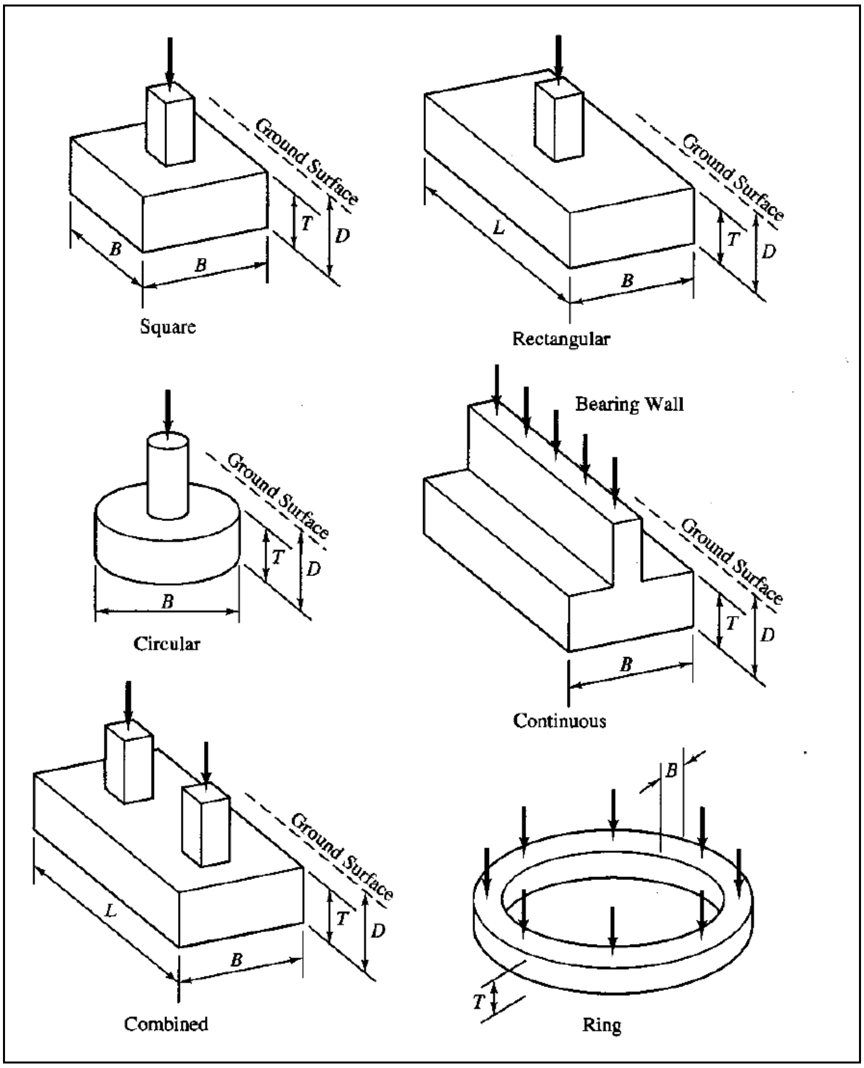


Figure 4.1. Different shapes of spread foundations (ibid).

Mat foundation is a large footing that can support columns and walls and can cover the whole footprint of the structure (Das, 2007). Mat foundations are suitable for such conditions:

- When the structural loads are large or when soil conditions are poor
- When there is a high chance of differential settlement occurrence in soil
- When the distribution of lateral loads is not uniform, causing the horizontal movements of spread foundations

The fact that mat foundation is structurally continuous and has flexural strength, allows it to cope with the current project challenges such as high horizontal loads, large structural loads and poor soil conditions (Coduto, 2001).

However, there are also difficulties related with the construction of the mat foundations. For instance, for big structures with heavy loadings, large mat foundations are required. Therefore, long construction time is required for hardening of concrete and also the large amount of material such as concrete and reinforcement steel will be used. The frost heave effect of soil can also become a serious problem by causing the damage to the footing. On top of that, the convenient weather is also necessary in order to avoid the delays in construction time.

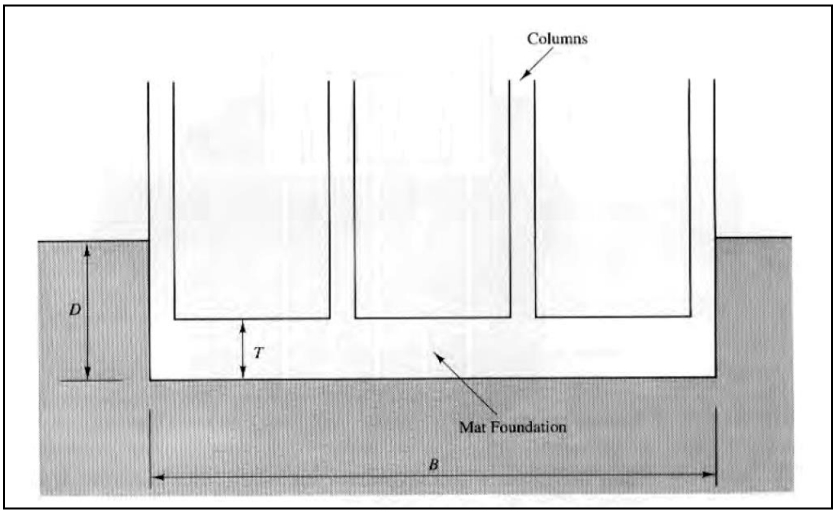


Figure 4.2. Mat foundation (ibid).

4.2.2 Deep foundations

Deep foundations transfer the structural loading to the deep layers of the soil. Due to the fact that in general, the strength of soil increases with the large depths, deep foundations are able to withstand large loads (ibid). Although, there are number of deep foundation types, two most common ones, pile foundation and drilled shaft foundations will be considered in this report.

Pile foundations are prefabricated members that are driven into the ground. The conditions where pile foundations are suitable are as follows (Das, 2007):

- When upper layers of the soil are not strong enough to support the load
- When the bedrock is not located at reasonable depth
- When the structure is prone to horizontal forces caused either by high wind or earthquake
- When the bottom of the structure is located below the groundwater level, leading to the occurrence of uplifting forces
- When soil is prone to swelling or shrinking

The main issues related with the pile foundations are the noises and vibrations produced by the hammers during the pile driving operations. The vibrations may cause some damages to the nearby structures, whereas the noise can be problematic for the residents that occupy buildings nearby. During the driving action of piles it is advised to provide an inspection, in order to check that piles are driven properly and do not suffer any significant damage. In addition, during the construction of pile foundations on the clayey soils, the ground heaving might be generated leading to the lateral movements of other driven piles (ibid).

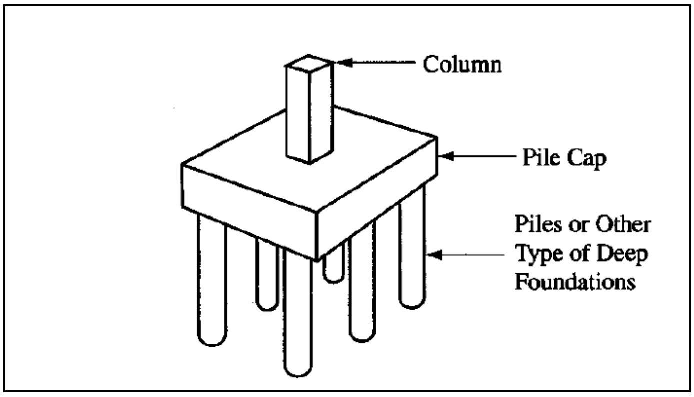


Figure 4.3. Group pile foundation (Coduto, 2001)

Drilled shaft foundations are in-situ constructed deep foundations which involve three steps of construction. Firstly, the hole is drilled into the ground. After that, the reinforced steels are installed and finally poured with concrete (ibid). Although, drilled shafts are used in situations similar to pile foundations, they have some advantages over pile foundations such as:

- In areas with hard soil layers, it is easier to construct drilled shafts than to drive piles
- Unlike pile foundation, do not produce noise and vibrations during construction
- One single drilled shaft can be constructed instead of group piles

The main drawbacks of drilled shafts are the complex construction, demand for close supervision and the convenient weather conditions during concreting, and high probability of causing ground loss and thus damage to other facilities located nearby (Das, 2007).

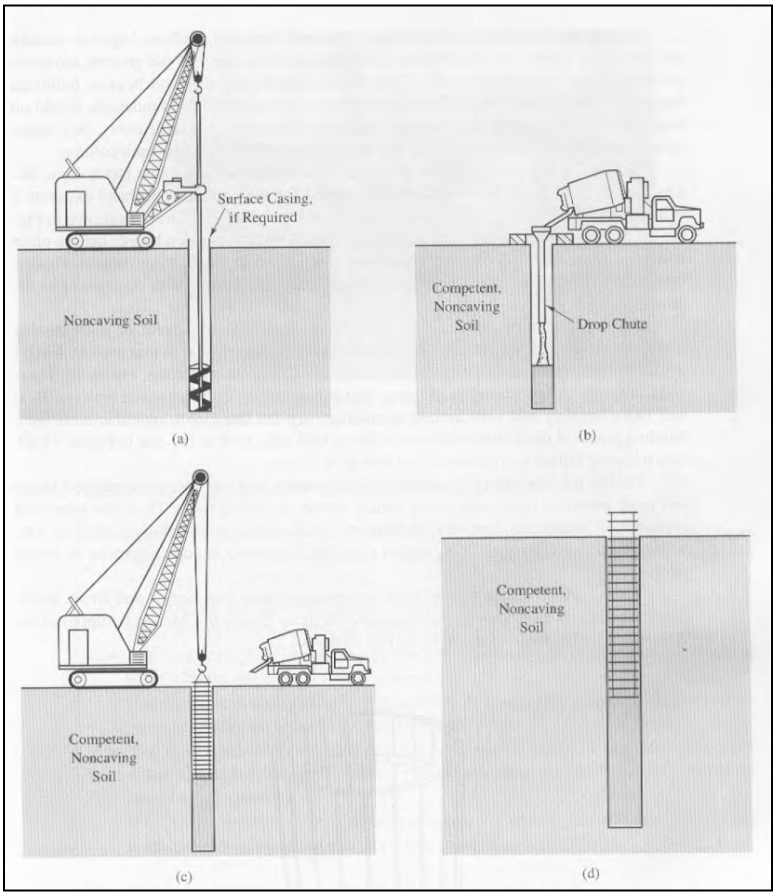


Figure 4.4. Drilled shaft construction (dry method): a) Drilling a bore; b) Concreting the bottom of the shaft; c) Placing steel reinforcements; d) Finishing the concreting (Coduto, 2001)

4.3 Choosing foundation type

In order to choose the suitable foundation for the project several factors such as the cost, ease of construction and suitability for the soil at the site will be considered. In the Table 4.5, the type and cost of construction equipment used for each type of foundation are provided. The values were taken as average and were obtained from the advertisement site named Satu.kz.

Table 4.5. Cost of equipment for each type of foundations

Foundation type	Equipment used
Spread footing	Excavator (backhoe) – 9000 KZT/h Concrete mixer truck – 1200 KZT/m^3 Autocrane – 8000 KZT/h
Mat foundation	Excavator (backhoe) – 9000 KZT/h Concrete mixer truck – 90000 KZT/h Autocrane – 8000 KZT/h or concrete pump – 25000 KZT/h
Pile foundation	Autocrane – 8000 KZT/h Concrete mixer truck – 90000 KZT/h Pile driving rig – 1500 KZT /m Pile transporting truck – 5000 KZT /h
Drilled shaft	Autocrane – 8000 KZT/h Concrete mixer truck – 90000 KZT/m^3 Drill rig – 1000 KZT/m

It has to be mentioned that usage of spread foundations in the project is undesirable, as the upper layers of the site soil profile are weak and are exposed to freezing and swelling. Moreover, the structure is large for the spread foundations and very likely to be exposed to high lateral loadings.

Mat foundations are more convenient for this project than spread footings, as they can be constructed on weak soils and withstand lateral movements caused by non-uniform horizontal loadings. However, some problems may still arise for the mat foundations. Firstly, because of the frost heave effect, foundation can suffer some damage. Also, due to the large size of the structure and wind loadings, the size of the mat foundations is more likely be large, leading to long concrete hardening time. Moreover, Astana climate with lots of precipitation, may also affect the construction time. In terms of cost of equipment used, the large mat foundation will require high amount of concrete and operation time, thus, significantly increasing the cost of equipment. So, even though, the mat foundation is suitable for soils with weak surface layers, its application for the project will be problematic in terms of frost heave effect, high cost and long construction time.

Pile foundations are suitable for the project as they can manage main issues of the project as lateral forces, weak surface layers of soil and frost heave, by transferring loads to deeper layers of the soil. In addition, the action of driving forces are relatively fast and do not require special excavations. Moreover, due to the fact that the site is located near Kabanbay Batyr Avenue, the noise and vibrations will not be problematic, as there are no dormitories, academic facilities or vulnerable structures nearby. The cost of equipment can be high if group piles will be used. As for group piles, the number of piles increase significantly, adding the cost for transportation and for driving operations. Moreover, the proper inspection has to be conducted during the pile driving.

Drilled shafts cope with the main concerns of the project as weak upper layers of soil, frost heave and lateral loads of the project in the same way as pile foundations. However, the construction of drilling shaft foundations may be problematic for this particular project. Firstly, the fact that water table is located only at 1.9m depth, will lead to the difficulties in construction of drilled shafts. Also, as it was previously mentioned, the groundwater and soil have high level of corrosive aggression to the steel. And this may affect the reinforcement of drilled shaft foundations during the construction operation, which may further lead to inadequate performance of the foundation. Moreover, the weather conditions of Astana city with high amount of precipitations may become another issue during concreting operations, causing the delays during the project construction. In addition, the need for the close supervision increases the risk of error occurrence during the construction operation. However, in terms of construction equipment cost drilled shafts appear to be more economic than pile foundations. This happens because only one bore has to be drilled, and no money is spent on transportation of precast piles.

So, it can be said that for this project pile foundations are the most appropriate option in terms of easy construction and suitability for soil conditions. Therefore, the further calculations and design procedures will be performed for pile foundations.

Pile foundations are usually made from three types of materials: steel, timber and concrete. In this project, the concrete pile foundations will be used as high corrosive aggression and high groundwater level decrease the strength of steel and timber piles respectively. In this project, the reinforced concrete piles with squared cross section will be considered.

4.4 Foundation design

For this project our team decided that pile foundations will be the most suitable option. Before starting the calculations, we have to define the loads that will be applied directly to the foundations. The loads such as axial loads, shear loads and moments at the bottom columns were obtained from the SAP software. The values of the loadings at the columns vary greatly, as columns under tower receive much higher loads than columns under the ground floor. Therefore, it was decided to divide the loadings into three groups as shown in Table 4.6.

Table 4.6. Load conditions of the building.

Load groups	**Condition 1**			**Condition 2**		
	Axial load, N (kN)	Shear load, Hx (kN)	Moment, My (kNm)	Axial load, N (kN)	Shear load, Hx (kN)	Moment, My (kNm)
Group 1	8097.78	0.527	1.5295	4167.308	113.176	400.4933
Group 2	3244.248	78.521	260.6941	2548.003	110.197	395.5296
Group 3	489.76	176.116	452.272	313.015	180.321	455.669

Condition 1 in table 4.6 corresponds to state when the axial load applied to the column is the highest. Whereas, condition 2 is the state with maximum moment and shear applied to the columns. Due to the fact that there are three groups of loads, three different foundation designs will be developed. Group 1 loads will be used for interior foundations under the tower, while group 2 loads will be applied for the design of tower's edge foundations. And finally, for foundations under the ground floors design will be developed according to group 3 loads.

Further foundation design procedures will be applied for the exterior columns under the tower with group 1 loads.

4.4.1 Bearing capacity

The first step is to calculate the ultimate load carrying capacity of one pile. In pile foundations load is transferred through pile point bearing and friction of soil-pile interface. So, the ultimate load carrying capacity of a pile can be calculated through following formula (Das, 2007):

$$Q_u = Q_p + Q_s \tag{4.1}$$

where

Q_u = load carrying capacity of a pile

Q_p = load carrying capacity of the pile point

Q_s = frictional resistance of the pile

Allowable load for a pile can be found by dividing the ultimate load carrying capacity of the pile by safety factor, which varies between 2.5 and 4 (ibid.).

$$Q_{all} = \frac{Q_u}{FS} \tag{4.2}$$

The safety factor of 3 is chosen for this project.

There are three ways of estimating Q_p, namely Meyerhof's method, Vesic's method and Coyle and Castello's method (ibid.). The average value of these three methods will be used for calculations.

Meyerhof's method for estimating the point bearing capacity of a pile in sand:

$$Q_p = A_p q_p = A_p q' N_q^* \leq A_p q_l \tag{4.3}$$

Where, A_p = pile tip area

c' = cohesion of the soil at pile tip

q_p = unit point resistance

q' = effective vertical stress at pile tip level

q_l = limiting point resistance, $q_l = 0.5 p_a N_q^* \tan\phi'$

ϕ' = effective soil friction angle of the bearing stratum

p_a = atmospheric pressure (=100 kN/m^2)

N_q^* = bearing capacity factor

N_q^* is found using Figure 4.5.

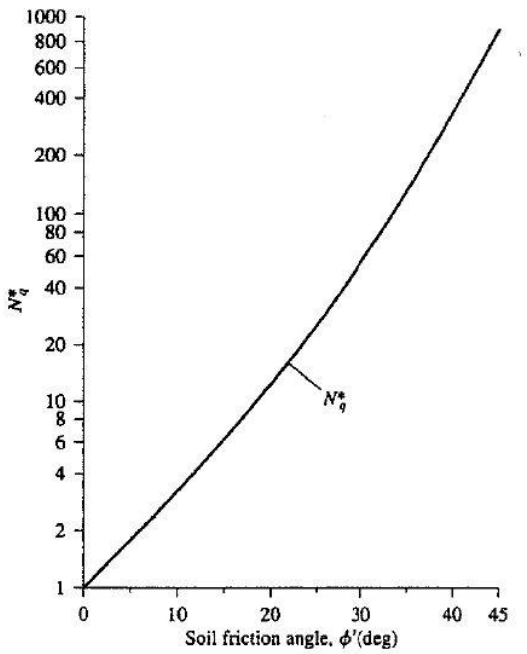


Figure 4.5. Variation of Nq* according to friction angle ϕ'

Vesic's method for estimating point bearing capacity of pile in sand:

$$Q_p = A_p q_p = A_p \sigma_o{}' N_\sigma^* \tag{4.4}$$

Where, A_p = pile tip area

σ_o' = mean effective normal ground stress at the level of the pile point, σ_o'= q'(1+2K_o)/3

K_o = earth pressure coefficient at rest = 1-sinϕ'

q_p = unit point resistance

N_σ* = bearing capacity factor, N_σ*=f(I_{rr})

I_{rr} = reduced rigidity index for the soil, I_{rr}= I_{rr}/(1+ I_r Δ)

I_r = rigidity index, I_r=E_s/(2(1+μ_s)q'tanϕ')

μ_s = Poisson's ratio of soil, μ_s=0.1+0.3(ϕ'-25)/20

E_s = modulus of elasticity of soil

Δ = average volumetric strain in the plastic zone below the pile point, Δ=0.005q'(1-(ϕ'-25)/20)/p_a

q' = effective vertical stress at pile tip level

q_l = limiting point resistance, q_l=0.5$p_a N_q$*tanϕ'

ϕ' = effective soil friction angle of the bearing stratum

p_a = atmospheric pressure (=100 kN/m^2)

N_σ* is found using Table 4.7.

Table 4.7. Nσ* based on Theory of Expansion Cavities

	I_{rr}									
ϕ'	10	20	40	60	80	100	200	300	400	500
35	37.65	53.30	75.22	91.91	105.92	118.22	166.14	202.64	233.27	260.15
	27.36	38.32	53.67	65.36	75.17	83.78	117.33	142.89	164.33	183.16
36	39.37	55.99	79.39	97.29	112.34	125.59	177.38	216.98	250.30	279.60
	29.60	41.68	58.68	71.69	82.62	92.24	129.87	158.65	182.85	204.14
37	41.17	58.81	83.77	102.94	119.10	133.34	189.25	232.17	268.36	300.26
	32.02	45.31	64.13	78.57	90.75	101.48	143.61	175.95	203.23	227.26
38	43.04	61.75	88.36	108.86	126.20	141.50	201.78	248.23	287.50	322.17
	34.63	49.24	70.03	86.05	99.60	111.56	158.65	194.94	225.62	252.71
39	44.99	64.83	93.17	115.09	133.66	150.09	215.01	265.23	307.78	345.41
	37.44	53.50	76.45	94.20	109.24	122.54	175.11	215.78	250.23	280.71
40	47.03	68.04	98.21	121.62	141.51	159.13	228.97	283.19	329.24	370.04
	40.47	58.10	83.40	103.05	119.74	134.52	193.13	238.62	277.26	311.50
41	49.16	71.41	103.49	128.48	149.75	168.63	243.69	302.17	351.95	396.12
	43.74	63.07	90.96	112.68	131.18	147.59	212.84	263.67	306.94	345.34
42	51.38	74.92	109.02	135.68	158.41	178.62	259.22	322.22	375.97	423.74
	47.27	68.46	99.16	123.16	143.64	161.83	234.40	291.13	339.52	382.53
43	53.70	78.60	114.82	143.23	167.51	189.13	275.59	343.40	401.36	452.96
	51.08	74.30	108.08	134.56	157.21	177.36	257.99	321.22	375.28	423.39
44	56.13	82.45	120.91	151.16	177.07	200.17	292.85	365.75	428.21	483.88
	55.20	80.62	117.76	146.97	172.00	194.31	283.80	354.20	414.51	468.28
45	58.66	86.48	127.28	159.48	187.12	211.79	311.04	389.35	456.57	516.58
	59.66	87.48	128.28	160.48	188.12	212.79	312.03	390.35	457.57	517.58
46	61.30	90.70	133.97	168.22	197.67	224.00	330.20	414.26	486.54	551.16
	64.48	94.92	139.73	175.20	205.70	232.96	342.94	429.98	504.82	571.74
47	64.07	95.12	140.99	177.40	208.77	236.85	350.41	440.54	518.20	587.72
	69.71	103.00	152.19	191.24	224.88	254.99	376.77	473.42	556.70	631.25
48	66.97	99.75	148.35	187.04	220.43	250.36	371.70	468.28	551.64	626.36
	75.38	111.78	165.76	208.73	245.81	279.06	413.82	521.08	613.65	696.64
49	70.01	104.60	156.09	197.17	232.70	264.58	394.15	497.56	586.96	667.21
	81.54	121.33	180.56	227.82	268.69	305.37	454.42	573.38	676.22	768.53
50	73.19	109.70	164.21	207.83	245.60	279.55	417.82	528.46	624.28	710.39
	88.23	131.73	196.70	248.68	293.70	334.15	498.94	630.80	744.99	847.61

Coyle and Castello's method for estimating the point bearing capacity of a pile:

$$Q_p = q'N_q^* A_p \tag{4.5}$$

Where, A_p = pile tip area

q' = effective vertical stress at pile tip level

N_q^* = bearing capacity factor

N_q^* is found using Figure 4.6.

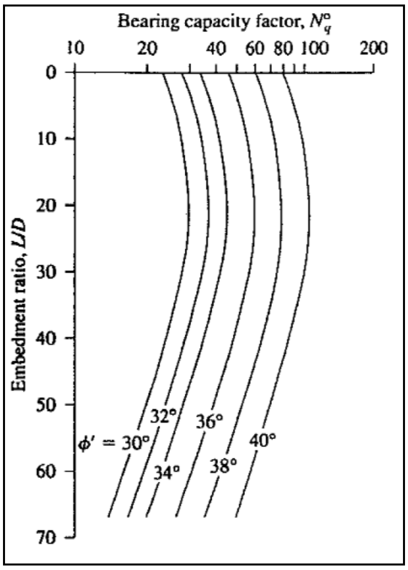


Figure 4.6. Variation of Nq* according to L/D (Coyle and Castello method)

In the case of Q_s, the cone penetration test results will be used for calculations. Frictional resistance can be estimated with formula given below (ibid.):

$$Q_s = \Sigma p \Delta L f \tag{4.6}$$

Where, p = pile section perimeter

ΔL = incremental pile length over which p and f are taken to be constant

f = unit friction resistance, (f=α'f_c). Variation of α' is found using Figures 4.7.

f_c = frictional resistance

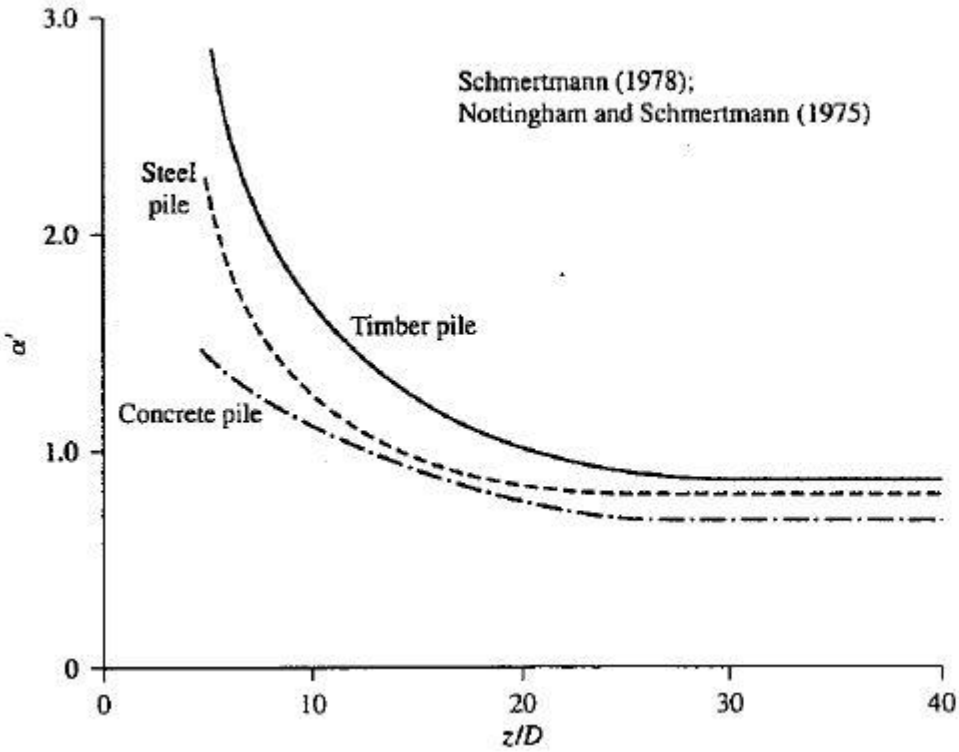


Figure 4.7. Variation of α' with embedment ratio for piles in sand: electric cone penetrometer

The load carrying capacity calculations will be performed for piles with different combinations of lengths and thicknesses. The pile width varies from 0.3m to 0.6m, with increments of about 0.05m. It is known that the gravel layer is located below 10.8m, and that there is a loam layer under the gravel layer. The maximum depth of 10.8m will be considered in order to maintain the high pile tip resistance and also because piles might be damaged during the driving action into the gravel layer.

