DESIGN STUDENT RESIDENCE BUILDING USING AERATED CONCRETE/BLOCK IN NAZARBAYEV UNIVERSITY, ASTANA, KAZAKHSTAN

(Capstone 2 Project)

Bachelor of Engineering

(Civil Engineering)

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DECLARATION

We hereby declare that this report entitled "*Design student residence building using aerated concrete/block in Nazarbayev University, Astana, Kazakhstan*" is the result of our own project work except for quotations and citations, which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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Abstract

This project is focused on the design of a student residence building using aerated concrete in Nazarbayev University, Astana, Kazakhstan. The shortage of residential places on the campus is one of the primary problems of the university. Thus, the design of the residence building for master students by DC Group is studied in this paper. Major part of the worldwide energy is consumed by residential and commercial buildings. Because of low thermal conductivity and light weight compared to normal concrete, aerated concrete is proposed in order to provide proper thermal insulation and prevent significant heat loss. Moreover, the project includes obtaining own mixture design through laboratory experiments, where aerated concrete blocks will be casted and tested for both mechanical and thermal properties. Structural, architectural and geotechnical literature was reviewed to ensure the safety, stability and serviceability of the building. Unique climate conditions of Astana are considered during the structural, architectural, and geotechnical analysis. In addition, DC Group provides preliminary cost estimation of the project. Future works are shown at the end of the report.

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1 Introduction

Nazarbayev University is one of the biggest universities in Astana, Kazakhstan. Unlike other universities it has a several number of dormitories located in one campus. However, number of students is growing each year and existing dormitories exceed their capacity, thus lead to the problem with residents' allocation. One of the solutions to this problem is to construct an additional dormitory. It was decided to builf dormitory for master students only because most of them are not Astana residents and occupy major part of dormitories. By manual calculation it was computed that about 300 master students are currently living in the dormitories. So, it was decided to design a dormitory for approximately 350 residents.

Several problems can be met due to the climatic conditions of Astana. Generally, Astana found as the second coldest capital in the world, with average annual precipitation 326 mm. Also, Astana has 280-300 windy days per year with average wind velocity 5.2 m/s, and highest value of 31 m/s observed at winter period (Ospanova, 2015). Thus, wind load need to be taken into account during the design stage. Also, weather conditions can affect structure durability and should be considered during the structural analysis stage.

Weather conditions of Astana play a huge role in the energy consumption of the buildings. According to United Nations Environment Program, 40% of global energy are consumed by buildings. Similarly, Committee of Atomic and Energy Supervision and Control claim that more than 30% of total energy in Kazakhstan are used by residential and commercial buildings. Incase of Kazakhstan, significant part of residential buildings was constructed in 1960-90s, and they were equipped with poor thermal insulation systems, so that 30% of heat are lost through walls and coatings. As a solution to the problem, this project suggests using Aerated Concrete blocks that have high insulating properties. AC blocks have low thermal conductivity; thus, the temperature is conserved and overall energy consumption is reduced. During project realization, AC blocks will be casted and tested by energy modelling software, along with overall architectural, structural, and geotechnical designing of a building.

2 Literature Review

2.1 Materials

2.1.1 Structural components' materials

According EN 1990 choice of suitable construction materials is one of the basic requirements that should be met for the design of structure. Building components can be divided into structural and non-structural elements, where the latter is supported by the former. These systems can be classified according to the materials used during construction. Basic structural system materials are concrete, steel, and wood. Considering basic design requirements specified by Eurocode, as safety, serviceability, robustness, reliability, durability, quality, and 50 years design life (for buildings), structural materials choice was narrowed down to concrete and steel. Concrete and steel structures are widespread due to their material characteristics that make them suitable in construction area. Their application methods are developing with the lapse of time, finding solutions to their not favorable properties. One of such developments is the introduction of reinforced concrete method by F. Hennebique in 1892, which gives an opportunity to increase the tensile strength of the concrete structures (Saba, 2013). Reinforced concrete (RC) and steel are mainly used for multi-story frame systems (FEMA, USDHS and NIBSSC, 2013). In order to choose the construction material between RC and steel, simple analysis of characteristics was conducted based on literature review.

- Strength to weight ratio

Concrete has restrictions in use as it has low tensile strength and ductility. Therefore, the material is brittle and needs reinforcement to increase its strength characteristics. In addition, it has significantly low strength to weight ratio, which means that the overall self-weight of the concrete structure is high. In the contrary, steel is a material with high tensile strength and low strength to weight ratio (Saba, 2013). Thus, steel is highly favorable in construction of structures requiring low dead load. In other words, if the project concern is the construction of high-rise buildings, steel is preferable over steel.

- Fire resistance and insulation

Fire resistance of construction materials is used to define the safety level of the building. One of the ways to describe the fire resistance is to observe thermal properties of the material, which are thermal conductivity, specific heat, and density. By comparing steel and RC by these properties, their fire resistances can be evaluated. Table 2.1 presents thermal properties of common construction materials (Vassart, 1991). It can be observed that the thermal conductivity of the steel much higher in comparison with concrete that results in lower critical temperature.

Table 2.1. Thermal material characteristics

Material	Temperature [°C]	λ [W/m/K]	ρ [kg/m³]	c_p [J/kg°K]
Normal weight concrete	20	2	2300	900
	200	1,63	2300	1022
	500	1,21	2300	1164
	1000	0,83	2300	1289
Light weight concrete	20	1	1500	840
	200	0,875	1500	840
	500	0,6875	1500	840
	1000	0,5	1500	840
Steel	20	54	7850	425
	200	47	7850	530
	500	37	7850	667
	1000	27	7850	650
Gypsum insulating material	20	0,035	128	800
	200	0,06	128	900
	500	0,12	128	1050
	1000	0,27	128	1100
Sealing cement	20	0,0483	200	751
	250	0,0681	200	954
	500	0,1128	200	1052
	800	0,2016	200	1059
CaSi board	20	0,0685	450	748
	250	0,0786	450	956
	450	0,0951	450	1060
	1050	0,157	450	1440
Wood	20	0,1	450	1113
	250	0,1	450	1125
	450	0,1	450	1135

In addition, thermal conductivity has an impact on insulation properties: the lower the thermal conductivity, the higher the insulation. Thus, from Table 2.1, RC is more appropriate than steel, as thermal conductivity of steel (= 54 W/mK) is considerably more than RC's (= 1 W/mK) at room temperature.

- Cost

In order to verify more suitable material in terms of cost, structural design of a component should be done under the same loading conditions. It could show the cost of the materials, in this case RC or steel, required for the same building construction. According to the cost comparison conducted by Merta, Kravanja, and Klansek (2008), who carried out a study on simply supported beams with span range of 5-30 m, RC beams are found to be cheaper till its span reaches 12 m. The Figures 2.1 and 2.2 illustrate the cost of RC and steel beams at different span length. Therefore, it can be assumed that for comparably short spans RC is more attractive in terms of cost.

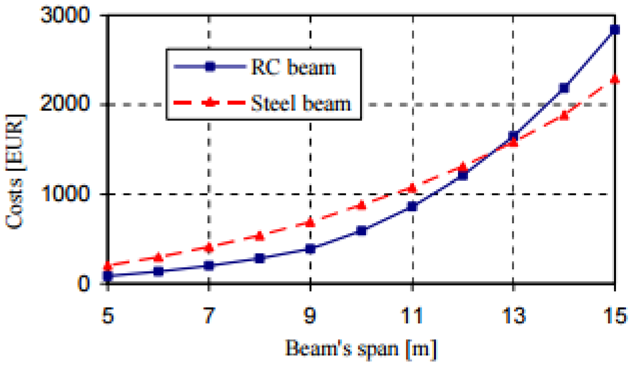


Figure 2.1. Cost of beam for spans 5-15 m

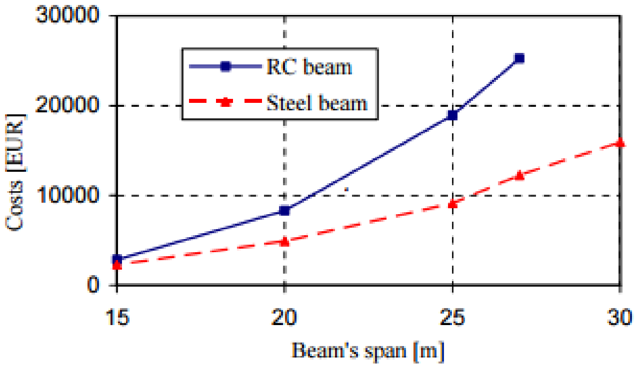


Figure 2.2. Cost of beam for spans 15-30 m

- Availability

Table 2.2 show the cement market data of the Kazakhstan. It can be seen that the cement production increased for approximately 2 mln tonnes, and consumption rised for 1 mlntonnes only. The import decreased till 2009 and since then it is maintained at approximately similar value. Whereas export indicators where not stable before 2010 (EY, 2013).

Table 2.2. Main parameters of the Kazakhstan cement market in 2005-2012

	2005	2006	2007	2008	2009	2010	2011	2012
Output, thousands of tonnes	4,181	4,880	5,699	5,837	5,694	6,683	5,500	6,300
Imports, thousands of tonnes	1,890	2,631	3,506	1,826	782	1,010	900	1,000
Exports, thousands of tonnes	4	1	-	131	25	199	200	200
Consumption, thousands of tonnes	6,067	7,510	9,205	7,532	6,451	7,494	6,200	7,100

In turn, Figure 2.3 illustrates steel production rate in Kazakhstan in last 5 years, where each bar correspond to the production in one month. It can be observed that steel manufacturing indicators were high in the end 2011 and start of 2012, and decreased significantly in 2013. The highest production in 2016 is approximately 0.38 mln tonnes (Trading Economics, 2016).

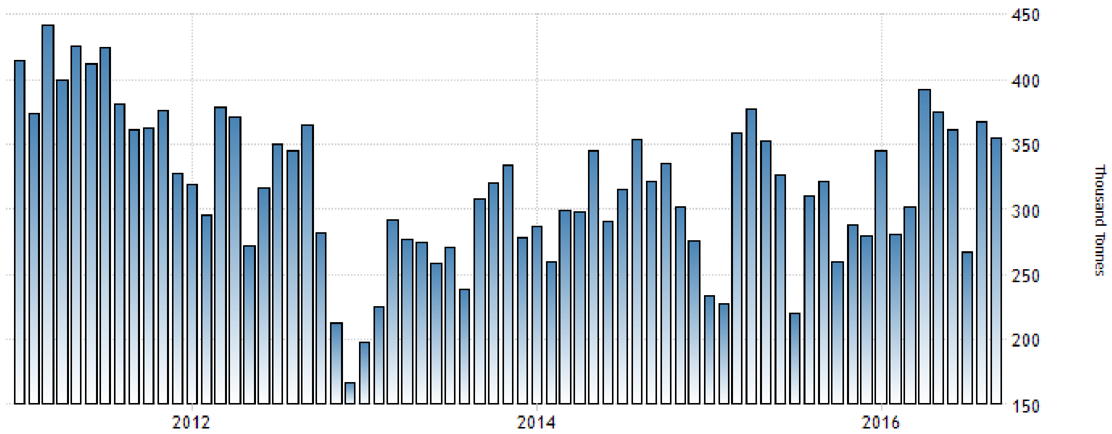


Figure 2.3. Steel production in Kazakhstan

Therefore, by comparing the production output of both materials, it can be assumed that the concrete availability is higher in Kazakhstan.

- Speed of construction

Sometimes, construction speed has a main role during project implementation. In terms of time-efficiency, steel is preferred over RC. Steel components are mostly prefabricated and can be assembled quickly by highly qualified labor, whereas concrete needs additional time for hardening, along with casting (Nunnally, 2013).

- Durability

Durability can be understood as an ability of structure to be within serviceability limits during its design period with reasonable maintenance (Kwan and Wong, n.d.). There are several factors affecting durability, such as structure type, construction materials and their characteristics, environmental conditions, and etc.

Construction materials are also susceptible to environmental conditions, causing deterioration. For example, RC structures subjected to acid attack, carbonation, and freeze-thaw cycles demonstrate low performance. Similarly, steel exposed weather suffers from different types of corrosion (Saba, 2013). Considering extremely bad weather conditions of Astana, concrete was chosen as material, which has better resistance against weather.

- Environmental considerations

Guggemos and Horvath (2005) state that comparison of steel and concrete structures should be carried by evaluating environmental emissions of the structure for the whole service life. Thus, the entire life cycle of the structure starting from construction to demolition phase should be assessed. Such comparison study revealed that concrete structure construction release CO_2, CO, NO_2, SO_2, and HC emissions as a result of longer installation works and equipment use. While steel structure construction tend to have heavy metal releases from erection and painting.

Figure 2.4. represent the service life environmental emissions of both structures. It illustrates that the results are comparable, as the none of the structures have dominance in majority of the sections.

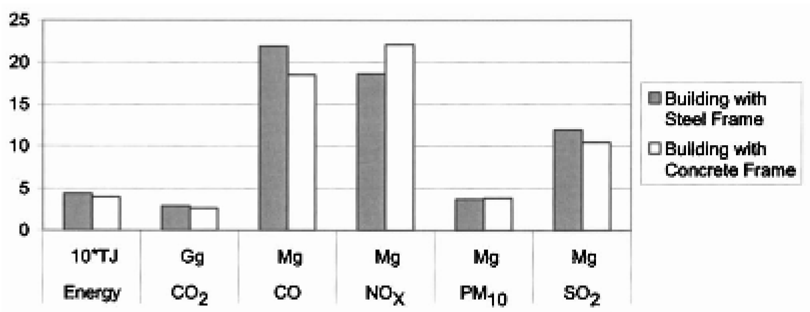


Figure 2.4. Overall life-cycle comparison

Summary of characteristics analysis is given by the Table 2.3.

Table 2.3. Steel and reinforced concrete comparison

Parameter	Steel	Reinforced Concrete
Strength to weight ratio	↑	↓
Fire resistance	↓	↑
Insulation	↓	↑
Attractive in terms of cost	↓	↑
Availability	↓	↑
Speed of construction	↑	↓
Durability	↓	↑
Environmental considerations	→	→

As a result of the comparison study, RC was chosen as the most appropriate material for construction, which also can be observed from Table 2.3. RC, in turn, can be casted in site or can be prefabricated at the plant and transported to the site.

- Quality

Generally, quality of cast-in-situ concrete is lower than precast concrete, as the latter is produced in specially equipped plant which maintains favorable conditions for casting. Moreover, precast concrete already has quality specifications, whereas during casting concrete on site additional quality tests are required. However, in the case of in-situ casting, reinforcement allocation of components is made by contractor’s judgement, which means that the component’s quality can be controlled. It is also convenient during maintenance works (Yee, 2001).

- Cost

In terms of cost, precast type is more favorable, as it needs less quantity of work by removing concrete forming, placing, finishing, and curing operations. However, such technique needs heavy equipment for lifting and installation works. Cast-in-place concrete, which requires the operations mentioned before, is assumed to be more expensive. Based on typical cost distribution of concrete construction, formworks constitute 40-60 % of the concrete construction cost and minimization of formwork cost is one of the important tasks (Nunnally, 2013). Thus, in-situ casting could be favorable for companies that reduce its cost by using forms repetitively.

- Time

Turai and Waghmare (2016) state that using prefabricated components is time-efficient, because it is does not need time for hardening, and saves time by eliminating several operations required in cast-in-place technique as well.

- Shape

One of the advantages of cast-in-situ concrete is the freedom in shape choice. Structure components can be casted in the desired shape by using formworks. In the contrary, the precast structure elements are produced having standard shapes (Nunnaly, 2013).

Considering the design specifications and comparison summary, cast-in-place reinforced concrete is chosen as material for beams, slabs, and columns construction.

2.1.2 Non-structural components' materials

Non-structural components are described as non-load-bearing building elements supported by the structure. Cladding, roofing, partitioning can be referred to as non-structural building components (FEMA, USDHS and NIBSSC, 2013). Main design factors during material choice for non-structural components are fire resistance, sound and thermal insulation properties, along with aesthetics and cost efficiency.

- Floor/Ceiling assembly

Ceiling would be assembled based on literature review of existing building floorings (BI Group, 2017). Figure 2.5 shows flooring layers and Table 2.4 illustrates layer names and parameters.

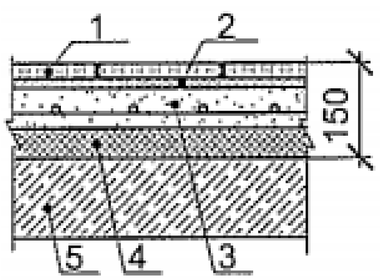


Figure 2.5. Floor/ceiling assembly

Table 2.4. Floor/ceiling layer properties

#	Layer name	Thickness (mm)	Density (kg/m³)	Load applied (N/m²)
1	Insulated laminate flooring	8	938	73,61
2	Self-leveling floor	2	1884	36,96
3	Cement/sand screed	50	2000	981,00
4	EPS Geofoam	90	18,88	16,67
5	Concrete slab	150	2400	3531,60
Total	Thickness =	300	Load =	4565,92

- Interior wall assembly

Interior wall assembly was also chosen from literature review (Figure 2.6) (BI group, 2017). Layer characteristics are given in Table 2.5.

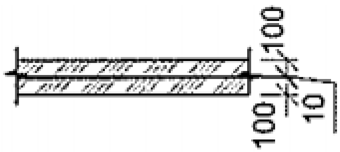


Figure 2.6. Interior wall assembly

Table 2.5. Interior wall layer parameters

#	Layer name	Thickness (mm)	Density (kg/m³)	Load applied (N/m²)
1	AC block	100	500	490,50
2	Air gap	10	0	0,00
3	AC block	100	500	490,50
Total	Wall thickness =	210	Load =	981,00

- Exterior wall assembly

Exterior walls were made up from gypsum, AC blocks with higher thickness, and stucco (Figure2.7, Table 2.6).

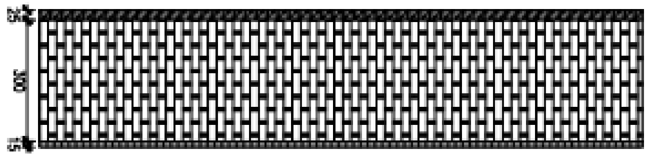


Figure 2.7 Exterior wall assembly

Table 2.6. Exterior wall layer parameters

#	Layer name	Thickness (mm)	Density (kg/m³)	Load applied (N/m²)
1	Stucco	25	1971	483,39
2	AC block	300	500	1471,50
3	Gypsum	15	2308	339,62
Total	Wall thickness =	340	Load =	2294,51

- Roofing assembly (Green roofing)

The system designation for green roofing was chosen as G2 with typical plants as sedum herbs perennials (Green Roof Technology, 2016). System layers are given in the Table 2.7.

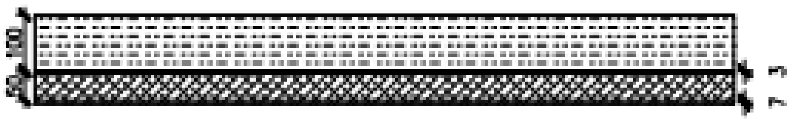


Figure 2.8 Roofing assembly

Table 2.7. Roofing assembly

#	Layer name	Thickness (mm)	Load applied (kN/m²)
1	Extensive soil mix	100	
2	Separation fabric	3	
3	Granular drainage	50	1,96
4	Protection mat	7	
Total	Floor thickness =	160	

2.1.3 Aerated concrete blocks

According to Wakili et al. (2015) aerated concrete (AC) is a lightweight concrete having low density and high porosity in comparison with other building materials, where air constitutes 20-90% of the total volume. AC is produced from sand, cementitious material, air-forming chemical, and water. Its parameters vary depending on the production methodology, having a density range of 93-1800 kg/m^3. The material is assumed to be commonly used, as it has both good mechanical and thermal characteristics (Jerman, 2013). In general, AC is widely known for its energy-efficient properties, as it has considerably low thermal conductivity. As the thickness of wall in building increase, thermal conductivity value is decreases. If low thermal conductivity material is used, the thickness of wall can be reduced. Aerated concrete offers high level of thermal insulation of buildings at low wall thickness, and low self-weight, respectively (Pruteanu and Vasilache, 2013). Therefore, application of aerated concrete in this project is emphasized by the purpose to reduce building energy consumption as wall blocks.

Aerated concrete is manufactured by entraining air voids deliberately, to come up with lightweight, cellular concrete form. It can be divided into 2 major types based on production method: foamed and aerated concrete (Newman, 2003).

Foamed concrete is produced by injecting a foaming agent into the base mix. In such mixture, no chemical reaction takes place, therefore, it is considered to be the most economical and controllable method of cellular concrete casting (Narayanan and Ramamurthy, 2000). The porosity of the concrete is achieved by adding foaming agent into the mix, which produces air voids that are included in the cement paste (Hamad, 2014). Consequently, foamed concrete characteristics directly depend on the foaming agent properties.

Next way of pore-formation in cellular concretes, is based on the formation of gas bubbles by reaction of chemicals, commonly aluminum powder, with the liquid cement mortar. Concentration of air voids and properties of hardened sample from this reaction depend on the alkalinity of mortar mix. Thus, sand with high silica content is favorable for aerated concrete production (Narayanan and Ramamurthy, 2000). Aluminum powder based porous concrete manufacturing method is assumed to be the best solution by Hamad (2014).

In turn, aerated concrete can be divided as autoclaved or non-autoclaved according to its curing method. Autoclaved aerated concrete is cured in a pressure chamber with high temperature and pressure, and the latter is cured under normal conditions. This project focuses on the Non-Autoclaved Aerated Concrete, because of equipment shortage.

Properties of AC blocks, aside from the thermal insulation that was mentioned before, are shown in the Table 2.8. It also provides comparison of AC blocks with other concrete block types, such as dense aggregate blocks and lightweight aggregate blocks.

Table 2.8. Concrete blocks characteristics comparison

Parameters	Dense aggregate blocks	Lightweight aggregate blocks	Aerated concrete blocks
Weight	↑	→	↓
Insulation	↓	↑	↑
Sound absorption	→	↑	↑
Fire resistivity	↑	↑	↑
Durability	↑	↑	→
Reusability	↑	↑	→
Compressive strength	↑	→	↓
Environmentally friendly	↓	↓	→
Typical thermal conductivity (W/mK)	0.70 - 1.28	0.25 - 0.60	0.09 - 0.40

Aerated concretes have high percentage of air voids and no coarse aggregates, which results in low compressive strength along with low density and weight (Table 2.9). Dense and lightweight aggregate blocks have the opposite characteristics, which are shown by Table 2.10. It can be observed that the maximum compressive strength of AC blocks, which is 5.47 MPa, is lower than the minimum of lightweight and dense aggregate blocks (7.3 MPa). However, insulative properties of AC are better than others that is illustrated by their thermal conductivities. Spence and Kultermann (2016) state that both thermal and sound insulation of the blocks are worse for denser units, as the air voids existence provide better isolation. All three materials are highly reusable, and environmentally conscious, respectively. (Yang and Lee, 2014 & Aggregate Industries, 2016)

Table 2.9. Aerated concrete parameters

Specimens	Flow (mm)	Defoamed depth (mm)	γ_d (kg/m³)	Compressive strength (MPa)			E_c (MPa)	ε_0	λ (W/m K)
				3 days	7 days	28 days			
G1-1000	225	0	493	1.42 (??)	1.96 (??)	2.45	1294	0.0024	0.145 (?)
G1-1250	230	0	496	1.93	2.56	2.85	1696	0.0023	0.138
G1-1500	250	0	477	1.43	1.87	2.46	1384	0.0022	0.134
G1-2000	255	0	477	1.39	1.92	2.34	1181	0.0018	0.137
G1-2500	265	0	487	1.33	2.01	2.42	1400	0.0022	0.127
G2-30	200	0	562	2.75	2.99	4.25	2395	0.0024	0.157
G2-27.5	200	0	570	2.88	3.41	4.32	2115	0.0026	0.160
G2-25	202	2	541	3.15	3.56	4.57	2045	0.0022	0.153
G2-22.5	266	3	545	3.11	3.04	4.17	2071	0.0024	0.163
G2-20	263	5	547	1.81	3.21	3.66	1740	0.0026	0.159
G3-400	235	3	425	1.57	2.01	2.07	1421	0.0020	0.118
G3-450	245	2	491	2.24	2.75	2.99	1562	0.0025	0.139
G3-500	248	2	531	2.57	3.07	3.42	2017	0.0022	0.142
G3-550	237	1	618	3.50	4.15	4.98	2753	0.0024	0.176
G3-600	240	0	674	4.83	4.90	5.88	2966	0.0025	0.199
G3-650	245	0	694	5.23	5.47	6.93	3252	0.0026	0.184

Table 2.10. Dense and lightweight aggregate blocks parameters

Size (mm)	440x215	440x215	290x215	440x215	440x215
Thickness (mm)	100	140	140	100	140
Density (kg/m³)	1950 (Dense)	1950 (Dense)	1950 (Dense)	1450 (Lightweight)	1450 (Lightweight)
Weight (kg)	18.73	26.22	17.28	14.13	19.78
Strength	7.3 and 10.4 N/mm²	7.3 and 10.4 N/mm²	7.3 and 10.4 N/mm²	7.3 and 10.4 N/mm²	7.3 and 10.4 N/mm²
Thermal conductivity (W/mK)	Int. 1.27 Ext. 1.37	Int. 1.27 Ext. 1.37	Int. 1.27 Ext. 1.37	Int. 0.78 Ext. 0.84	Int. 0.78 Ext. 0.84

AC block has a range of positive characteristics, including light weight, high thermal and sound insulation, and fire resistivity. It also has some drawbacks as low compressive strength, so that it can not be used as a load bearing material.

As given project focuses on energy consumption reduction, aerated concrete blocks were chosen for construction. To conduct energy consumption study for further introduction with aerated concrete, it was decided to carry out AC blocks casting laboratory works and software energy modelling. Therefore, different mixture proportions were compared making an emphasis on their thermal conductivity to choose the most appropriate in terms of energy conservation. In general, within aerated concrete mixture dry basis of the ingredients constitute approximately 70% of the total mass, while other 30% is water (Ropelewski and Neufeld, 1999). The Table 2.11 below illustrates the hardened properties for different mix designs.

Table 2.11. Hardened properties comparison

Sources	Density (kg/m³)	Thermal conductivity (W/m°C)	Compressive strength (MPa)
Ecoton (2016)	500	0,109	4,5
Wongkeo et al. (2012)	1457	0,57	9,5
Newman and Choo (2003)	450	0,12	3,2
Aruova and Dauzhanov (2014)	797	0,219	5,4

By comparing the data presented in Table 2.11, mixture design proposed by Ecoton Company was chosen as a reference for further examinations. The mix proportions for casting 1 m³ aerated concrete are shown in Table 2.12.

Table 2.12. Aerated concrete mix proportions provided by Ecoton (V = 1 m³)

Component	Amount	Units
Fine sand mixture	467	kg
Lime	94	kg
Cement	93	kg
Gypsum	20	kg
Aluminium powder	433	g
Water	64	l
Waste	167	kg

However, during laboratory testing the fact that Ecoton casts Autoclaved Aerated Concrete (AAC) should be taken into account. It is supposed that the hardened characteristics of Non-Autoclaved Aerated Concrete would be slightly lower in comparison with AAC, which will be defined and proved during consequent project stages.

2.2 Architectural design

2.2.1 Main Function of the Building

Building primarily designed as a dormitory for master students in Nazarbayev University. As it was mentioned above, it was designed in order to accommodate 350 students. Also, several facilities, such as market, hair shop, canteen and etc. will be placed on the first floor.

2.2.2 Site Location

Generally, it was required to build dormitory on the campus of Nazarbayev University. Location area for the dormitory in Astana is illustrated in Figure 2.7. The dimensions of selected territory are 70m x 70m. Selected area was analyzed by following criteria:

- Noise
- View
- Access to academic building and parking area

Proposed dormitory is located 212 m away from the Kabanbay Batyr Ave and 410 m away from the Turan Ave, so that noise will not disturb residents. Also, surrounding environment of the building is found pleasant for view, because park is located from the left side, existing dormitories from the right and front side, and the Nazarbayev University Intellectual School from back side of the building. Since, it is located 500 m away from the main entrance and 100 m away from the parking area, location of the building can be assumed as open accessible and convenient. Moreover, it was oriented in the s way that sun touches almost every corner of the building. The orientation of building is shown in Figure 2.9.

Figure 2.9. Location of the building

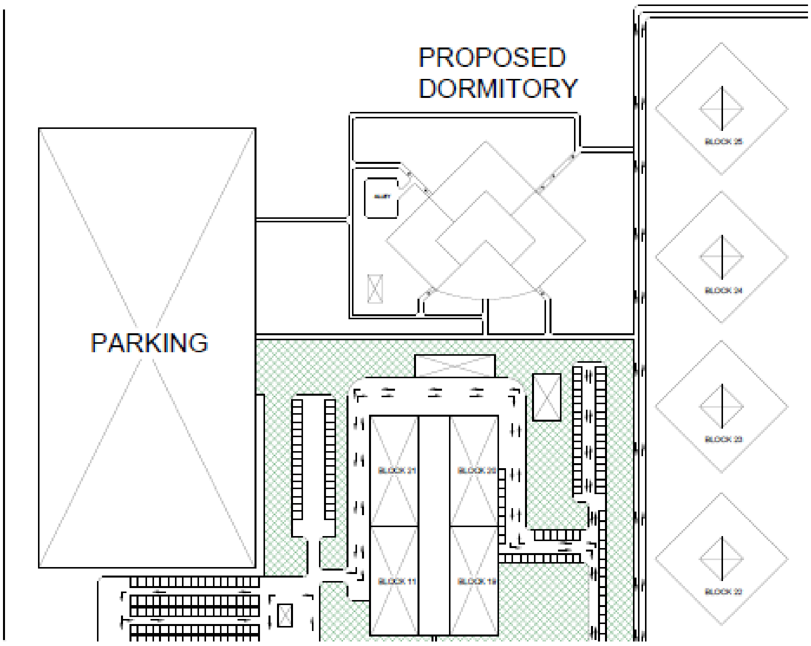


Figure 2.10. Site Layout

2.2.3 Design of Building Geometry

At the beginning of the project several types of buildings geometries were considered and compared. Some of them are represented in Figure 2.11. They were analyzed based on aesthetics, ease of construction, and harmony along with surrounding buildings.

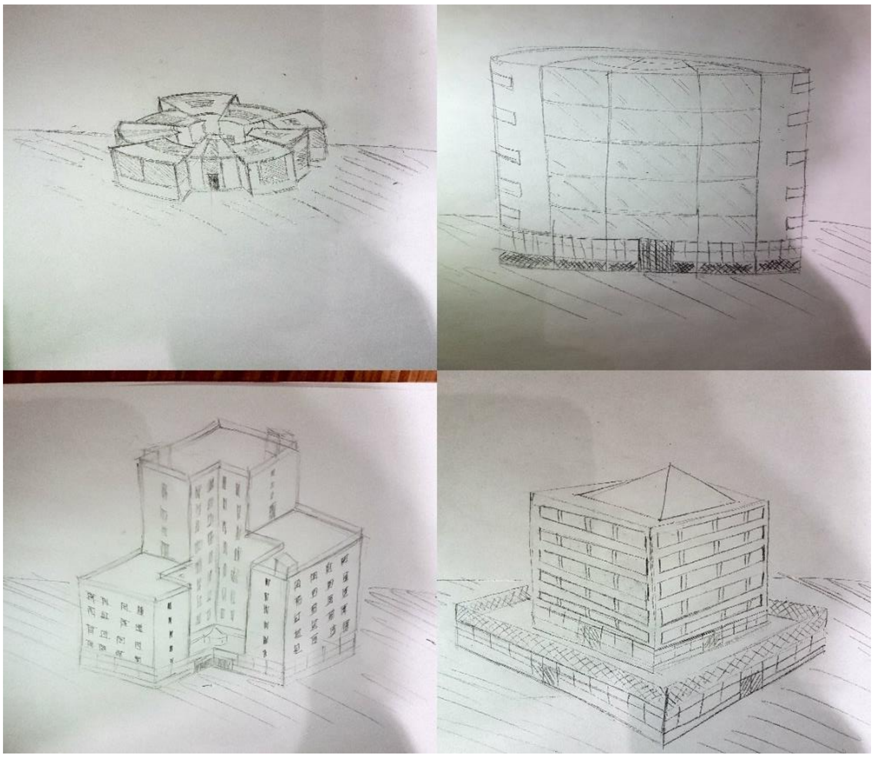


Figure 2.11. Preliminary design sketches

Nazarbayev University is one of the most prestigious universities in Kazakhstan, where a lot of different international conferences and forums are held, which attracts significant number of guests from other countries. The proposed dormitory is located almost in the middle of campus and it is high enough to be seen from each corner of the university territory; thus, it has to provide aesthetically pleasant and presentable view. Despite the potential challenges during structural and geotechnical design, a unique L-shape building geometry was chosen. By using SketchUp Pro software 3D model of the building was created. Several views of the building obtained from the SketchUp software are represented in Figure 2.12

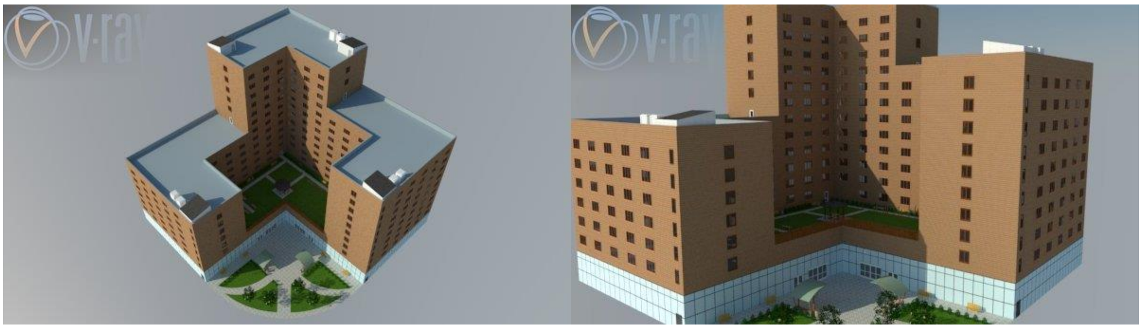


Figure 2.12. 3D model of the building

In terms of dormitory infrastructure, based on the experience of existing ones, the proposed one is developed and has following developments:

- Better noise control
- Larger rooms
- Option to choose between single, double, and family rooms

2.2.4 Building category

According to the International Building Code (IBC), Section 310, dormitory categorized as a Residential Group (R-2) and can be defined as a residential occupancy containing sleeping units or more than two dwelling units where the occupants are primarily permanent in nature. Also, our dormitory can be defined as a Type IA construction (Section 504, IBC), because it’s height more than 48.768 m. Heights of the Blocks are: 8.8 m, 25.8 m, 39.4 m, 50.9 m. Type IA is classified in the UN (Unlimited) category, so it is no limitations regarding height, number of stories, and area.

2.2.4.1 Use and occupancy

Gross area of first = 4 * (30*15+15*15) = 2700 m^2. However, according to the IBC, 40% of the gross area should be subtracted. Thus, total usable area of first floor = 2700 - 2700*0.4 = 1620 m^2. In Section 1004 of IBC, maximum floor area allowance per person for residential buildings is 200 sq. ft. = 18.58 m^2 per occupant. Number of people on the first floor = 1620/18.58 = 87 ≈ 90. In addition, based on the design of the residential floor total number of residents = 385. Total Number of people in building = 475.

2.2.4.2 First floor space allocation

Total usable area of first floor equals to 1620 m^2. It was designed to include market (102m^2), hair shop (52 m^2), two service room (53 m^2), WC (160 m^2), security station and security room (80 m^2), canteen (94 m^2), pharmacy (34 m^2), medical room (64 m^2), multifunctional room (70 m^2), office (40 m^2), four technical room (68 m^2), and fire control room (36.5 m^2). Figure 2.13 illustrates room allocation on the 1st floor.

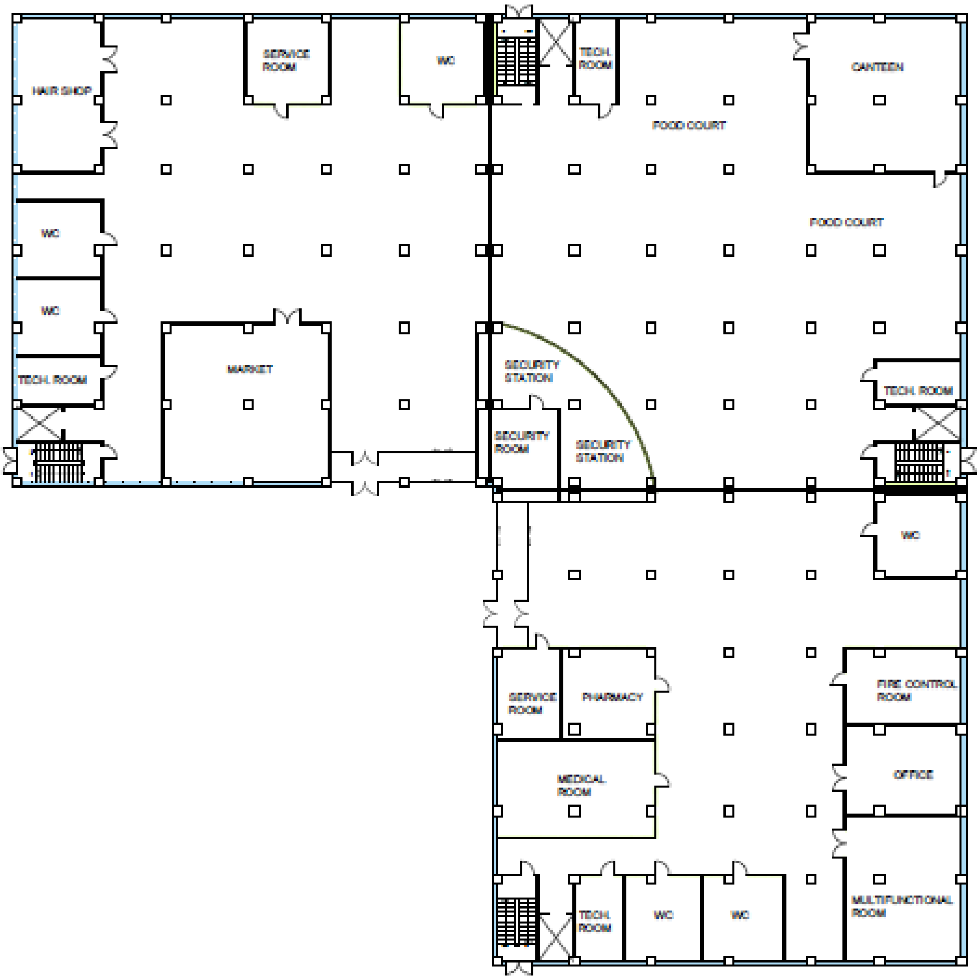


Figure 2.13. First floor plan

2.2.4.3 Resident floor space allocation

Total area of residential floor for one building is 675 m^2. For one floor it was decided to locate single, double, and family room with total area 26 m^2, 40 m^2, and 80 m^2 respectively. Also, laundry with total area 23 m^2 and kitchen with total area 28 m^2 will be installed on each floor of each section of the building. Residential floor plan can be seen from Figure 2.14.

Figure 2.14. Residential floor plan

2.2.4.4 Special detailed calculations

2.2.4.4.1 Number of elevators

Miller states that for every 4180.6 m^2 of total usable area one elevator should be installed. Thus, means that for total 14175 m^2, minimum 4 elevators are required. Also, by using KONE Quick Traffic elevator calculator (2016) results shown on Figure 2.15 were obtained. As it can be seen from the Figure 2.7 the total number of elevator required is 3. It was decided to use 4 elevators in the designed dormitory. The greater number of elevators was used in order to increase safety.