With the help of Excel program the ultimate load carrying capacity of piles with different thicknesses were calculated. The results can be observed in Appendix C. These values were divided by safety factor of 3 in order to derive the allowable load carrying capacity of one pile (Das, 2007). Further, the allowable load carrying capacities of group piles are calculated. It is found by multiplying the Q_{all} by group pile efficiency and by number of piles in a group.

Group pile efficiency can be calculated with the following formula (ibid.):

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{2(n_1+n_2-2)d+4B}{pn_1n_2} \tag{4.7}$$

Where, η = group efficiency

$Q_{g(u)}$ = ultimate load-bearing capacity of the group pile

Q_u = ultimate load-bearing capacity of each pile without group effect

B = width of a pile

d = minimum center-to-center pile spacing, typically d=3B

n_1, n_2 = number of rows and columns in pile groups

If η<1, then Qg(u)= ηΣQu

If η≥1, then Qg(u)= ΣQu

The group efficiency values for typical group pile arrangements can be seen in table 4.8.

Table 4.8. Group efficiency values for typical group pile arrangements.

Pile width	**Group pile arrangement**				
	1x2	***2x2***	***3x1***	***3x2***	***3x3***
0.3	1.25	1	1.333	0.9167	0.778
0.35	1.25	1	1.333	0.9167	0.778
0.4	1.25	1	1.333	0.9167	0.778
0.45	1.25	1	1.333	0.9167	0.778
0.5	1.25	1	1.333	0.9167	0.778
0.55	1.25	1	1.333	0.9167	0.778
0.6	1.25	1	1.333	0.9167	0.778

For foundations under the interior columns of the tower the maximum axial load is about 8100 kN. According to the tables in appendix C, 3x2 group piles with width of 0.6m at depth larger than 9m or 3x3 group piles with width of 0,55m at depth of 7m or more can withstand the applied axial load. Finally, 3x2 group piles with width of 0.6m were chosen as more economical option, whereas the depth will be considered as 10.8m for more conservative design.

Table 4.9. Pile capacity for pile with width of 600mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)
6	6132.884	3248.586	1886.956	1858.450	2331.330	1065.286	3396.616	1132
7	6947.852	3248.586	2020.954	2195.641	2488.394	1353.766	3842.160	1281
8	7762.820	3248.586	2140.124	2486.791	2625.167	1516.966	4142.133	1381
9	8577.788	3248.586	2247.897	2822.129	2772.871	1680.166	4453.037	1484

10	9392.756	3248.586	2351.196	3130.919	2910.233	1843.366	4753.599	1585
11	10088.214	3248.586	2426.799	3348.243	3007.876	1973.926	4981.802	1661

Table 4.10. Pile capacity of group piles with width of 600mm.

Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
3396.616	1132	2264.411	4528.82166	6227.356	7927.702
3842.160	1281	2561.44	5122.879515	7044.215	8967.601
4142.133	1381	2761.422	5522.8438	7594.186	9667.738
4453.037	1484	2968.691	5937.3823	8164.198	10393.39
4753.599	1585	3169.066	6338.132422	8715.249	11094.9
4981.802	1661	3321.201	6642.402747	9133.636	11627.53

4.4.2 Lateral load for group piles

The next step is to determine the lateral load resistance of the group piles. Firstly, the applied lateral load on a single pile has to be estimated with the following formula (Design of piled foundations, 2010):

$$H = \frac{(H_x^2 + H_y^2)^{1/2}}{R} \qquad (4.8)$$

Where, H = lateral load on a single pile

R = number of piles in a group

H_x= horizontal load on a pile cap in x direction

H_y= horizontal load on a pile cap in y direction

From the table 4.6, it is known that maximum horizontal load in x direction is about 111 kNm for foundations under interior columns of the tower, whereas in y direction it is zero. By using 4.8, lateral load on a single pile:

$$H = \frac{(114^2 + 0)^{0.5}}{6} = 19\ kN$$

Now, the allowable lateral load has to be estimated with the Brom's method (Das, 2007). Firstly, the characteristic length of the soil pile system has to be determined:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \qquad (4.9)$$

Where, T = characteristic length of the pile-soil system

E_p = modulus of elasticity in the pile material, $E_p = 4700\sqrt{f'_c}$

f_c = specified 28-day compressive strength of concrete

I_p = moment of inertia of pile section, $I_p = \frac{B^4}{12}$

n_h = constant of modulus of horizontal subgrade reaction

n_h is found using table 4.11

Table 4.11. Constant modulus of horizontal subgrade reaction for different soil types

Soil	n_h	
	kN/m³	lb/in³
Dry or moist sand		
Loose	1800–2200	6.5–8.0
Medium	5500–7000	20–25
Dense	15,000–18,000	55–65
Submerged sand		
Loose	1000–1400	3.5–5.0
Medium	3500–4500	12–18
Dense	9000–12,000	32–45

Assuming n_h = 4000kN/m^3 and f_c = 40MPa, and then estimating I_p = 0.00521m^4 and E_p =29725.41kN/m^2, T can be found with equation 4.9:

$$T = \sqrt[5]{\frac{29725.41 * 0.00521}{4000}} = 0.522m$$

If L ≥ 5T, pile is considered as a long pile. In our case, the pile is long, as L/T = 19.16. Next step is to determine the yield moment, in order to find the ultimate lateral resistance from figure 4.8.

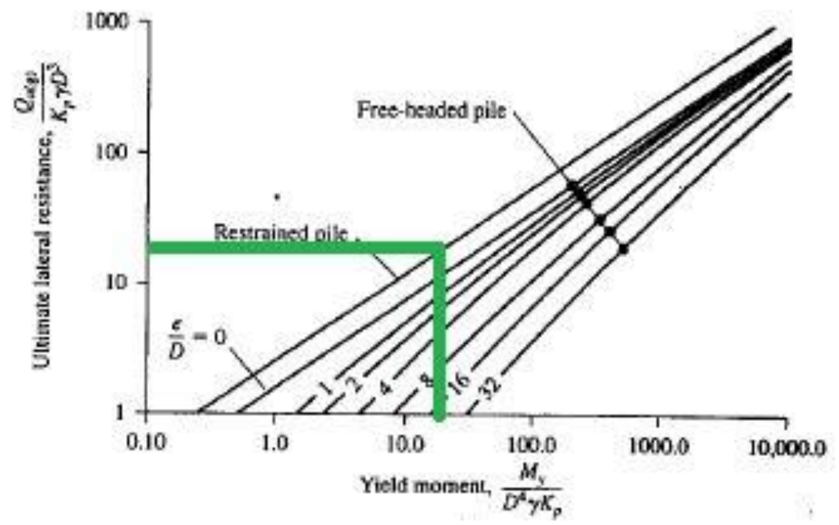


Figure 4.8. Brom's solution for ultimate lateral resistance of long piles in sand.

Yield moment is found by the following term:

$$Yield\ moment = \frac{M_y}{\gamma K_p B^4} \tag{4.10}$$

Where, M_y = yield moment for a pile

γ = unit weight of soil

K_p = Rankine passive earth pressure coefficient, $K_p = \tan^2(45+\phi'/2)$

B = width of a pile

In this case, ϕ'= 38 degrees, γ = 19.61, M_y = 401kNm and B = 0.6m, by finding the K_p and using 4.10 we find:

$$Yield\ moment = \frac{401}{19.61 * 4.2 * 0.6^4} = 37.57$$

From figure 4.8, we can see that $\frac{Q_{u(g)}}{\gamma K_p B^3}$ term is approximately equal to 20. Hence, the ultimate lateral resistance can be estimated:

$$Q_{u(g)} = 20 * 19.61 * 4.2 * 0.6^3 = 355.8\ kN$$

Allowable lateral resistance can be found by dividing $Q_{u(g)}$ by safety factor of 3.

$$Qall(g) = \frac{Q_{u(g)}}{FS} = \frac{355.8}{3} = 118.6\ kN$$

As $Q_{all(g)}$ = 118.6 is more than lateral load on a single pile (19 kN), which means that pile can resist the lateral load.

4.4.3 Pile cap dimensions design

According to Das (2007), the pile-to-pile spacing varies between 2.5B to 3.5B. In current project, the spacing between piles will be taken as 2.5B. The distance from pile center to edge and the height of the pile cap are advised to be 1.5B. As the width of the pile is 0.6m, the pile-to-pile spacing will be equal to 1.5m, pile to edge distance and the height of pile cap will be 0.9m. As a result, a pile cap with dimensions 4800mm x 3300mm x 900mm is obtained as shown in figure 4.9.

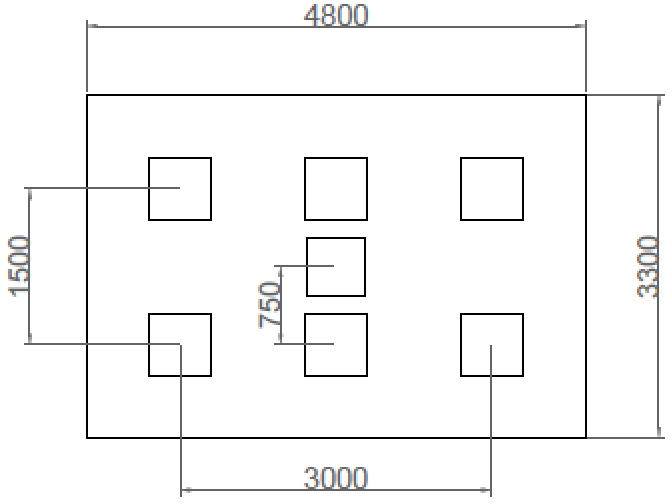


Figure 4.9. 3x2 pile cap dimensions

4.4.4 Vertical load for group piles

The slab for the first floor is located at 2.7m above ground level, and the distance between slab and pile cap is filled with backfill.

W = pile cap weight + backfill weight + slab weight = 4.8m*3.3m*0.9m*24kN/m3 + 4.8m*3.3m*1.8m*20kN/m3 + 4.8m*3.3m*1.5m*24kN/m3 = 969.408 kN.

Total vertical load on group of piles will be (Design of piled foundations, 2010):

$$P = N + W = 8100 + 969.41 = 9069.41\ kN \leq 9133.64\ kN\ OK \qquad (4.11)$$

In case, P is greater than allowable load carrying capacity of group piles, pile cap has to be recalculated with more number of piles or with wider piles.

Now, the maximum vertical load on a single pile can be found with the following formula (ibid.):

$$Q = \left(\frac{P}{R}\right) \pm \left(\frac{M_{xx}y}{I_{xx}}\right) \pm \left(\frac{M_{yy}x}{I_{yy}}\right) \qquad (4.12)$$

Where, Q = vertical load on single pile

P = Total vertical load on group of piles

R = number of piles in group

M_{xx} = moment on a pile cap about x-x axis, $M_{xx} = M_x + Ne_y + H_y h + M_x^*$

M_{yy} = moment on a pile cap about y-y axis, $M_{yy} = M_y + Ne_x + H_x h + M_y^*$

x = distance from column centre to pile centre along x axis

y = distance from column centre to pile centre along y axis

$I_{xx} = \sum y^2$ about x-x axis

$I_{yy} = \sum x^2$ about y-y axis

M_x = moment on a pile cap about x axis

N = vertical load exerted from column

e_y = eccentricity of vertical load on a pile cap in y direction

H_y = horizontal loads on pile cap in y direction

h = overall height of pile cap

M_x^* = moment about the x axis due to eccentric surcharge on pile cap

M_{xx} = 0 in this project, as M_x, e_y, H_y and M_x^* are all equal to zero.

$$M_{yy} = M_y + Ne_x + H_x h + M_y^* = 1.55 + 0 + 0.55 * 0.9 + 0 = 2.045\ kNm$$

$$Q_{max} = \left(\frac{P}{R}\right) + \left(\frac{M_{xx}y}{I_{xx}}\right) + \left(\frac{M_{yy}x}{I_{yy}}\right) = \left(\frac{9069.41}{6}\right) + 0 + \left(\frac{2.045 * 1.5}{4 * 1.5^2}\right) = 1511.91\ kN$$

$$Q_{min} = \left(\frac{P}{R}\right) - \left(\frac{M_{xx}y}{I_{xx}}\right) - \left(\frac{M_{yy}x}{I_{yy}}\right) = \left(\frac{9069.41}{6}\right) - 0 - \left(\frac{2.045 * 1.5}{4 * 1.5^2}\right) = 1511.23\ kN$$

Maximum vertical load on single pile is equal to 1511.91 kN, and it is less than the allowable load capacity on a single pile (1660.6 kN). This means that piles can support the vertical load applied on the foundation.

In case, when Q_{max} will be larger than Q_{all}, design with piles of larger width or with more piles has to be recalculated.

4.4.5 Pile cap reinforcement design

In order to perform the reinforcement design, the bending moments on critical sections of pile cap have to be calculated. Critical sections are shown in figure 4.10.

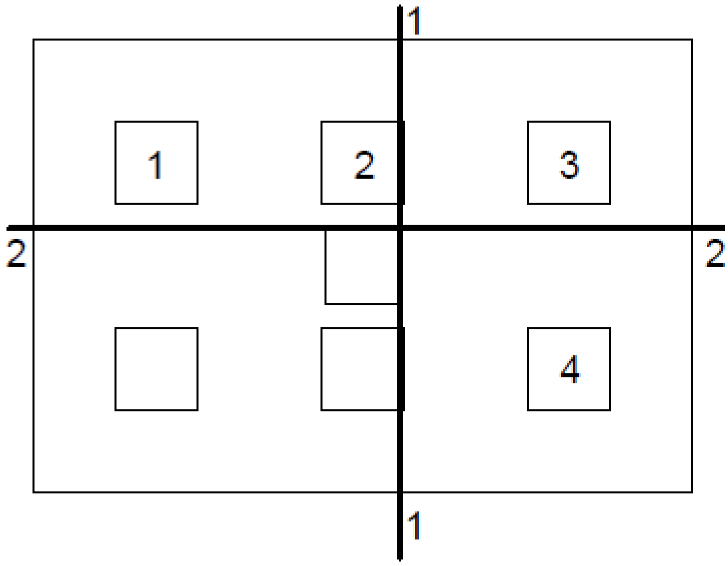


Figure 4.10. Critical sections of 3x2 pile foundation

Bending moments about 1-1 section are composed of two components as follows (ibid.):

$$M_{11} = M'_{11} + M''_{11} \tag{4.13}$$

Where, M_{11} = total bending moment about 1-1 section of pile cap

M_{11}' = bending moment due to dead load about 1-1 section of pile cap

M_{11}'' = bending moment due to pile reactions about 1-1 section of pile cap

Dead load of pile cap + backfill + slab = 0.9*24 + 1.8*20 + 0.15*24 = 61.2 kN/m^2

$$M'_{11} = \frac{3.3 * 61.2 * 2.125^2}{2} = 455.988\ kNm$$

$$M'_{22} = \frac{4.8 * 61.2 * 1.375^2}{2} = 277.695\ kNm$$

$M''_{11} = 1.225 * (Q_3 + Q_4) = 1.225 * (1511.91 + 1511.91) = 3704.18\ kNm$

$M''_{22} = 0.475 * (Q_1 + Q_2 + Q_3) = 1.225 * (1511.23 + 1511.57 + 1511.91)$
$= 2153.99\ kNm$

$M_{11} = 455.988 + 3704.18 = 4160.16\ kNm$

$M_{22} = 277.695 + 2153.99 = 2431.98\ kNm$

After finding the bending moments, the effective depths can be estimated. Two sides of the pile cap will have different reinforcement designs.

<u>Reinforcement for b = 3.3m</u>

Assume that the cover of reinforcement is 90mm and the diameter of bars is 20mm. Effective depth of pile cap is calculated as follows:

$$d_x = 900(height) - 90(cover) - 10(half\ bar\ diameter) = 800mm$$

The concrete with C40 grade and 40N/m^2 compressive strength, and steel with 460 N/m^2 yield strength will be used in this project.

$$K = \frac{M_{11}}{f_{cu}bd_x^2} = \frac{4160.16 * 10^6}{40 * 3300 * 800^2} = 0.0492$$

$$z = d_x\left(0.5 + \sqrt{0.25 - \frac{K}{0.9}}\right) = d_x\left(0.5 + \sqrt{0.25 - \frac{0.0492}{0.9}}\right) = 0.94d_x \leq 0.95d_x$$

$$A_{st} = \frac{M_{11}}{0.87 * f_y z} = \frac{4160.16 * 10^6}{0.87 * 460 * 0.94 * 800} = 13.8 * 10^3 mm^2$$

Area of 20mm bar diameter is 314mm^2, and number of bars equals to 13800/314=43.95. Thus, 44 bars with 20mm diameter will be used on side with b=3300mm.

The spacing between bars can be calculated as follows:

$$Bar\ spacing = \frac{3300(width) - 2 * 90(cover) - 20 * (44 - 1)}{44 - 1} = 52.56$$

The spacing between bar will be taken as 55mm.

<u>Reinforcement for b = 4.8m</u>

Assume that the cover of reinforcement is 90mm and the diameter of bars is 16mm.

$$d_y = 900(height) - 90(cover) - 20(bar\ diameter) - 8(half\ bar\ diameter) = 782mm$$

$$K = \frac{M_{22}}{f_{cu}bd_y^2} = \frac{2431.98 * 10^6}{40 * 4800 * 782^2} = 0.0207$$

$$z = d_x\left(0.5 + \sqrt{0.25 - \frac{0.0207}{0.9}}\right) = 0.97d_x > 0.95d_x; z = 0.95d_x$$

$$A_{st} = \frac{M_{22}}{0.87 * f_y z} = \frac{2431.98 * 10^6}{0.87 * 460 * 0.95 * 782} = 8.2 * 10^3 mm^2$$

Area of 16mm bar diameter is 201mm^2, and number of bars equals to 8200/201=40.79. Thus, 41 bars with 16mm diameter will be used on side with b=4800mm.

$$Bar\ spacing = \frac{4800(width) - 2 * 90(cover) - 16 * (41 - 1)}{41 - 1} = 99.5$$

The spacing between bar will be taken as 100mm.

4.4.6 Punching shear stress in pile cap

$$U_1 = perimter\ of\ column = 2200\ mm$$

$$U_2 = perimter\ on\ punching\ shear\ critical\ plane\ for\ pile\ load$$
$$= (900 + 0.3B + 1.5d_x) + \left(900 + 0.3B + 1.5d_y\right) = 4533\ mm$$

$$Column\ punching\ shear\ stress = \frac{N}{U_1 d} = \frac{8100 * 1000}{2200 * 0.5 * (800 + 782)} = 4.65;\ less\ than\ 0.8\sqrt{f_{cu}} = 5.06 \frac{kN}{m^2}\ OK \tag{4.14}$$

$$Punching\ shear\ stress\ at\ pile\ perimeter = \frac{Q_{max}}{U_1 d_x} = \frac{1511.91 * 1000}{4 * 600 * 800} = 0.787 < 5.06 \frac{kN}{m^2}\ OK \tag{4.15}$$

$$Pile\ punching\ shear\ stress = \frac{Q_{max}}{U_2 d} = \frac{1511.91 * 1000}{4533 * 0.5 * (800 + 782)} = 0.422 < 5.06 \frac{kN}{m^2}\ OK \tag{4.16}$$

4.4.7 Pile reinforcement design

The area of reinforcement in piles has to meet the following conition (ibid.):

$$\frac{100\ A_{sc}}{A_c} \geq 0.4 \tag{4.17}$$

Where, A_{sc} = total area of reinforcement in a pile

A_c = net area of concrete in a pile cross-section

$$A_c = 600 * 600 = 360000\ mm^2$$

$$A_{sc} \geq 360000 * \frac{0.4}{100} = 1440\ mm^2$$

For piles with 600mm width, 12 bars of 20mm diameter with overall area of $3768mm^2$ ($>1440\ mm^2$ OK)will be used as a reinforcement.

4.4.8 Settlement of group piles

Total settlement of one pile under load can be calculated with the following formula (Das, 2007):

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)} \tag{4.18}$$

Where, $s_{e(1)}$ = elastic settlement of pile

$s_{e(2)}$ = settlement of pile caused by the load at the pile tip

$s_{e(3)}$ = settlement of pile caused by the load transmitted along the pile shaft

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{wp})L}{A_p E_p} \tag{4.19}$$

Where, Q_{wp} = load carried at the pile point under working load condition

Q_{ws} = load carried by frictional (skin) resistance under working load contition

A_p = area of cross section of pile

L = length of pile

E_p = modulus of elasticity of the pile material

For conservative calculation, Q_{max} will be used as a load exerted on a pile in further estimations.

$$s_{e(1)} = \frac{1511.91 * 10.8}{0.36 * 29725.41} = 1.526\ mm$$

$$s_{e(2)} = \frac{q_{wp}B}{E_s}(1 - \mu_s^2)I_{wp} \tag{4.20}$$

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

B = width or diameter of pile

μ_s = Poisson's ratio of soil,

I_{wp} = influence factor ≈0.85

$$s_{e(2)} = \frac{1511.91*0.6}{0.36*23000000}\left(1-\left(0.1+\frac{0.3}{20}*(38-25)\right)^2\right)*0.85 = 8.5*10^{-5}\ mm$$

$$s_{e(3)} = \frac{(Q_{ws})B}{pLE_s}(1-\mu_s^2)I_{ws} \tag{4.21}$$

Where, p = perimeter of the pile

L = embedded length of pile

I_{ws} = influence factor, =2+0.35√(L/D)

$$s_{e(3)} = \frac{1511.91*0.6}{2.4*10.8*23000000}(1-0.295^2)*\left(2+0.35*\sqrt{\frac{10.8}{0.6}}\right)$$
$$= 4.84*10^{-6}\ mm$$

$$s_e = 1.526 + 8.5*10^{-5} + 4.84*10^{-6} = 1.526\ mm$$

Elastic settlement of group piles is estimated with the following formula (ibid.):

$$s_{g(e)} = \sqrt{\frac{B_g}{B}}\,s_e \tag{4.22}$$

Where, Bg = width of group pile section, =(n_2-1)d+2(D/2)

B = width or diameter of each pile in the group

$$s_{g(e)} = \sqrt{\frac{2.1}{0.6}}*1.526 = 2.855\ mm$$

Consolidation settlement will not be calculated in this project, as the soil profile mostly consists of sand and the long term settlement will have negligible value.

4.5 Other foundation designs

Apart from 3x2 group pile design under interior columns of tower, there are two more designs that were developed for the project. 2x2 group pile foundation under edge columns of tower and 2x2 group pile foundation under columns of ground floor use group 1 loads and group 2 loads (from table 4.6) for the design respectively. The same procedures as in previous sections were implemented for the designing of these two foundations. The tables 4.12 and 4.13, show the main design specifications of both foundations.

Table 4.12. Main characteristics of foundation under edge columns of tower.

Main characteristics	Values	Design limits	Accordance
Pile width, B (m)	0.5	-	-
Pile depth, L (m)	10	-	-
Number of piles, R	4	-	-
Pile cap length (m)	2.75	-	-
Pile cap width (m)	2.75	-	-
Pile cap height (m)	0.75	-	-
Pile-to-pile spacing (m)	1.25	-	-
Horizontal load on a pile, H (kN)	27.75	< 137.4	OK
Total vertical load on group of piles, P (kN)	3758.3	< 4506.63	OK
Maximum vertical load on a pile, Qmax (kN)	1067.57	< 1126.66	OK
Reinforcement at 1st side of pile cap	#11 Ø20 @240	-	-
Reinforcement area at 1st side of pile cap (mm2)	3454	> 3431.98	OK
Reinforcement at 2nd side of pile cap	#16 Ø16 @156	-	-
Reinforcement area at 2nd side of pile cap (mm2)	3216	> 3156.83	OK
Column punching shear stress (kN/m2)	2.34	< 5.06	OK
Punching shear stress at pile perimeter (kN/m2)	0.82	< 5.06	OK
Pile punching shear stress (kN/m2)	0.447	< 5.06	OK
Pile reinforcement	#8 Ø16	-	-
Pile reinfrocement area (mm2)	1608	> 1000	OK

Settlement of group piles, Sge (mm)	3.8	-	-

Table 4.12. Main characteristics of foundation under edge columns of tower.