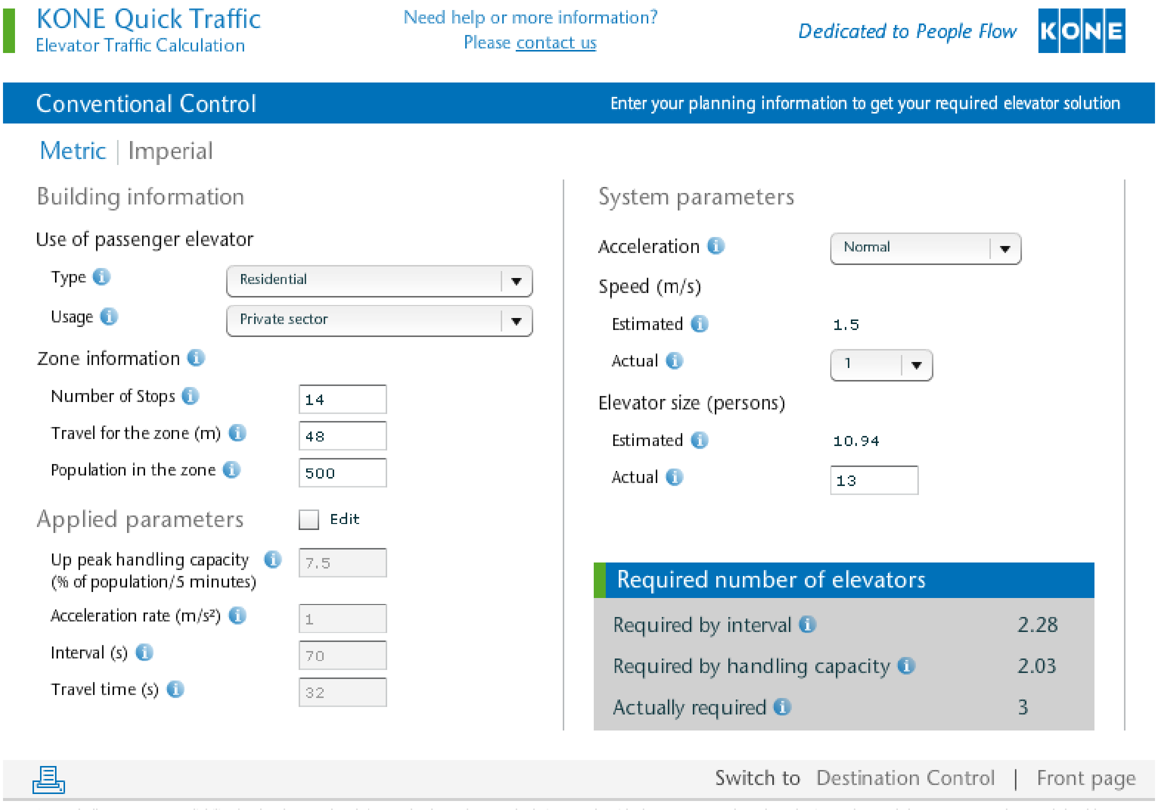


Figure 2.15. Elevator number calculation

2.2.4.5 Stairs

Width of each stair should be more than the 44 inches (1118 mm) for occupant load more than 50 (IBC, Section 1009.4). A stairway should have a headroom clearance of 80 inches (2032 mm) measured vertically (IBC, Section 1009.5). Stair riser height should be more than 4 inches (102 mm) but less than 7 inches (158 mm). Risers' height measured vertically between the nosings of adjacent treads (IBC, Section 1009.7.2). Also, according to the IBC, 1009.7.2, depth of one stair should be minimum 11 inches (279 mm). Stairway landing should be installed at the top and bottom of each stairway and should have more or equal dimensions as the stairway width (IBC, Section 1009.8). Three or more exit access doorways should be provided for the building with occupant capacity 500 and greater. Moreover, one additional exit doorway should be added so that if one will be blocked, other will be available (IBC, Section 1007.1.2). Thus, by observing all standards listed above, 4 evacuation stairways will be installed in the dormitory.

2.2.4.6 Corridors

As it can be seen from the Table 2.13 (IBC, Section 1020.2) for the residential buildings minimum required width for corridor is 44 inches (1118 mm). Thus, in the designed building the minimum 1850 mm corridor was used.

Table 2.13. Minimum corridor width

OCCUPANCY	MINIMUM WIDTH (inches)
Any facilities not listed below	44
Access to and utilization of mechanical, plumbing or electrical systems or equipment	24
With an occupant load of less than 50	36
Within a *dwelling unit*	36
In Group E with a *corridor* having an occupant load of 100 or more	72
In *corridors* and areas serving stretcher traffic in occupancies where patients receive outpatient medical care that causes the patient to be incapable of self-preservation	72
Group I-2 in areas where required for bed movement	96

2.2.4.7 Fire protection system

Different types of codes were used during the designing process. One of the main codes that were used is an International Fire Code (IFC). IFC, Section 905.5.1 states that occupying is prohibited before the required fire detection, alarm, and supervision system have been installed and tested.

2.2.4.7.1 Shaft enclosure

For dormitory with total height less than 420 feet (128 000 mm), fire-resistance rating of the fire barriers enclosing vertical shafts can be reduced to 1 hour, when automatic sprinklers are installed inside the shafts at the top levels (IBC, Section 403.2.1.2)

2.2.4.7.2 Automatic sprinkler system

Designed building should be equipped by an automatic sprinkler system. Installation of the sprinklers should be done according to the National Fire Protection Association (HFPA). HFPA 25 system were selected for the designed building. As it can be seen from the Table 2.14, HFPA 25 is a water based fire protection system. Moreover, automatic sprinkler system shall have a sprinkler control valve supervisory switch and-flow-initiating device provided for each floor that is monitored by the building's fire alarm system (IBC, Section 3008.2.2).

Table 2.14. Fire Protection System Maintance Standards

SYSTEM	STANDARD
Portable fire extinguishers	NFPA 10
Carbon dioxide fire-extinguishing system	NFPA 12
Halon 1301 fire-extinguishing systems	NFPA 12A
Dry-chemical extinguishing systems	NFPA 17
Wet-chemical extinguishing systems	NFPA 17A
Water-based fire protection systems	NFPA 25
Fire alarm systems	NFPA 72
Smoke and heat vents	NFPA 204
Water-mist systems	NFPA 750
Clean-agent extinguishing systems	NFPA 2001

2.2.4.7.3 Emergency system

The whole dormitory will be equipped by the emergency system equipment, which include smoke detection, fire alarm system, standpipe system, emergency voice/alarm communication system, emergency responder radio coverage, and fire command center (IBC, Section 403.4). Minimum dimensions for the fire command center should be 200 sq. ft. (19 m^2) (IBC, Section 911.1.3). Dormitory provides a room with total area of 28.9 m^2.

2.2.4.7.4 Smoke removal

Each building should be equipped with the ventilation system in order to remove products of combustion (IBC, Section 403.4.7). According to the IBC, Section 403, manually operated windows or panels should be installed every 50 ft. (15 240 mm) intervals. The area of each operable window should be more than 40 sq. ft. (3.7 m^2). Also, air - handling system, that provides one exhaust air change every 15 minutes for the area, should be installed in the designed building (IBC, Section 403.4.8).

2.2.4.7.5 Means of egress and evacuation

As it was mentioned before, 4 evacuation stairways with exits will be installed in the designed building. As it can be seen from the Figure 2.16, due to the irregular shape of the building maximum travel distance to the evacuation stairway is 28 m. According to the IBC, Chapter 10, maximum distance from the remote door to evacuation stairway should be 30 m and from the remote corner to the stairway should be 90 m. Both parameters were met during the design stage.

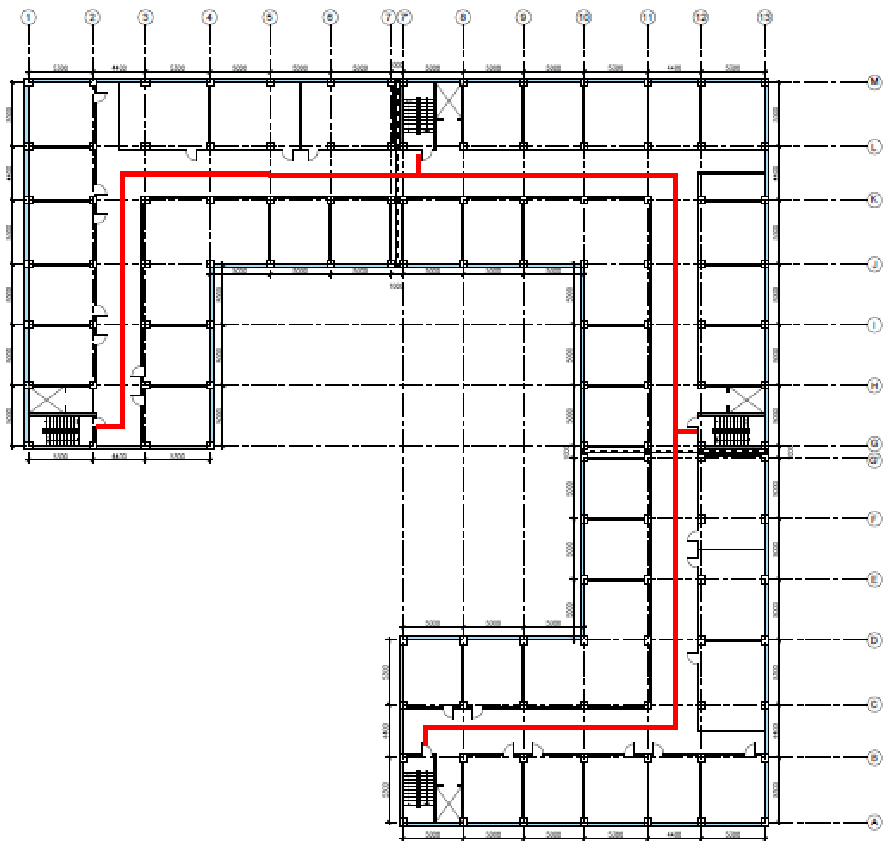


Figure 2.16. Fire Evacuation Plan

2.2.5 Interior Environment

2.2.5.1 Ventilation

According to the IBC, mechanical ventilation should be installed in the dormitories. As it is mentioned in IBC ventilation with power more than 30 Cubic feet per minute per person (cfm) should be placed in the bedrooms, bathrooms and living rooms.

2.2.5.2 Lighting

Every building should be provided with natural light and the minimum net glazed area shall be minimum 8 percent of the floor net area (1205, IBC). According to the IBC every living room should be provided with windows, which total area shall be more or equal to 2.32 m^2. The artificial light with average illumination of 10 foot-candles (107 lux) will be installed at the height of 2800 mm on the residential floors and at the height of 4800mm on first floor.

In addition, stairways will be illuminated by the 1 foot-candle (11 lux) lightbulb according to the IBC.

2.2.5.3 Sound Transmission

In order to provide good sound isolation aerated concrete was used. As it illustrated in section 2.1.2 air layer is placed between two layers of AC. This way of interior wall design will provide both thermal and sound insulation.

2.2.5.4 Minimum Room Width

As it is mentioned in IBC rooms in dormitories should be more than 2134 mm in any plan dimension (1208, IBC).

2.2.5.5 Minimum Ceiling Height

According to the IBC, habitable spaces, occupiable spaces and corridors should have a ceiling more than 2286 mm above the floor level. In addition, bathrooms, laundry, kitchens, and storage rooms need to have ceiling 2134 mm above the floor.

2.3 Structural design

2.3.1 Structural Design Loads

2.3.1.1 Dead Load

Dead load can be described as a self-weight of all permanent structural and non-structural components of the building that includes:

- Weight of members;
- Weight of all construction materials attached to the building and permanently supported by members;
- Weight of permanent partitions;

- Weight of fixed service equipment.

2.3.1.2 Live load

Live loads are mainly caused by the occupancy of the building which includes:

- Normal use by person;
- Furniture and moveable objects;
- Anticipating rare events, such as concentrations of persons or furniture, or the moving or stacking of object which may occur during reorganization or redecoration.

Table 2.15 below contains live loads expected to be imposed on the building

Table 2.15. Live loads

Category of Loaded Area	Occupancy or use	Live load, kN/m2
A	Rooms Stairs	1.5 to 2.0 2.0 to 4.0
C1	Foodcourt Reception	2.0 to 3.0
C2	conference room	3.0 to 4.0
C3	first floor corridors residential floor corridors	3.0 to 5.0
D1	market	4.0 to 5.0
H	roof	0 to 1.0
I	rooftop	5 to 7.5

2.3.1.3 Snow Load

Snow load calculation will be done by following regulations provided by EN 1991-1-3:2003. This document (EN 1991-1-3:2003) has been prepared by Technical Committee CENITC250 "Structural Eurocodes", the Secretariat of which is held by BSI (British Standard Institution). The document also provides information about determining the values of loads due to snow to be used for the structural design of buildings and civil engineering works.

2.3.1.3.1 Terms and Definitions

- Undrifted snow load on the roof – load arrangement which describes the uniformly distributed snow load on the roof, affected only by the shape of the roof, before any redistribution of snow due to other climatic actions.
- Drifted snow load on the roof – load arrangement which describes the snow load distribution resulting from snow having been moved from one location to another location on a roof, e.g. by the action of the wind.
- Roof snow load shape coefficient – ratio of the snow load on the roof to the undrifted snow load on the ground, without the influence of exposure and thermal effects.
- Thermal coefficient – reduction of snow load on roofs as a function of the heat flux through the roof, causing snow melting.
- Exposure coefficient – reduction or increase of load on a roof of an unheated building, as a fraction of the characteristic snow load on the ground.
- Load due to exceptional snow drift – load arrangement which describes the load of the snow layer on the roof resulting from a snow deposition pattern which has an exceptionally infrequent likelihood of occurring.

2.3.1.3.2 Symbols

Table 2.16 below contains all the symbols that will be used in further load calculation explanation.

Table 2.16. Symbols

Latin upper case letters	
C_e	Exposure coefficient
C_t	Thermal coefficient
C_{esl}	Coefficient for exceptional snow loads
A	Site altitude above sea level, m
S_e	Snow load per meter length due to overhang, kN/m
F_s	Force per meter length exerted by a sliding mass of snow, kN/m
Latin lower case letters	
b	Width of construction work, m
d	Depth of the snow layer, m
h	Height of construction work, m

k	Coefficient to take account of the irregular shape of snow
l_s	Length of snow drift or snow loaded area, m
s	Snow load on the roof, kN/m^2
s_k	Characteristic value of snow on the ground at the relevant site, kN/m^2
s_{Ad}	Design value of exceptional snow load on the ground, kN/m^2
Greek Lower case letters	
α	Pitch of roof, measured from horzontal, ^{0}
β	Angle between the horizontal and the tangent to the curve for a cylindrical roof, ^{0}
γ	Weight density of snow, kN/m^3
μ	Snow load shape coefficient
ψ_0	Factor for combination value of a variable action
ψ_1	Factor for frequent value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action

2.3.1.3.3 Design situation

As Astana is located in the region with strong winds and frequent and high amount of snow precipitation, the design situation is assumed to be location with both exceptional snow falls and exceptional snow drifts. For such location following design situations are applied:

a) The transient/persistent design situation for both undrifted and drifted snow load arrangements;
b) The accidental design situation for both undrifted and drifted snow load arrangement;

2.3.1.3.4 Snow load on roofs

The design performed should recognize that snow can be accumulated on roof in different patterns due to number of factors.

a) The shape of the roof;
b) Thermal properties of the roof;
c) Surface roughness of the roof;
d) The amount of heat generated under the roof;
e) The proximity of nearby buildings;

f) The surrounding terrain;

g) The local meteorological climate, particularly, windiness, temperature variations, and precipitation probability as rain or snow.

2.3.1.3.4.1 Snow load calculation

a) For the persistent/transient design situations:

$$s = \mu_i C_e C_t s_k \qquad \text{eq(2.3.1)}$$

b) For the accidental design situation where exceptional snow load is assumed as the accidental action:

$$s = \mu_i C_e C_t s_{Ad} \qquad \text{eq(2.3.2)}$$

c) For the accidental design situation where exceptional snow drift is assumed as accidental action:

$$s = \mu_i s_k \qquad \text{eq(2.3.3)}$$

2.3.1.3.4.2 Exposure coefficient

Dependence of Exposure coefficient on the topography is illustrated in the Table 2.17 below.

Table 2.17. Recommended values of Ce for different topographies

Topography	Ce
Windswept	0.8
Normal	1.0
Sheltered	1.2
Windswept topography: flat unobstructed areas exposed on all sides by wind, or little shelter afforded by terrain, higher construction works or trees. Normal topography: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees. Sheltered topography: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.	

2.3.1.3.4.3 Thermal coefficient

Thermal coefficient has value less than one in case of roofs with high thermal transmittance (>1 W/m^2K) such as glass covered roofs. For all other cases:

$C_t = 1.0$

2.3.1.3.4.4 Roof shape coefficients

The roof of the building to be constructed is chosen to be monopitch roof. Such kind of roof gives opportunity for snow to not fall on the roofs of lower sections, but slide outside of the building area. The roof shape coefficient can be determined from the following Figure 2.15 and Table 2.18.

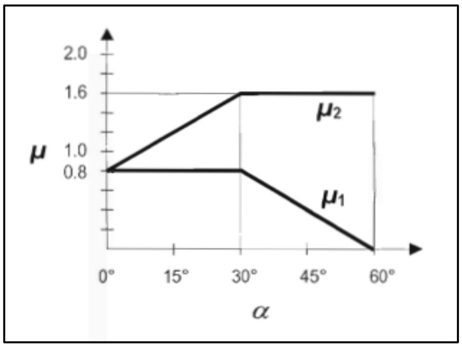


Figure 2.17. Snow load shape coefficient

Table 2.18. Snow load shape coefficient

Angle of pitch of roof α	$0^O \leq \alpha \leq 30^O$	$30^O \leq \alpha \leq 60^O$	$\alpha \geq 60^O$
μ1	0.8	0.8(60-α)/30	0.0
μ2	0.8+0.8α/30	1.6	..

2.3.1.3.4.5 Roof abutting and close to taller construction works

Because the building will have sections with different height the snow load shape coefficient should be calculated as follows:

$\mu_1 = 0.8$ (assuming the lower roof is flat) eq(2.3.4)

$\mu_2 = \mu_s + \mu_w$ eq(2.3.5)

where:

μ_s is the snow load shape coefficient due to sliding of snow from the upper roof

For $\alpha \leq 15^o$, $\mu_s = 0$

μ_w is the snow load shape coefficient due to wind

$\mu_w = (b1 + b2)/2h \leq \gamma h/s_k$ eq(2.3.6)

2.3.1.4 Wind Load

Wind load calculation will be done by following regulations provided by EN 1991-1-4:2005. Guidance on the calculation of natural wind actions on buildings structure, its parts, and elements attached to the structure is provided in the document. It was also prepared by Technical Committee CENIT250 "Structural Eurocode".

2.3.1.4.1 Definitions

Fundamental basic wind velocity – the 10 minute mean wind velocity with an annual risk of being exceeded of 0, 02, irrespective of wind direction, at a height of 10m above flat open country terrain and accounting for altitude effects (if required).

Basic wind velocity - the fundamental basic wind velocity modified to account for the direction of the wind being considered and the season (if required).

Mean wind velocity - the basic wind velocity modified to account for the effect of terrain roughness and orography.

Pressure coefficient - external pressure coefficients give the effect of the wind on the external surfaces of buildings; internal pressure coefficients give the effect of the wind on the internal surfaces of buildings.

Force coefficient - force coefficients give the overall effect of the wind on a structure, structural element or component as a whole, including friction, if not specifically excluded.

Background response factor - the background factor allowing the lack of full correlation of the pressure on the structure surface.

Resonance response factor - the resonance response factor allowing turbulence in resonance with the vibration mode.

2.3.1.4.2 Symbols

Table 2.19 below contains all the symbols that will be used in further load calculation explanation

Table 2.19. Symbols

Latin upper case letters	
A_{ref}	Reference area
B2	Background response part
F_w	Resultant wind force
R2	Reosnant response part
Latin lower case letters	
c_{dir}	directional factor
$c_c(z)$	exposure factor
c_d	dynamic factor
c_t	force coefficient
c_{pe}	external pressure coefficient
c_{pi}	internal pressure coefficient
$c_{p,net}$	net pressure coefficient
c_{prob}	probability factor
c_r	roughness factor
c_o	orography factor
c_s	size factor
c_{season}	seasonal factor
k_i	turbulence factor
k_p	peak factor
k_e	terrain factor
v_m	mean wind velocity
v_b	basic wind velocity
w	wind pressure
z	height above ground
z_0	roughness length
Greek lower case letters	
ρ	air density

2.3.1.4.3 Wind velocity and velocity pressure

In order to calculate wind load simplified set of pressures or forces which have equivalent extreme effect of the turbulent wind should be used. According to EN 1990-4-1-1, wind actions are classified as variable fixed actions and determined based on values of wind velocity or the velocity pressure. The effect of the wind load on the structure depends on several factors such as size, shape and dynamic properties of the structure.

2.3.1.4.3.1 Basis for Calculation

The wind velocity and the velocity pressure include mean and fluctuating component. The mean wind velocity V_m should be determined from the basic wind velocity V_b which depends on the wind climate. The fluctuating component of the wind is represented by the turbulence intensity that will be defined later.

2.3.1.4.3.2 Basic values

The basic wind velocity:

$$v_b = c_{dir}\, c_{season}\, v_{b,0} \qquad \text{eq(2.3.7)}$$

2.3.1.4.3.3 Mean wind

Variation with height z:

$$v_m(z) = c_r(z)\, c_0(z)\, v_b \qquad \text{eq(2.3.8)}$$

The recommended procedure for the calculation the roughness factor at height z:

$$c_r(z) = k_r \cdot ln\left(\frac{z}{z_0}\right) \quad \text{for } z_{min} \leq z \leq z_{max} \qquad \text{eq(2.3.9)}$$

$$k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07}$$

$$c_r(z) = c_r(z_{min}) \qquad \text{for } z < z_{min}$$

Values for z_0 and z_{min} are taken from the Table 2.20 below; z_{max} is to be taken as 200 meters.

Table 2.20. Terrain categories and terrain parameters

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	0.3	5

	heights (such as villages, suburban terrain, permanent forest)		
IV	Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m	1.0	10

Orography factor c_0 should be taken into account in cases where orography increases wind velocities by more than 5%. In other cases orography factor is taken as 1.0.

2.3.1.4.3.4 Wind turbulence

The turbulence intensity $I_v(z)$ is calculated by dividing the standard deviation of the turbulence to the mean wind velocity. Standard deviation of the turbulence:

$$\sigma_v = k_r \cdot v_b \cdot k_l \qquad \text{eq(2.3.10)}$$

The recommended rules for the determination of $I_v(z)$

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_l}{c_0(z) \cdot ln(z/z_0)} \quad \text{for} \quad z_{min} \leq z \leq z_{max} \qquad \text{eq(2.3.11)}$$

$$I_v(z) = I_v(z_{min}) \quad \text{for} \; z < z_{min}$$

2.3.1.4.3.5 Peak velocity pressure

The peak velocity pressure qp(z) at height z in terms of mean and short-term velocity fluctuations:

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b \qquad \text{eq(2.3.12)}$$

Exposure factor: $c_e(z) = \frac{q_p(z)}{q_b}$ eq(2.3.13)

Basic velocity pressure: $q_b = \frac{1}{2} \cdot \rho \cdot v_b^2$ eq(2.3.14)

2.3.1.4.4 Wind Actions

2.3.1.4.4.1 General

Following Table 2.21 contains the summary of the wind action calculation procedure.

Table 2.21. Calculation procedure for the determination of wind actions

Parameter
peak velocity pressure q_p
basic wind velocity v_b
reference height z_e
terrain category
charcteristic peak velocity pressure q_p

turbulence intencity I_v mean wind velocity v_m orography coefficient $c_0(z)$ roughness coefficient $c_r(z)$
Wind pressure, e.g. for cladding, fixing and structural parts external pressure coefficient c_{pe} internal pressure coefficient c_{pi} net pressure coefficient c_{pnet} external wind pressure: $we=q_pc_{pe}$ internal wind pressure: $wi=q_pc_{pi}$
Wind forces on structures, e.g. for overall wind effects structural factor: c_sc_d wind force F_w calculated from force coefficients wind force F_w calculated from pressure coefficients

2.3.1.4.4.2 Wind pressures on surfaces

Wind pressures acting on the external and internal surfaces can be determined using equations below.

$$w_e = q_p(z_e) \cdot c_{pe} \qquad \text{eq(2.3.15)}$$

$$w_i = q_p(z_i) \cdot c_{pi} \qquad \text{eq(2.3.16)}$$

Figure 2.16 below provides examples of pressures on surface of roof and wall elements. Pressures, directed towards the surface are considered as positive, and pressures with opposite direction considered as negative.

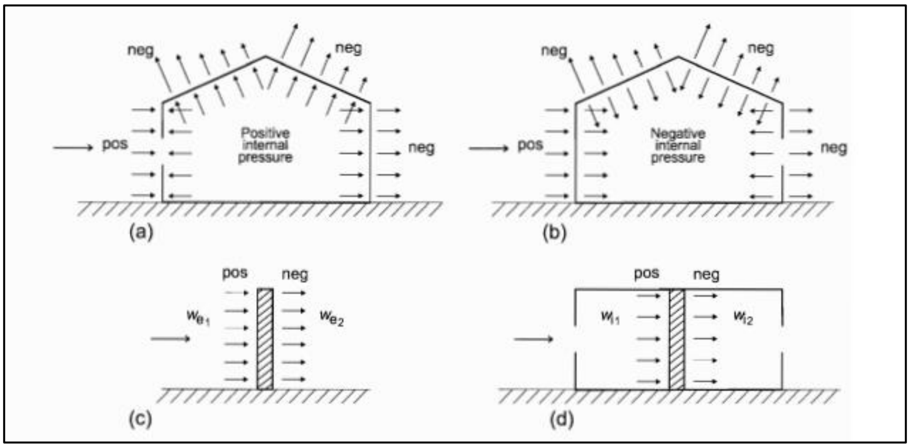


Figure 2.18. Pressures on surfaces

2.3.1.4.4.3 Wind forces

The wind force Fw can be obtained directly using eq(2.3.17).

$F_w = c_s c_d \cdot c_t \cdot q_p(z_e) \cdot A_{ref}$ eq(2.3.17)

or by vectorial summation over the individual structural elements.

$F_w = c_s c_d \cdot \sum_{elements} c_t \cdot q_p(z_e) \cdot A_{ref}$ eq(2.3.18)

2.3.1.4.4.4 Determination of structural factor cscd

For the following cases $c_s c_d$ may be taken as 1.0:

a) For buildings with a height less than 15 m.

b) For facade and roof elements having a natural frequency greater than 5 Hz

c) For framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth.

d) For chimneys with circular cross-sections whose height is less than 60 m and 6,5 times the diameter.

For all other cases $c_s c_d$ calculated following detailed procedure:

$c_s c_d = \frac{1+2 \cdot k_p \cdot I_v(z_s) \cdot \sqrt{B^2+R^2}}{1+7 \cdot I_v(z_s)}$ eq(2.3.19)

2.3.1.5 Load Combinations

According to BS EN 1990:2002 Eurocode – Basis of Structural Design load combination varies whether the structure is considered as rigid body (EQU) or a structural element (STR).

2.3.1.5.1 Load Combination EQU

$Design\ Load = \sum_{j \geq 1} \gamma_{G,j} G_{k,j}$ "+" $\gamma_{Q,1} Q_{k,1}$ " + " $\sum_{i>1} \gamma_{Q,1} \psi_{0,1} Q_{k,i}$ eq(2.3.20)

The load factors:

- $\gamma_{G,j}$ = 1.10 (unfavourable), 0.90 (favourable)
- $Q_{k,1}$ is the leading variable action
- $\gamma_{Q,1}$ = 1.50 (unfavourable), 1.00 (favourable)
- $Q_{k,I}$ are accompanying variable actions
- $\gamma_{Q,I}$ = 1.50 (unfavourable), 1.00 (favourable)

2.3.1.5.2 Load Combination STR

$Design\ Load = \sum_{j \geq 1} \gamma_{G,j} G_{k,j}$ "+" $\gamma_{Q,1} Q_{k,1}$ " + " $\sum_{i>1} \gamma_{Q,1} \psi_{0,1} Q_{k,i}$ eq(2.3.21)

Or alternatively

$Design\ Load = \sum_{j \geq 1} \gamma_{G,j} G_{k,j}$ "+" $\gamma_{Q,1} \psi_{0,1} Q_{k,1}$ " + " $\sum_{i>1} \gamma_{Q,1} \psi_{0,1} Q_{k,i}$ eq(2.3.22)

$$Design\ Load = \sum_{j\geq 1} \xi \gamma_{G,j} G_{k,j}\ \text{"+"}\ \gamma_{Q,1} Q_{k,1}\text{" + "} \sum_{i>1} \gamma_{Q,1} \psi_{0,1} Q_{k,i} \qquad \text{eq(2.3.23)}$$

ξ = reduction factor for unfavourable permanent actions G

The load factors:

- $\gamma_{G,j}$ = 1.35 (unfavourable), 1.00 (favourable)
- $Q_{k,1}$ is the leading variable action
- $\gamma_{Q,1}$ = 1.50 (unfavourable), 1.00 (favourable)
- $Q_{k,I}$ are accompanying variable actions
- $\gamma_{Q,I}$ = 1.50 (unfavourable), 1.00 (favourable)
- ξ = 0.85

2.3.2 Structural Analysis

2.3.2.1 Hand Calculations

In order to proceed to member design of the building it is needed to obtain internal stresses, forces, and deflections occurred due to external loadings calculated in the way presented in above section. Commonly, buildings with reinforced concrete structures are considered as statically indeterminate, because of the fact that columns and beams are poured as continuous members through the joints and over supports. For solving such structures simple force and moment equilibrium equations are insufficient. Number of structural analysis theories and methods which require satisfaction of equilibrium and compatibility are helpful in solving indeterminate structures, taking into account that system to be solved undergone some assumptions and simplifications.

2.3.2.2 Approximate Analysis

Approximate analysis is the analysis of the structure when it is already brought to simpler model. Performing such analysis makes it possible to obtain preliminary design of the structure members. After this, more detailed and complicated analysis can be done and design can be improved.

- Vertical Load on Building Frames

It cannot be definitely said that actual connection of beam or girder with column is extremely rigid or flexible, it is somewhere between. In case of extremely stiff connection (fixed support) zero moment points located at the distance 0.21L from the edges of the girder (see Figure 2.17)

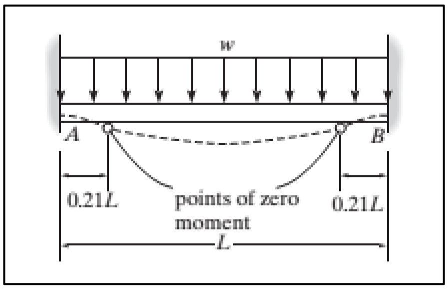


Figure 2.19. Fixed supported

In case of simply supported girder, points of zero moment are located at the supports (see Figure 2.18).

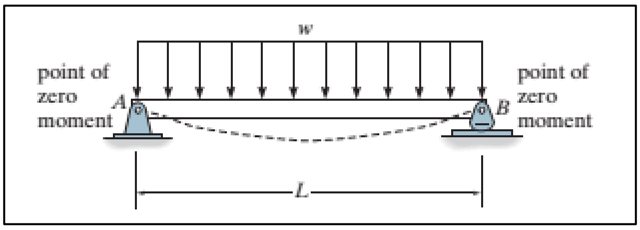


Figure 2.20. Simply supported

Because the actual connection is neither fixed nor simply supported, it is assumed that zero moment points located at the middle of between two extremes, 0.1L. From the following Figures 2.19 and 2.20 approximate case and simplified model of the system can be found.

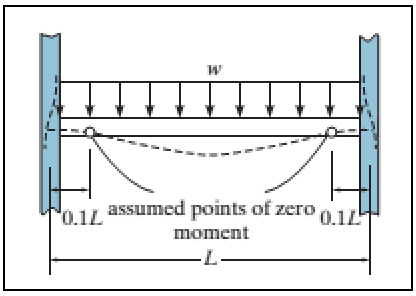


Figure 2.21. Approximate case

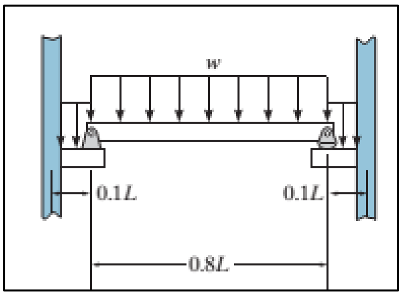


Figure 2.22. Model

- Lateral Loads on Building Frames

Shear forces, axial forces and bending moments in members of the building frame also caused by the lateral loadings such as wind actions. One of the approximate methods that allows to determine internal forces in the frame due to lateral loadings is called cantilever method. This method assumes that the frame behaves similar way as cantilever beam during under lateral loadings (see Figure 2.21)

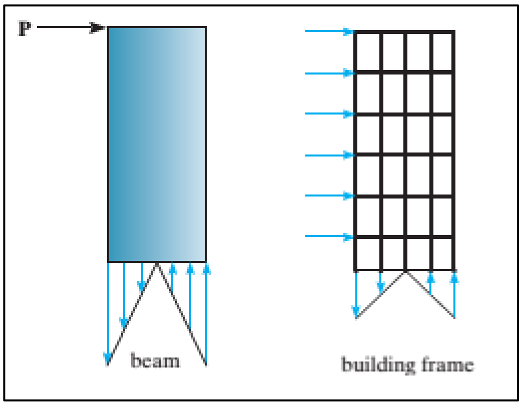


Figure 2.23. Cantilever method

Following assumptions are applied to a fixed-supported frame:

- Hinge is placed at the center of each girder as it is the location of a point of zero moment;
- Hinge is placed at the center of each column as it is the location of a point of zero moment;
- The axial stress in a column is proportional to its distance from the centroid of the cross-sectional areas of the columns a a given floor level. In case of columns with equal cross-sectional areas, axial force in column is proportional to its distance from the centroid.

There is one more approximate analysis method useful in determining internal forces due to lateral loads called portal method. Assumptions made in this method:

- Points of inflection occur at approximately the center of each girder.
- Columns carry equal shear loads.
- Interior columns carry represent the effect of two portal columns and therefore carry twice the shear as the exterior columns (see Figure 2.24).

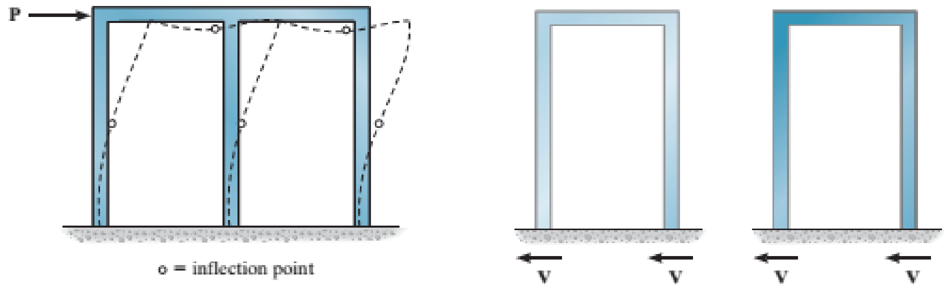


Figure 2.24 Portal method

2.3.2.3 Software Calculations

Even though there are approximated methods to obtain internal forces in the frame, it is better to use software programs to determine more accurate values of internal forces in building that consist of large number of frames. SAP 2000 is chosen by DC Group to perform structural analysis of the student residence building. It is a stand-alone finite-element-based structural program for the analysis and design of civil structures. The software provides user friendly interface as well as analytical techniques needed to do the most complex projects.

Furthermore, model created both in 2D and 3D frame at SAP 2000 presents the physical reality as the software is object based. For example, in a model beam with multiple framing into it is generated as a single object as it would be in the real world. Also, connection between members provided by meshing and handled internally by the program.

2.3.3 Structural Member Design

Structural member design procedure will be done in accordance with EN 1991-1-1:2004, the document prepared by Technical Committee CENT/TC250 «Structural Eurocodes». It was generated to provide guidance in design of buildings and civil engineering works in plain, reinforced, and prestressed concrete.

2.3.3.1 General Design Considerations

Following sections will include general code requirements that should be taken into account during design of structural members.

2.3.3.1.1 Concrete cover

The shortest distance between the surface of the reinforcement and concrete surface is defined as concrete cover. The value for nominal cover is summation of a minimum cover, c_{min}, and an allowance in design for deviation, Δc_{dev}.

$$c_{nom} = c_{min} + ,\Delta c_{dev} \qquad \text{eq(2.3.24)}$$

Minimum concrete cover, $c_{min,}$ is provided to guarantee safe transmission of bond forces, protection of the steel corrosion, and an adequate fire resistance. More detailed calculation procedure for concrete cover calculation can be found in Figures B1 to B3 of the Appendix B.

2.3.3.1.2 Spacing of bars

Bars should be placed in way that concrete will be properly compacted and adequate bonding will be provided.

The clear distance between parallel placed bars or horizontal layers of parallel bars should be greater than k_1*bar diameter, (d_g+k_2 mm) or 20mm. d_g is the maximum size of aggregate, and recommended values for k_1 and k_2 are 1 and 5 mm respectively.

2.3.3.1.3 Permissible mandrel diameters for bent bars

Bars should be bent such that they will not crack due to bending and will not cause failure of the concrete inside the bent. Figure B4 in the Appendix B provides information on minimum mandrel diameter.

2.3.3.1.4 Anchorage details

The anchorage of longitudinal reinforcement including reinforcing bars, wires or welded mesh fabrics should be designed in the way that the bond forces appropriately transmitted to the concrete without longitudinal cracking or spalling. Additionally, transverse reinforcement can be provided if needed.

The anchorage of link and shear reinforcement should be made as bends and hooks, or by welded transverse reinforcement. More particular information on anchorage can be found in Appendix B.

2.3.3.1.5 Laps and mechanical couplers

Lap requirements:

- The transmission of the forces from one bar to the next should be assured;
- Spalling of the concrete in the neighbourhood of the joints should not occur;
- Large cracks which affect the performance of the structure should not occur;
- Should not be located in areas of high moments;
- Should be arranged symmetrically at any section;
- Clear distance between lapped bars should be smaller than 4φ or 50 mm;
- The longitudinal distance between adjacent laps should be greater than 0.3 times the lap length;

2.3.3.1.6 Shear reinforcement of members

$V_{Rd,c}$ is the design shear resistance of the member without shear reinforcement.
$V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement.
$V_{Rd,max}$ is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

The shear resistence of a member with shear reinforcement is equal to:

$$V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td} \qquad \text{eq(2.3.25)}$$

When $V_{Ed} \leq V_{Rd,c}$, no shear reinforcement is required.

In case when $V_{Ed} \geq V_{Rd,c}$, shear reinforcement should be provided appropriately.

Design process for members that require shear reinforcement is provided in Figures B6 to B12 of Appendix B.

2.3.3.2 Beam Design

Following steps describe the process of designing rectangular beam.

Step 1: Estimation of effective depth d (8%~10% of span length).

Step 2: Calculation of depth h of the beam with estimated bar diameter, link diameter and cover.

Step 3: Assuming beam width as 0.4~0.6 of beam depth taking into account the fire resistance requirement.

Step 4: Calculation of self-weight of the beam.

Step 5: Calculation of design moment based on all loads

Step 6: Determination of λ, η and δ based on f_{ck}

Step 7: Calculation of the k_{max} from Table 4.8 (see Appendix B Figure B13) and then $d_{min} = (M/k_{max}bf_{ck})^{1/2}$

Step 8: Adjusting beam depth based on values found in step 6.

Step 9: Comparison of self-weight with the adjusted depth against the initial assumption Repeating steps 1 to 6 in case of difference larger than 5%.

Step 10: Calculating $k = M/(bd^2f_{ck})$

Step 11: Calculation of level arm $z/d = 0.5[1+(1-3k/\eta)^{1/2}]$

Step 12: Calculation of required steel $A_s = \gamma_s M/(f_{yk}z)$

Step 13: Checking for minimum and maximum amount of steel

Step 14: Sizing and arranging the reinforcing steel in the beam section.

More detailed information on the reinforcement of the beam is provided in Figures B14 to B20 of the Appendix B.

2.3.3.3 Slab Design

Following steps describe design process of the one-way slab.

Step 1: Determination of minimum thickness of the slab (assuming no deflection) $h = L/20$.

Step 2: Calculation of design moment $M_u = w_u*L^2/8$.

Step 3: Calculating non-prestressed reinforcement ratio $R_n = M_u/\varphi bd^2$.

Step 4: Checking calculated value for ρ.

Step 5: Determination of reinforcement amount $A_s = \rho bd$.

Step 6: Checking of bar spacing

Step 7: Calculation of the amount of transverse reinforcement $A_s = 0.0018bd$.

Step 8: Checking obtained steel amount for minimum and maximum limits.

Step 9: Sizing and arranging the reinforcing steel in slab sections.

More detailed information on the reinforcement of the slab can be found in Figures B20 to B25 of Appendix B.

2.3.3.4 Column Design

RC column can be designed by following steps.

Step 1: Find trial section by considering axial force N_u only

Step 2: Calculation of N/bhf_{ck} and M/bh^2f_{ck}

Step 3: Finding reinforcement ratio $\rho = A_sf_y/bhf_{ck}$ from design chart

Step 4: Checking code requirement for reinforcement

Step 5: Select links based on code requirements

Code requirements can be found in Figures B26 and B27 of Appendix B.