Main pile characteristics	**Values**	**Design limits**	**Accordance**
Pile width, B (m)	0.35	-	-
Pile depth, L (m)	8	-	-
Number of piles, R	4	-	-
Pile cap length (m)	2	-	-
Pile cap width (m)	2	-	-
Pile cap height (m)	0.55	-	-
Pile-to-pile spacing (m)	0.9	-	-
Horizontal load on a pile, H (kN)	45.25	< 106.03	OK
Total vertical load on group of piles, P (kN)	739.2	< 2102.886	OK
Maximum vertical load on a pile, Qmax (kN)	495.36	< 525.722	OK
Reinforcement at 1st side of pile cap	#6 Ø20 @345	-	-
Reinforcement area at 1st side of pile cap (mm2)	1884	> 1835.56	OK
Reinforcement at 2nd side of pile cap	#8 Ø12 @250	-	-
Reinforcement area at 2nd side of pile cap (mm2)	904.3	> 893.15	OK
Column punching shear stress (kN/m2)	0.514	< 5.06	OK
Punching shear stress at pile perimeter (kN/m2)	1	< 5.06	OK
Pile punching shear stress (kN/m2)	0.425	< 5.06	OK
Pile reinforcement	#4 Ø16	-	-
Pile reinfrocement area (mm2)	804	> 490	OK
Settlement of group piles, Sge (mm)	6.28	-	-

It can be observed that all specifications satisfy all design requirements and limits. Therefore, it can be said that design procedures were accurately performed. The side and top views with relevant reinforcement details of the foundations are provided in CAD drawings.

5 Project Management

5.1 Review of estimation

Prior to any planning or designing work, the client needs to know the rough cost of the project. Since in the design requirements there were no information about the cost, it is challenging to predict the exact budget for the project. In order to proceed to the next stage and identify the feasibility, cost estimation should be used. Cost estimate is a critical and essential process at the entire project since the total cost will be predicted with insufficient data and will involve control of the expenditures till the completion of the project to prevent overrun of the budget. Since it is rough estimations, the large margin of errors will occur that leads to the contingency; therefore enhancing the accuracy of the cost estimations is essential. The following section will cover different methods of cost estimation including top-down and bottom-up with its procedures and calculations to get the best appropriate cost of the project.

Approximate cost estimation, also known as top-down, preliminary estimate and order of magnitude, is performed in a conceptual phase where detailed design and any technical drawings are not prepared. It is used to obtain rough cost estimations using practical knowledge and historical data (Woo et al., 2001). The main resources of the conceptual estimate are time, cost and information. Considering the fact that availability of all these resources is restricted, the complexity of this stage should be mentioned (Jensen, 2009).

- Square meter method

Since the construction cost of the building in Kazakhstan is not available, different countries around the world were considered. Analyzing similar existing projects and its unit costs with inflation rate, the unit costs were found and illustrated in Table 5.1.2. Moreover, neighboring countries such as Russia and China were considered, since financial and economical spheres of Kazakhstan are interdependent with these countries. In conformity with the article 282 of the Civil Code of the Republic of Kazakhstan (2015), all financial liabilities in the Kazakhstan shall be expressed in tenge, hence the all costs obtained in this project will be in the national currency. In addition to that the currency rate was taken as 1$ = 340 tg (The World's Trusted Currency Authority, 2016). Comparing them and taking neighboring Russian value, the unit cost of 700 000 KZT was obtained (Nedvizhimost' Kazakhstana, 2016).

In order to determine the total cost of the building, the unit cost is multiplied to the total area of the academic building.

$$\text{Cost} = \text{total area } (m^2) * \text{unit cost } (T) = 16149\ m^2 * 700\ 000 =$$
$$= 11\ 304\ 300\ 000 \text{ KZT } (\sim \$\ 33 \text{ million})$$

Table 5.1. The unit costs of construction in the world.

Country	Cost per m^2 (KZT)	Inflation rate (2011-2016)	Cost per m^2 in 2016 (KZT)
USA	789000	1.46	797460
UK	721700	1.46	729610
Russia	719780	1.46	721230
Ukraine	712370	1.46	719460
China	703950	1.46	714680
Europe	771430	1.46	723760

The bottom-up approach, quantity take-off, is developed when all of the resources needed are identified and work packages are provided. It involves Work-Breakdown Structure that will cover details from the lowest to the highest level. After identifying, categorizing and organizing the estimated tasks, quantity takeoff is originated. This process covers the determination of the work quantity that needs to be performed on the project. The work quantity sections for the project are as following (Elbeltagi, 2007):

1. Site Preparation;
2. Excavating;
3. Foundation Construction (pile driving);
4. Superstructure (1st ground floor, 2-10 typical tower floors, roofing);
5. Interior (partitioning, flooring, ceiling, elevators, HVAC system and etc.)
6. Exterior (curtain wall)

Each work can be separated by the units of labor, material and equipment. Material Cost includes the cost of material used during the construction and provided by the supplier. Labor Cost involves determining hourly wage rate, crew productivity and labor cost while the equipment cost is needed to calculate the cost of equipment used in terms of ownership/rent and maintenance. In order to estimate them, the following considerations should be taken into account (Gould and Joyce, 2011):

- Efficiency rate
- Weather conditions
- Specific conditions of the work;
- Duration and frequency.

The quantity take-off sheet is provided in Table below.

The total cost of the project is determined by:

Total Cost = Material Cost + Labor Cost + Equipment Cost

The Unified Norms and Prices of the Republic of Kazakhstan have similar cost estimation methods that are regulated locally. According to this UNaP RK (2011), the basis for determining the exact estimated cost is as follows:

- the project and working documentation (drawings, statements of volume construction and installation works), specifications and schedules for equipment; main decisions on the organization and sequence of construction; additional explanatory notes to the project materials);
- operating estimate-regulatory framework;

The estimated cost of the construction in accordance with the technological structure of capital investment and the order of the activities of the construction organizations is determined by the following elements:

- construction works;
- installation works;
- the cost of the equipment operation, furniture and etc;
- other costs (design, survey and research work, preparation of the operational works).

Table 5.2. Quantity Take-Off

	Item Quantity	Units	Cost of Material		Cost of Labor		Cost of Equipment		Item Estimate
Earthwork									
Site Grading, Clearing and Levelling	15000	m2					Bulldozer (KZT/h)	10 000 KZT	
					Crew (KZT/m2)	1 000 KZT	Loader (KZT/h)	8 000 KZT	
				- KZT	**Total Cost of Labor**	1 500 000 KZT	**Total Cost of Equipment**	144 000 KZT	**1 644 000 KZT**
Site Backfill&Excavation	11432	m3	Coarse-grained soil (KZT/m3)	1 500 KZT	Production rate (m3/h)	60			
					Crew (KZT/h)	15 000 KZT	Dozer (KZT/h)	7 000 KZT	
					Volume (m3)	11432	Backhoe (KZT/h)	10 000 KZT	
			Total Cost of Material	**17 148 000 KZT**	**Total Cost of Labor**	**2 858 000 KZT**	**Total Cost of Equipment**	**3 239 067 KZT**	**23 245 067 KZT**
0									
Checkpoint for employees (security, transport check-in point, turnstile, tracking system)	1	pcs							**5 008 000 KZT**
Temporary Staff Office	1	pcs							**8 127 000 KZT**
Temporary Foreman's Office	1	pcs							**3 937 500 KZT**
Temporary Canteen for workers	1	pcs							**8 505 000 KZT**
Temporary Toilets, Dressing and Shower rooms	1	pcs							**4 315 500 KZT**
Foundation Construction									
Pile Installation			Piles C90.30-6 (KZT/pcs)	30 000 KZT	Crew (KZT/h)	10000	Autocrane (KZT/h)	8 000 KZT	
			Quantity (pcs)	692			Concrete mixer truck (KZT/h)	90 000 KZT	
Pile Cap Installation			Concrete (KZT/m3)	13 800 KZT	Crew (KZT/h)	9000	Pile driving rig (KZT/m)	1 500 KZT	
			Volume (m3)	870			Pile transporting truck (KZT/h)	5 000 KZT	
Pile Cap Reinforcement Installation			Rebar D=20mm (KZT/t)	147 000 KZT	Crew (KZT/h)	15000	Working hours (h)	500 KZT	
			Quantity (t)	30			Length of pile driving (m)	7 536 KZT	
			Rebar D=16mm (KZT/t)	131 320 KZT					
			Quantity (t)	6					
			Rebar D=12mm (KZT/t)	128 640 KZT					
			Quantity (t)	2					
Binding of reinforcing cages					Crew (KZT/h)	22000			
			Total Cost of Material	**126 655**	**Total Cost of Labor**	**28 000 000**	**Total Cost of Equipment**	**62 804 000 KZT**	**217 459 200 KZT**

				200 KZT		**KZT**			
Superstructure									
Beam (0.5x0.3x5.5)	1408	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 6mm (t)	4.6			Production rate (m3/h)	35	
			Rebar, 6 mm (KZT/t)	105 000 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 10 mm (t)	5.3			Production rate (m3/h)	3	
			Rebar, 10 mm (KZT/t)	112 000 KZT					
			Rebar, 12mm (t)	6.78					
			Rebar, 12 mm (KZT/t)	128 640 KZT					
			Rebar, 16 mm (t)	4.78					
			Rebar, 16 mm (KZT/t)	131 320 KZT					
			Rebar, 20 mm (t)	15.23					
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25mm (t)	20.3					
			Rebar, 25mm (KZT/t)	162 000 KZT					
			Rebar, 32mm (t)	20.3					
			Rebar, 32 mm (KZT/t)	187 200 KZT					
			Volume (m3/unit)	0.825	Volume (m3/unit)	0.825	Volume (m3/unit)	0.825	
			Total Cost of Material	**194 146 248 KZT**	**Total Cost of Labor**	**23 232 000 KZT**	**Total Cost of Equipment**	**9 691 063 KZT**	**300 349 677 KZT**
Beam (0.6x0.3x5.5)	896	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 6mm (t)	2.4			Production rate (m3/h)	35	
			Rebar, 6 mm (KZT/t)	105 000 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 10 mm (t)	1.7			Production rate (m3/h)	3	
			Rebar, 10 mm (KZT/t)	112 000 KZT					
			Rebar, 12mm (t)	3.75					
			Rebar, 12 mm (KZT/t)	128 640 KZT					
			Rebar, 16 mm (t)	20.2					
			Rebar, 16 mm (KZT/t)	131 320 KZT					
			Rebar, 20 mm (t)	9.8					
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25mm (t)	20.5					
			Rebar, 25mm (KZT/t)	162 000 KZT					**165 947 632 KZT**

			Rebar, 32mm (t)	16.789					
			Rebar, 32 mm (KZT/t)	187 200 KZT					
			Volume (m3/unit)	0.825	Volume (m3/unit)	0.99	Volume (m3/unit)	0.99	
			Total Cost of Material	**138 069 232 KZT**	**Total Cost of Labor**	**17 740 800 KZT**	**Total Cost of Equipment**	**10 137 600 KZT**	
Beam (0.6x0.3x6.0)	576	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 10 mm (t)	5.25			Production rate (m3/h)	35	
			Rebar, 10 mm (KZT/t)	112 000 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 12mm (t)	8.75			Production rate (m3/h)	3	
			Rebar, 12 mm (KZT/t)	128 640 KZT					
			Rebar, 16 mm (t)	8.5					
			Rebar, 16 mm (KZT/t)	131 320 KZT					
			Rebar, 20 mm (t)	9.5					
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25mm (t)	18.5					
			Rebar, 25mm (KZT/t)	162 000 KZT					
			Rebar, 32mm (t)	18.5					
			Rebar, 32 mm (KZT/t)	187 000 KZT					
			Volume (m3/unit)	0.825	Volume (m3/unit)	1.08	Volume (m3/unit)	1.08	
			Total Cost of Material	**99 956 100 KZT**	**Total Cost of Labor**	**49 766 400 KZT**	**Total Cost of Equipment**	**7 109 486 KZT**	**156 831 986 KZT**
Beam (0.5x0.3x6.0)	998	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	120 000 KZT	
			Rebar, 6mm (t)	10			Production rate (m3/h)	35	
			Rebar, 6 mm (KZT/t)	105 000 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 10 mm (t)	10			Production rate (m3/h)	3	
			Rebar, 10 mm (KZT/t)	112 000 KZT					
			Rebar, 12mm (t)	20.5					
			Rebar, 12 mm (KZT/t)	128 640 KZT					
			Rebar, 16 mm (t)	9.8					
			Rebar, 16 mm (KZT/t)	131 320 KZT					
			Rebar, 20 mm (t)	15.5					
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25mm (t)	18.785					
			Rebar, 25mm (KZT/t)	162 000 KZT					**198 649 683 KZT**

			Rebar, 32mm (t)	30.75					
			Rebar, 32 mm (KZT/t)	187 000 KZT					
			Volume (m3/unit)	0.825	Volume (m3/unit)	0.9	Volume (m3/unit)	0.9	
			Total Cost of Material	**170 420 540 KZT**	**Total Cost of Labor**	**17 964 000 KZT**	**Total Cost of Equipment**	**10 265 143 KZT**	
Beam (0.4x0.2x5.5)	985	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	31 000 KZT	
			Rebar, 16 mm (t)	9.5			Production rate (m3/h)	35	
			Rebar, 16 mm (KZT/t)	131 320 KZT			Concrete truck mixer (KZT/h)	6 000 KZT	
							Production rate (m3/h)	3	
			Volume (m3/unit)	0.44	Volume (m3/unit)	0.44	Volume (m3/unit)	0.44	
			Total Cost of Material	**61 517 600 KZT**	**Total Cost of Labor**	**8 668 000 KZT**	**Total Cost of Equipment**	**1 250 669 KZT**	**97 440 268 KZT**
Beam (0.4x0.2x6.0)	756	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	120 000 KZT	
			Rebar, 16 mm (t)	8.5			Production rate (m3/h)	35	
			Rebar, 16 mm (KZT/t)	131 320 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
							Production rate (m3/h)	3	
			Volume (m3/unit)	0.48	Volume (m3/unit)	0.48	Volume (m3/unit)	0.48	
			Total Cost of Material	**51 805 200 KZT**	**Total Cost of Labor**	**7 257 600 KZT**	**Total Cost of Equipment**	**4 147 200 KZT**	**63 210 000 KZT**
Column (0.3x0.3x3)	768	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 16 mm (t)	8			Production rate (m3/h)	35	
			Rebar, 16 mm (KZT/t)	131 320 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 20 mm (t)	8			Production rate (m3/h)	3	
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25 mm (t)	8					
			Rebar, 25 mm (KZT/t)	162 000 KZT					
			Rebar, 32 mm (t)	8					
			Rebar, 32 mm (KZT/t)	187 000 KZT					
			Volume (m3/unit)	0.27	Volume (m3/unit)	0.27	Volume (m3/unit)	0.27	
			Total Cost of Material	**78 801 280 KZT**	**Total Cost of Labor**	**4 147 200 KZT**	**Total Cost of Equipment**	**1 729 975 KZT**	**84 678 455 KZT**
Column (0.45x0.45x3)	1152	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 16 mm (t)	8			Production rate (m3/h)	35	
			Rebar, 16 mm (KZT/t)	131 320 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 20 mm (t)	24.5			Production rate (m3/h)	3	
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25 mm (t)	9					**102 484 715 KZT**

			Rebar, 25 mm (KZT/t)	162 000 KZT					
			Rebar, 32 mm (t)	9					
			Rebar, 32 mm (KZT/t)	187 000 KZT					
			Volume (m3/unit)	0.6075	Volume (m3/unit)	0.6075	Volume (m3/unit)	0.6075	
			Total Cost of Material	**82 649 250 KZT**	**Total Cost of Labor**	**13 996 800 KZT**	**Total Cost of Equipment**	**5 838 665 KZT**	
Column (0.55x0.55x3)	504	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 16 mm (t)	20			Production rate (m3/h)	35	
			Rebar, 16 mm (KZT/t)	131 320 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 20 mm (t)	18			Production rate (m3/h)	3	
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25 mm (t)	12					
			Rebar, 25 mm (KZT/t)	162 000 KZT					
			Rebar, 32 mm (t)	8					
			Rebar, 32 mm (KZT/t)	187 000 KZT					
			Volume (m3/unit)	0.9075	Volume (m3/unit)	0.9075	Volume (m3/unit)	0.9075	
			Total Cost of Material	**90 145 464 KZT**	**Total Cost of Labor**	**9 147 600 KZT**	**Total Cost of Equipment**	**3 815 856 KZT**	**103 108 920 KZT**
Column (0.2x0.2x3)	800	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 16 mm (t)	2.785			Production rate (m3/h)	72	
			Rebar, 16 mm (KZT/t)	131 320 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
			Rebar, 20 mm (t)	15			Production rate (m3/h)	3	
			Rebar, 20 mm (KZT/t)	147 000 KZT					
			Rebar, 25 mm (t)	8.56					
			Rebar, 25 mm (KZT/t)	162 000 KZT					
			Rebar, 32 mm (t)	15					
			Rebar, 32 mm (KZT/t)	187 000 KZT					
			Volume (m3/unit)	0.12	Volume (m3/unit)	0.12	Volume (m3/unit)	0.12	
			Total Cost of Material	**56 610 723 KZT**	**Total Cost of Labor**	**1 920 000 KZT**	**Total Cost of Equipment**	**784 000 KZT**	**59 314 723 KZT**
Slab	9	pcs	Concrete (KZT/m3)	13 800 KZT	Crew (KZT/m3)	20 000 KZT	Truck mounted concrete pump (KZT/h)	12 000 KZT	
			Rebar, 8 mm (t)	245			Production rate (m3/h)	35	
			Rebar, 8mm (KZT/t)	108 000 KZT			Concrete truck mixer (KZT/h)	24 000 KZT	
							Production rate (m3/h)	3	
			Volume (m3/unit)	230	Volume (m3/unit)	230	Volume (m3/unit)	230	**443 851 714 KZT**

			Total Cost of Material	**385 182 000 KZT**	**Total Cost of Labor**	**41 400 000 KZT**	**Total Cost of Equipment**	**17 269 714 KZT**	
Exterior&Interior Works									
Walls									
Brickwork	12543	m2	Bricks 250x60x65 (KZT/pcs)	70	Crew (KZT/m2)	7 000 KZT			
Plastering	12543	m2	Plastering Mixture 25kg (KZT/bag)	3 000 KZT					
Gypsum Plastering	2120	m2	Drywall 1200x2500 (KZT/m3)	1 500 KZT					
			Total Cost of Material	**1 290 432 000 KZT**	**Total Cost of Labor**	**235 310 000 KZT**			**1 525 742 000 KZT**
Flooring									
Screeding	17120	m2	Cement (KZT/bag)	1 200 KZT	Crew (KZT/m2)	1 500 KZT			
Tile Placing	17000	m2	Floor tile (KZT/m2)	25 000 KZT					
			Total Cost of Material	**434 000 000 KZT**	**Total Cost of Labor**	**51 360 000 KZT**			**485 360 000 KZT**
Ceiling									
Panel Placing	17100	m2	Decorative panels 1,22x2,44 m2 (KZT/pcs)	15 000 KZT	Crew (KZT/m2)	5 000 KZT			
			Total Cost of Material	**86 363 636 KZT**	**Total Cost of Labor**	**85 500 000 KZT**			**171 863 636 KZT**
Façade Works									
Panel Installation	1260	m2	Façade Panels (KZT/m2)	5 000 KZT	Crew (KZT/m2)	15 000 KZT			
Glass Installation	4224	m2	Stained glass (KZT/m2)	45 000 KZT					
			Total Cost of Material	**196 380 000 KZT**	**Total Cost of Labor**	**63 360 000 KZT**			**259 740 000 KZT**
Roofing									
Parapet Installation	300	m3	Autoclaved aerated concrete roof panels 400x300x600 mm3 (KZT/m3)	13 800 KZT	Crew (KZT/m3)	12 000 KZT			
Flooring&Waterproofing	4200	m2	Three-Layer Torch Down Roofing (KZT/m2)	2 500 KZT					**75 200 000 KZT**
			Total Cost of Material	**68 000 000 KZT**	**Total Cost of Labor**	**7 200 000 KZT**			
Other Costs (Plumbing, Electrical&Mechanical Works, etc)									**4 000 000 000 KZT**
Total									**10 256 749 000 KZT**

5.2 Economic analysis

Economic evaluation of the construction project is one of the main preliminary analyses that have to be studied in terms of the determination of the project's feasibility. It involves the calculation elements as cash flow, measure of worth over time, payback period. Economic analysis steps are performed by the following procedure in the Figure 5.1. The first step is to identify the necessity of the project and then the estimation of cash flows, expected life and other parameters. As the cash inflows stand revenues, incomes, savings, and receipts, whereas cash outflows are costs, expenses and taxes. Thirdly, the client's project selection criteria including PW (present worth), NPV (net present value), ROR (rate of return), payback period are determined. Finally, the economic analysis considering noneconomic factors, sensitivity analysis and risk assessments are developed (Leland 2014).

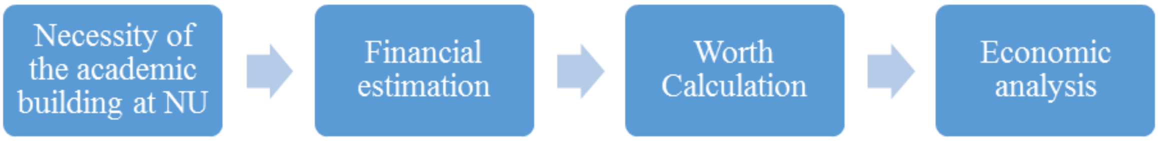


Figure 5.1.Preliminary Project Evaluation procedure.

Step 1. Necessity of the project

Academic Building for Nazarbayev University is in high demand, since the number of the enrolled students increase each year. According to the Table 5.4, it can be seen that for the last years, the number of registered and accepted students for only foundation course grows considerably (Statistics of Nazarbayev University, nu.edu.kz). Hence, it has experienced significant enrollment growth in the past few years, and this growth is expected to continue. Moreover, there are many accepted undergraduate, graduate, PhD and other professional programs. Additionally, since the University stands for the center of the education in Astana, there are many scientific conferences and forums that are hold at the territory of the University. Hence, the client needs additional academic building that will sufficiently accommodate this projected growth and fulfill the missions of the University.

Table 5.3. Statistics admitted to NU

	2013	2014	2015
Number of registered applicants	3290	3362	3380
Number of accepted students	535	548	700

The construction of academic building will generate approximately 3000 full and part-time jobs over the development, construction and operation of the project. In addition to that each year 500 students will be graduated and additional workforce will be created. This will contribute to the local economy of the city. Moreover, it will attract investing from other sources, which will also improve well-being of the city.

Step 2. Financial Estimation

Cost estimations are provided in the previous sub-section.

Step 3. Worth Calculation

In order to analyze the economic efficiency of the project, revenues and expenses of the University have to be determined. For the income of the University stand operating governmental budget including governmental grant (cost of underground and graduate degrees), investment income and all others while for the expenditures stand labor cost (salaries for staff were shown in Table 2, Appendix D), costs of goods sold, equipment maintenance, operational supplies, scholarships, bursaries and other utility costs. Studying at this academic building will cost 6 200 000 KZT for undergraduate and 9 450 000 KZT for graduate students (Tengrinews 2016). Table 5.4 shows annual income and costs over the operation of the University.

Table 5.4. Annual revenues and expenditures

Tuition income	2 925 000 000 KZT
Investment income	2 800 000 000 KZT
Other incomes	220 000 000 KZT
Total revenue	**5 945 000 000 KZT**
Labor expenses	2 967 920 000 KZT
Students living expenditures	890 376 000 KZT
Costs of goods sold, equipment maintenance and other	415 508 800 KZT
Operational supplies and expenses	593 584 000 KZT
Scholarships, bursaries and other	296 792 000 KZT

Other utility costs	178 075 200 KZT
Total expenses	**4 842 256 000 KZT**

To analyze the feasibility of the project, Net Present Value has to be determined. Since the total cost of the project is 11 304 300 000 KZT and bid price of 12 000 000 000 KZT, it will be considered as the invested capital (IC). NPV is the discounted cash flow for the life of the project (or planned period) deducting the initial capital investment (Sullivan 2014).

NPV = $-12\ 000\ 000\ 000 + 1\ 102\ 700\ 000(P/A\ , 18\%, 12)$ = 3 849 392 000 KZT

Internal Rate of Return (IRR) is the discount rate making the net present value of zero or the rate of growth a project is expected to generate. Having the greater IRR than MARR (Minimum Acceptable Rate of Return) which is taken as 20% means that the project is profitable (ibid). IRR was calculated using Excel function (Table 5.6).

Table 5.5. IRR Calculation

EOY	Cash Flow
0	-12 000 000 000 KZT
1	1 102 700 000 KZT
2	1 102 700 000 KZT
3	1 102 700 000 KZT
4	1 102 700 000 KZT
5	1 102 700 000 KZT
6	1 102 700 000 KZT
7	1 102 700 000 KZT
8	1 102 700 000 KZT
9	1 102 700 000 KZT
10	1 102 700 000 KZT
11	1 102 700 000 KZT
12	1 102 700 000 KZT
13	1 102 700 000 KZT
14	1 102 700 000 KZT
15	1 102 700 000 KZT
IRR	27%

Benefit/Cost ratio (B/C ratio) is the proportion of present/annual/future value of benefits to the present/annual/future value of costs. When the B/C ratio is greater or equal to 1 than the project is economically feasible, otherwise it is not.