2.3.4 Deflection check for beams and slabs

After reinforcement of beams is designed, it was necessary to check it for deflection. Firstly, it is necessary to obtain percentage of tension reinforcement (ρ) and if necessary percentage of compression reinforcement and (ρ′):

$$\rho = \frac{A_s}{b*d} \qquad \text{eq(2.3.26)}$$

$$\rho' = \frac{A_s'}{b*d} \qquad \text{eq(2.3.27)}$$

Where,

A_s – area of tension reinforcement

A_s' - area of compression reinforcement

b – beam width

b – beam depth

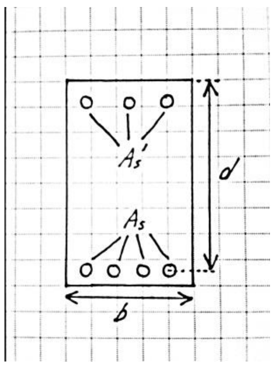


Figure 2.25 Cross section of a beam

The next step is to determine basic l/d based on the following equation:

For $\rho \le \rho_0$, $l/d = k * [11 + \frac{1.5*\sqrt{f_{ck}}*\rho_0}{\rho} + 3.2 * \sqrt{f_{ck}} * \left(\frac{\rho_0}{\rho} - 1\right)^{1.5}]$; eq(2.3.28)

For $\rho > \rho_0$, $l/d = k * [11 + \frac{1.5 * \sqrt{f_{ck}} * \rho_0}{(\rho - \rho')} + \frac{\sqrt{f_{ck}}}{12} * \sqrt{\frac{\rho_0}{\rho}}]$; eq(2.3.29)

Where,

f_{ck} – characteristic cube strength

$\rho_0 = \frac{\sqrt{f_{ck}}}{1000}$

k = 1.0 for simply supported span

k = 1.5 for interior span

k = 1.3 for end span

k = 0.4 for cantilevers

Finally, obtained value of basic l/d is compared to actual l/d, and the actual one has to be no more than basic one.

2.4 Geotechnical design

Geotechnical engineering is one of the most crucial branches of civil engineering, concerning construction, occurring under the ground surface. One of the main purposes of geotechnical engineering is to design structure foundations. According to Das (2011), in the design of foundations, the main factors to be considered are: the load transferred by the structure to the foundation, the local building code requirements, and the soil behavior that will support the foundation system.

2.4.1 Soil condition

Since the proposed student residence building is located on the Campus of Nazarbayev University, soil data was obtained from the report provided by LLP “KaragandaGIIZ and Co”. Table 2.22 represents soil profile of Nazarbayev University campus area. Table 2.23 indicates mechanical properties of soil layers. The water table is found to be at the level of 1.5-2.3 m (Ospanova, 2015).

Table 2.22. Soil profile

#	Soil layer	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
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, medium density, saturated
5	Sand and gravel	5,3-8,5	1,8-7,1	Brown and grey, medium density, saturated, some interlayers of loam
6	Gravel	10,3-11,3	0,4-1,9	Grey brown, saturated
7	Loam	11,1-14,1	1,3-5,9	reddish, yellowish, some interlayers of clay and insignificant presence of ballast

Table 2.23. Mechanical properties of soil layers

#	Soil layer	Density (g/cm3)	Cohesion (kPa)	Friction angle (ϕ)	Modulus of elasticity (MPa)
1	Backfill	1,87	-	-	-
2	Loam	1,97	18	22	19
3	Medium sand	1,92	2	35	17
4	Coarse sand	2	1	38	21
5	Sand and gravel	2	1	38	21
6	Gravel	2,05			23
7	Loam	1,93	34	32	18

2.4.2 Selection criteria

The selection of a particular foundation type is generally based on the following factors:

- Bearing capacity failure: the foundation of the structure must be safe against a bearing capacity failure
- Settlement: the settlement of the foundation must not exceed the maximum allowable settlement

- Quality: the appropriate material of adequate quality must be used for foundation, so it is not subjected to deterioration
- Adequate strength: the foundation must be designed with sufficient strength to withstand loads applied by structure; besides, the foundation must be properly constructed, based on the design specifications.
- Adverse soil changes: the foundation must be designed considering potential long-term adverse soil changes.
- Seismic forces: the foundation must be able to support the structure in case of earthquakes preventing excessive settlement and lateral movement.

2.4.3 Shallow foundations

Shallow foundation is usually used when the load applied by the structure will not induce significant settlement of the underlying soil layers. In general, shallow foundations are more economically beneficial but cannot be used for high-rise buildings. Mainly, shallow foundations can be divided into two groups: 1) spread footings, combined footings, and strip footings; 2) mat foundations.

- Spread Footings, Combined Footings, and Strip Footings

These types of shallow foundations are used more often compared to mat foundations (Figure 2.26).

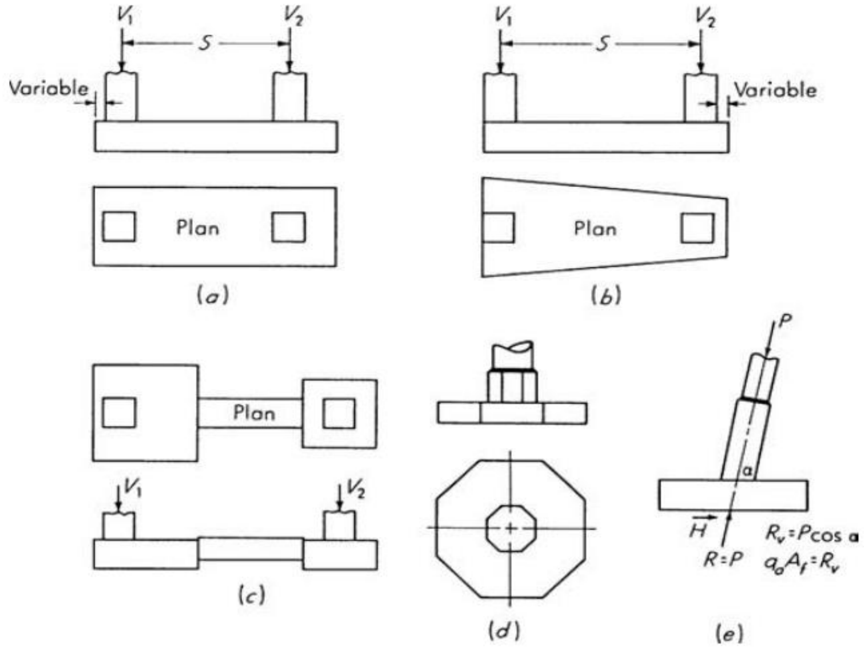


Figure 2.26 Examples of shallow foundations: a) combined footing; b) combined trapezoidal footing; c) cantilever or strip footing; d) octagonal footing; e) eccentric loaded footing

2.4.4 Mat foundations

When foundation supports more than one line of columns, it is called mat foundation (Das 2011). Mat foundations are usually for the following cases:

- Large individual footing: mat foundations are usually selected as a foundation type when the sum of individual footing areas is more than a half of the total foundation area.
- Cavities or compressible lenses: when the exploration of subsurface shows that there is a possibility of not uniform settlement due to small cavities or compressible lenses below the foundation, mat foundation is appropriate solution, because it can span over the cavities and weak lenses, and provide more uniform settlement condition.
- Shallow settlements: mat foundation is recommended to use when shallow settlements are dominating, and, consequently, mat foundation is able to minimize differential settlements.
- Hydrostatic uplift: mat foundation is recommended to use when the foundation will be subjected to hydrostatic uplift forces.

Typical mat foundation variations are shown in the Figure 2.27

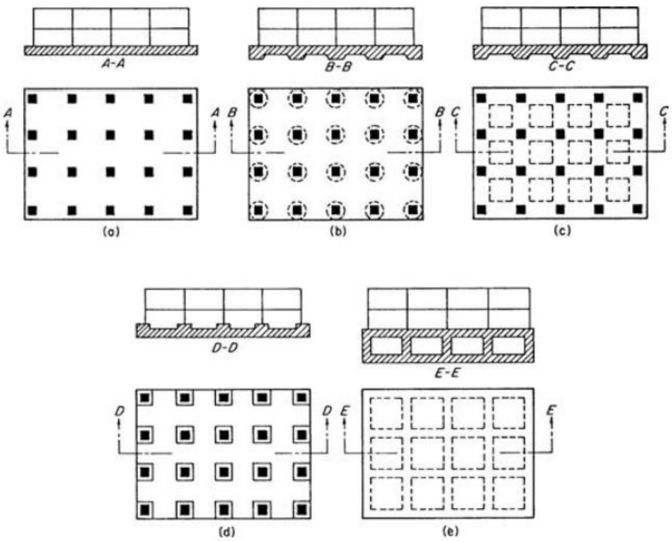


Figure 2.27. Some types of mat foundations: a) flat plate; b) plate thickened under columns; c) beam-and-slab; d) plate with pedestals; e) basement walls as part of mat

2.4.5 Shallow foundation alternatives

According to Day (2006), if large settlement is expected, there are some other options for foundation support or soil stabilization to be considered:

- Grading: this operation includes removing compressible soil and replacing it with structural fill. This operation is beneficial only if the compressible soil layer is near ground surface, and water table is below compressible soil layer.
- Surcharge: if there is an underlying compressible cohesive soil layer, the site can be surcharged with a fill placed at the ground level. In order to accelerate the consolidation process, vertical drains can be installed. Once the compressible cohesive soil layer has consolidated enough, the surcharge is removed.
- Densification of soil: loose or soft soil layer can be densified through dynamic compaction or compaction grouting.
- Floating foundation: to balance the structure weight soil can be removed and an underground basement is constructed.

2.4.6 Deep foundations

The most widely used type of deep foundations is pile foundation. According to Coduto (2001), piles can be described as long, slender, column-like members usually made of steel, concrete, or timber. Piles are usually driven into soil in specific arrangement and are used as a support for reinforced concrete pile caps or a mat foundation. Figure 2.28 represents some typical pile configurations.

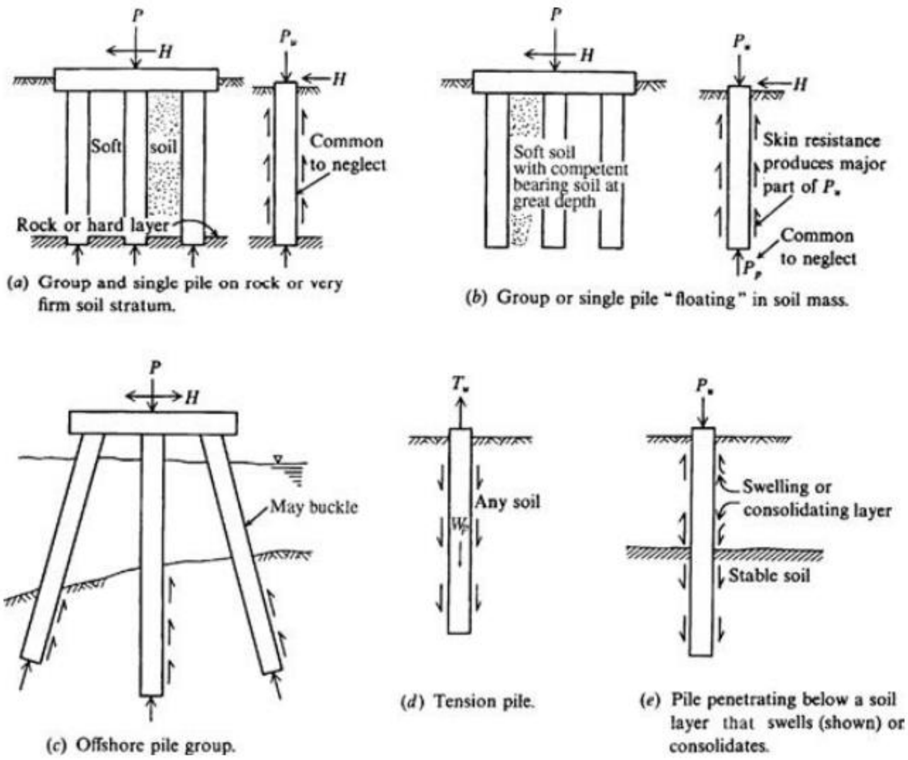


Figure 2.28. Typcial pile configuration

In terms of support capacity, piles can be classified into some types:

- End-bearing pile: the support capacity of this type of pile is derived from the resistance of the foundation layer the pile's tip lies on. End-bearing piles are usually used when hard rock layer underlie a soft upper soil layer (Figure 2.29).
- Friction pile: the support capacity of this type of pile is derived from the resistance of the soil friction and adhesion appeared along the pile length. Friction piles are, usually, used, when the soil is soft and the end-bearing capacity is small (Figure 2.29).
- Combined end-bearing and friction pile: the support capacity of this type is derived from both the resistance of the soil friction and adhesion, and the end-bearing resistance.
- Batter pile: is, usually, used to resist the lateral loads and is driven at an angle inclined to the vertical (Figure 2.30).

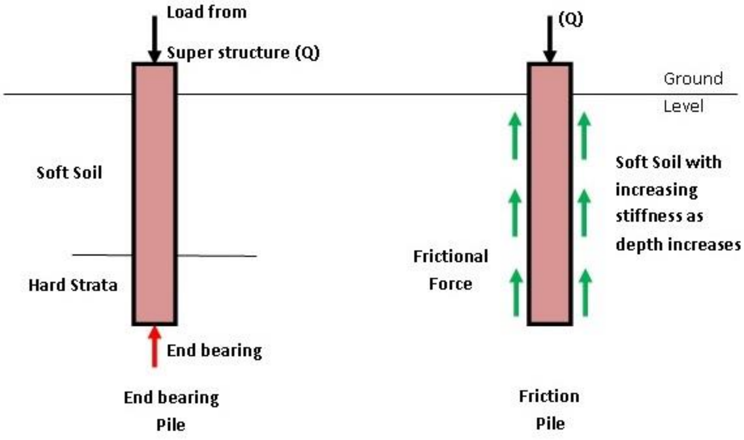


Figure 2.29. End bearing pile and Friction pile

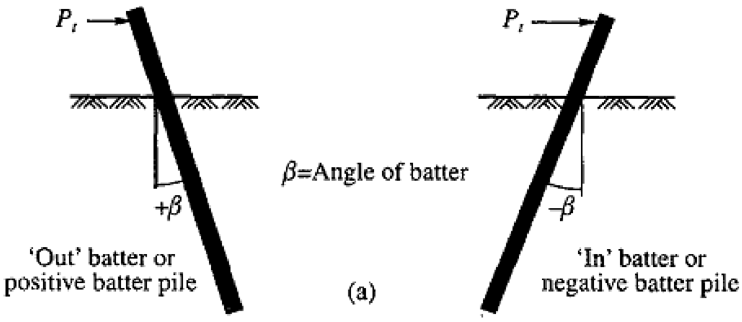


Figure 2.30. Batter pile

Table 2.24 presents comparison of pile types.

Table 2.24. Comparison of different pile types

Pile type	Timber	Steel	Precast concrete (including prestressed)	Cast in place (shells driven without mandrel)	Composite
Optimum length	9-20 m	12-50 m	12-15 m for precast 18-30 m for prestressed	9-25 m	18-36 m
Optimum load range	130-225 kN	350-1050 kN	350-3500 kN	450-700 kN	250-725 kN
Disadvantages	-vulnerable to damage in hard driving -difficult to splice	-vulnerable to corrosion -can be damaged or deflected by major obstructions	-difficult to handle unless prestressed -high initial cost -prestressed difficult to splice	-hard to splice after concreting -considerable displacement	-difficult to achieve good joint between two materials
Advantages	-low initial cost -easy to handle	-easy to splice -high capacity -able to penetrate through light obstructions -small displacement	-high capacity -hard driving possible -corrosion resistance can be reached	-can be redriven -shell not easily damaged	-considerable length can be provided at comparatively low cost
Remarks	-best suited for friction pile in granular material	-best suited for end bearing on rock	-cylindrical piles are suited for bending resistance	-best suited for friction piles of medium length	-the weakest of any material used shall govern allowable stress and capacity

2.4.7 Selection of foundation type

Since the proposed student residence building is considered as a high-rise building, deep foundations have to be used. According to soil profile shown in Table 2.22, no rock layer was identified in first 15 meters underground. Therefore, friction piles are to be used, where the support capacity is derived from the resistance of the soil friction and adhesion appeared along the pile length. Based on the comparison described in Table 2.24, precast concrete is the most appropriate, in terms of capacity reaching up to 3500 kN. Considering the height of the building, high capacity is preferable. Moreover, since Astana's climate is known to have high exposure to wind, piles will be subjected to lateral loads. This is another reason of choosing precast concrete pile, because they are suited for bending resistance.

2.4.8 Selection of installation technique

Since precast concrete pile is chosen, driving pile method will be used (Figure 2.31). The types of hammer used in pile driving are shown in Figure 2.32: a) the drop hammer; b) the single-acting air or steam hammer; c) the double-acting and differential air or steam hammer; d) the diesel hammer (Das, 2011).

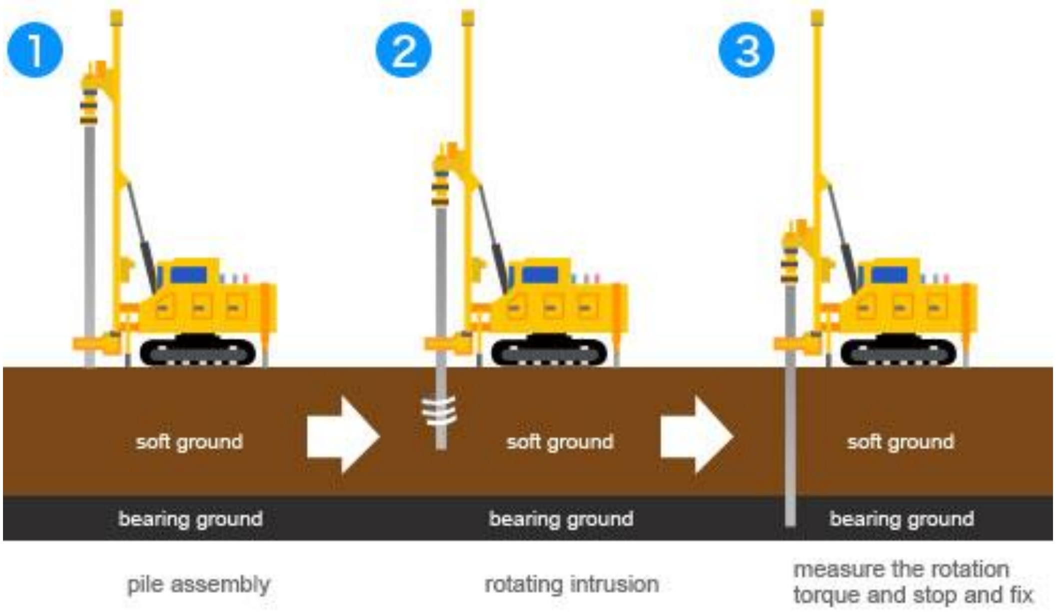


Figure 2.31. Driving pile method

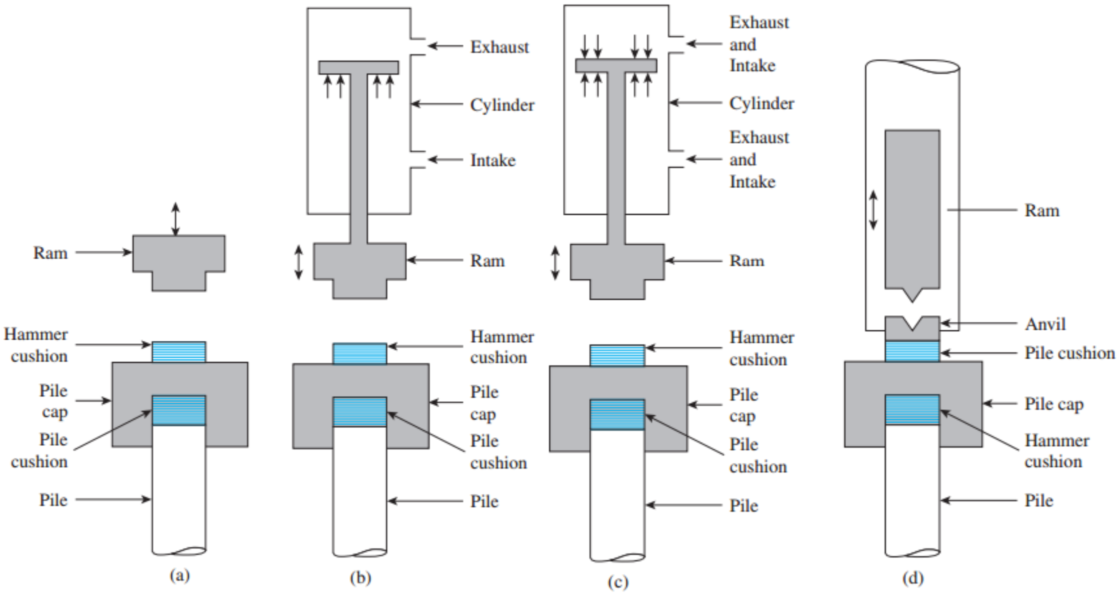


Figure 2.32. Hammer types

Table 2.25 represents average efficiencies of different hammer types. Diesel hammer is found to be the most efficient one (Fleming et al, 2008). However, in terms of availability and environmental considerations, hydraulic hammers are assumed to be better, as they produce less amount of pollutants. In addition, this type of hammer is widely used in Kazakhstan, which leads to the solution of choosing hydraulic hammers.

Table 2.25. Hammer efficiencies

Hammer type	Average efficiency
Single and double acting hammer	77,5%
Drop hammer	85,0%
Diesel hammer	80,0%

2.4.9 Pile design

EN 1997 states that the equilibrium equation to satisfy ultimate limit state design of piles is,

$$F_{c;d} \leq R_{c;d} \qquad \text{eq(2.4.1.)}$$

Where,

$F_{c;d}$ – design axial compression load

$R_{c;d}$ – pile compressive design resistance

The design axial compression load will be identified in structural analysis section, whereas the pile compressive design resistance will be estimated in following section 2.4.9.1.

2.4.9.1 Pile capacity estimation

The ultimate load-carrying capacity Q_u of a pile can be calculated using (Figure 2.33):

$$Q_u = Q_p + Q_s \qquad \text{eq(2.4.2.)}$$

Where,

Q_p – load-carrying capacity of the pile tip

Q_s – frictional resistance derived from the soil-pile interface

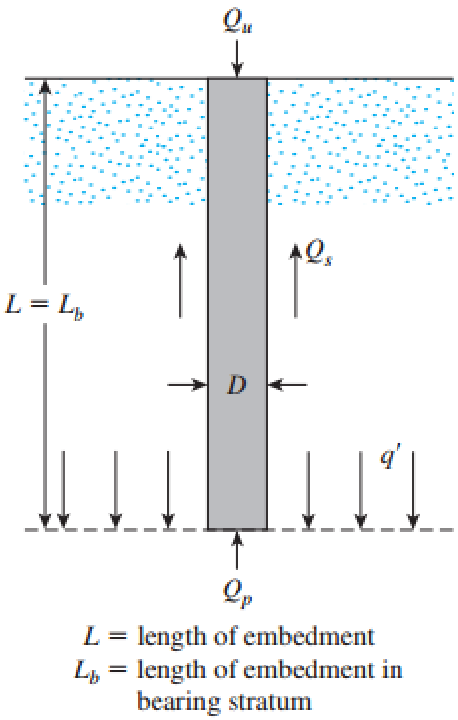


Figure 2.33. Ultimate load-carrying capacity of a pile

The point bearing of a pile can be obtained using:

$$Q_p = A_p * q_p = A_p * (c' * N_c^* + q' * N_q^*) \qquad \text{eq(2.4.3.)}$$

Where,

A_p – area of a pile tip

c′ - cohesion of the soil supporting the tip

q_p – unit point resistance

q' - effective vertical stress at the level of the pile tip

N_c^*, N_q^* - the bearing capacity factors

The frictional resistance can be written as:

$$Q_s = \sum p * \Delta L * f \qquad \text{eq(2.4.4.)}$$

Where,

p – perimeter of the pile section

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

f – unit friction resistance at any depth z

Allowable load can be calculated as:

$$Q_{all} = \frac{Q_u}{FS} \qquad \text{eq(2.4.5.)}$$

Where,

Q_{all} – allowable load-carrying capacity for each pile

FS – safety factor, usually 2.5-4.0

The above mentioned Q_p and Q_s can be calculated with corresponding equations for sand and clay. For sand, Q_p can be estimated by Meyerhof's method, which is described as,

$$Q_p = A_p * q_p = A_p * q' * N_q^* \leq A_p * q_t \qquad \text{eq(2.4.6.)}$$

$A_p q_t$ is a limiting value for Q_p, or Q_p should not exceed this value, and,

$$q_t = 0.5 p_a N_q^* \tan \varphi' \qquad \text{eq(2.4.7.)}$$

Where,

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

N_q^* - bearing capacity factor (Appendix C)

Frictional resistance, Q_s can be found using eq(2.4.4) given above, by substituting

$$f = K \sigma_o' \tan \delta' \qquad \text{eq(2.4.8.)}$$

Where,

K – effective earth pressure coefficient (Appendix C)

σ'o – effective vertical stress at the depth under consideration

δ' – soil-pile friction angle (=0.8φ' according to Coyle and Castello)

In case of presence of cone penetration test results, Q_p and Q_s can be estimated by using correlations with these results. For pile tip resistance,

$$Q_p = A_p * q_p = A_p * q_c, \qquad \text{eq(2.4.9.)}$$

Where,

q_c is cone penetration resistance

Skin resistance can be calculated using,

$$Q_s = \sum p * \Delta L * f = \sum p * \Delta L \alpha' f_c, \qquad \text{eq(2.4.10.)}$$

Where,

f_c is frictional resistance obtained by cone penetration

α' can be obtained from figure given in Appendix C.

2.4.9.2 Group piles

Generally, piles are grouped together and a pile cap is constructed above group piles to provide better load bearing capacity. Group piles should be designed properly so that the individual load-bearing capacities of the piles are not reduced. The factor contributing to this is the spacing between piles, which is about 3-3.5D. When the piles are placed too close, the pile stresses transmitted to the soil overlap and result in the reduction of load-bearing capacity.

Assuming that the total number of piles in a group = n_1*n_2 in the Figure 2.34,

$$L_g = (n_1 - 1)d + 2\left(\frac{D}{2}\right) \qquad \text{eq(2.4.11.)}$$

$$B_g = (n_2 - 1)d + 2\left(\frac{D}{2}\right) \qquad \text{eq(2.4.12.)}$$

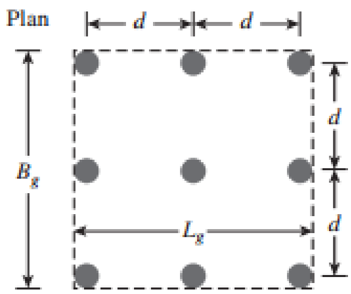


Figure 2.34 Group piles

The efficiency of group piles is described by,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} \qquad \text{eq(2.4.13.)}$$

Where,

η – group efficiency

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

Q_u – ultimate load-bearing capacity of individual pile

Group piles can act in two ways: as a block ($V=L_g* B_g*L$), or as individual piles. Thus, by simplifying the equation with assumption that group piles in sand act as a block,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{2(n_1+n_2-2)d+4D}{pn_1n_2} \qquad \text{eq(2.4.14.)}$$

2.4.9.3 Pile cap design

According Whitaker (1976), in order to conduct analysis of pile loads and pile caps, the vertical and horizontal loads on a pile need to be determined. For rigid pile cap, vertical load on a pile group (P) can be calculated by,

$$P = N + weight\ of\ pile\ cap + weight\ of\ backfill\ on\ pile\ cap, \qquad \text{eq(2.4.15.)}$$

Where,

N is combined vertical load on a pile cap

M_x is combined moment about x-x

M_y is combined moment about y-y

h is the depth of pile cap

In turn, vertical and horizontal loads on a pile are equal to,

$$Vertical\ load\ on\ a\ pile = \frac{P}{R} \pm \frac{M_{xx}y}{I_{xx}} \pm \frac{M_{yy}x}{I_{yy}}, \qquad eq(2.4.16.)$$

Where,

R is the number of piles in a group

M_{xx} is moment about x-x on pile group ($=M_x + Ne_y + H_y h$)

M_{yy} is moment about y-y on pile group ($=M_y + Ne_x + H_x h$)

I_{xx} is moment of inertia about x-x axis ($=\Sigma y^2$)

I_{yy} is moment of inertia about y-y axis ($=\Sigma x^2$)

$$Horizontal\ load\ on\ any\ pile = \frac{(H_x^2 + H_y^2)^{1/2}}{R}, \qquad eq(2.4.17.)$$

Where,

H_x is combined horizontal load on a pile cap in x-x direction

H_y is combined horizontal load on a pile cap in y-y direction

Figure 2.35 below shows loads on a pile cap.

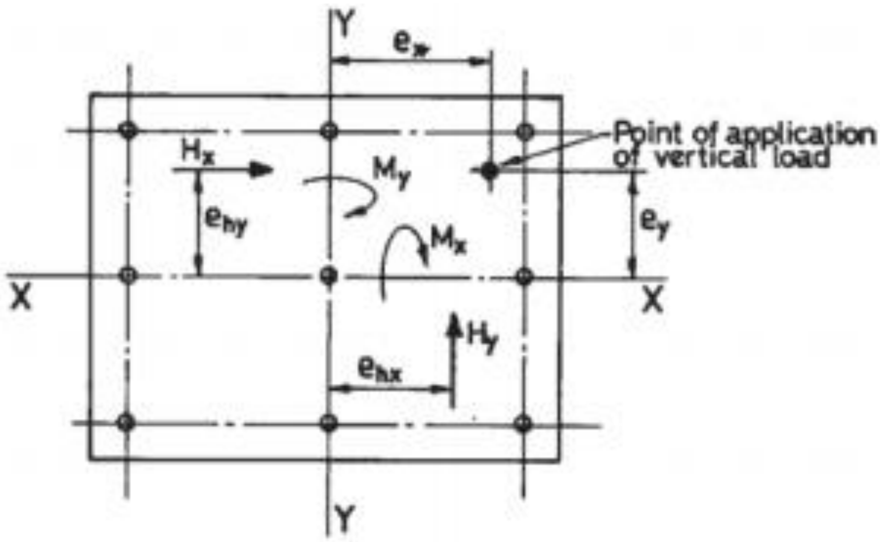


Figure 2.35. Plan view of loads and eccentricity on pile cap

Next step is reinforcement design for the pile cap. The cover to reinforcement depends on the concentration of the sulfates in the soil, and can be found in table given in Appendix C. Before determination of reinforcement area K ($= M / (f_{cu}bd^2)$) should be checked:

$K = \frac{M}{f_{cu} b d^2} \leq 0.156,$ eq(2.4.18.)

Where,

f_{cu} is concrete characteristic cube strength at 28 days

b is width of section over which moment acts

d is effective depth to tension reinforcement

If K ≤ 0.156, then the depth of pile cap need to be increased. Area of reinforcement can be derived from,

$A_{st} = \frac{M}{0.87 f_y z},$ eq(2.4.19.)

Where,

$z = d\left[0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)}\right] \leq 0.95d,$ eq(2.4.20.)

If pile spacing is ≥ 3D, punching shear check is required. The shear stress has to be less than $0.8(f_{cu})^{1/2}$ or 5 N/mm^2. The parameters needed for punching shear check are illustrated in Figure 2.34.

$v = \frac{P}{Ud} \leq 0.8\sqrt{f_{cu}} \leq 5\,\frac{N}{mm^2},$ eq(2.4.21.)

Where,

U is perimeter at punching shear plane

P is ultimate vertical column load or ultimate vertical pile reaction

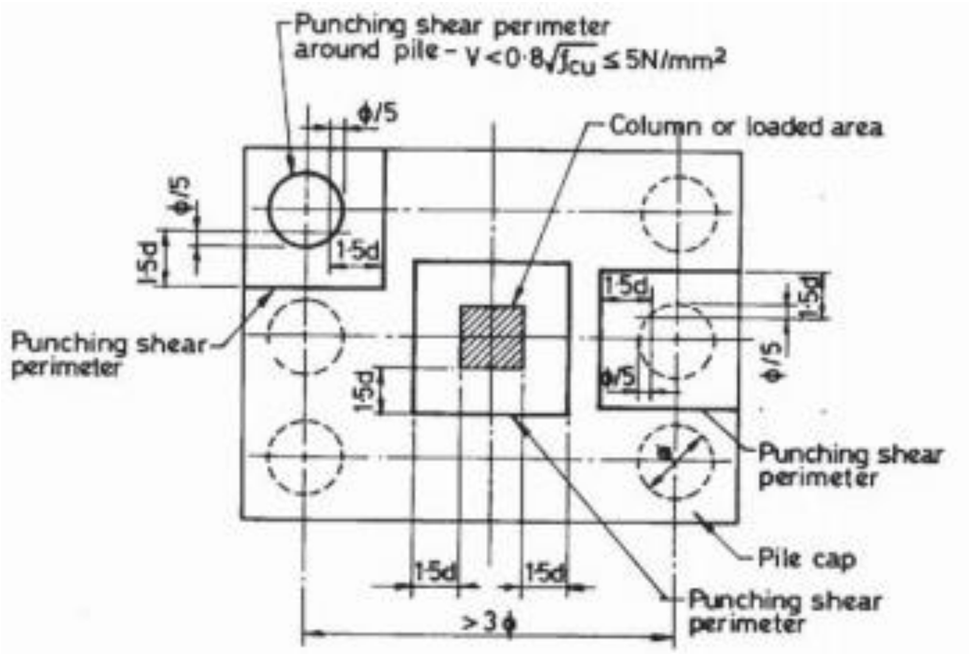


Figure 2.36. Perimeters for punching shear check in a pile cap

2.4.9.4 Settlement estimation

Since the soil profile of chosen location does not contain clay, therefore, consolidation settlement can be ignored. Thus, the elastic settlement needs to be calculated. According to Das (2011), the settlement of a pile group with equal working load on piles depends on the width of the group and pile spacing,

$$s_{g(e)} = \sqrt{\frac{B_g}{D}}\, s_e, \qquad \text{eq(2.4.22.)}$$

Where,

$s_{g(e)}$ is elastic settlement of group piles

B_g is width of group pile section

D is width or diameter of each pile in a group

s_e is elastic settlement of each pile at comparable working load

Total elastic settlement of a pile subjected to vertical load Q_w, can be obtained by equation:

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)}, \qquad \text{eq(2.4.23.)}$$

Where,

$s_{e(1)}$ is elastic settlement of pile

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

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

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}, \qquad \text{eq(2.4.24.)}$$

Where,

Q_{wp} is load carried at the pile point under working load condition

Q_{ws} is load carried by frictional (skin) resistance under working load condition

A is area of cross section of pile

L is length of pile

E is modulus of elasticity of pile material

ξ varies between 0.5 – 0.67, depending on the distribution of skin resistance along the pile shaft

$$s_{e(2)} = \frac{q_{wp} D}{E_s}(1 - \mu_s^2) I_{wp}, \qquad \text{eq(2.4.25.)}$$

Where,

q_{wp} is point load per unit area at the pile point ($=Q_{wp}/A_p$)

E_s is modulus of elasticity of soil at or below the pile point

μ_s is Poisson's ratio of soil

I_{wp} is influence factor (≈ 0.85)

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL}\right)\frac{D}{E_s}(1 - \mu_s^2) I_{ws}, \qquad \text{eq(2.4.26.)}$$

Where,

p is perimeter of pile

L is embedded length of pile

I_{ws} is influence factor ($= 2 + 0.35(L/D)^{1/2}$)

Allowable settlement can be figured out by tables given in the Appendix C (Ricceri and Soranzo, 1985).

2.5 Energy modelling

In order to prove the energy efficiency of buildings with aerated concrete compared to the ones with normal concrete the simulation of one story of proposed residence building can be conducted using «Energy Plus» software with the help of its auxiliary application «OpenStudio» software and «SketchUp». In terms of hand calculations, heat loss through the walls during cold period and heat transfer of construction wall during warm period can be estimated.

2.5.1 Energy Plus Simulation

- Energy Plus Overview

Energy Plus is the building energy simulation software widely used by engineers and architects to model energy as well as to simulate water use in buildings. This simulation tool is developed and funded by the U.S. Department of Energy Building Technologies Office.

According to U.S. DOE (2015), Energy Plus program's main feature is integrated simultaneous solution of thermal zones and calculation of heating and cooling loads to maintain temperature set points year round. There is a large database of weather files for a variety of cities over the world.

Heat transfer algorithms are set heat balance-based solution, where radiant and convective effects of both interior and exterior are considered and thermal loads are obtained simultaneously on an hourly basis. Moreover, air movement between zones is accounted through combined heat and mass balance model.

Since Energy Plus is not a user interface, but just a simulation engine, it needs some auxiliary programs. U.S. DOE mostly uses OpenStudio software development kit (Building technologies office: EnergyPlus energy simulation software, 2015). In this energy modelling, SketchUp integrated OpenStudio plugin is used for more use friendly view and simplified input functions. For drawing the building geometry with all necessary fenestration SketchUp program tools are used (Figure 2.24).

Analysis is performed for the schedule with desired time steps defined by a user.

Figure 2.37. Energy Plus, Open Studio, and SketchUp softwares

2.5.2 Heat Loss Calculation

Another way to prove the efficiency in using aerated concrete is calculating heat loss through the walls of the building.

Degree Day is the index of fuel consumption demonstrating how many degrees F the mean temperature fell below 65 degrees F for the day.

Heating Degree Days (HDD65) is used to calculate the amount of energy needed for residential space heating during the cool season.

Heat Loss per Degree Day is the loss per day with a one degree between inside and outside temperature.

To calculate the Heat Loss per Degree Day the following formula can be used:

$$Q = \frac{[A]*[T_{inside} - T_{outside}]}{R} * 24\, {}^{hr}/_{day} \qquad \text{eq(2.5.1)}$$

Where,

A – Total Wall Area [ft^2]

$T_{inside} - T_{outside} = 1°F$

R – Thermal Resistance [hr*ft^2*F/Btu]

To obtain Thermal Resistance value the following equation is used:

$$R = \frac{l}{\lambda} \qquad \text{eq(2.5.2)}$$

Where,

l - Thickness of normal concrete layer [ft]

λ – Thermal conductivity [Btu*in/hr*ft^2*F]

2.5.3 Heat transfer through construction wall

One dimensional steady state model was taken as a base for heat transfer problem. The wall system is composed of two convection elements on each wall surfaces and one heat conduction element inside the wall. The wall is assumed to be homogeneous material with constant thermal conductivity value.

The heat transfer model can be interpreted in terms of electrical circuit model, where heat flow (q) is a current; voltage represents temperature (T), and representation of constant current source is heat source. Thermal resistance (R) is represented by electrical resistance. As it can be seen from the Figure 2.25, two convection elements are connected with one conduction element in series.