B/C ratio = $\frac{5\,945\,000\,000}{4\,842\,256\,000} = 1.23$

Payback period (PB) is the time when the expenses are recovered and is determined by dividing the cost of the project to the annual cash flow (ibid).

Payback period = $\frac{12\,000\,000\,000}{1\,102\,700\,000} = 10.9$ years

Step 4. Analysis

After the values of NPV (net present value), IRR (internal rate of return), B/C benefit over costs, the payback period are determined, the project's profitability can be analyzed. IF the NPV value is positive, IRR is in accepted range, B/C ratio is more than 1 and payback period is not large, then it can be said that the project is profitable.

Table 5.6. Economic analysis

NPV	3 849 392 000 KZT
IRR	27%
B/C ratio	1.23
Payback period	10.9 years

From Table 5.6, it can be seen that NPV value is positive, obtained IRR is more than 20%, B/C ratio is also more than 1 and payback period is appropriate; hence, the project is feasible.

5.3 Risk assessment

The construction process is full of risks that may cause both positive and negative effects on duration, quality and thus, the cost of the project. The risk management is systematic and quantitative method of identifying, analyzing and elimination of the potential risks in order to meet goals of the projects. Applying the principles of risk management improves the construction project management processes and increases the effectiveness of the resource uses.

Any construction process is very complex and full of unique uncertainties that vary from one project to another. These uncertainties and potential risks can inflict irreparable harm to the whole project. Therefore, it is essential to analyze and manage all risks in order to deal effectively with unexpected outcomes of any activity. In this section the risk management will be discussed in detail.

The risk management process contains of following steps: risk identification, risk assessment, risk mitigation and risk monitoring.

Risk identification

The first and the most crucial steps of risk management is risk identification since in this step all potential risks are identified according to their sources and types (Carbone and Tippett 2004). It gives comprehensive information about the conditions when risky event may happen in construction. There are many approaches to classify risks, one of which is to classify risks according to their origin, i.e. internal and external risks (Tah and Carr 2000). While internal risks are the uncertainties that are under the control of main stakeholders (client, contractors and consultants), the external ones cannot be prevented by the key stakeholders. Since the construction of academic building is local project, some of external risks, such as unawareness of social and economic conditions, unfamiliarity of procedural formalities and governing authority are successfully eliminated. All potential risks are included in table 5.7 below.

Table 5.7. Risk categories

	Categories	Likelihood 1 (rare) – 5 (very frequent)	Impact 1 (very low) – 5 (very high)
Design risks			
D1	Errors in design and omissions	4	5
D2	Insufficient detailing	3	4
D3	Conflicting with Eurocode and SNaR	2	5
D4	Stakeholder request late changes	3	3
External risks			
Ex1	Public objection	1	3
Ex2	Inflation and raised tax	1	4
Ex3	Law and local standard change	1	3
Environmental risks			
En1	Air and land pollution with dust	3	5

En2	Incomplete environmental analysis	2	4
En3	Hazardous materials	3	5
Organizational risks			
O1	Lack of experience in workforce	3	3
O2	Lack of communication	3	4
O3	Delays in delivery of materials	3	3
Project management risks			
PM1	Schedule delays	4	4
PM2	Project team conflicts	3	3
Construction risks			
C1	Construction cost overruns	4	4
C2	Human health and safety	2	5
C3	Equipment breakdown	3	3
C4	Undocumented changes	3	4

Risk Assessment

The next step of risk management is the risk assessment that basically means to calculate the risk scores after all risks have been determined. The risk scores found in table 5.7 are assigned for individual risk factors in required risk categories. The risk matrix shown in figure 5.2 integrates the risk scores of impact and likelihood, as a result, all risks are prioritized. Using risk matrix method, Risk Score can be determined by (Vaidyanathan 2013):

Risk Score = Occurrence * Outcome

Impact	Rare	Occasional	Somewhat frequent	Frequent	Very frequent
Very high		D3, C2	En1, En3		
High	Ex2	En2	D2, O2, C4	D1, PM1, C1	
Moderate	Ex1, Ex3		D4, O1, O3, PM2, C3		
Low					
Very Low					

Likelihood

Figure 5.2. Risk Matrix

From the results of risk matrix it can be seen that under the design risks category, it is need to pay attention on errors in design. It has both frequent likelihood and high impact on project objectives. Under environmental risks category, the use of hazardous material is recognized to be the most critical risk. Construction materials that can cause long term harm to the environment are very dangerous. Moreover, according to the risk matrix, it can be seen that the dust, air and land pollution and transportation resources are in the red zone and extremely need the mitigation measures. Energy consumption, noise, waste generation, site conditions are also should be considered. Therefore, effective environmental assessment must be performed for the aim of reduction of harmful environmental effects, improvement of the sustainable performance and enhancement of environmentally friendly materials (Zolfagharian et al. 2012). The most important risk under organizational risks category is breakdown in communication. Since the construction process is very complex process that involves various specialists, the effective communication between contractors, owner and supplier is the key factor in achieving common goals. Schedule delays, that is under project management risk category entails a chain of negative consequences such as unexpected expenses in case the contract contains a penalty for schedule delay. Construction cost from construction risks category has frequent likelihood and high impact to the owner, since any additional expenses are undesirable. As a result, threats that lie within red, yellow and green zones should be treated in first, second and third order, accordingly.

Risk Mitigation

Once all potential risks are identified and measured, it is essential to identify how they will be dealt with. Mitigation plan for all risks are tabulated below.

Table 5.8. Risk response

	Categories	Risk value	Mitigation plan
Design risks			
D1	Errors in design and omissions	20	Involve only certified and experiences engineers, enhance quality control procedure along with performance control

			procedure
D2	Insufficient detailing	12	Ensure detailed drawings in order to follow correct procedures on site
D3	Conflicting with Eurocode and SNaR	10	Consult with engineers
D4	Stakeholder request late changes	9	Enhance communication between stakeholders and engineers
External risks			
Ex1	Public objection	3	Promote the slightly enhanced design, compromise on common decision
Ex2	Inflation and raised tax	4	Provide less costly alternatives
Ex3	Law and local standard change	3	Provide back-up plans, consult with lawyers
Environmental risks			
En1	Air and land pollution with dust	15	Clean site from construction garbage on time
En2	Incomplete environmental analysis	8	Conduct comprehensive environmental analysis
En3	Hazardous materials	15	Test material
Organizational risks			
O1	Lack of experience in workforce	9	Hire experienced workforce
O2	Lack of communication	12	Carry out workshops and teambuilding to improve the communication
O3	Delays in delivery of materials	9	Monitor deliverables and improve transportation
Project management risks			
PM1	Schedule delays	16	Increase workforce
PM2	Project team conflicts	9	Carry out workshops and teambuilding to improve the communication
Construction risks			

C1	Construction cost overruns	16	Monitor the accurate purchase, quantity take-off and check-lists
C2	Human health and safety	10	Provide medical assurance and conduct safety trainings
C3	Equipment breakdown	9	Repair or purchase
C4	Undocumented changes	12	Provide supervisors to monitor and control the construction order

Mitigation measures to protect air pollution can be reduction of exposed areas to minimize the dust or spraying by the water the places that generate the dust. Moreover, waste management system should be well elaborated. Land pollution can be maintained by the efficient stabilization of altered soil structure, use of drip trays, storing and handling the soil and other methods of minimization. Protection of the human environment can be developed by proper management operations such as timing the construction so that it will not disturb the neighboring residents. Moreover, safety precautions including fencing the construction site and others should also be taken into account (Scoping guidance on EIA of projects, 2001).

According to the Environmental Code of Republic of Kazakhstan (EC 30-200, 2016), design and construction of buildings and other settlements should ensure the most favorable conditions for life, work and rest of the population, taking into account environmental, sanitary and epidemiological requirements and environmental safety. Moreover, when planning and building, their sanitation, safe handling of waste production and consumption, creating green and protection zones must be considered and implemented. Construction and reconstruction of buildings and other structures are carried out only in the presence of the approving conclusions of the state ecological examination and sanitary-epidemiological expertise and in compliance with environmental quality standards (EC 29-202, 2016). Hence, it is essential that construction process of the academic building follow all the regulations, otherwise, it leads to the significant issues.

Once the risk mitigation plan has been developed, the risk monitoring step took place. All potential risks should be continuously monitored along with updating the risk categories, risk matrix and risk mitigation plan throughout the construction process.

5.4 Scheduling

Determining the duration of the project is also an important criterion for the building owner since they should predict the cash flow according to the duration. Construction duration can be defined by scheduling as the time given for the contractor from the site works to the completion of the project (Gould and Joyce 2011). It can easily be affected by different factors including the size of the project, academic building functions, height, location and complexity of the project. In order to determine the exact duration, the construction scheduling must be performed. Construction scheduling involves the coordination of many activities without delays. Scheduling process can be divided by the following steps (ibid):

- Identification of construction activities (site, work, foundation, exterior and interior works and etc.);
- Determination of the activity sequence (in what order activities occur);
- Determination of the each activity duration;
- Performance of scheduling calculations;
- Revision and adjustments of the scheduling;
- Control of the activities.

Analyzing the similar projects, it was determined that the duration of the project will be approximately 20 months. To estimate the duration, the Work-Breakdown-Structure was developed for the construction of the academic building and, respectively, Gantt chart was prepared using Microsoft Project software. According to this chart, the duration of the project was 446 days which is almost 20 months. The working time is considered as standard time from starting 9.00 to 18.00 with lunch time of 1 hour and almost 23 working days for one month.

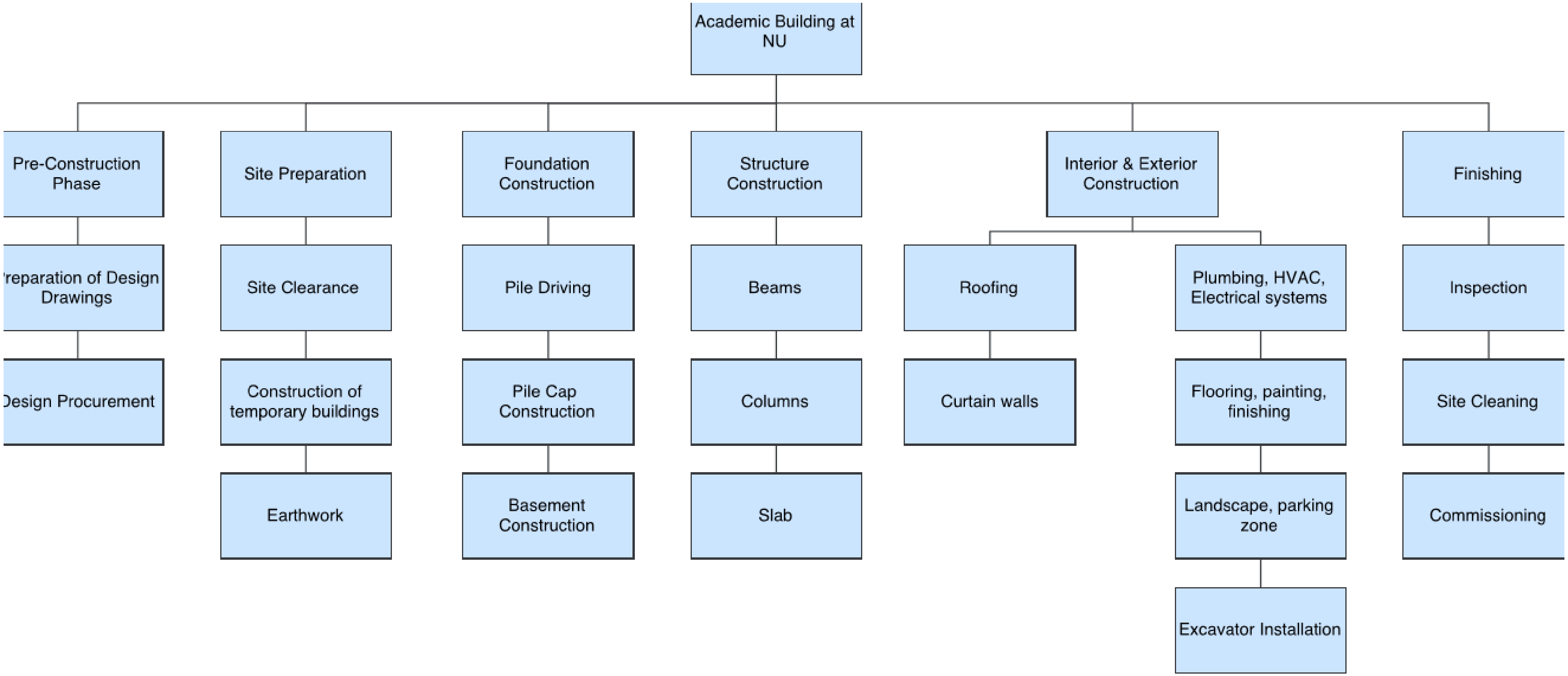


Figure 5.3. WBS structure for the project

Table 5.9. Project Scheduling

ID	Task Name	Duration	Start	Finish	Predecessors
0	**NU**	**446 days**	**Mon 03.04.17**	**Tue 18.12.18**	
1	**Academic Building at NU**	**446 days**	**Mon 03.04.17**	**Tue 18.12.18**	
2	**Pre-Construction**	**15 days**	**Mon 03.04.17**	**Mon 24.04.17**	
3	Receiving Permission Documents	5 days	Mon 03.04.17	Mon 10.04.17	
4	Preliminary Shop Drawings Preparation	10 days	Mon 10.04.17	Mon 24.04.17	3
5	Project Schedule Preparation	5 days	Mon 10.04.17	Mon 17.04.17	3
6	Hire Labors	3 days	Mon 17.04.17	Thu 20.04.17	5
7	**Site Earthwork**	**26 days**	**Mon 24.04.17**	**Tue 30.05.17**	**2**
8	Site Fencing	2 days	Mon 24.04.17	Wed 26.04.17	
9	Site Grading, Clearing and Levelling	10 days	Wed 26.04.17	Wed 10.05.17	8
10	Site Excavation and Backfill	14 days	Wed 10.05.17	Tue 30.05.17	9
11	**Site Mobilization**	**10 days**	**Tue 30.05.17**	**Tue 13.06.17**	**10**
12	Construction of Checkpoint for employees (security, transport check-in point, turnstile, tracking system)	3 days	Tue 30.05.17	Fri 02.06.17	10
13	Construction of Temporary Staff Office	7 days	Tue 30.05.17	Thu 08.06.17	10
14	Construction of Temporary Foreman's Office	7 days	Tue 30.05.17	Thu 08.06.17	10
15	Construction of Temporary Canteen for workers	10 days	Tue 30.05.17	Tue 13.06.17	10
16	Construction of Temporary Toilets, Dressing and Shower rooms	3 days	Tue 30.05.17	Fri 02.06.17	10
17	**Foundation Construction**	**72 days**	**Tue 13.06.17**	**Thu 21.09.17**	**11**
18	Pile Installation	21 days	Tue 13.06.17	Wed 12.07.17	11
19	Pile Cap Reinforcement Installation	14 days	Wed 12.07.17	Tue 01.08.17	18
20	Binding of reinforcing cages	14 days	Tue 01.08.17	Mon 21.08.17	19
21	Pile Cap Pouring&Curing	23 days	Mon 21.08.17	Thu 21.09.17	20
22	**Superstructure Construction**	**189 days**	**Thu 21.09.17**	**Wed 13.06.18**	**17**
23	**Ground Floor**	**21 days**	**Thu 21.09.17**	**Fri 20.10.17**	**17**
24	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Thu 21.09.17	Mon 02.10.17	17
25	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Mon 02.10.17	Wed 11.10.17	24

ID	Task Name	Duration	Start	Finish	Predecessors
26	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Wed 11.10.17	Fri 20.10.17	25
27	**2nd Floor**	**21 days**	**Fri 20.10.17**	**Mon 20.11.17**	
28	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Fri 20.10.17	Tue 31.10.17	26
29	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Tue 31.10.17	Thu 09.11.17	28
30	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Thu 09.11.17	Mon 20.11.17	29
31	**3rd Floor**	**21 days**	**Mon 20.11.17**	**Tue 19.12.17**	
32	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Mon 20.11.17	Wed 29.11.17	30
33	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Wed 29.11.17	Fri 08.12.17	32
34	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Fri 08.12.17	Tue 19.12.17	33
35	**4th Floor**	**21 days**	**Tue 19.12.17**	**Wed 17.01.18**	
36	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Tue 19.12.17	Thu 28.12.17	34
37	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Thu 28.12.17	Mon 08.01.18	36
38	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Mon 08.01.18	Wed 17.01.18	37
39	**5th Floor**	**21 days**	**Wed 17.01.18**	**Thu 15.02.18**	
40	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Wed 17.01.18	Fri 26.01.18	38
41	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Fri 26.01.18	Tue 06.02.18	40
42	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Tue 06.02.18	Thu 15.02.18	41
43	**6th Floor**	**21 days**	**Thu 15.02.18**	**Fri 16.03.18**	
44	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Thu 15.02.18	Mon 26.02.18	42

ID	Task Name	Duration	Start	Finish	Predecessors
45	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Mon 26.02.18	Wed 07.03.18	44
46	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Wed 07.03.18	Fri 16.03.18	45
47	**7th Floor**	**21 days**	**Fri 16.03.18**	**Mon 16.04.18**	
48	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Fri 16.03.18	Tue 27.03.18	46
49	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Tue 27.03.18	Thu 05.04.18	48
50	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Thu 05.04.18	Mon 16.04.18	49
51	**8th Floor**	**21 days**	**Mon 16.04.18**	**Tue 15.05.18**	
52	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Mon 16.04.18	Wed 25.04.18	50
53	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Wed 25.04.18	Fri 04.05.18	52
54	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Fri 04.05.18	Tue 15.05.18	53
55	**9th Floor**	**21 days**	**Tue 15.05.18**	**Wed 13.06.18**	
56	Columns: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Tue 15.05.18	Thu 24.05.18	54
57	Beams: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Thu 24.05.18	Mon 04.06.18	56
58	Slabs: Forming, Bending&Binding of Rebars, Pouring&Curing	7 days	Mon 04.06.18	Wed 13.06.18	57
59	**Exterior&Interior Works**	**120 days**	**Wed 13.06.18**	**Wed 28.11.18**	
60	External Walls: Brickwork	37 days	Wed 13.06.18	Fri 03.08.18	58
61	Internal Walls: Plastering, Gypsum Plastering	46 days	Fri 03.08.18	Mon 08.10.18	60
62	Floors: Screeding, Tile Placing	23 days	Mon 08.10.18	Thu 08.11.18	61
63	Ceilings: Panel Placing	14 days	Thu 08.11.18	Wed 28.11.18	62
64	**Roofing Works**	**7 days**	**Wed 13.06.18**	**Fri 22.06.18**	
65	Flooring&Waterproofing	7 days	Wed 13.06.18	Fri 22.06.18	58
66	Parapet Installation	5 days	Wed 13.06.18	Wed 20.06.18	58

ID	Task Name	Duration	Start	Finish	Predecessors
67	**Façade Works**	**23 days**	**Fri 03.08.18**	**Wed 05.09.18**	
68	Panel Installation	23 days	Fri 03.08.18	Wed 05.09.18	60
69	Glass Installation	23 days	Fri 03.08.18	Wed 05.09.18	60
70	Plumbing Works: Water System, Sewage and etc.	23 days	Fri 03.08.18	Wed 05.09.18	60
71	Electrical Works: Elevator&Transformer Installation, Lighting and etc.	23 days	Fri 03.08.18	Wed 05.09.18	60
72	HVAC: Heating, Ventilating, Air Conditioning and etc.	23 days	Fri 03.08.18	Wed 05.09.18	60
73	Landscape Works: Asphalt Covering, Green Zone and etc.	7 days	Wed 05.09.18	Fri 14.09.18	69
74	**Finishing Works**	**14 days**	**Wed 28.11.18**	**Tue 18.12.18**	
75	Inspection and Commissioning	5 days	Wed 28.11.18	Wed 05.12.18	63
76	Final Cleaning	5 days	Wed 28.11.18	Wed 05.12.18	63
77	Furniture Installation	7 days	Wed 05.12.18	Fri 14.12.18	76
78	Temporary Stations Removing	2 days	Fri 14.12.18	Tue 18.12.18	77

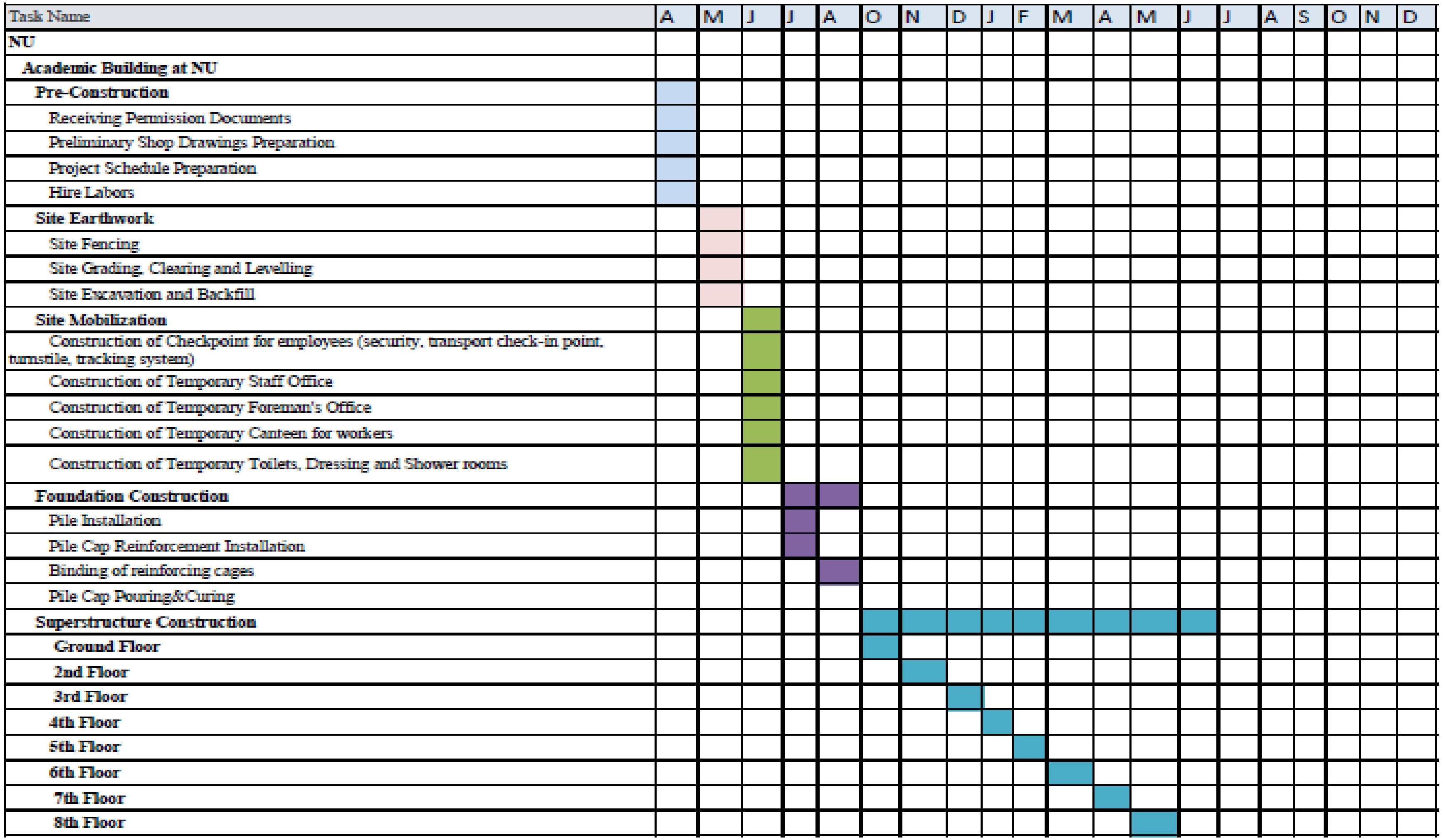


Figure 5.4. Gantt Chart

9th Floor																				
Exterior&Interior Works																				
External Walls: Brickwork																				
Internal Walls: Plastering, Gypsum Plastering																				
Floors: Screeding, Tile Placing																				
Ceilings: Panel Placing																				
Roofing Works																				
Flooring&Waterproofing																				
Parapet Installation																				
Façade Works																				
Panel Installation																				
Glass Installation																				
Plumbing Works: Water System, Sewage and etc.																				
Electrical Works: Elevator&Transformer Installation, Lighting and etc.																				
HVAC: Heating, Ventilating, Air Conditioning and etc.																				
Landscape Works: Asphalt Covering, Green Zone and etc.																				
Finishing Works																				
Inspection and Commissioning																				
Final Cleaning																				
Furniture Installation																				
Temporary Stations Removing																				

5.5 Quality management

Quality, time and cost are the main constraints of the successful construction projects that achieve the primary objectives of the construction sphere. Quality management is used to provide the environment where the construction processes are effectively implemented leading to the successful performance of the project. The aim of the performing this management system is to prevent defects that might be occurred during construction, satisfy the customer needs and provide continuous improvement. Quality management system involves the following processes (Mane and Patil 2015):

- Quality Planning;
- Quality Assurance;
- Quality Control.

Quality can be defined in terms of the construction as a conformance to the customer requirements. Quality Planning involves an identification of quality standards that are suitable for the project and preparation of quality plan. One of the world's largest standard developers is International Organization for Standardization (ISO). ISO 9000 is the package of standards and guidelines that are referred to the quality management. It involves eight quality management principles that include customer focus, leadership, involvement of projects, process approach, system approach, continuous improvement, decision-making and supplier relationships. Construction projects have to follow the regulations that are discussed in these standards. After that, quality plan is prepared. In order to develop, it, firstly, the project information, requirements of clients, risk register and schedule data are gathered. Secondly, according to the economic analysis provided, quality planning activities are prepared. Then important quality metrics should be identified and, lastly, improvement plan is developed (Vaidyanathan 2013).