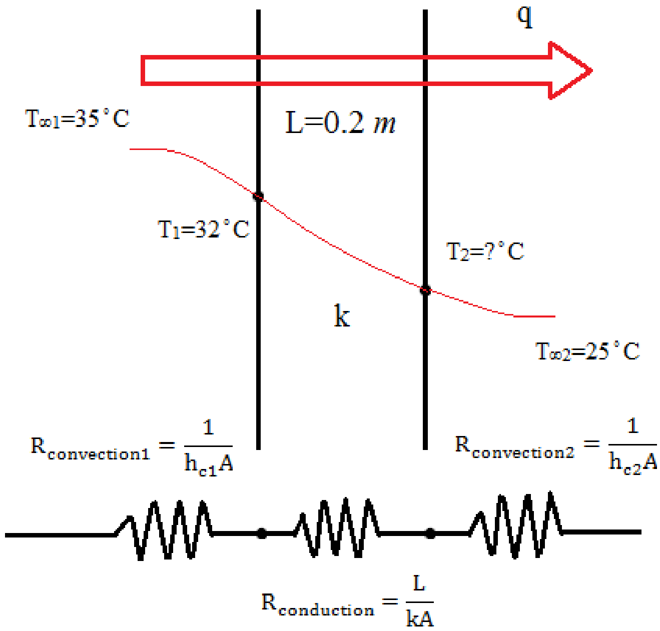


Figure 2.38. One dimensional steady state heat transfer model of wall

- Convection Resistance

Based on Newton’s Law of Cooling Convection Resistance can be defined as:

$$q = h_c * A_c * (T_s - T_\infty) \qquad \text{eq(2.5.3)}$$

Eq(2.5.3.) can be written as:

$$q = \frac{(T_S - T_\infty)}{R_{convection}} \qquad \text{eq(2.5.4)}$$

From two equations above, convection resistance is defined as:

$$R_{convection} = \frac{1}{h_c * A_s} \qquad \text{eq(2.5.5)}$$

In heat transfer at a surface within a fluid, the Nusselts number is defined as:

$$Nu = \frac{h_c * L}{k} \qquad \text{eq(2.5.6)}$$

Where,

L – characteristic length

k – thermal conductivity of the fluid

hc – convective heat transfer coefficient fluid

Consequently, the average convective heat transfer for isothermal surface is:

$$\overline{h_c} = \frac{\overline{Nu} * k}{L} \qquad \text{eq(2.5.7)}$$

In convective heat transfer, Rayleigh number is defined as:

$$Ra = Gr * Pr \qquad \text{eq(2.5.8)}$$

Where,

Gr – Grashof number

Pr – Prandtl number

To obtain Grashof number the following formula is used:

$$Gr_1 = \frac{g * \beta * (T_{\infty 1} - T_1) * L^3}{\nu^2} \qquad \text{eq(2.5.9)}$$

Where,

g – gravitational acceleration

ν - kinematic viscosity

β - coefficient of volume expansion, for ideal gas $\beta = \frac{1}{T}$

Nusselts number for the air is determined based on the Rayleigh number:

If $10^{-1} < Ra < 10^{4}$, Nu is determined through the «Correlation of heated vertical plate» chart

If $10^4 < Ra < 10^{12}$,

$$Nu = C * (Ra)^a \qquad \text{eq(2.5.10)}$$

Where,

C = 0.59, a = 1/4 when $10^4 < Ra < 10^9$

C = 0.13, a = 1/3 when $10^9 < Ra < 10^{12}$

The properties of gases such as Pr and ν at atmospheric pressure can be found using the table provided by Holman (1997) based on the mean film temperature $T_{mean} = \frac{T_\infty + T_S}{2}$

- Conduction Resistance

Considering a wall of homogeneous material, constant thermal conductivity, and a constant temperature at both surfaces, one dimensional steady state conduction can be defined as:

$$q = k * A * \frac{(T_1 - T_2)}{L} \qquad \text{eq(2.5.11)}$$

Where,

k – thermal conductivity of the wall material

A – cross section area of the wall

T – uniform surface temperature

It also can be written as:

$$q = \frac{(T_1 - T_2)}{R_{conduction}} \qquad \text{eq(2.5.12)}$$

From the two equations above, conduction resistance can be defined as:

$$R_{conduction} = \frac{L}{k * A} \qquad \text{eq(2.5.13)}$$

3 Development of Aerated Concrete for Building Energy Performance Analysis

3.1 Materials' properties

Laboratory works were carried out to further investigate insulative properties of AC blocks. Materials required for experimental part were provided by Ecoton company. The company is using fine aggregates from 3 various sources: Karasar, Korgalzhyn, and Red Flag (Figure 3.1). The samples differ by silica content in the sand (from left to right: highest to lowest).

Figure 3.1. Sand samples (sources from left to right: Karasar, Red Flag, Korgalzhyn)

In their mix design, Ecoton is using finely milled mixture of 3 fine aggregates to reduce the cost and maintain the highest properties (Figure 3.2). The proportions of all samples are the same and equal to 33.3%.

Figure 3.2. Milled sand mixture

Material properties investigations were conducted first. Due to the lack of time, characteristics of fine aggregates were examined only. The list of experiments on sand include moisture content specification (Table 3.1), particle size distribution (Appendix A), absorption capacity determination (Table 3.2), and specific gravity investigation (Table 3.3).

Table 3.1. Moisture contents of fine aggregates

Materials	Fine/Coarse	Wt (Stock+bowl)	Wt (od+bowl)	Wt (bowl)	Wt (stock)	Wt (od)	Moisture Conent
		(g)	(g)	(g)	(g)	(g)	(%)
Karasar (yellow)	Fine aggr.	403,0	400,0	112,0	291,0	288,0	1,04
Red flag (Brown)	Fine aggr.	298,0	294,0	111,0	187,0	183,0	2,19
Korgalzhyn (Grey)	Fine aggr.	319,0	313,0	106,0	213,0	207,0	2,90

Table 3.2. Absorption capacities of fine aggregates

Materials	Wt (bowl)	Wt (SSD+bowl)	Wt (SSD)	Wt (od+bowl)	Wt (od)	AC (%)	AC (%)
	(g)	(g)	(g)	(g)	(g)		avg
Karasar (yellow)	161,0	461,0	300,0	460,0	299,0	0,33	0,33
	163,0	463,0	300,0	462,0	299,0	0,33	
Red flag (brown)	194,0	494,0	300,0	478,0	284,0	5,63	6,20
	161,0	461,0	300,0	442,0	281,0	6,76	
Korgalzhyn (grey)	159,0	396,0	237,0	390,0	231,0	2,60	2,34
	162,0	406,0	244,0	401,0	239,0	2,09	

Table 3.3. Specific gravities of fine aggregates

Materials	Wt (pyc +water)	Wt (bowl)	Wt (SSD +bowl)	Wt (SSD)	Wt (syc+water +SSD)	Wt (od+bowl)	Wt (od)	SG (od)	SG (SSD)	SG (app.)
	(g)	(g)	(g)	(g)	(g)	(g)	(g)			
Karasar (yellow)	657,0	161,0	461,0	300,0	858,0	460,0	299,0	3,02	3,03	3,05
	660,0	163,0	463,0	300,0	845,0	462,0	299,0	2,60	2,61	2,62
Red flag (brown)	678,0	194,0	494,0	300,0	858,0	478,0	284,0	2,37	2,50	2,73
	649,0	161,0	461,0	300,0	825,0	442,0	281,0	2,27	2,42	2,68
Korgalzhyn (grey)	647,0	159,0	396,0	237,0	791,0	390,0	231,0	2,48	2,55	2,66
	671,0	162,0	406,0	244,0	821,0	401,0	239,0	2,54	2,60	2,69

3.2 Aerated concrete mix design

It was decided to design 5 different mixes with water-cementitious material ratio w/c = 0.6 (by weight) having the given instructions:

1. Cementitious material (cement, lime, gypsum) + sand 1 (yellow) + aluminum powder + water
2. Cementitious material (cement, lime, gypsum) + sand 2 (brown) + aluminum powder + water
3. Cementitious material (cement, lime, gypsum) + sand 3 (grey) + aluminum powder + water
4. Cementitious material (cement, lime, gypsum) + sand 1, 2, and 3 + aluminum powder + water
5. Normal concrete

Aerated concrete casting process can be described by Figure 3.3. Figure shows summarized illustration of information given in the literature review.

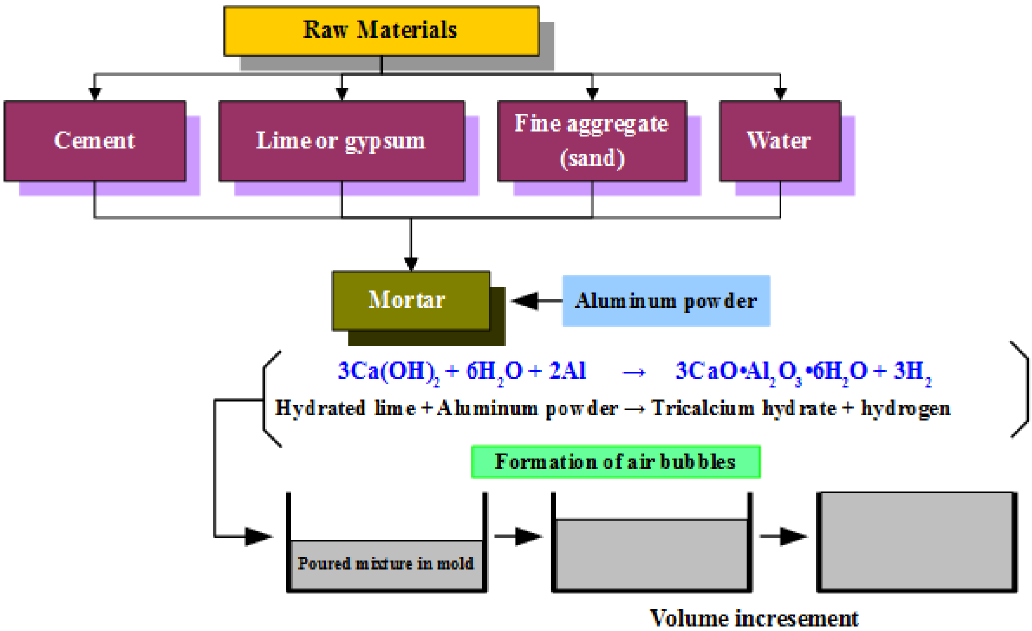


Figure 3.3. Process of aerated concrete casting

Following the literature review, mixture proportion with reference to Ecoton company was designed first for mix 1. The fresh concrete characteristics of this mix were not appropriate, therefore new mix design was developed. New mixture proportion (mix 0) with w/c = 0.58 is presented in Table 3.4. An assumption of 80% filling was made. Calculations were carried out for 4 cubic and 3 beam samples with dimensions

0.1x0.1x0.1 m^3 and 0.04x0.04x0.16 m^3, respectively for each mix. The batch was mixed in 3 stages due to mixer dimensions. The total batch amount was calculated considering 20% loss (eq 3.2.1)

$$V_b = 1.2 * (4 * (0.1)^3 + 3 * (0.04)^2 * 0.16), \qquad \text{eq (3.2.1)}$$

$$= 1.2 * 4.77 * 10^{-3} = 5.72 * 10^{-3}\ m^3$$

Table 3.4. Mixture proportions for yellow sand (w/c = 0.58) (mix 0)

Ingredients	Percentages	Volume, m^3	Mass, kg	80% filling	1/3 of 80%	Moisture adjusted, kg
Sand	42,00%	2,40E-03	6,753	5,402	1,801	1,807
PC	10,35%	5,92E-04	1,859	1,488	0,496	0,496
Lime	10,35%	5,92E-04	1,303	1,042	0,347	0,347
Gypsum	2,30%	1,32E-04	0,305	0,244	0,081	0,081
Water	35,00%	2,00E-03	2,003	1,602	0,534	0,515
Aluminum powder	0,06%	3,43E-06	0,009	0,007	0,002	0,002

The mix following Table 3.4 proportions had low workability, thus the next mix (1) was designed with higher w/c (=0.69). This mix design was further developed for other mixes, as the properties of fresh concrete were satisfactory. Tables 3.5-3.9 illustrate mixture proportions for mixes 1-5.

Table 3.5 Mixture proportions for yellow sand (w/c = 0.69) (mix 1)

Ingredients	Percentages	Volume, m^3	Mass, kg	80% filling	1/3 of 80%	Moisture adjusted, kg
Sand	42,00%	2,40E-03	6,753	5,402	1,801	1,807
PC	9,23%	5,28E-04	1,657	1,326	0,442	0,442
Lime	9,23%	5,28E-04	1,161	0,929	0,310	0,310
Gypsum	2,05%	1,17E-04	0,272	0,218	0,073	0,073
Water	37,50%	2,15E-03	2,146	1,716	0,572	0,553
Aluminum powder	0,06%	3,43E-06	0,009	0,007	0,002	0,002

Aerated concrete mixing was conducted following the sequence illustrated by Figure 3.4 using mortar mixing machine. Firstly, water and cement were mixed, then the lime was added and mixed (both stages were carried out at slow speed for 30 s). Next steps were also mixed at slow speed for 30 s, by adding gypsum and cement, then sand. At the end overall mix was blended for 1.5 min with 1 min pause at medium speed.

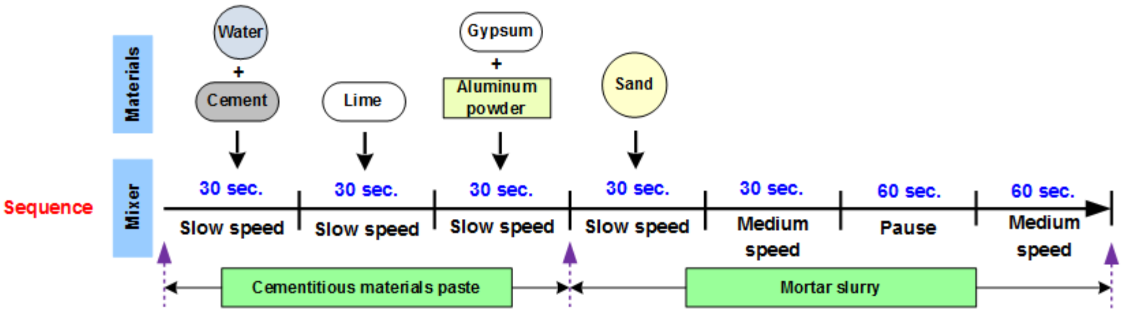


Figure 3.4. Mixing procedure

Further experiments were conducted for 9 cubic and 6 beam specimens, to get more accurate results. Final mixture proportions are given in the Appendix A.

$$V_b = 1.2 * (9 * (0.1)^3 + 6 * (0.04)^2 * 0.16) = 12.64 * 10^{-3}\ m^3$$

3.3 Casted concrete properties

In order to identify the amount of air voids in the specimen, assumption of 80% filling was made first. Consequently, to investigate actual filling percentage, both fresh and hardened unit weights were recorded to calculate the resulted loss. Aerated concrete properties right after casting were recorded to Table 3.6, and hardened properties excluding and including lost material are shown by Table 3.7.

Table 3.6. Fresh concrete properties for mix 0 and 1

Sample Number	$W_{bucket+concrete}$	W_{bucket}	$W_{concrete}$	V_{bucket}	Unit weight	Unit weight (avg.)	Unit weight
	g	g	kg	m³	kg/m³	kg/m³	lb/ft³
1	9540,0	8200,0	1,340	0,001	1675,0	1727,1	104,6
2	9462,0	8110,0	1,352	0,001	1690,0		105,5
3	9623,0	8170,0	1,453	0,001	1816,3		113,4

Table 3.7. Hardened concrete properties for mix 0 and 1 (without lost material)

Mixure	Sample Number	$W_{bucket+concrete}$	W_{bucket}	$W_{concrete}$	V_{bucket}	Unit weight	Unit weight (avg.)	Unit weight
		g	g	kg	m³	kg/m³	kg/m³	lb/ft³
Mixture 1	1	9402,0	8155,0	1,2	0,001	1247,0	1206,7	77,8
	2	9194,0	8041,0	1,2	0,001	1153,0		72,0
	3	9487,0	8267,0	1,2	0,001	1220,0		76,2
Mixture 2	1	9196,0	8200,0	1,0	0,001	996,0	993,7	62,2
	2	9106,0	8110,0	1,0	0,001	996,0		62,2
	3	9159,0	8170,0	1,0	0,001	989,0		61,7

The result show that average amount of material lost is 270 g, consequently the volume of sample considering loss is, $V = \frac{1264*0.001}{994} = 1.272 * 10^{-3}\ m^3$. The percentage lost is $\frac{1.272-1}{1} * 100\% = 27.2\%$. Therefore, filling percentage for the next mixes could be taken as 80% - 20% = 60% (assuming errors).

In order to avoid errors during strength test, the samples were dried at 40°C for 2 hours before conducting strength tests. The Figures 3.5-3.8 illustrate summaries of strength tests. It can be observed that both compressive and flexural strengths of the normal concrete are much higher than of aerated concrete. Among aerated concrete samples, mix 2 has the highest compressive strength, while mix 1 has the lowest. However, from flexural strength comparison mix 3 has the highest value, and mix 1 has the lowest.

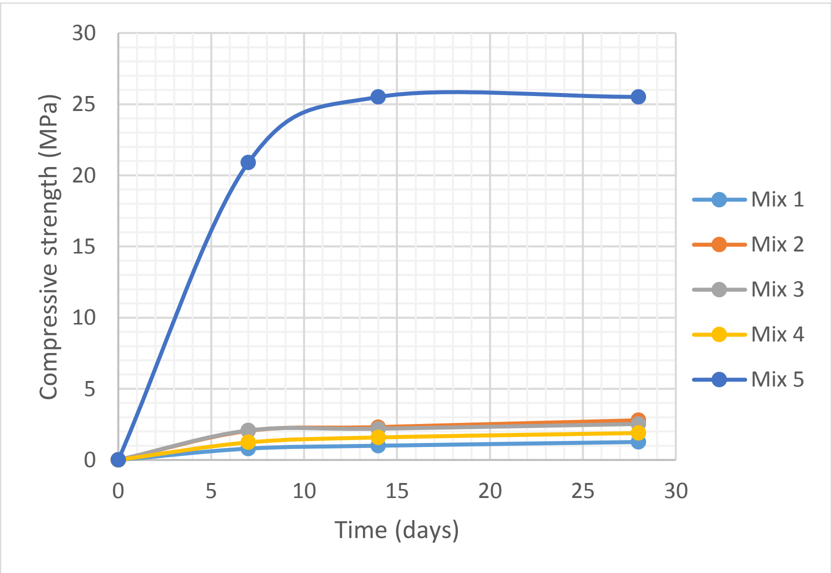


Figure 3.5 Comparison of compressive strengths of all 5 mixes

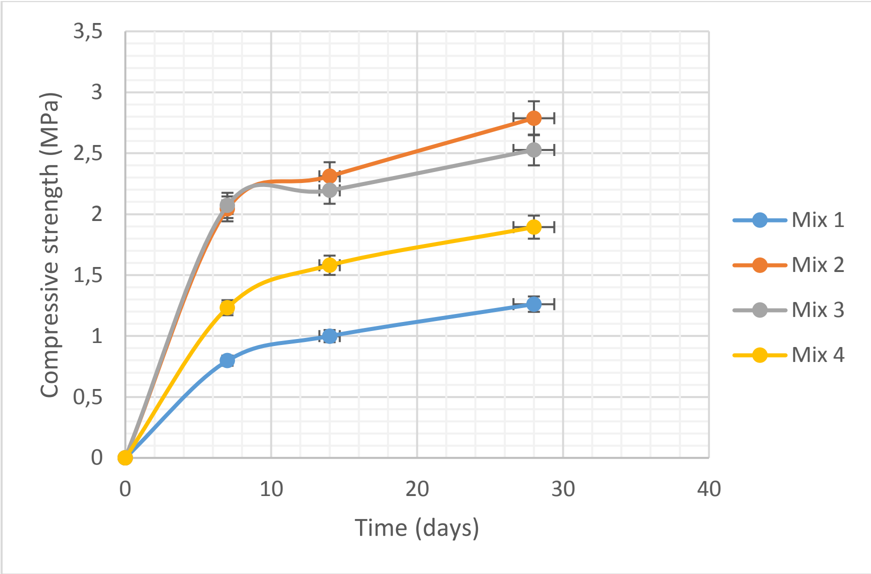


Figure 3.6 Comparison of compressive strengths of aerated concrete mixes

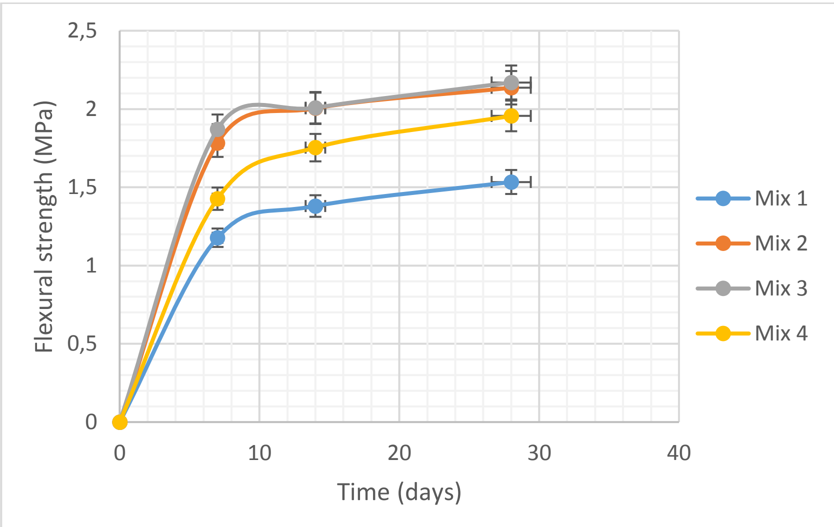


Figure 3.7 Comparison of flexural strengths of all 5 mixes

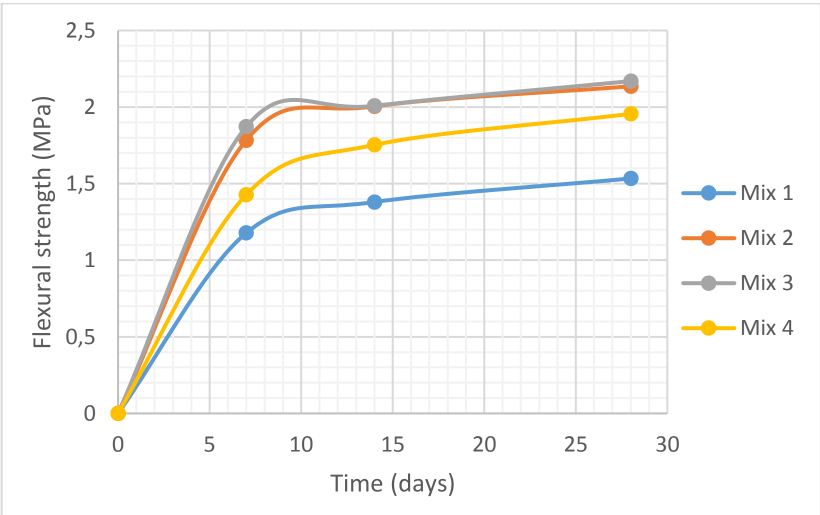


Figure 3.8 Comparison of flexural strengths of aerated concrete mixes

3.4 Thermal conductivity tests

To conduct thermal conductivity tests, samples with 150x150x30 mm dimensions were casted to fit into device (Figure 3.9). The results of test are given in Table 3.8. It also shows densities of samples, and their change with time. Figures 3.10-3.11 represent comparison of thermal conductivities for mixes. It can be observed that thermal conductivity of normal concrete is much higher comparing to aerated concrete mixes. Between aircretes, mix 4 illustrates the lowest thermal conductivity.