Quality assurance is used to estimate the project performance in a timely manner and ensure the customer that the project satisfies the quality standards. It is performed by creating inspection forms, checklists and periodically monitoring each phase of the work by material quality and system function (Caldeira 2012).

Lastly, quality control is the verification that the project outcomes meet the acceptable quality. It can be controlled by the following methods:

- Cause-and-effect diagrams;

- Run charts;
- Process maps;
- Control charts;
- Reviews and others.

Quality management plan is prepared for the academic building and shown in Appendix D.

5.6 Occupational safety and health

Since the construction site is one of the hazardous places to work, injuries, illnesses and even incidents of fatalities can occur. Hence, the safety and health of workers must be guaranteed and know for certain that all construction processes are not risky for them. In order to ensure that International Labor Organization developed international guidelines, handbooks and standards that prevents accidents during construction (ELCOSH 2009). Kazakhstan has its own Occupational Safety and Health Standard in construction (SNaR RK 1.03-05-2011). One of the most common labor standards is the Occupational Safety and Health in construction industry. It was designed to guide managers in terms of safety and how to inspect the overall construction. It includes key concepts of safety and health measurements, risk assessments, monitoring, planning of inspection programs and evaluation. Moreover, rules to use construction equipment, working at height, excavation, earthwork and underground works, wearing personal protective equipment and others are considered. According to this OSH standard, there are 2 categories of common risks in construction: hazards/risks that may cause occupational accidents immediately and hazards/risks that may cause occupational illnesses in a long term. In order to prevent that, construction company must have manager/inspector of OSH and independent OSH organization that will monitor working conditions (International Labor Organization 1992).

During the construction period of academic building, all the measures will be considered according to these international and local OSH standards. With the help of these documents, effective inspection program and performing on-site OSH inspections at the construction site will be realized.

6 Conclusion and Recommendations

Milky Way (MW) Company offered detailed design of the 10-story academic facility at Nazarbayev University in Astana, Kazakhstan. Architectural, structural, geotechnical and management analyses were performed based on internationally and locally accepted standards. It can be stated that the aim of the project was successfully met.

Structural loads including wind, snow, live and dead loads have been calculated in order to estimate the total horizontal and vertical loads applied on the structure. These loads have been calculated according to the Eurocode. As they had been calculated manually, the obtained values were set into SAP2000 in order to analyze and design. Before the starting design part, it is important to make the frame analysis. By comparing the hand calculations with SAP2000 results, it was found that the output is close and has allowable error. Secondly, by using the SAP2000, the design of section by providing the minimum required reinforcement was done. Additionally, hand calculations were done to prove that SAP2000 results are reasonable and appropriate. Also it contains the reinforcement detailing as shear reinforcement, lab length, cracking and deflection check. Finally, it was estimated that all beam do not need deflection check because they are in the allowable range, and similarly, with the slab deflection check. The appendix contains the reinforcement and shear detailing of beams and columns for the critical frame with rebar sizes. All results have been taken from SAP2000; the rebaring was manually calculated and arranged which can be seen in CAD drawings.

Geotechnical analysis includes detailed foundation design with its specifications and drawings. First of all, soil profile of the site with physical-mechanical properties of soil layers and test results were analyzed. Foundation type was chosen based on the applied loads including wind, dead, live and snow loads. The first challenge of geotechnical analysis was due to high axial loads which were properly performed by appropriate sizing of piles. The second challenge involved significant difference of loads among tower and ground floor; therefore, various designs were performed. Since the building is high-rise and due to windy climate of the city, it undergoes high horizontal loads. Hence, the footings were checked for horizontal resistance.

Project planning and management of the project is an essential part of the project development since it is directly influence the completion of the building. It was developed by computing top-down and bottom-up estimates, performing economic analysis, identifying related risks, scheduling and preparing temporary site layout drawings. Moreover, it includes quality management and occupational safety and health measurements. From the analysis, it was determined that the project is economically and financially feasible.

For further development of the project, some recommendations should be considered. 3D model is able to provide more accurate reinforcement and interior beam detailing. In addition, it can decrease effect of wind load by decreasing interstorey drift. Regarding geotechnical design, more detailed analysis with Plaxis software can be developed. Moreover, quantity take-off estimate should be performed using computer softwares. Project planning can be improved by using Primavera which will enhance accurate scheduling.

7 Reference List

American Concrete Institue (2014) ‘ACI standard building code requirements for reinforced concrete (ACI 318-56)’, ACI Journal Proceedings, 104(5). doi: 10.14359/11656

Caldeira E. (2012). Construction Quality Assurance/Quality Control Blog. First Time Quality.http://www.firsttimequality.com/Blog/bid/75546/How-to-Write-a-Construction-Quality-Control-Plan

Coduto D. (2001). Foundation design: principles and practices. New Jersey: Prentice Hall.

Construction Norms and Regulations of Republic of Kazakhstan (2007) CNaR RK 2.01.07-85* p1.20. Concrete and reinforced concrete structures. Astana, Ministry of Industry and Trade of the Republic of Kazakhstan: Construction and housing committee.

Das B. (2007). Principles of Foundation Engineering.

Design of piled foundations. (2010). Group piles. Available from: https://akkhrakh.files.wordpress.com/2011/09/7-design-of-piled-foundation.pdf [Accessed: 20th March 2017]

Elbeltagi E. (2007). Construction Management: Cost Estimating. Mansoura University, Egypt. http://deltauniv.edu.eg/new/engineering/wp-content/uploads/Cost-Ch3.pdf

Electronic Library of Construction Occupational Safety and Health (ELCOSH). (2009). Inspecting Occupational Safety and Health in the Construction Industry. http://www.elcosh.org/document/802/d001018/Inspecting%2BOccupational%2BSafety%2Band%2BHealth%2Bin%2Bthe%2BConstruction%2BIndustry.html?show_text=1

Eurocode (2004) EN 1992-1-1-2004. Design of concrete structures - Part 1-1 : General rules and rules for buildings. Brussels, European Committee for Standardization.

Eurocode (2004) EN 1992-1-2-2004. Design of concrete structures - Part 1-2: General rules - Structural fire design. Brussels, European Committee for Standardization.

Goncalves, M. D., Roussenau, M. Z.. (2007). Design Considerations for Curtain Wall Parapets in Cold Climates. Available from: http://www.cebq.org/documents/CurtainWallParapetsinColdClimates.pdf. [Accessed: 15th September 2016]

International Labor Organization. (1992). Safety and health in construction. Geneva, Switzerland. http://www.ilo.org/wcmsp5/groups/public/---ed_protect/---protrav/---safework/documents/normativeinstrument/wcms_107826.pdf

Jensen K. (2009). What Is the Difference Between Estimated Costs & a Rough Order of Magnitude? Chron. http://smallbusiness.chron.com/difference-between-estimated-costs-rough-order-magnitude-80955.html

Kone. (2015). Elevator Traffic Calculations. Available from: https://toolbox.kone.com/media/mpb/frontpage_mpb/Quick%20Traffic.html?rdrsr c=/media/mpb/frontpage_mpb/Quick%20Traffic.html&rdrtrg=https://toolbox.kon e.com/media/mpb/frontpage_mpb/Quick%20Traffic.html. [Accessed: 25th September 2015].

Leland B. (2014). Basics of engineering economy. 2nd edition, McGraw-Hill. ISBN 9781259080760

Li X., Zhu Y., Zhang Z. (2010). "An LCA-based environmental impact assessment model for construction processes". Building and Environment 45:766-775.

Mane P.P. and Patil J.R. (2015). Quality Management System at Construction Project: A Questionnaire Survey. Journal of Engineering Research and Applications. ISSN : 2248-9622, Vol. 5, Issue 3, (Part-3) March 2015, pp.126-130 http://www.ijera.com/papers/Vol5_issue3/Part%20-%203/W50303126130.pdf

McCormac, J. C., Brown R. H. (2014) Design of Reinforced Concrete. ISBN: 978-1-118-12984-5. Ninth Edition. New York, Wiley.

Miller, N. (2012). Estimating Office Space per Worker. Available from: https://www.sandiego.edu/pipeline/documents/EstimatingOfficeSpaceRequirementsMay12012.pdf. [Accessed: 15th September 2015].

Ospanova G. E. (2015). “Technical report”.

Owens. G. W., Knowles. P. (1994) Steel Designers Manual. The Steel Construction Institute. London, ELBS Blackwell Scientific Publishers.

Rakov, M., (2011) Wind Atlas of Kazakhstan. Project PROON. Available from: http://www.windenergy.kz/files/1298019306_file.pdf

Rempel A.W, J.S. Wettlaufer and M.G. Worster. (2004). “Premelting dynamics in a continuum model of frost heave”. Fluid mech 498: 227.

Scoping guidance on the environmental impact assessment of projects. (2001). A1: Scoping the Environmental Impacts of Construction Works. https://www.gov.uk/government/uploads/system/uploads/attachment_data/file/297123/geho0411btrg-e-e.pdf

Sullivan W. G. (2014). Engineering Economy. 16th ed., Global ed., Pearson Education Limited. ISBN 9781292019499

The Black Book. Version 6. (2015) Guidance to the setting out the seatings. Available from: http://www.chartareaseating.com/pdfs/CAS_Blackbook_CAS_Version%20V6_15.pdf. [Accessed: 10th October 2016]

Woo S, Jung Y, Lee Y, Lee B. (2001). Approximate Cost Estimating System For Construction of steel mill. Construction and Economy Research Institute of Korea http://www.iaarc.org/publications/fulltext/isarc2001-ACE_20.pdf

Zolfagharian S., Zourbakhsh M., Irizarry J., Ressang A., Gheisari M. (2012). Environmental Impacts Assessment on Construction Sites. Construction Research Congress. http://rebar.ecn.purdue.edu/crc2012/papers/pdfs/-21.pdf

Appendix A

Flooring layers:

- Floor – the top layer of the floor, directly exposed to operational impacts;
- Intermediate layer – a layer of the floor connecting the coating to the underlying layer of the floor;
- Waterproofing – prevents penetration through the floor of wastewater and other fluids, as well as protecting the entire floor structure from groundwater;
- Screed – serves to align the surface of the underlying layer of the floor, to cover various pipelines;
- Insulation – reduces the overall thermal conductivity of the floor;
- Sound insulation – p revents the penetration of impact noise in the room, or out of it;
- Underlayment – distributes the load on the subgrade;
- Subgrade – natural or artificial floor support, perceiving all the loads transmitted from the floor.

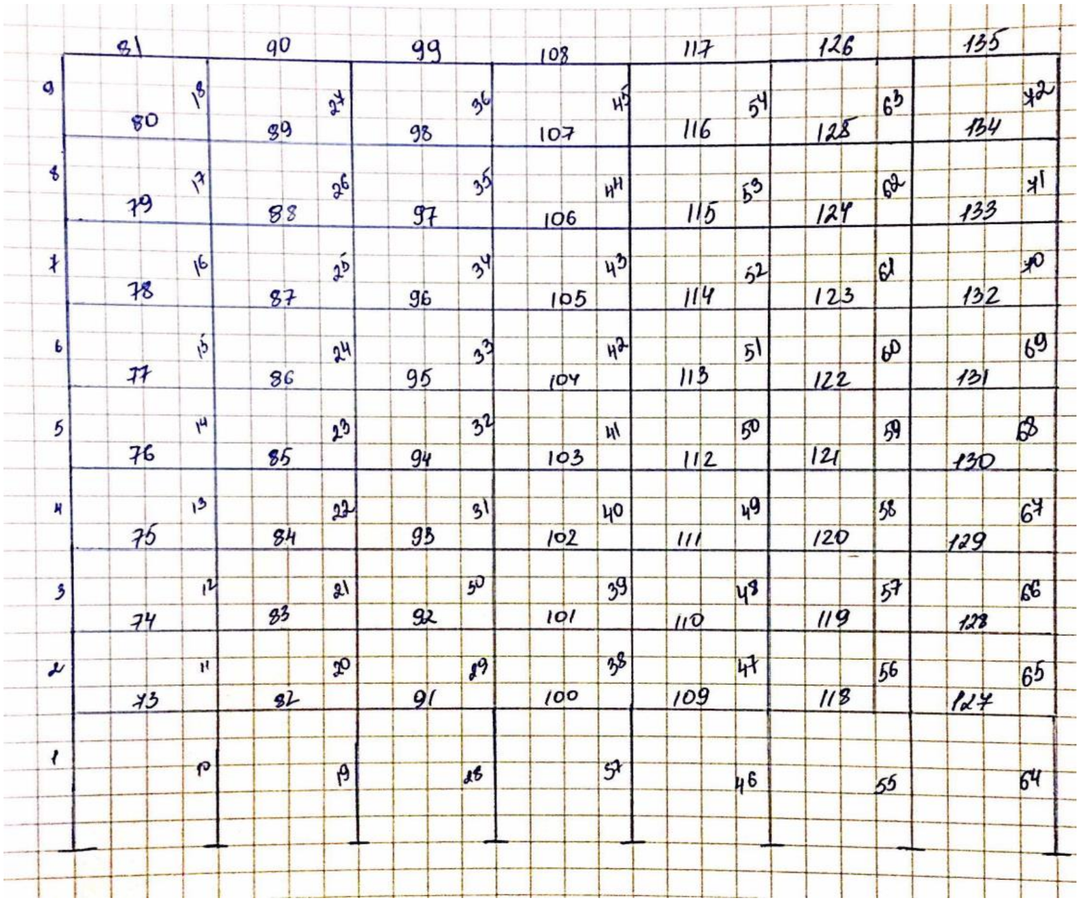


Figure 1. Numerated frame members of the building

Table 1. Comparison of SAP2000 and hand calculations of axial, shear and moment forces under wind load

	Station, m	Axial force (SAP2000) , kN	Axial force (hand), kN	Error for axial force	Shear force (SAP2000), kN	Shear force (hand), kN	Error for shear force	Moment (SAP2000), kN*m	Moment (hand), kN*m	Error for moment
1	0	243.6	192.9	20.8	97.0	36.8	62.1	396.8	331.1	16.6
	3.15	243.6	192.9	20.8	97.0	36.8	62.1	105.7	115.9	9.6
	6.3	243.6	192.9	20.8	97.0	36.8	62.1	-185.4	-115.9	37.5
2	0	161.0	151.6	5.8	36.1	33.0	8.5	55.3	54.4	1.6
	1.65	161.0	151.6	5.8	36.1	33.0	8.5	-4.2		

	3.3	161.0	151.6	5.8	36.1	33.0	8.5	-63.7	-54.4	14.6
3	0	123.0	120.8	1.8	29.9	29.4	1.4	42.6	48.6	14.2
	1.65	123.0	120.8	1.8	29.9	29.4	1.4	-6.7		
	3.3	123.0	120.8	1.8	29.9	29.4	1.4	-56.0	-48.6	13.2
4	0	88.7	87.4	1.5	28.1	26.5	5.9	40.7	43.7	7.3
	1.65	88.7	87.4	1.5	28.1	26.5	5.9	-5.7		
	3.3	88.7	87.4	1.5	28.1	26.5	5.9	-52.2	-43.7	16.2
5	0	60.7	60.9	0.4	21.3	20.7	2.8	27.8	34.2	22.9
	1.65	60.7	60.9	0.4	21.3	20.7	2.8	-7.4		
	3.3	60.7	60.9	0.4	21.3	20.7	2.8	-42.6	-34.2	19.6
6	0	38.7	38.9	0.4	16.0	15.2	4.6	19.6	25.2	28.4
	1.65	38.7	38.9	0.4	16.0	15.2	4.6	-3.5		
	3.3	38.7	38.9	0.4	16.0	15.2	4.6	-26.6	-25.2	5.3
7	0	21.2	21.2	0.2	15.2	15.0	1.4	24.4	24.8	1.4
	1.65	21.2	21.2	0.2	15.2	15.0	1.4	-0.7		
	3.3	21.2	21.2	0.2	15.2	15.0	1.4	-25.8	-24.8	4.0
8	0	8.8	8.7	0.9	8.6	8.3	3.5	12.6	13.7	9.0
	1.65	8.8	8.7	0.9	8.6	8.3	3.5	-1.6		
	3.3	8.8	8.7	0.9	8.6	8.3	3.5	-15.8	-13.7	13.5
9	0	2.1	2.0	3.4	3.6	3.5	3.3	4.8	5.8	20.1
	1.65	2.1	2.0	3.4	3.6	3.5	3.3	-1.2		
	3.3	2.1	2.0	3.4	3.6	3.5	3.3	-7.1	-5.8	18.5
10	0	-5.4	0		65.3	73.6	12.7	234.5	231.7	1.2
	3.15	-5.4	0		65.3	73.6	12.7	38.7		
	6.3	-5.4	0		65.3	73.6	12.7	-157.1	-231.7	47.5
11	0	-1.0	0		63.8	64.0	0.4	147.0	105.6	28.2
	1.65	-1.0	0		63.8	64.0	0.4	17.0		
	3.3	-1.0	0		63.8	64.0	0.4	-112.9	-105.6	6.5
12	0	-0.4	0		60.7	58.9	3.1	90.7	97.2	7.2
	1.65	-0.4	0		60.7	58.9	3.1	-9.6		
	3.3	-0.4	0		60.7	58.9	3.1	-109.8	-97.2	11.5
13	0	-0.3	0		48.8	48.9	0.3	73.6	80.7	9.7
	1.65	-0.3	0		48.8	48.9	0.3	-6.8		
	3.3	-0.3	0		48.8	48.9	0.3	-87.3	-80.7	7.6
14	0	-0.2	0		39.9	39.5	1.2	60.7	65.1	7.3
	1.65	-0.2	0		39.9	39.5	1.2	-5.2		
	3.3	-0.2	0		39.9	39.5	1.2	-71.1	-65.1	8.4
15	0	-0.1	0		31.8	30.5	4.2	47.0	50.3	7.0

	1.65	-0.1	0		31.8	30.5	4.2	-5.5		
	3.3	-0.1	0		31.8	30.5	4.2	-58.0	-50.3	13.2
16	0	-0.2	0		20.9	22.0	5.5	33.1	36.4	9.8
	1.65	-0.2	0		20.9	22.0	5.5	-1.4		
	3.3	-0.2	0		20.9	22.0	5.5	-35.9	-36.4	1.5
17	0	-0.1	0		13.8	14.1	2.4	22.0	23.3	5.8
	1.65	-0.1	0		13.8	14.1	2.4	-0.7		
	3.3	-0.1	0		13.8	14.1	2.4	-23.5	-23.3	0.9
18	0	0.3	0		6.5	6.8	4.3	10.0	11.2	12.1
	1.65	0.3	0		6.5	6.8	4.3	-0.7		
	3.3	0.3	0		6.5	6.8	4.3	-11.4	-11.2	2.1
19	0	-2.2	0		54.4	59.6	9.6	212.6	187.6	11.7
	3.15	-2.2	0		54.4	59.6	9.6	49.5		
	6.3	-2.2	0		54.4	59.6	9.6	-113.6	-187.6	65.1
20	0	-2.3	0		53.1	57.0	7.4	81.2	94.0	15.8
	1.65	-2.3	0		53.1	57.0	7.4	-6.4		
	3.3	-2.3	0		53.1	57.0	7.4	-94.0	-94.0	0.0
21	0	-1.1	0		60.9	58.9	3.4	95.7	97.2	1.5
	1.65	-1.1	0		60.9	58.9	3.4	-4.8		
	3.3	-1.1	0		60.9	58.9	3.4	-105.4	-97.2	7.8
22	0	-1.1	0		47.4	48.9	3.2	71.5	80.7	12.9
	1.65	-1.1	0		47.4	48.9	3.2	-6.7		
	3.3	-1.1	0		47.4	48.9	3.2	-84.9	-80.7	5.0
23	0	-0.4	0		39.9	39.5	1.2	58.6	65.1	11.1
	1.65	-0.4	0		39.9	39.5	1.2	-5.6		
	3.3	-0.4	0		39.9	39.5	1.2	-69.9	-65.1	6.8
24	0	-0.3	0		31.0	30.5	1.5	45.8	50.3	9.8
	1.65	-0.3	0		31.0	30.5	1.5	-5.3		
	3.3	-0.3	0		31.0	30.5	1.5	-56.4	-50.3	10.8
25	0	-0.2	0		20.6	21.0	2.3	32.7	34.7	6.2
	1.65	-0.2	0		20.6	21.0	2.3	-1.3		
	3.3	-0.2	0		20.6	21.0	2.3	-35.2	-34.7	1.4
26	0	-0.2	0		13.6	14.0	3.3	21.5	23.1	7.8
	1.65	-0.2	0		13.6	14.0	3.3	-0.9		
	3.3	-0.2	0		13.6	14.0	3.3	-23.3	-23.1	1.0
27	0	-0.4	0		6.8	6.8	1.1	10.4	11.2	7.3
	1.65	-0.4	0		6.8	6.8	1.1	-0.9		
	3.3	-0.4	0		6.8	6.8	1.1	-12.2	-11.2	7.8
28	0	-0.5	0		54.1	61.6	13.9	211.6	193.9	8.4
	3.15	-0.5	0		54.1	61.6	13.9	49.4		
	6.3	-0.5	0		54.1	61.6	13.9	-112.8	-193.9	71.9
29	0	-0.3	0		52.3	54.0	3.3	80.5	89.1	10.6

	1.65	-0.3	0		52.3	54.0	3.3	-5.7		
	3.3	-0.3	0		52.3	54.0	3.3	-92.0	-89.1	3.1
30	0	-0.3	0		57.1	57.9	1.4	89.0	95.5	7.4
	1.65	-0.3	0		57.1	57.9	1.4	-5.2		
	3.3	-0.3	0		57.1	57.9	1.4	-99.4	-95.5	4.0
31	0	-0.2	0		46.5	46.9	0.9	70.4	77.4	9.9
	1.65	-0.2	0		46.5	46.9	0.9	-6.3		
	3.3	-0.2	0		46.5	46.9	0.9	-83.0	-77.4	6.8
32	0	-0.2	0		37.7	38.5	2.0	56.5	63.5	12.4
	1.65	-0.2	0		37.7	38.5	2.0	-5.8		
	3.3	-0.2	0		37.7	38.5	2.0	-68.0	-63.5	6.6
33	0	-0.2	0		29.8	30.1	1.0	44.0	49.7	13.0
	1.65	-0.2	0		29.8	30.1	1.0	-5.2		
	3.3	-0.2	0		29.8	30.1	1.0	-54.4	-49.7	8.7
34	0	-0.2	0		20.4	20.0	1.7	32.4	33.1	2.2
	1.65	-0.2	0		20.4	20.0	1.7	-1.3		
	3.3	-0.2	0		20.4	20.0	1.7	-34.9	-33.1	5.2
35	0	-0.1	0		13.4	13.1	2.0	21.1	22.1	4.5
	1.65	-0.1	0		13.4	13.1	2.0	-0.9		
	3.3	-0.1	0		13.4	13.1	2.0	-23.0	-22.1	4.0
36	0	-0.1	0		6.7	6.8	1.5	10.1	11.2	10.4
	1.65	-0.1	0		6.7	6.8	1.5	-0.9		
	3.3	-0.1	0		6.7	6.8	1.5	-11.9	-11.2	5.6
37	0	0.1	0		53.9	61.6	14.3	210.9	184.7	12.4
	3.15	0.1	0		53.9	61.6	14.3	49.3		
	6.3	0.1	0		53.9	61.6	14.3	-112.3	-184.7	64.4
38	0	0.2	0		52.2	54.0	3.4	80.4	89.1	10.8
	1.65	0.2	0		52.2	54.0	3.4	-5.7		
	3.3	0.2	0		52.2	54.0	3.4	-91.8	-89.1	3.0
39	0	0.3	0		56.8	57.9	1.8	88.6	95.5	7.8
	1.65	0.3	0		56.8	57.9	1.8	-5.2		
	3.3	0.3	0		56.8	57.9	1.8	-99.0	-95.5	3.5
40	0	0.3	0		46.5	46.9	0.8	70.6	77.4	9.7
	1.65	0.3	0		46.5	46.9	0.8	-6.2		
	3.3	0.3	0		46.5	46.9	0.8	-83.0	-77.4	6.7
41	0	0.3	0		37.8	38.5	1.8	56.6	63.5	12.2
	1.65	0.3	0		37.8	38.5	1.8	-5.8		
	3.3	0.3	0		37.8	38.5	1.8	-68.1	-63.5	6.7
42	0	0.3	0		29.8	30.1	0.9	44.0	49.7	12.9
	1.65	0.3	0		29.8	30.1	0.9	-5.3		
	3.3	0.3	0		29.8	30.1	0.9	-54.5	-49.7	8.8
43	0	0.2	0		20.4	20.0	1.7	32.4	33.1	2.1

	1.65	0.2	0		20.4	20.0	1.7	-1.3		
	3.3	0.2	0		20.4	20.0	1.7	-34.9	-33.1	5.2
44	0	0.2	0		13.4	13.1	2.1	21.2	21.6	2.3
	1.65	0.2	0		13.4	13.1	2.1	-0.9		
	3.3	0.2	0		13.4	13.1	2.1	-23.1	-21.6	6.3
45	0	0.1	0		6.7	6.8	1.2	10.1	11.2	10.0
	1.65	0.1	0		6.7	6.8	1.2	-0.9		
	3.3	0.1	0		6.7	6.8	1.2	-11.9	-11.2	5.9
46	0	3.9	0		53.8	61.6	14.5	210.4	184.7	12.2
	3.15	3.9	0		53.8	61.6	14.5	49.1		
	6.3	3.9	0		53.8	61.6	14.5	-112.3	-184.7	64.4
47	0	2.1	0		52.9	54.0	2.1	80.5	89.1	10.7
	1.65	2.1	0		52.9	54.0	2.1	-5.7		
	3.3	2.1	0		52.9	54.0	2.1	-91.9	-89.1	3.0
48	0	1.1	0		60.1	57.9	3.7	88.7	95.5	7.6
	1.65	1.1	0		60.1	57.9	3.7	-5.1		
	3.3	1.1	0		60.1	57.9	3.7	-99.0	-95.5	3.5
49	0	1.3	0		47.5	46.9	1.2	70.5	77.4	9.8
	1.65	1.3	0		47.5	46.9	1.2	-6.3		
	3.3	1.3	0		47.5	46.9	1.2	-83.4	-77.4	7.2
50	0	0.6	0		39.1	38.5	1.5	56.6	63.5	12.1
	1.65	0.6	0		39.1	38.5	1.5	-5.8		
	3.3	0.6	0		39.1	38.5	1.5	-68.1	-63.5	6.8
51	0	0.4	0		31.1	30.1	3.1	44.1	49.7	12.6
	1.65	0.4	0		31.1	30.1	3.1	-5.3		
	3.3	0.4	0		31.1	30.1	3.1	-54.2	-49.7	8.3
52	0	0.2	0		20.6	20.0	2.8	32.4	33.1	2.0
	1.65	0.2	0		20.6	20.0	2.8	-1.3		
	3.3	0.2	0		20.6	20.0	2.8	-35.1	-33.1	5.7
53	0	0.2	0		13.6	13.1	3.6	21.2	21.6	2.1
	1.65	0.2	0		13.6	13.1	3.6	-1.0		
	3.3	0.2	0		13.6	13.1	3.6	-23.1	-21.6	6.5
54	0	0.4	0		6.9	6.8	1.7	10.5	11.2	6.6
	1.65	0.4	0		6.9	6.8	1.7	-0.9		
	3.3	0.4	0		6.9	6.8	1.7	-12.2	-11.2	8.4
55	0	5.7	0		63.8	61.6	3.5	229.9	184.7	19.7
	3.15	5.7	0		63.8	61.6	3.5	38.5		
	6.3	5.7	0		63.8	61.6	3.5	-152.8	-184.7	20.8
56	0	5.5	0		77.4	54.0	30.2	143.7	89.1	38.0
	1.65	5.5	0		77.4	54.0	30.2	16.1		
	3.3	5.5	0		77.4	54.0	30.2	-111.6	-89.1	20.1
57	0	5.0	0		59.5	57.9	2.7	89.2	95.5	7.1

	1.65	5.0	0		59.5	57.9	2.7	-9.0		
	3.3	5.0	0		59.5	57.9	2.7	-107.1	-95.5	10.8
58	0	4.4	0		48.9	46.9	4.0	74.2	77.4	4.4
	1.65	4.4	0		48.9	46.9	4.0	-6.5		
	3.3	4.4	0		48.9	46.9	4.0	-87.1	-77.4	11.2
59	0	3.2	0		40.1	38.5	4.1	61.0	63.5	4.0
	1.65	3.2	0		40.1	38.5	4.1	-5.1		
	3.3	3.2	0		40.1	38.5	4.1	-71.3	-63.5	11.0
60	0	3.0	0		31.9	30.1	5.7	47.2	49.7	5.3
	1.65	3.0	0		31.9	30.1	5.7	-5.5		
	3.3	3.0	0		31.9	30.1	5.7	-58.2	-49.7	14.5
61	0	2.1	0		20.9	20.0	4.3	33.2	33.1	0.4
	1.65	2.1	0		20.9	20.0	4.3	-1.4		
	3.3	2.1	0		20.9	20.0	4.3	-35.9	-33.1	7.9
62	0	0.1	0		13.8	13.1	5.2	22.1	21.6	2.0
	1.65	0.1	0		13.8	13.1	5.2	-0.7		
	3.3	0.1	0		13.8	13.1	5.2	-23.6	-21.6	8.4
63	0	-0.3	0		6.5	6.8	3.5	10.0	11.2	11.1
	1.65	-0.3	0		6.5	6.8	3.5	-0.7		
	3.3	-0.3	0		6.5	6.8	3.5	-11.5	-11.2	2.6
64	0	-239.3	-201.6	15.8	72.8	58.8	19.2	290.1	176.3	39.2
	3.15	-239.3	-201.6	15.8	72.8	58.8	19.2	71.8		
	6.3	-239.3	-201.6	15.8	72.8	58.8	19.2	-146.5	-176.3	20.3
65	0	-160.1	-164.0	2.4	45.3	41.3	8.8	81.5	68.1	16.5
	1.65	-160.1	-164.0	2.4	45.3	41.3	8.8	6.8		
	3.3	-160.1	-164.0	2.4	45.3	41.3	8.8	-67.9	-68.1	0.4
66	0	-122.7	-121.9	0.6	27.1	27.4	1.2	37.2	45.3	21.9
	1.65	-122.7	-121.9	0.6	27.1	27.4	1.2	-7.6		
	3.3	-122.7	-121.	0.6	27.1	27.4	1.2	-52.3	-45.3	13.5

			9							
67	0	-88.9	-88.7	0.2	28.7	28.5	0.9	42.6	47.0	10.3
	1.65	-88.9	-88.7	0.2	28.7	28.5	0.9	-4.8		
	3.3	-88.9	-88.7	0.2	28.7	28.5	0.9	-52.2	-47.0	10.0
68	0	-60.8	-60.4	0.7	21.4	20.7	3.1	27.9	34.2	22.5
	1.65	-60.8	-60.4	0.7	21.4	20.7	3.1	-7.4		
	3.3	-60.8	-60.4	0.7	21.4	20.7	3.1	-42.7	-34.2	19.9
69	0	-38.9	-38.6	0.6	14.0	14.2	1.5	19.7	23.5	19.6
	1.65	-38.9	-38.6	0.6	14.0	14.2	1.5	-3.5		
	3.3	-38.9	-38.6	0.6	14.0	14.2	1.5	-26.7	-23.5	11.9
70	0	-21.3	-21.0	1.4	15.3	15.0	1.6	24.5	24.8	1.3
	1.65	-21.3	-21.0	1.4	15.3	15.0	1.6	-0.7		
	3.3	-21.3	-21.0	1.4	15.3	15.0	1.6	-25.9	-24.8	4.2
71	0	-8.8	-8.7	1.2	8.6	8.1	6.7	12.6	13.3	5.2
	1.65	-8.8	-8.7	1.2	8.6	8.1	6.7	-1.6		
	3.3	-8.8	-8.7	1.2	8.6	8.1	6.7	-15.9	-13.3	16.4
72	0	-2.1	-2.0	3.8	3.7	3.4	7.4	4.9	5.6	14.8
	1.65	-2.1	-2.0	3.8	3.7	3.4	7.4	-1.2		
	3.3	-2.1	-2.0	3.8	3.7	3.4	7.4	-7.2	-5.6	22.1
73	0				45.7	41.3	9.6	240.7	113.5	52.8
	5.5				45.7	41.3	9.6	-213.9	-113.5	46.9
74	0				38.0	36.9	3.0	106.3	101.4	4.6
	5.5				38.0	36.9	3.0	-102.7	-101.4	1.3
75	0				34.3	34.3	0.2	96.7	94.4	2.3
	5.5				34.3	34.3	0.2	-91.9	-94.4	2.7
76	0				28.1	27.5	1.9	80.0	75.7	5.4
	5.5				28.1	27.5	1.9	-74.3	-75.7	1.9
77	0				21.9	21.0	4.2	62.2	57.7	7.1
	5.5				21.9	21.0	4.2	-58.4	-57.7	1.1
78	0				17.5	17.8	1.3	51.0	48.8	4.2
	5.5				17.5	17.8	1.3	-45.5	-48.8	7.3
79	0				12.4	11.9	4.6	38.4	32.6	15.1
	5.5				12.4	11.9	4.6	-29.9	-32.6	9.0
80	0				6.7	6.5	3.1	20.6	17.8	13.7
	5.5				6.7	6.5	3.1	-16.1	-17.8	10.6
81	0				2.1	2.0	3.4	7.1	5.6	21.4
	5.5				2.1	2.0	3.4	-4.4	-5.6	26.1
82	0				34.2	41.3	20.7	90.1	113.5	25.9
	5.5				34.2	41.3	20.7	-97.9	-113.5	15.9
83	0				36.4	36.9	1.2	100.9	101.4	0.5
	5.5				36.4	36.9	1.2	-99.4	-	2.0

									101.4	
84	0				33.3	34.3	3.0	91.5	94.4	3.2
	5.5				33.3	34.3	3.0	-91.8	-94.4	2.8
85	0				26.9	27.5	2.4	73.7	75.7	2.7
	5.5				26.9	27.5	2.4	-74.2	-75.7	2.0
86	0				21.7	21.0	3.5	59.7	57.7	3.3
	5.5				21.7	21.0	3.5	-59.8	-57.7	3.5
87	0				16.6	17.8	6.8	45.6	48.8	7.1
	5.5				16.6	17.8	6.8	-45.9	-48.8	6.3
88	0				10.3	10.9	5.1	28.0	29.8	6.6
	5.5				10.3	10.9	5.1	-28.8	-29.8	3.5
89	0				6.3	6.5	2.8	17.4	17.8	2.2
	5.5				6.3	6.5	2.8	-17.2	-17.8	3.5
90	0				2.4	2.3	3.4	7.0	6.4	8.4
	5.5				2.4	2.3	3.4	-6.3	-6.4	2.1
91	0				32.3	37.8	17.2	96.9	113.5	17.1
	6				32.3	37.8	17.2	-96.8	-113.5	17.3
92	0				30.1	31.8	5.4	90.4	95.4	5.5
	6				30.1	31.8	5.4	-90.5	-95.4	5.4
93	0				28.4	28.6	1.0	85.1	85.9	1.0
	6				28.4	28.6	1.0	-85.2	-85.9	0.8
94	0				23.2	23.3	0.5	69.4	69.9	0.8
	6				23.2	23.3	0.5	-69.8	-69.9	0.1
95	0				18.7	19.2	3.1	55.8	57.7	3.4
	6				18.7	19.2	3.1	-56.1	-57.7	2.9
96	0				14.5	14.4	0.0	43.2	43.3	0.3
	6				14.5	14.4	0.0	-43.5	-43.3	0.5
97	0				9.3	9.6	3.3	27.9	28.9	3.7
	6				9.3	9.6	3.3	-28.1	-28.9	2.8
98	0				5.5	5.7	3.8	16.5	17.2	4.4
	6				5.5	5.7	3.8	-16.7	-17.2	3.0
99	0				2.0	2.0	1.8	5.9	5.9	0.0
	6				2.0	2.0	1.8	-6.1	-5.9	3.3
100	0				32.2	37.8	17.7	96.5	113.5	17.6
	6				32.2	37.8	17.7	-96.4	-113.5	17.7
101	0				30.1	31.8	5.6	90.4	95.4	5.5
	6				30.1	31.8	5.6	-90.3	-95.4	5.6
102	0				28.2	29.6	5.0	84.8	88.9	4.9
	6				28.2	29.6	5.0	-84.7	-88.9	5.0
103	0				23.2	24.3	4.6	69.7	72.9	4.6
	6				23.2	24.3	4.6	-69.7	-72.9	4.6
104	0				18.6	19.2	3.3	55.9	57.7	3.3

	6				18.6	19.2	3.3	-55.9	-57.7	3.2
105	0				14.4	14.4	0.1	43.3	43.3	0.1
	6				14.4	14.4	0.1	-43.3	-43.3	0.0
106	0				9.3	9.6	3.8	27.9	28.9	3.7
	6				9.3	9.6	3.8	-27.9	-28.9	3.6
107	0				5.5	5.7	4.9	16.4	17.2	5.1
	6				5.5	5.7	4.9	-16.4	-17.2	4.9
108	0				1.9	2.0	1.8	5.8	5.9	1.4
	6				1.9	2.0	1.8	-5.8	-5.9	1.7
109	0				32.1	37.8	17.9	96.4	113.5	17.7
	6				32.1	37.8	17.9	-96.2	-113.5	18.0
110	0				30.0	31.8	6.0	90.1	95.4	5.9
	6				30.0	31.8	6.0	-89.8	-95.4	6.2
111	0				28.3	29.6	5.0	84.8	88.9	4.9
	6				28.3	29.6	5.0	-84.7	-88.9	5.0
112	0				23.2	23.3	0.4	69.9	69.9	0.0
	6				23.2	23.3	0.4	-69.5	-72.9	4.9
113	0				18.7	18.2	2.4	56.1	54.7	2.5
	6				18.7	18.2	2.4	-56.0	-57.7	3.0
114	0				14.5	14.4	0.2	43.6	43.3	0.6
	6				14.5	14.4	0.2	-43.3	-43.3	0.0
115	0				9.4	9.6	3.1	28.2	28.9	2.6
	6				9.4	9.6	3.1	-28.0	-28.9	3.2
116	0				5.6	5.7	3.4	16.8	17.2	2.6
	6				5.6	5.7	3.4	-16.6	-17.2	3.6
117	0				2.0	2.0	2.3	6.3	5.9	6.7
	6				2.0	2.0	2.3	-7.1	-5.9	16.9
118	0				33.9	33.4	1.6	97.1	91.8	5.4
	5.5				33.9	33.4	1.6	-89.5	-91.8	2.6
119	0				36.0	35.2	2.0	98.3	96.9	1.4
	5.5				36.0	35.2	2.0	-99.6	-96.9	2.7
120	0				33.1	32.3	2.2	91.1	88.9	2.4
	5.5				33.1	32.3	2.2	-90.7	-88.9	2.0
121	0				26.9	26.5	1.5	74.2	72.9	1.7
	5.5				26.9	26.5	1.5	-73.8	-72.9	1.2
122	0				21.8	23.9	9.5	60.0	65.7	9.5
	5.5				21.8	23.9	9.5	-59.9	-65.7	9.7
123	0				16.7	16.9	1.0	46.0	46.3	0.8
	5.5				16.7	16.9	1.0	-45.8	-46.3	1.1
124	0				10.4	10.9	4.7	28.9	29.8	3.2
	5.5				10.4	10.9	4.7	-28.1	-29.8	6.0
125	0				6.3	6.3	1.0	17.3	17.2	0.4

	5.5				6.3	6.3	1.0	-17.4	-17.2	1.1
126	0				2.4	2.0	16.7	6.3	5.6	11.4
	5.5				2.4	2.0	16.7	-7.1	-5.6	21.1
127	0				79.1	41.3	47.8	207.1	113.5	45.2
	5.5				79.1	41.3	47.8	-228.0	-113.5	50.2
128	0				37.5	36.9	1.7	101.2	101.4	0.2
	5.5				37.5	36.9	1.7	-105.0	-101.4	3.4
129	0				33.7	34.3	1.8	90.6	94.4	4.2
	5.5				33.7	34.3	1.8	-94.9	-94.4	0.5
130	0				28.1	27.5	2.1	74.4	75.7	1.7
	5.5				28.1	27.5	2.1	-80.1	-75.7	5.5
131	0				22.0	21.0	4.5	58.5	57.7	1.3
	5.5				22.0	21.0	4.5	-62.3	-57.7	7.4
132	0				17.6	17.8	1.0	45.6	48.8	7.1
	5.5				17.6	17.8	1.0	-51.1	-48.8	4.5
133	0				12.4	11.9	4.8	29.9	32.6	9.0
	5.5				12.4	11.9	4.8	-38.5	-32.6	15.3
134	0				6.7	6.5	3.6	16.1	17.9	11.0
	5.5				6.7	6.5	3.6	-20.8	-17.9	13.9
135	0				2.1	2.0	4.2	4.5	5.6	25.4
	5.5				2.1	2.0	4.2	-7.1	-5.6	21.1

Appendix B-2

Table 1. Influence factors

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	a1=1.0	a1=1.0
	other than straight	a1=0.7 if Cd>3d otherwise a1=1.0	
Concrete cover	straight	a2=1-0.15(Cd-d)/d>0.7<1.0	a2=1.0
	Other than straight	a2=1-0.15(Cd-3d)/d>0.7<1.0	a2=1.0
Confinement by transverse reinforcement not welded to main reinforcement	All types	a3=1-Kƛ>0.7<1.0	a3=1.0
Confinement by welded transverse reinforcement	Alt types, position and sizes	a4=0.7	a4=0.7
Condinement by transverse pressure	All types	A5=1-0.04p>0.7<1.0	
Where,	ƛ=(∑Ast-∑Ast,min)/As Ast ∑Ast cross-section area of the transverse reinforcement along the design anchorage length lbd ∑Ast,min section area of the minimum transverse reinforcement =0.25 As for beam and 0 for slabs As area of a single anchorage bar with maximum bar diameter K=0.05 for middle rebar p transverse pressure at ultimate limit state along lbd		

Table 2. Anchorage and lap lengths for concrete class C25/30 (mm)

		Bond condition	Reinforcement in tension, bar diameter, d(mm)								Reinforcement in compression
			8	10	12	16	20	25	32	40	
Anchorage length, lbd	Straight bars only	Good	230	320	410	600	780	1010	1300	1760	40d
		Poor	330	450	580	850	1120	1450	1850	2510	58d
	Other bars	Good	320	410	490	650	810	1010	1300	1760	40d
		Poor	460	580	700	930	1160	1450	1850	2510	58d
Lap length, lbd	50% lapped in one direction (a6=1.4)	Good	320	440	570	830	1090	1420	1810	2460	57d
		Poor	460	630	820	1190	1560	2020	2590	3520	81d
	100% lapped in one direction (a6=1.5)	Good	340	470	610	890	1170	1520	1940	2640	61d
		Poor	490	680	870	1270	1670	2170	2770	3770	87d

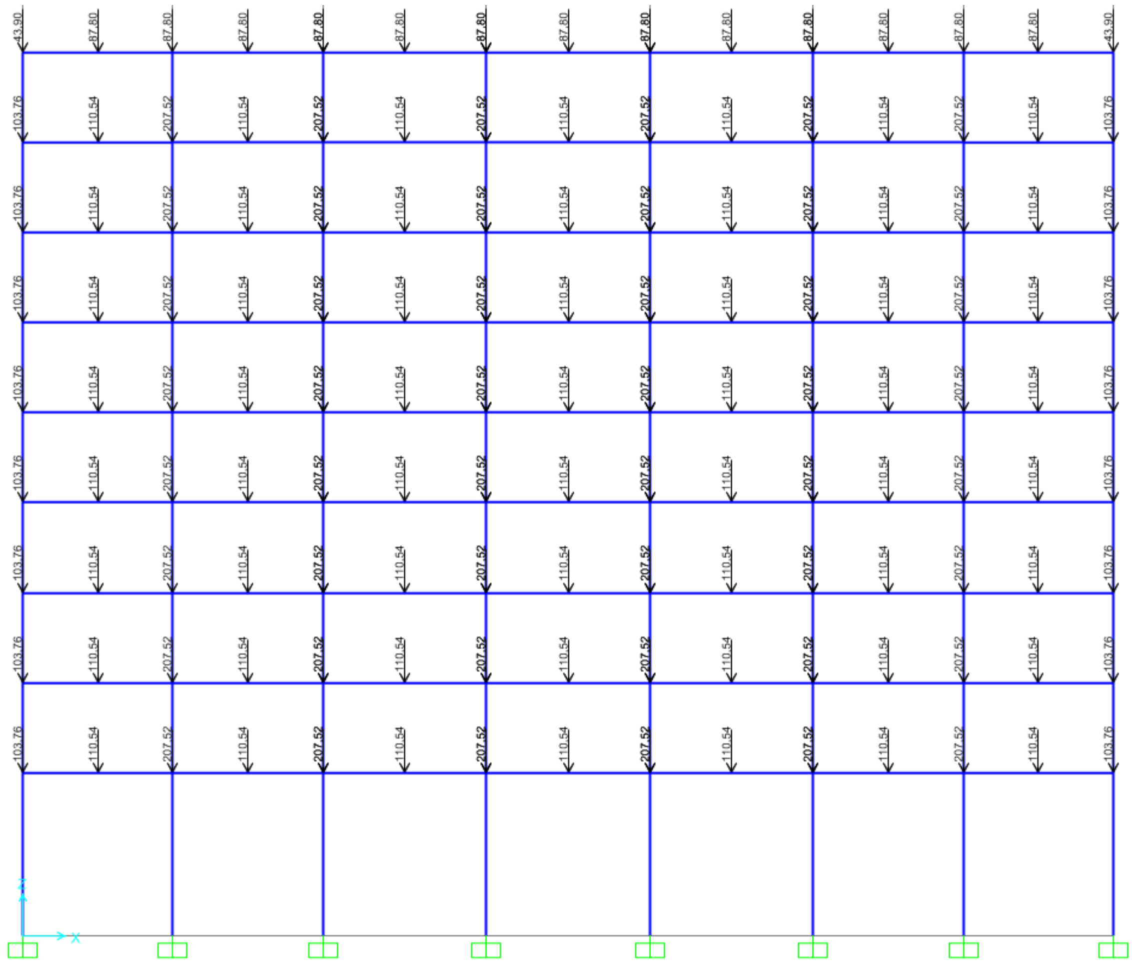


Figure 2. Dead load applied in Z-Y frame

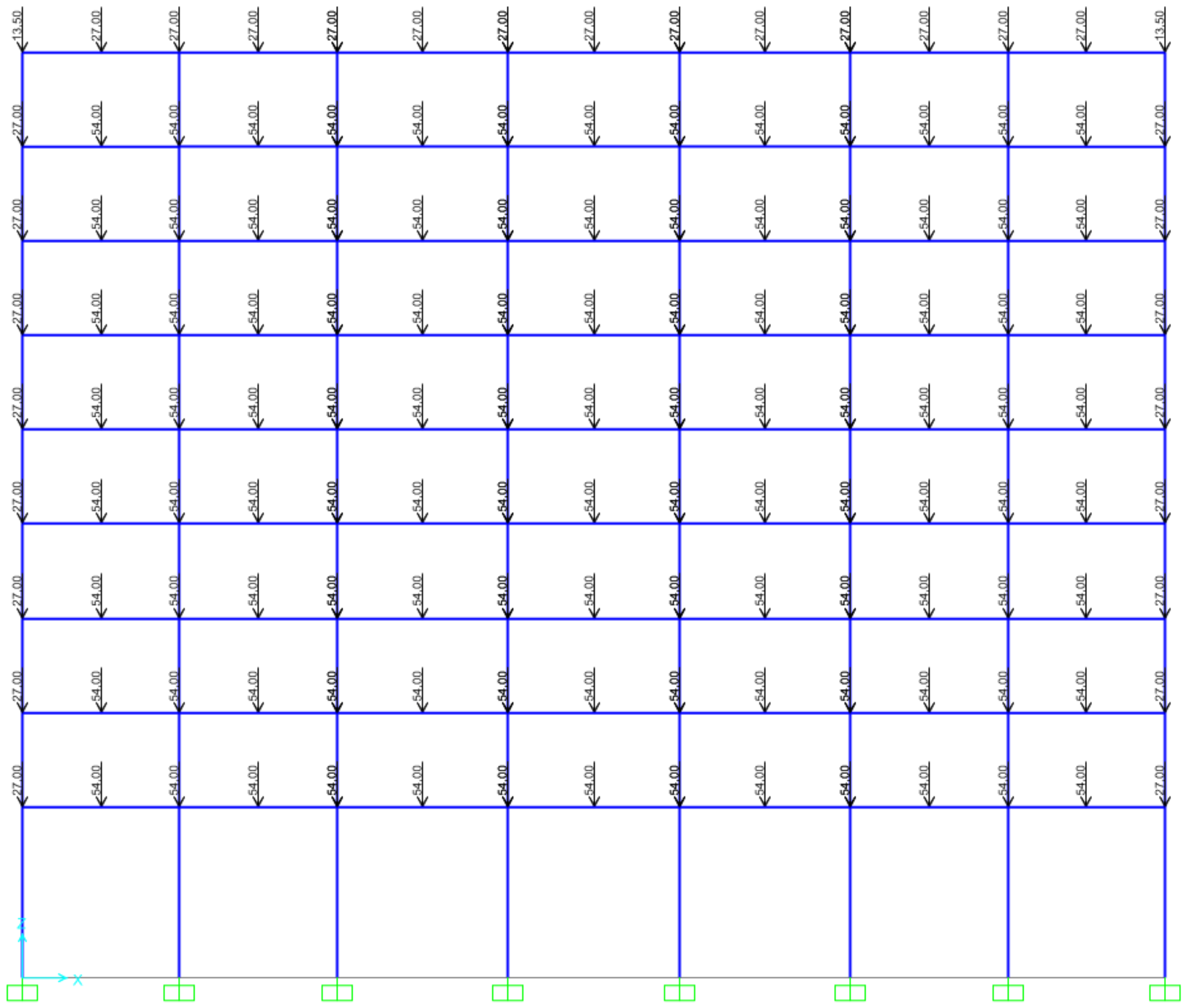


Figure 3. Live load applied in Z-Y frame

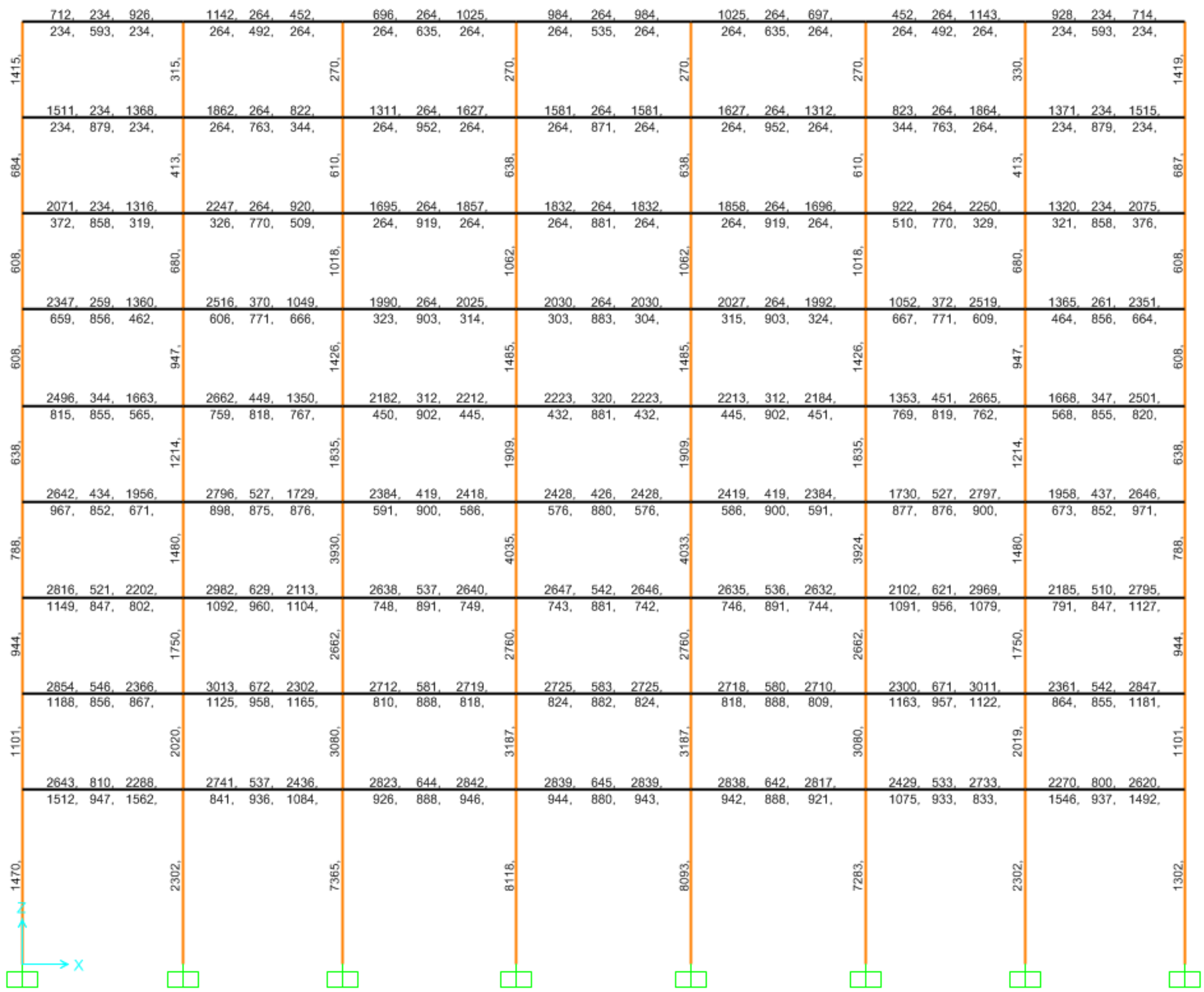


Figure 4. Design section for Z-y frame

SAP2000 results for halls

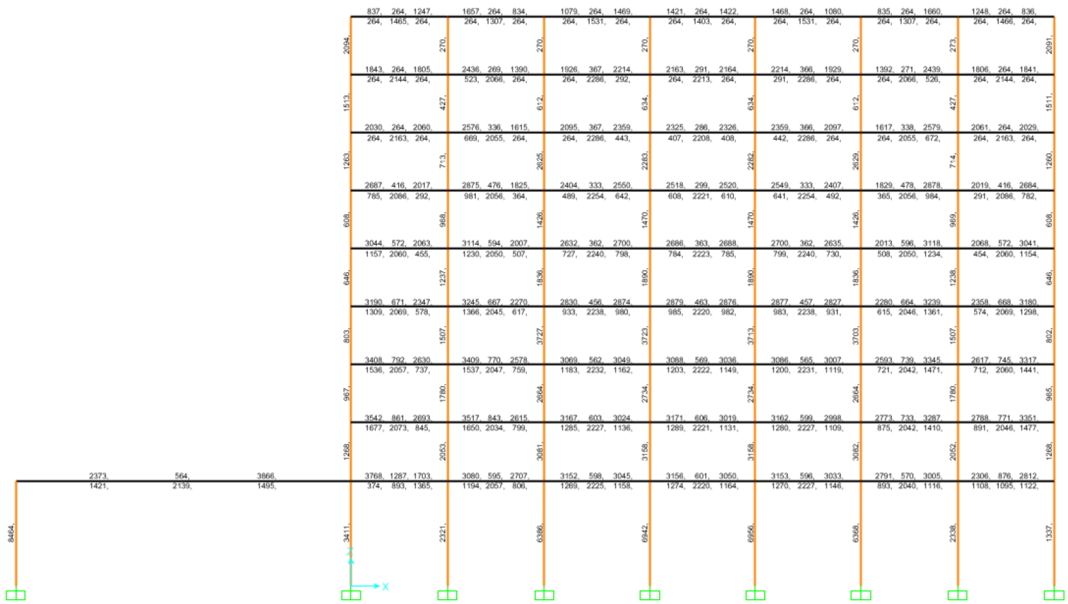


Figure 4. Large hall (19.070m) SAP2000 analyses and check, design results.

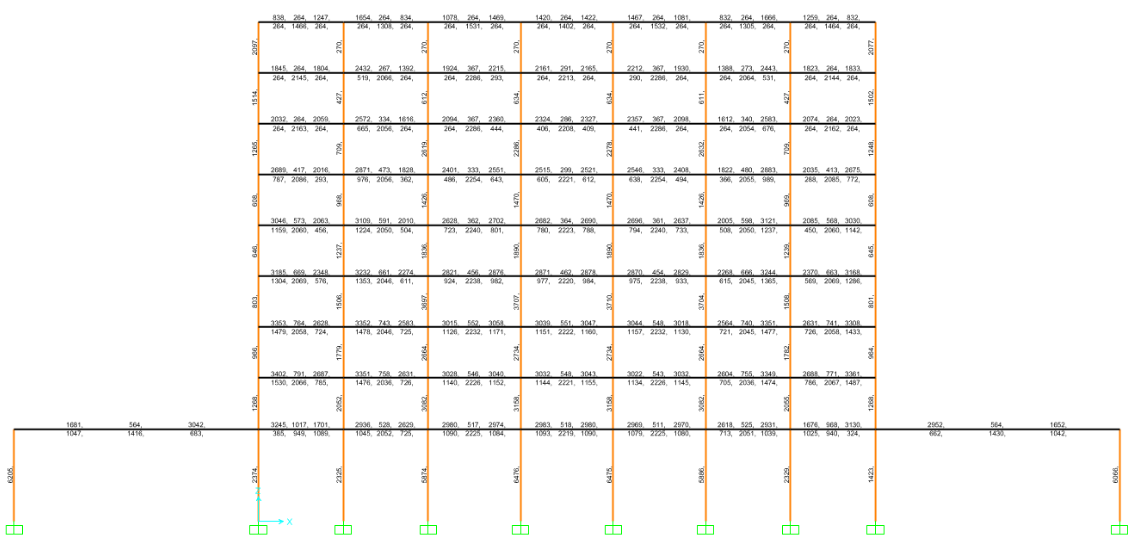


Figure 5. Small (15.080m) hall SAP2000 analyses and check, design results.

Appendix B-3

Table 1. Column detailing

A1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	9	2924.99	3220	10.086	4@20 + 4@25
8	8	1576.524	1608	1.997	8@16
7	7	1377.619	1608	16.723	8@16
6	6	2025	1963	-3.062	4@25
5	5	2025	1963	-3.062	4@25
4	4	2025	1963	-3.062	4@25
3	3	3025	3216	6.314	4@32
2	2	3025	3216	6.314	4@32
1	1	3025	3216	6.314	4@32

B1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	18	900	904	0.444	8@12
8	17	900	904	0.444	8@12
7	16	900	904	0.444	8@12
6	15	2025	1963	-3.062	4@25
5	14	2025	1963	-3.062	4@25
4	13	2025	1963	-3.062	4@25
3	12	3025	3216	6.314	4@32
2	11	3025	3216	6.314	4@32
1	10	3025	3216	6.314	4@32

C1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	27	900	904	0.444	8@12
8	26	900	904	0.444	8@12
7	25	1018.124	1256	23.364	4@20
6	24	2025	1963	-3.062	4@25
5	23	3929.726	3926	-0.095	8@25
4	22	2025	1963	-3.062	4@25
3	21	3025	3216	6.314	4@32
2	20	3180	3216	1.132	4@32
1	19	7365.344	8243	11.916	4@40+4@32

D1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	36	900	904	0.444	8@12
8	35	900	904	0.444	8@12
7	34	1019	1256	23.258	4@20
6	33	2025	1963	-3.062	4@25
5	32	2025	1963	-3.062	4@25
4	31	4034.771	3926	-2.696	8@25
3	30	3025	3216	6.314	4@32
2	29	3187.427	3216	0.896	4@32
1	28	8117.989	8243	1.540	4@40+4@32

E1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	45	900	904	0.444	8@12
8	44	900	904	0.444	8@12
7	43	1061.823	1256	18.287	4@20
6	42	2025	1963	-3.062	4@25
5	41	2025	1963	-3.062	4@25
4	40	4032.734	3926	-2.647	8@25
3	39	3025	3216	6.314	4@32
2	38	3187.427	3216	0.896	4@32
1	37	8093.471	8243	1.848	4@40+4@32

F1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	54	900	904	0.444	8@12
8	53	900	904	0.444	8@12
7	52	1019	1256	23.258	4@20
6	51	2025	1963	-3.062	4@25
5	50	2025	1963	-3.062	4@25
4	49	3180	3216	1.132	4@32
3	48	3180	3216	1.132	4@32
2	47	3574.19	3927	9.871	8@25
1	46	7282.572	6990	-4.017	4@40+4@25

G1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	63	900	904	0.444	8@12
8	62	900	904	0.444	8@12
7	61	900	904	0.444	8@12
6	60	2025	1963	-3.062	4@25
5	59	2025	1963	-3.062	4@25
4	58	2025	1963	-3.062	4@25
3	57	3025	3216	6.314	4@32
2	56	3025	3216	6.314	4@32
1	55	3025	3216	6.314	4@32

H1

Floor	Column #	Area Required (mm2)	Area Provided (mm2)	Error %	Rebar
9	72	2971.172	3216	8.240	4@32
8	71	1568.196	1608	2.538	8@16
7	70	1374.874	1608	16.956	8@16
6	69	2025	1963	-3.062	4@25
5	68	2025	1963	-3.062	4@25
4	67	2025	1963	-3.062	4@25
3	66	3025	3216	6.314	4@32
2	65	3025	3216	6.314	4@32
1	64	3025	3216	6.314	4@32

Table 2. Beam detailing

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	768.55	2@12+1@20+3@10	776	0.97	486.292	2@12+1@20	540	11.04	1606.616	2@12+1@20+2@25+1@12	1635	1.77
9	Bottom	731.45	2@20+1@12	741	1.31	2065.993	2@20+1@12+3@25	2211	7.02	749.266	2@20+1@12+3@6	826	####
	Top	1607.6	2@12+1@20+2@25+1@12	1635	1.71	495.4	2@12+1@20	540	9.00	2301.355	2@12+1@20+2@25+1@32	2325	1.03
8	Bottom	749.68	2@20+1@16	829	10.58	2808.712	2@20+1@16+2@32+1@25	2930	4.32	1033.598	2@20+1@16+3@10	1065	3.04
	Top	1703.7	2@12+1@20+2@25+1@16	1723	1.13	512.566	2@12+1@20	540	5.35	2398.848	2@12+1@20+2@32+1@20	2464	2.72
7	Bottom	790.64	2@20+1@16	829	4.85	2815.712	2@20+1@16+2@32+1@25	2930	4.06	1071.235	2@20+1@16+3@12	1168	9.03
	Top	1700.1	1@20+2@12+1@32+2@16	1746	2.70	530.715	2@12+1@20	540	1.75	2481.301	1@20+2@12+2@32+1@25	2641	6.44
6	Bottom	789.1	2@20+1@16	829	5.06	2843.199	2@20+1@16+2@32+1@25	2930	3.05	1111.189	2@20+1@16+3@12	1168	5.11
	Top	3045.1	2@12+1@25+3@32	3127	2.69	664.278	2@12+1@25	717	7.94	2217.418	2@12+1@25+2@32	2327	4.94
5	Bottom	1410.6	2@25+1@10+2@16	1462	3.64	2626.655	2@25+1@10+2@32	2670	1.65	1000.729	2@25+1@10	1061	5.97
	Top	3021.6	2@20+1@16+3@32	3239	7.20	782.046	2@20+1@16	829	6.00	1671.763	2@20+1@16+3@20	1772	6.00
4	Bottom	1406.7	2@16+1@25+3@16	1496	6.35	1899.164	2@16+1@25+2@25+1@12	1988	4.68	805.763	2@16+1@25	893	####
	Top	2896.2	2@20+1@16+3@32	3239	11.83	782.046	2@20+1@16	829	6.00	1734.531	2@20+1@16+3@20	1772	2.16
3	Bottom	1353	2@16+1@25+3@16	1496	10.57	1909.946	2@16+1@25+2@25+1@12	1988	4.09	834.758	2@16+1@25	893	6.98
	Top	2806.6	2@20+1@16+2@32+1@25	2930	4.40	782.046	2@20+1@16	829	6.00	2004.408	2@20+1@16+2@25+1@16	2012	0.38
2	Bottom	1314.3	2@16+1@25+3@16	1496	13.83	1904.38	2@16+1@25+2@25+1@12	1988	4.39	834.758	2@16+1@25	893	6.98
	Top	2887.3	2@20+1@16+2@32+1@25	2930	1.48	782.046	2@20+1@16	829	6.00	2530.834	2@20+1@16+2@32+1@12	2552	0.84
1	Bottom	1349.1	2@25+1@25+3@8	1447	7.25	1935.506	2@25+1@20+2@20+1@12	2037	5.24	1200.473	2@25+1@20	1296	7.96

Shear V		Spacing and rebar distribution along the beam			
9	Beam length	0-1.8	1.8-2.75	2.75-5.5	
		250mm at 8	275mm at 8	200mm at 8	
8	Beam length	0-1.375	1.375-1.8	1.8-2.75	2.75-5.5
		175mm at 8	275mm at 10	250mm at 10	150mm at 8
7	Beam length	0-1.8	1.8-2.75	2.75-4.12	4.12-5.5
		250mm at 10	175mm at 8	250mm at 10	225mm at 10
6	Beam length	0-1.8	1.8-2.75	2.75-5.5	
		250mm at 10	175mm at 8	225mm at 10	
5	Beam length	0-0.9	0.9-2.75	2.75-4.5	4.5-5.5
		300mm at 12	225mm at 10	175mm at 8	250mm at 10
4	Beam length	0-1.8	1.8-2.75	2.75-3.2	3.2-5.5
		150mm at 8	250mm at 10	250mm at 8	200mm at 8
3	Beam length	0-0.9	0.9-2.75	2.75-4.1	4.1-5.5
		150mm at 8	250mm at 10	200mm at 8	300mm at 10
2	Beam length	0-0.9	0.9-2.75	2.75-4.1	4.1-5.5
		150mm at 8	250mm at 10	300mm at 10	275mm at 10
1	Beam length	0-1.3	1.3-2.75	2.75-3.6	3.6-5.5
		150mm at 8	250mm at 10	175mm at 8	250mm at 10

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	1602.6	2@12+1@20+2@25+1@12	1635	2.02	486.292	2@12+1@20	540	11.04	1690.193	2@12+1@20+2@25+1@16	1723	1.94
9	Bottom	747.54	2@20+1@16	829	10.90	1650.475	2@20+1@16+3@20	1772	7.36	784.895	2@20+1@16	829	5.62
	Top	2403	2@12+1@20+2@32+1@20	2464	2.54	530.66	2@12+1@20	540	1.76	2481.07	1@20+2@12+2@32+1@25	2641	6.45
8	Bottom	1072.8	2@25+1@12	1095	2.07	2548.607	2@25+1@12+3@25	2565	0.64	1111.07	2@25+1@12+3@6	1180	6.19
	Top	2520.4	2@8+1@25+2@32+1@25	2693	6.85	551.341	2@8+1@25	592	7.37	2569.171	2@8+1@25+2@32+1@25	2693	4.82
7	Bottom	1131.5	2@25+1@16	1183	4.56	2546.16	2@25+1@16+3@25	2653	4.20	1156.799	2@25+1@16	1183	2.26
	Top	2705.9	2@8+1@25+3@32	3002	10.94	583.561	2@8+1@25	592	1.45	2668.415	2@8+1@25+2@32+1@25	2693	0.92
6	Bottom	1228.5	2@25+1@20	1296	5.49	2540.079	4@25+2@20	2588	1.89	1208.765	2@25+1@20	1296	7.22
	Top	2707.8	2@20+1@32+3@25	2902	7.17	608.782	2@20	628	3.16	2812.343	2@20+1@32+3@25	2902	3.19
5	Bottom	1229.5	2@25+1@20	1296	5.41	2557.558	4@25+2@20	2588	1.19	1285.006	2@25+1@20	1296	0.86
	Top	2665.9	2@20+1@32+3@25	2902	8.86	614.933	2@20	628	2.12	2838.246	2@20+1@32+3@25	2902	2.25
4	Bottom	1207.4	2@25+1@20	1296	7.34	2562.544	4@25+2@20	2588	0.99	1295.841	2@25+1@20	1296	0.01
	Top	2270.8	2@20+1@16+3@25	2299	1.24	782.046	2@20+1@16	829	6.00	2359.317	2@20+1@16+2@32	2439	3.38
3	Bottom	1078.4	2@20+1@25	1119	3.76	1868.469	4@20+1@25+1@16	1948	4.26	1117.926	2@20+1@25	1119	0.10
	Top	2359.7	3@20+1@16+2@32	2753	16.67	782.046	2@20+1@16	829	6.00	2483.499	3@20+1@16+2@32	2753	####
2	Bottom	1118.1	2@25+1@20	1296	15.91	1867.217	2@25+1@20+3@16	1899	1.70	1172.99	2@25+1@20	1296	####
	Top	2627.4	3@20+1@16+2@32	2753	4.78	782.046	2@20+1@16	829	6.00	2724.958	3@20+1@16+2@32	2753	1.03
1	Bottom	1236.3	2@25+1@20	1296	4.83	1858.019	2@25+1@20+3@16	1899	2.21	1278.855	2@25+1@20	1296	1.34

Shear V		Spacing and rebar distribution along the beam		
9	Beam length	0-1.8	1.8-3.2	3.2-5.5
		225mm at 8	250mm at 8	225mm at 8
8	Beam length	0-2.3	2.3-3.2	3.2-5.5
		250mm at 10	275mm at 10	250mm at 10
7	Beam length	0-1.3	1.3-4.125	4.125-5.5
		225mm at 10	250mm at 10	225mm at 10
6	Beam length	0-1.8	1.8-3.2	3.2-5.5
		225mm at 10	250mm at 10	225mm at 10
5	Beam length	0-1.8	1.8-3.2	3.2-5.5
		225mm at 10	250mm at 10	225mm at 10
4	Beam length	0-1.8	1.8-3.2	3.2-5.5
		225mm at 10	250mm at 10	225mm at 10
3	Beam length	0-1.8	1.8-3.2	3.2-5.5
		275mm at 10	300mm at 10	275mm at 10
2	Beam length	0-1.3	1.3-3.2	3.2-5.5
		250mm at 10	275mm at 10	250mm at 10
1	Beam length	0-1.3	1.3-3.2	3.2-5.5
		250mm at 10	275mm at 10	250mm at 10

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	1776.9	2@16+1@12+2@25+1@20	1811	1.92	486.292	2@16+1@12	515	5.90	1847.072	2@16+1@12+3@25	1985	7.47
9	Bottom	821.48	3@20	943	14.79	1864.335	3@20+2@25	1924	3.20	850.755	3@20	943	####
	Top	2623.8	3@16+2@32+1@25	2704	3.06	574.827	3@16	603	4.90	2668.869	3@16+2@32+1@25	2704	1.32
8	Bottom	1185.4	2@25+1@20	1296	9.33	2781.348	2@25+1@20+2@32	2906	4.48	1209.004	2@25+1@20	1296	7.20
	Top	2719.9	3@16+3@32	3013	10.78	595.016	3@16	603	1.34	2754.271	3@16+3@32	3013	9.39
7	Bottom	1235.9	3@25	1470	18.94	2781.411	6@25	2950	6.06	1254.117	3@25	1470	####
	Top	2825	2@20+3@32	3038	7.54	615.361	2@20	628	2.05	2840.05	2@20+3@32	3038	6.97
6	Bottom	1291.8	3@25	1470	13.80	2780.574	6@25	2950	6.09	1299.805	3@25	1470	####
	Top	2907.1	2@16+1@20+3@32	3126	7.53	639.643	2@16+1@20	716	11.94	2942.053	2@16+1@20+3@32	3126	6.25
5	Bottom	1335.8	3@25	1470	10.05	2780.201	6@25	2950	6.11	1354.639	3@25	1470	8.52
	Top	2937	2@16+1@20+3@32	3126	6.43	647.779	2@16+1@20	716	10.53	2976.137	2@16+1@20+3@32	3126	5.04
4	Bottom	1351.9	3@25	1470	8.73	2779.979	6@25	2950	6.12	1373.087	3@25	1470	7.06
	Top	2440.5	2@20+1@16+2@25+1@32	2615	7.15	782.046	2@20+1@16	829	6.00	2461.959	2@20+1@16+2@25+1@32	2615	6.22
3	Bottom	1154	2@25+1@20	1296	12.31	2052.254	2@25+3@20+1@16	2125	3.54	1163.469	2@25+1@20	1296	####
	Top	2531.2	2@20+1@16+2@32+1@12	2552	0.82	782.046	2@20+1@16	829	6.00	2483.499	2@20+1@16+2@32+1@12	2552	2.76
2	Bottom	1194	2@25+1@20	1296	8.54	1867.217	2@25+3@20+1@16	2125	13.81	1172.99	2@25+1@20	1296	####
	Top	2627.4	2@20+2@16+2@32	2640	0.48	782.046	2@20+1@16	829	6.00	2572.878	2@20+2@16+2@32	2640	2.61
1	Bottom	1236.3	2@25+1@20	1296	4.83	2053.345	2@25+3@20+1@16	2125	3.49	1212.364	2@25+1@20	1296	6.90

Shear V		Spacing and rebar distribution along the beam		
9	Beam length	0-1.5	1.5-4.0	4.0-6.0
		200mm at 8	225mm at 8	200mm at 8
8	Beam length	0-2.5	2.5-3.5	3.5-6.0
		250mm at 10	175mm at 8	250mm at 10
7	Beam length	0-1.5	1.5-4.0	4.0-6.0
		225mm at 10	250mm at 10	225mm at 10
6	Beam length	0-2.0	2.0-4.5	4.5-6.0
		225mm at 10	250mm at 10	225mm at 10
5	Beam length	0-2.0	2.0-4.5	4.5-6.0
		225mm at 10	250mm at 10	225mm at 10
4	Beam length	0-2.5	2.5-4.0	4.0-6.0
		225mm at 10	250mm at 10	225mm at 10
3	Beam length	0-2.5	2.5-4.5	4.5-6.0
		175mm at 8	300mm at 10	175mm at 8
2	Beam length	0-2.5	2.5-3.5	3.5-6.0
		275mm at 10	300mm at 10	275mm at 10
1	Beam length	0-2.0	2.0-4.5	4.5-6.0
		250mm at 10	275mm at 10	250mm at 10

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	1824.3	2@16+1@12+3@25	1985	8.81	486.292	2@16+1@12	515	5.90	1824.9	2@16+1@12+3@25	1985	8.77
9	Bottom	841.28	3@20	943	12.09	1850.746	3@20+2@25	1924	3.96	841.529	3@20	943	####
	Top	2651.8	3@16+2@32+1@25	2704	1.97	570.878	3@16	603	5.63	2652.132	3@16+2@32+1@25	2704	1.96
8	Bottom	1200	2@25+1@20	1296	8.00	2776.576	2@25+1@20+2@32	2906	4.66	1200.205	2@25+1@20	1296	7.98
	Top	2743	3@16+3@32	3013	9.85	592.337	3@16	603	1.80	2742.895	3@16+3@32	3013	9.85
7	Bottom	1248.1	3@25	1470	17.78	2775.312	6@25	2950	6.29	1248.086	3@25	1470	####
	Top	2838.9	2@20+3@32	3038	7.01	615.082	2@20	628	2.10	2838.751	2@20+3@32	3038	7.02
6	Bottom	1299.2	3@25	1470	13.15	2773.583	6@25	2950	6.36	1299.111	3@25	1470	####
	Top	2927.6	2@16+1@20+3@32	3126	6.78	636.206	2@16+1@20	716	12.54	2927.47	2@16+1@20+3@32	3126	6.78
5	Bottom	1346.9	3@25	1470	9.14	2776.485	6@25	2950	6.25	1346.766	3@25	1470	9.15
	Top	2955.1	2@16+1@20+3@32	3126	5.78	642.753	2@16+1@20	716	11.40	2954.676	2@16+1@20+3@32	3126	5.80
4	Bottom	1361.7	3@25	1470	7.95	2777.463	6@25	2950	6.21	1361.464	3@25	1470	7.97
	Top	2454	2@20+1@16+2@25+1@32	2615	6.56	782.046	2@20+1@16	829	6.00	2453.607	2@20+1@16+2@25+1@32	2615	6.58
3	Bottom	1160	2@25+1@20	1296	11.73	2048.277	2@25+3@20+1@16	2125	3.75	1159.774	2@25+1@20	1296	####
	Top	2555.6	2@20+1@16+2@25+1@32	2615	2.33	782.046	2@20+1@16	829	6.00	2555.699	2@20+1@16+2@25+1@32	2615	2.32
2	Bottom	1204.8	2@25+1@20	1296	7.57	2049.698	2@25+3@20+1@16	2125	3.67	1204.813	2@25+1@20	1296	7.57
	Top	2803.9	2@20+1@16+2@32+1@25	2930	4.50	782.046	2@20+1@16	829	6.00	2803.211	2@20+1@16+2@32+1@25	2930	4.52
1	Bottom	1313.1	3@25	1470	11.95	2048.314	3@25+2@20+1@16	2299	12.24	1312.817	3@25	1470	####

Shear V		Spacing and rebar distribution along the beam		
9	Beam length	0-2.0	2.0-4.0	4.0-6.0
		200mm at 8	225mm at 8	200mm at 8
8	Beam length	0-2.0	2.0-4.0	4.0-6.0
		250mm at 10	275mm at 10	250mm at 10
7	Beam length	0-1.5	1.5-4.5	4.5-6.0
		225mm at 10	250mm at 10	225mm at 10
6	Beam length	0-2.0	2.0-4.0	4.0-6.0
		225mm at 10	250mm at 10	225mm at 10
5	Beam length	0-2.0	2.0-4.0	4.0-6.0
		225mm at 10	250mm at 10	225mm at 10
4	Beam length	0-2.0	2.0-4.0	4.0-6.0
		225mm at 10	250mm at 10	225mm at 10
3	Beam length	0-2.0	2.0-4.0	4.0-6.0
		175mm at 8	300mm at 10	175mm at 8
2	Beam length	0-2.5	2.5-3.5	3.5-6.0
		175mm at 8	300mm at 10	175mm at 8
1	Beam length	0-2.0	2.0-4.0	4.0-6.0
		250mm at 10	275mm at 10	250mm at 10

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	1849.9	2@16+1@12+3@25	1985	7.30	486.292	2@16+1@12	515	5.90	1780.892	2@16+1@12+3@25	1985	####
9	Bottom	851.95	3@20	943	10.69	1864.335	3@20+2@25	1924	3.20	823.136	3@20	943	####
	Top	2670.7	3@16+2@32+1@25	2704	1.25	575.26	3@16	603	4.82	2626.42	3@16+2@32+1@25	2704	2.95
8	Bottom	1210	2@25+1@20	1296	7.11	2781.348	2@25+1@20+2@32	2906	4.48	1186.716	2@25+1@20	1296	9.21
	Top	2754	3@16+3@32	3013	9.40	594.952	3@16	603	1.35	2719.488	3@16+3@32	3013	####
7	Bottom	1254	3@25	1470	17.23	2781.411	6@25	2950	6.06	1235.698	3@25	1470	####
	Top	2839.5	2@20+3@32	3038	6.99	615.225	2@20	628	2.08	2824.204	2@20+3@32	3038	7.57
6	Bottom	1299.5	3@25	1470	13.12	2780.574	6@25	2950	6.09	1291.337	3@25	1470	####
	Top	2941.3	2@16+1@20+3@32	3126	6.28	639.456	2@16+1@20	716	11.97	2905.989	2@16+1@20+3@32	3126	7.57
5	Bottom	1354.2	3@25	1470	8.55	2780.201	6@25	2950	6.11	1335.189	3@25	1470	####
	Top	2974.1	2@16+1@20+3@32	3126	5.11	647.299	2@16+1@20	716	10.61	2934.167	2@16+1@20+3@32	3126	6.54
4	Bottom	1372	3@25	1470	7.14	2779.979	6@25	2950	6.12	1350.38	3@25	1470	8.86
	Top	2459.9	2@20+1@16+2@25+1@32	2615	6.30	782.046	2@20+1@16	829	6.00	2437.643	2@20+1@16+2@25+1@32	2615	7.28
3	Bottom	1162.6	2@25+1@20	1296	11.48	2052.254	2@25+3@20+1@16	2125	3.54	1152.706	2@25+1@20	1296	####
	Top	2573.5	2@20+1@16+2@25+1@32	2615	1.61	782.046	2@20+1@16	829	6.00	2532.112	2@20+1@16+2@25+1@32	2615	3.27
2	Bottom	1212.7	2@25+1@20	1296	6.87	2053.345	2@25+3@20+1@16	2125	3.49	1194.433	2@25+1@20	1296	8.50
	Top	2813.1	2@20+1@16+2@32+1@25	2930	4.15	782.046	2@20+1@16	829	6.00	2763.481	2@20+1@16+2@32+1@25	2930	6.03
1	Bottom	1317.1	3@25	1470	11.61	2055.006	2@25+3@20+1@16	2125	3.41	1295.595	3@25	1470	####

Shear V		Spacing and rebar distribution along the beam		
9	Beam length	0-1.5	1.5-4.5	4.5-6.0
		200mm at 8	225mm at 8	200mm at 8
8	Beam length	0-2.5	2.5-4.0	4.0-6.0
		250mm at 10	275mm at 10	250mm at 10
7	Beam length	0-2.5	2.5-4.0	4.0-6.0
		250mm at 10	275mm at 10	250mm at 10
6	Beam length	0-2.0	2.0-4.0	4.0-6.0
		150mm at 8	250mm at 10	150mm at 8
5	Beam length	0-2.0	2.0-4.0	4.0-6.0
		150mm at 8	250mm at 10	150mm at 8
4	Beam length	0-2.0	2.0-4.0	4.0-6.0
		150mm at 8	250mm at 10	150mm at 8
3	Beam length	0-2.0	2.0-4.0	4.0-6.0
		275mm at 10	300mm at 10	275mm at 10
2	Beam length	0-3.0	3.0-4.5	4.5-6.0
		275mm at 10	300mm at 10	275mm at 10
1	Beam length	0-2.0	2.0-4.0	4.0-6.0
		250mm at 10	275mm at 10	250mm at 10

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	1696.8	2@12+1@20+2@25+1@16	1723	1.54	486.292	2@12+1@20	540	11.04	1610.617	2@12+1@20+2@25+1@16	1723	6.98
9	Bottom	787.72	2@20+1@16	829	5.24	1650.475	2@20+1@16+3@20	1772	7.36	750.98	2@20+1@16	829	####
	Top	2463.3	2@12+1@20+2@32+1@20	2464	0.03	531.649	2@12+1@20	540	1.57	2409.754	1@20+2@12+2@32+1@25	2641	9.60
8	Bottom	1113.3	2@25+1@12+3@6	1180	5.99	2548.607	2@25+1@12+3@25	2565	0.64	1075.409	2@25+1@12+3@6	1180	9.72
	Top	2568.3	2@8+1@25+2@32+1@25	2693	4.85	551.146	2@8+1@25	592	7.41	2519.429	2@8+1@25+2@32+1@25	2693	6.89
7	Bottom	1156.4	2@25+1@16	1183	2.30	2546.16	2@25+1@16+3@25	2653	4.20	1130.934	2@25+1@16	1183	4.60
	Top	2666.9	2@8+1@25+3@32	3002	12.57	583.132	2@8+1@25	592	1.52	2704.034	2@8+1@25+3@32	3002	####
6	Bottom	1208	2@25+1@20	1296	7.29	2540.079	4@25+2@20	2588	1.89	1227.535	2@25+1@20	1296	5.58
	Top	2810.5	2@20+1@32+3@25	2902	3.26	608.341	2@20	628	3.23	2705.698	2@20+1@32+3@25	2902	7.26
5	Bottom	1284	2@25+1@20	1296	0.93	2557.558	4@25+2@20	2588	1.19	1228.413	2@25+1@20	1296	5.50
	Top	2833.6	2@20+1@32+3@25	2902	2.41	613.831	2@20	628	2.31	2660.659	2@20+1@32+3@25	2902	9.07
4	Bottom	1295.4	2@25+1@20	1296	0.05	2562.544	4@25+2@20	2588	0.99	1204.686	2@25+1@20	1296	7.58
	Top	2354.3	2@20+1@16+2@32	2439	3.60	782.046	2@20+1@16	829	6.00	2264.896	2@20+1@16+2@32	2439	7.69
3	Bottom	1115.7	2@20+1@25	1119	0.29	1868.469	4@20+1@25+1@16	1948	4.26	1075.783	2@20+1@25	1119	4.02
	Top	2485	3@20+1@16+2@32	2753	10.78	782.046	2@20+1@16	829	6.00	2361.483	3@20+1@16+2@32	2753	####
2	Bottom	1173.7	2@25+1@20	1296	10.42	1867.217	2@25+1@20+3@16	1899	1.70	1118.89	2@25+1@20	1296	####
	Top	2717.2	3@20+1@16+2@32	2753	1.32	782.046	2@20+1@16	829	6.00	2618.714	3@20+1@16+2@32	2753	5.13
1	Bottom	1275.5	2@25+1@20	1296	1.61	1858.019	2@25+1@20+3@16	1899	2.21	1232.471	2@25+1@20	1296	5.15

Shear V		Spacing and rebar distribution along the beam		
9	Beam length	0-1.8	1.8-3.2	3.5-5.5
		200mm at 8	225mm at 8	200mm at 8
8	Beam length	0-2.3	2.3-3.6	3.6-5.5
		250mm at 10	275mm at 10	250mm at 10
7	Beam length	0-1.8	1.8-3.6	3.6-5.5
		225mm at 10	250mm at 10	225mm at 10
6	Beam length	0-1.8	1.8-3.6	3.6-5.5
		225mm at 10	250mm at 10	225mm at 10
5	Beam length	0-1.8	1.8-3.6	3.6-5.5
		225mm at 10	250mm at 10	225mm at 10
4	Beam length	0-1.8	1.8-3.6	3.6-5.5
		225mm at 10	250mm at 10	225mm at 10
3	Beam length	0-2.2	2.2-3.6	3.6-5.5
		275mm at 10	300mm at 10	275mm at 10
2	Beam length	0-1.3	1.3-4.5	4.5-5.5
		250mm at 10	275mm at 10	250mm at 10
1	Beam length	0-1.3	1.3-4.5	4.5-5.5
		250mm at 10	275mm at 10	250mm at 10

Floor		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
	Top	1614.9	2@12+1@20+2@25+1@12	1635	1.24	486.292	2@12+1@20	540	11.04	774.717	2@12+1@20+3@10	776	0.17
9	Bottom	752.82	2@20+1@12+3@6	825.9	9.71	2065.993	2@20+1@12+3@25	2211	7.02	586.292	2@20	628	7.11
	Top	2308.6	2@12+1@20+2@25+1@32	2325	0.71	496.685	2@12+1@20	540	8.72	1613.551	2@12+1@20+2@25+1@12	1635	1.33
8	Bottom	1036.4	2@20+1@16+3@10	1065	2.76	2808.712	2@20+1@16+2@32+1@25	2930	4.32	752.236	2@20+1@16	829	####
	Top	2397.5	2@12+1@20+2@32+1@20	2464	2.77	512.327	2@12+1@20	540	5.40	1702.684	2@12+1@20+2@25+1@16	1723	1.19
7	Bottom	1070.7	2@20+1@16+3@12	1168	9.09	2815.712	2@20+1@16+2@32+1@25	2930	4.06	790.188	2@20+1@16	829	4.91
	Top	2480.3	1@20+2@12+2@32+1@25	2641	6.48	530.491	2@12+1@20	540	1.79	1698.603	1@20+2@12+1@32+2@16	1746	2.79
6	Bottom	1110.7	2@20+1@16+3@12	1168	5.16	2843.199	2@20+1@16+2@32+1@25	2930	3.05	788.46	2@20+1@16	829	5.14
	Top	2212.7	2@12+1@25+2@32	2327	5.17	662.965	2@12+1@25	717	8.15	3039.643	2@12+1@25+3@32	3127	2.87
5	Bottom	998.86	2@25+1@10	1061	6.17	2626.655	2@25+1@10+2@32	2670	1.65	1407.628	2@25+1@10+2@16	1462	3.86
	Top	1662.5	2@20+1@16+3@20	1772	6.59	782.046	2@20+1@16	829	6.00	3009.188	2@20+1@16+3@32	3239	7.64
4	Bottom	801.47	2@16+1@25	893	11.42	1899.164	2@16+1@25+2@25+1@12	1988	4.68	1401.384	2@16+1@25+3@16	1496	6.75
	Top	1727.5	2@20+1@16+3@20	1772	2.57	782.046	2@20+1@16	829	6.00	2887.313	2@20+1@16+3@32	3239	####
3	Bottom	831.54	2@16+1@25	893	7.39	1909.946	2@16+1@25+2@25+1@12	1988	4.09	1349.126	2@16+1@25+3@16	1496	####
	Top	2008.2	2@20+1@16+2@25+1@16	2012	0.19	782.046	2@20+1@16	829	6.00	2812.388	2@20+1@16+2@32+1@25	2930	4.18
2	Bottom	890.05	2@16+1@25	893	0.33	1904.38	2@16+1@25+2@25+1@12	1988	4.39	1316.788	2@16+1@25+3@16	1496	####
	Top	2533.7	2@20+1@16+2@32+1@12	2552	0.72	782.046	2@20+1@16	829	6.00	2871.653	2@20+1@16+2@32+1@25	2930	2.03
1	Bottom	1195.1	2@25+1@20	1296	8.44	1935.506	2@25+1@20+2@20+1@12	2037	5.24	1342.38	2@25+1@25+3@8	1447	7.79

Shear V		Spacing and rebar distribution along the beam		
9	Beam length	0-2.7	2.7-5.5	
		300mm at 10	250mm at 8	
8	Beam length	0-1.8	1.8-2.7	2.7-5.5
		225mm at 10	250mm at 10	275mm at 10
7	Beam length	0-1.8	1.8-2.7	2.7-5.5
		225mm at 10	250mm at 10	275mm at 10
6	Beam length	0-1.8	1.8-2.7	2.7-5.5
		225mm at 10	250mm at 10	275mm at 10
5	Beam length	0-1.3	1.3-2.7	2.7-5.5
		250mm at 10	275mm at 10	250mm at 10
4	Beam length	0-2.2	2.2-2.7	2.7-5.5
		300mm at 10	225mm at 8	250mm at 10
3	Beam length	0-2.2	2.2-2.7	2.7-5.5
		300mm at 10	225mm at 8	250mm at 10
2	Beam length	0-2.7	2.7-5.5	
		300mm at 10	250mm at 10	
1	Beam length	0-1.8	1.8-2.7	2.7-5.5
		250mm at 10	275mm at 10	250mm at 10

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Table 3. Large halls beam detailing

		Required As	Rebar	Area provided As	% Error	Required	Rebar	Area provided	% Error	Required	Rebar	Area provided	% Error
Large beam	Top	14527.32	2@40+4@32+6@40	14540	0.09	5778.65	2@40+4@32	5780	0.02	2576.89	2@40	2610	1.28
19.070	Bottom	10344.56	4@32+6@32+2@40	10570	2.18	7906.78	4@32+6@32	8050	1.81	3221.76	4@32	3230	0.26
Small beam	Top	13887.046	4@25+2@32+8@40	14144	1.85	4044.961	5@25+2@32	4060	0.37	2254.834	5@25	2450	8.66
15.085	Bottom	11697.635	6@25+2@40+6@32	11858	1.37	6228.56	6@25+2@40	6720	7.89	2924.959	6@25	2950	0.86

Shear V		Spacing/rebar distribution along the beam		
	Beam length	1.0-5.0	5.0-12.0	12.0-19.07
		200mm at 8	300mm at 8	275mm at 8
	Beam length	1.0-5.0	5.0-10.0	10-15.085
		250mm at 12	275mm at 10	300mm at 8

Table 4. End span provided steel

End span										
Level	As, required at supports (mm2/m)	detail	As, provided at supports (mm2/m)	As, required at midspan (mm2/m)	detail	As, provided at midspan (mm2/m)	p	l/d, EC2	l/d	Percentage overdesign
9	182.71	8mm@275mm	183.00	182.71	8mm@275mm	183.00	0.00145	127.27	21.83	0.16
8	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
7	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
6	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
5	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
4	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
3	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
2	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42
1	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	127.27	21.83	7.42

Table 5. Interior span provided steel

Interior span										
Level	As, required at supports (mm2/m)	detail	As, provided at supports (mm2/m)	As, required at midspan (mm2/m)	detail	As, provided at midspan (mm2/m)	p	l/d, EC2	l/d	Percentage overdesign
9	181.67	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	0.73
8	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
7	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
6	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
5	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
4	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
3	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
2	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42
1	170.35	8mm@275mm	183.00	170.35	8mm@275mm	183.00	0.00145	146.85	23.81	7.42

Appendix C

Table C.1. Pile capacity of piles with width = 300mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	1533.221	812.146	471.739	517.711	600.532	362.702	963.234	321.0782	642.1563	1284.313	1765.994	2248.189
7	1736.963	812.146	505.238	601.546	639.643	467.612	1107.256	369.0853	738.1706	1476.341	2030.043	2584.335
8	1940.705	812.146	535.031	672.106	673.094	533.912	1207.007	402.3356	804.6712	1609.342	2212.926	2817.154
9	2144.447	812.146	561.974	696.249	690.123	600.212	1290.336	430.1119	860.2238	1720.448	2365.701	3011.643
10	2348.189	812.146	587.799	721.738	707.228	666.512	1373.740	457.9134	915.8267	1831.653	2518.615	3206.309

Table C.2. Pile capacity of piles with width = 350mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	2086.884	1105.422	642.089	704.662	817.391	461.264	1278.654	426.2182	852.4363	1704.873	2344.285	2984.38
7	2364.200	1105.422	687.686	818.770	870.626	583.659	1454.284	484.7615	969.523	1939.046	2666.285	3394.3
8	2641.515	1105.422	728.237	914.810	916.156	661.009	1577.165	525.7216	1051.443	2102.886	2891.574	3681.103
9	2918.831	1105.422	764.909	947.672	939.334	738.359	1677.693	559.231	1118.462	2236.924	3075.882	3915.735
10	3196.146	1105.422	800.060	982.365	962.615	815.709	1778.324	592.7747	1185.549	2371.099	3260.379	4150.608

Table C.3. Pile capacity of piles with width = 400mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	2725.726	1443.816	838.647	920.375	1067.613	571.770	1639.382	546.4607	1092.921	2185.843	3005.643	3826.318
7	3087.934	1443.816	898.202	1069.414	1137.144	722.002	1859.146	619.7152	1239.43	2478.861	3408.558	4339.246
8	3450.142	1443.816	951.166	1194.854	1196.612	813.122	2009.734	669.9113	1339.823	2679.645	3684.646	4690.719
9	3812.350	1443.816	999.065	1237.776	1226.886	904.242	2131.127	710.3758	1420.752	2841.503	3907.209	4974.051
10	4174.558	1443.816	1044.976	1283.089	1257.294	995.362	2252.655	750.8851	1501.77	3003.54	4130.018	5257.697

Table C.4. Pile capacity of piles with width = 450mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	3449.747	1827.329	1061.412	1164.850	1351.197	692.390	2043.587	681.1957	1362.391	2724.783	3746.712	4769.732
7	3908.167	1827.329	1136.787	1353.478	1439.198	873.200	2312.398	770.7992	1541.598	3083.197	4239.55	5397.136
8	4366.586	1827.329	1203.820	1512.238	1514.462	980.300	2494.762	831.5874	1663.175	3326.349	4573.897	5822.775
9	4825.006	1827.329	1264.442	1566.560	1552.777	1087.400	2640.177	880.0591	1760.118	3520.236	4840.501	6162.174
10	5283.425	1827.329	1322.548	1623.910	1591.262	1194.500	2785.762	928.5874	1857.175	3714.35	5107.417	6501.969

Table C.5. Pile capacity of piles with width = 500mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	4258.947	2255.962	1310.386	1438.086	1668.145	818.602	2486.747	828.9156	1657.831	3315.662	4559.201	5804.067
7	4824.897	2255.962	1403.440	1670.960	1776.787	1032.952	2809.739	936.5798	1873.16	3746.319	5151.376	6557.932
8	5390.847	2255.962	1486.197	1866.960	1869.707	1160.452	3030.159	1010.053	2020.106	4040.211	5555.493	7072.39
9	5956.797	2255.962	1561.040	1934.025	1917.009	1287.952	3204.961	1068.32	2136.641	4273.281	5875.976	7480.379
10	6522.747	2255.962	1632.775	2004.827	1964.521	1415.452	3379.973	1126.658	2253.316	4506.631	6196.843	7888.858

Table C.6. Pile capacity of piles with width = 550mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	5153.326	2729.714	1585.567	1583.923	1966.401	936.762	2903.163	967.721151	1935.442	3870.885	5322.66	6775.984
7	5838.125	2729.714	1698.163	1844.949	2090.942	1187.342	3278.284	1092.76129	2185.523	4371.045	6010.406	7651.515
8	6522.925	2729.714	1798.299	2117.833	2215.282	1336.942	3552.224	1184.07465	2368.149	4736.299	6512.647	8290.891
9	7207.724	2729.714	1888.858	2371.373	2329.982	1486.542	3816.524	1272.17455	2544.349	5088.698	6997.214	8907.766
10	7892.524	2729.714	1975.658	2630.841	2445.404	1636.142	4081.546	1360.51546	2721.031	5442.062	7483.107	9526.329

Table C.7. Pile capacity of piles with width = 600mm.

Pile length (m)	Qp, Meyerhof (kN)	Qp, Meyerhof limit (kN)	Qp, Vesic (kN)	Qp, Coyle & Castello (kN)	Qp avg (kN)	Qs (kN)	Qult (kN)	Qall (kN)	1x2 group pile capacity (kN)	2x2 group pile capacity (kN)	3x2 group pile capacity (kN)	3x3 group pile capacity (kN)
6	6132.884	3248.586	1886.956	1858.450	2331.330	1065.286	3396.616	1132	2264.411	4528.82166	6227.356	7927.702
7	6947.852	3248.586	2020.954	2195.641	2488.394	1353.766	3842.160	1281	2561.44	5122.879515	7044.215	8967.601
8	7762.820	3248.586	2140.124	2486.791	2625.167	1516.966	4142.133	1381	2761.422	5522.8438	7594.186	9667.738
9	8577.788	3248.586	2247.897	2822.129	2772.871	1680.166	4453.037	1484	2968.691	5937.3823	8164.198	10393.39
10	9392.756	3248.586	2351.196	3130.919	2910.233	1843.366	4753.599	1585	3169.066	6338.132422	8715.249	11094.9
10.8	10088.214	3248.586	2426.799	3348.243	3007.876	1973.926	4981.802	1661	3321.201	6642.402747	9133.636	11627.53

Appendix D

Milky Way Company

Quality Plan Document

1. **Quality Policy and Standards**

   ISO 9000 Quality management standards

2. **Project Quality Definition**

   As the end outcome, the client gets the constructed academic building according to the project scope, performance and value by CNaS and other standards.

3. **Deliverables and Acceptance Criteria**

   Is the project budget within the identified range?

   Are the construction works follow the international and local standards?

   Is the used material has the same quality as in the contract?

   Is the occupational safety and health of the workers are satisfied?

   Are the construction processes (excavating, foundation construction) within the proposed duration?

4. **Quality Assurance**

   - To ensure the quality is built into the project process by holding trainings and inspections activities
   - To ensure testing and inspection by qualification testings of the material, soil structure and etc.
   - To ensure the project scope accurately reflects the customer needs by four-phase inspection program (preparatory, initial, follow-up, completion)
   - To ensure project plan is followed by providing daily construction reports (checklists)
   - To ensure quality control of deliverables

5. **Project Monitoring and Control**

   Project monitoring is supervised by the Quality Control Staff and by the independent third party. It can be directed by creating Quality Control Test for 10, 15 days or weekly construction reports. Quality audit should be performed by financial experts to ensure the project comply with organizational and constructional processes, standards.

Table 1. Salaries for staff

Position	Number of positions	Unit salary per year KZT	Salary per year, KZT
Office director	1	4 800 000 KZT	4 800 000 KZT
Deputy director	4	3 600 000 KZT	14 400 000 KZT
HR Department director	1	3 600 000 KZT	3 600 000 KZT
HR inspector	1	1 800 000 KZT	1 800 000 KZT
Secretary	2	1 800 000 KZT	3 600 000 KZT
Dean of Department	1	60 000 000 KZT	60 000 000 KZT
Vice Dean of Department	1	52 000 000 KZT	52 000 000 KZT
Professors	6	450 000 000 KZT	2 700 000 000 KZT
Associate Professors	10	22 000 000 KZT	220 000 000 KZT
Assistant Professor	18	18 000 000 KZT	324 000 000 KZT
Teaching Assistants	30	2 400 000 KZT	72 000 000 KZT
Research Assistants	20	3 000 000 KZT	60 000 000 KZT
IT manager	3	2 160 000 KZT	6 480 000 KZT
Cleaners	39	720 000 KZT	28 080 000 KZT
Cloakroom attendants	3	600 000 KZT	1 800 000 KZT
Elevetor Operator	1	960 000 KZT	960 000 KZT
Maintenance engineers	8	1 800 000 KZT	14 400 000 KZT
Total			3 567 920 000 KZT

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