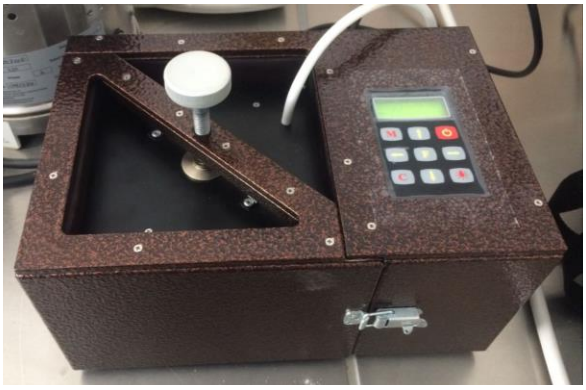


Figure 3.9 Thermal conductivity measuring device

Table 3.8 Thermal conductivity and density of mixes

Mix	Sample	Density (g/cm³)			Thermal Conductivity (W/mK)	
		7days	14days	28days	7	14
1	1	1,262	1,284	1,359	0,3172	0,3359
	2	1,275	1,269	1,279	0,3201	0,3375
	3	1,254	1,277	1,332		
	average	1,264	1,277	1,323	0,3187	0,3367
2	1	1,527	1,612	1,623	0,2304	0,3743
	2	1,616	1,628	1,653	0,3560	0,2667
	3	1,653	1,627	1,636		
	average	1,599	1,622	1,637	0,2932	0,3205
3	1	1,456	1,547	1,510	0,2587	0,2764
	2	1,541	1,597	1,574	0,3007	0,3084
	3	1,567	1,460	1,538		
	average	1,521	1,535	1,541	0,2797	0,2924
4	1	1,482	1,481	1,436	0,2020	0,2200
	2	1,459	1,438	1,414	0,1600	0,1733
	3	1,447	1,473	1,442		
	average	1,463	1,464	1,431	0,1810	0,1967
5	1	2,325	2,292	2,296	1,500	1,500
	2	2,351	2,301	2,295	1,500	1,500
	3	2,298	2,305	2,169		
	average	2,325	2,299	2,253	1,500	1,500

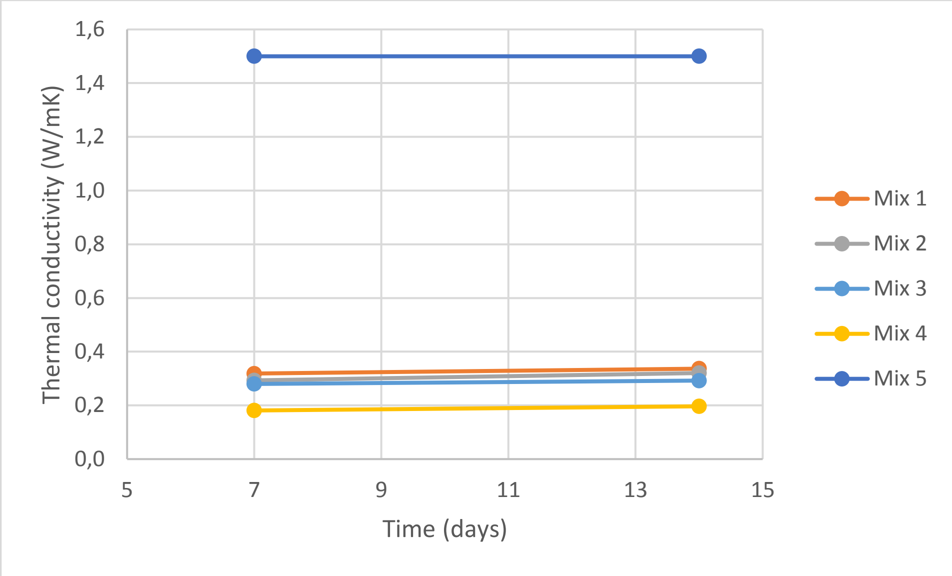


Figure 3.10 Comparison of thermal conductivities of all 5 mixes

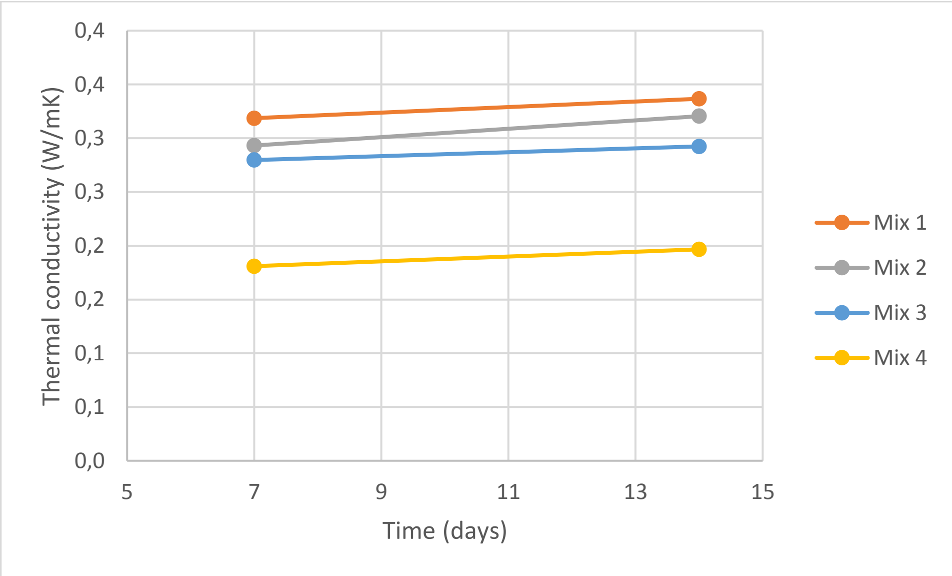


Figure 3.11 Comparison of thermal conductivities of aerated concrete mixes

Figure 3.12 shows the relationship among compressive strength, density, and porosity of aerated concrete (AC) mixture cured for 28 days. In general, compressive strength, density, and porosity are closely related to each other. With the increase in the density of aerated concrete, the compressive strength increases. As the porosity of the aerated concrete increases, the compressive strength decreases. For example, the strength of mixture 2 with the porosity of 47.0 % was 2786.8 kPa, whereas that of mixture 4 with 59.1% porosity was 1893.5 kPa.

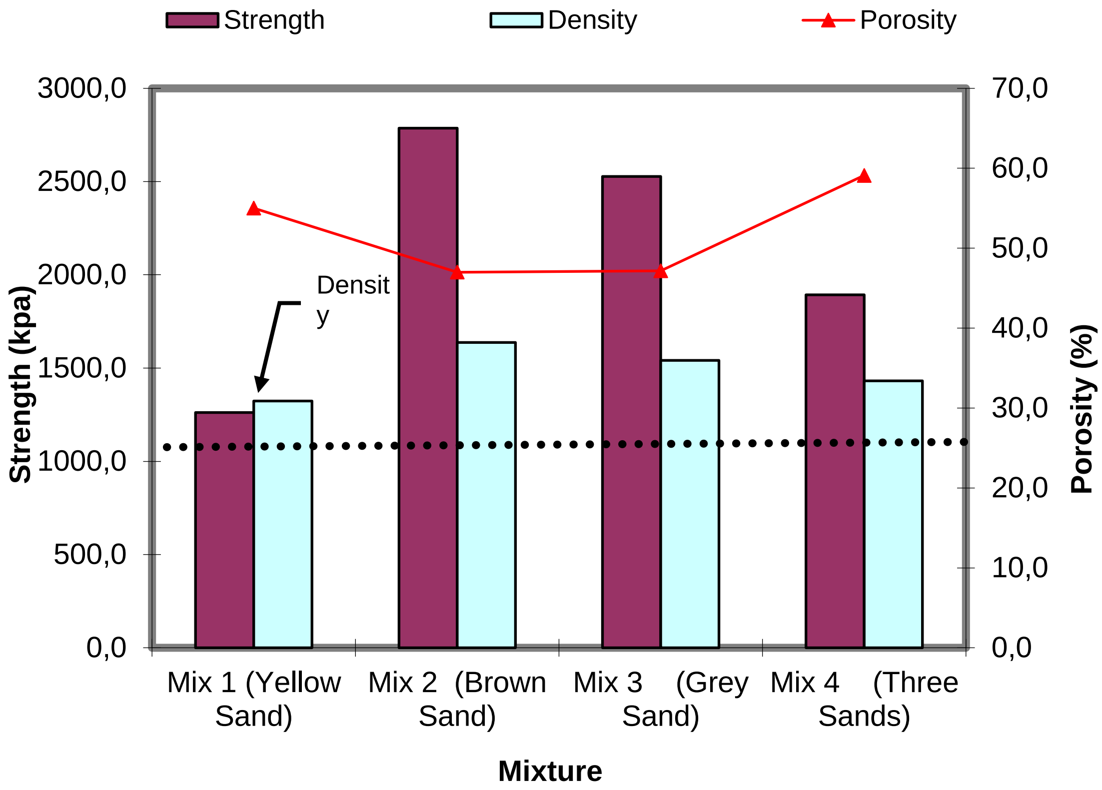


Figure 3.12 Relationship between Strength, Density, and Porosity

Figure 3.13 illustrate the relationship between porosity and thermal conductivity of mixes. It can be observed that for mixes 2-4, value of thermal conductivity decrease, as the porosity increases. However, thermal conductivity of mix 1 is the highest, despite the fact that it has the second highest porosity in the mixture. This result may be contributed to large size of pores and open-connectivity of pores.

There are many factors to influence thermal conductivity of concrete. These includes size, connectivity, and shape of pores. Machrafi (2015) reported that thermal conductivity is considerably reduced when pore size passes from macro- to nano-pores. Bhattacharjee (2004) also reported that concrete which has open pore cells has higher thermal conductivity than that of enclosed pore because open cell concrete has more chance to be saturated.

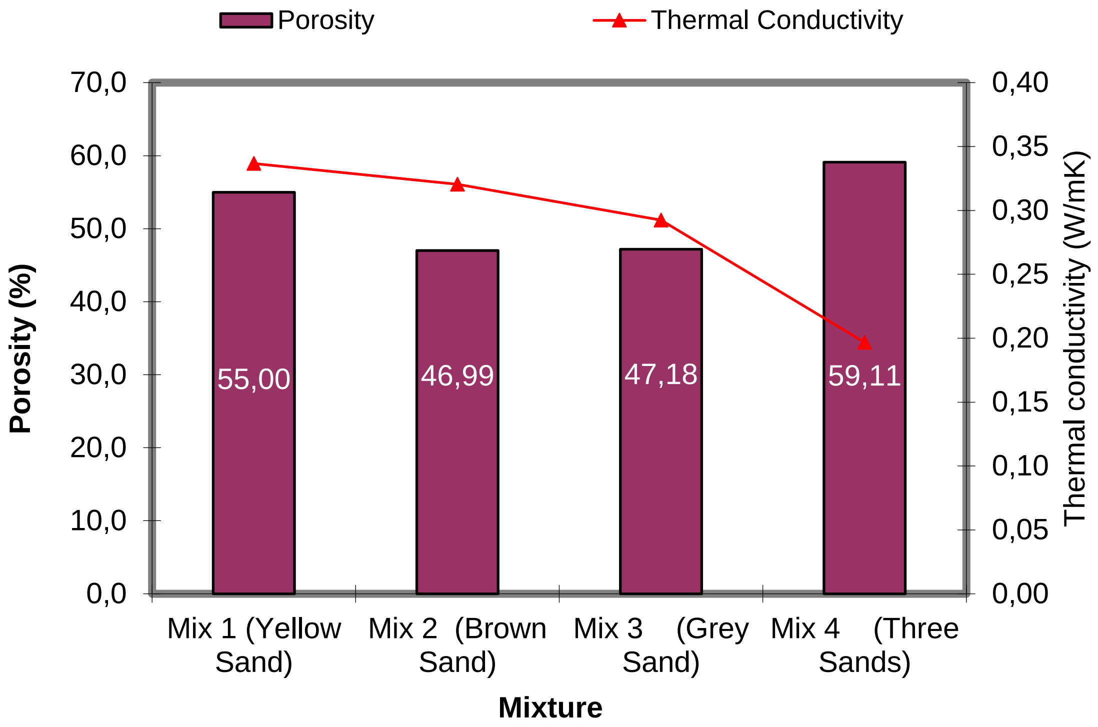


Figure 3.13 Porosity vs Thermal conductivity

4 Evaluation of Building Energy Performance (Case Study)

4.1 Energy Plus simulation

Analysis steps

Analysis is performed using the Energy Plus software as a simulation engine, OpenStudio as a user interface where all inputs are defined. Meanwhile, SketchUp is used as a drawing tool to create building's geometry. Mainly, performance of concrete walls of 5 different mixes will be evaluated and compared.

Case #1

- Creating building's geometry

The simulation of the whole building was performed 5 times for each mix. One simulation took around 6 hours. Figure 4.1 represents the building geometry created using Open Studio Plug In in SketchUp.

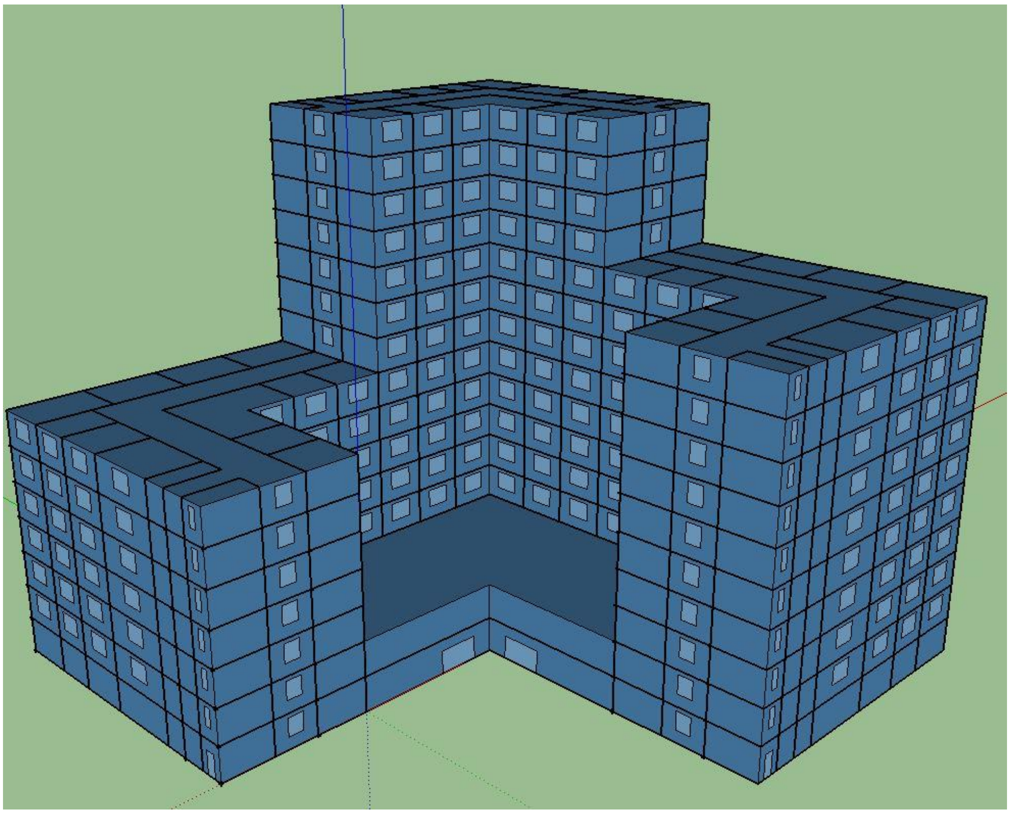


Figure 4.1 Building Geometry

Typical residence floorplan is shown in Figure 4.2. Name and area of each space is tabulated in Table 4.1. For more detailed dimensions please refer to technical drawings.

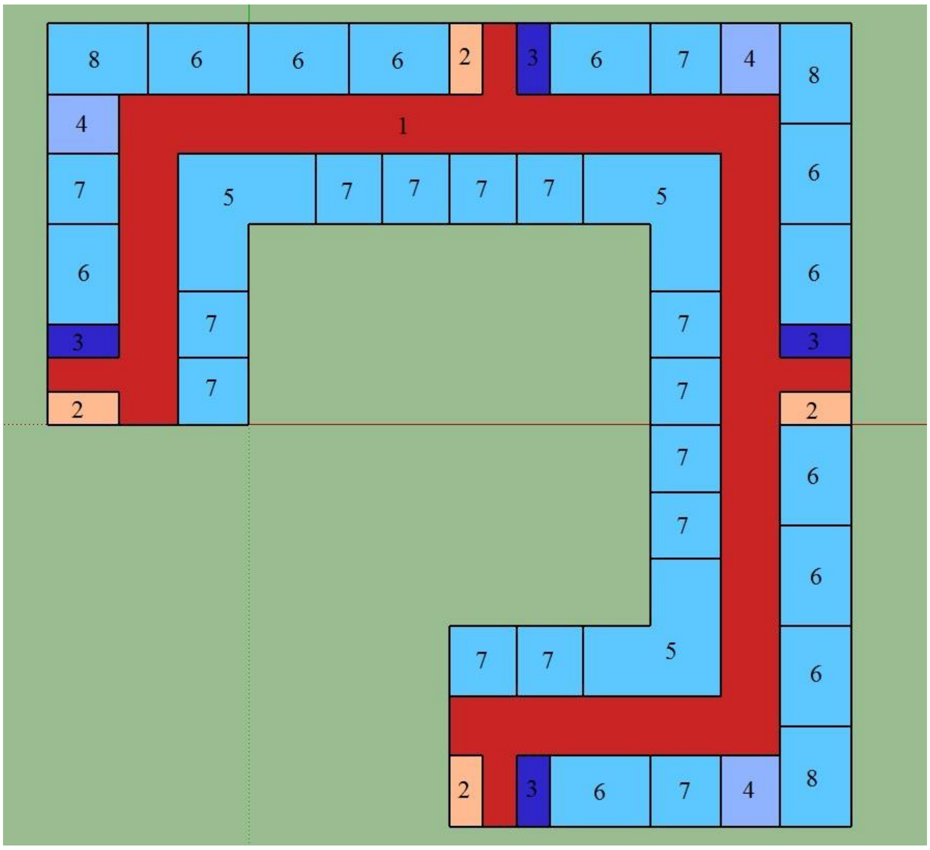


Figure 4.2 Building Space Diagram

Table 4.1. Spaces designation

#	Space	Area [m^2]	Conditioned [Y/N]
1	Corridor	663	yes
2	Stairs	13	no
3	Technical Room	13	yes
4	Laundry	23	yes
5	Family Room	80	yes

6	Double Room	40	yes
7	Single Room	26	yes
8	Kitchen	28	yes

Table 4.2. Window Dimensions

Window type	Length [m]	Height [m]
1 (all rooms)	2.31	1.8
2 (corridor)	1.55	1.8
3 (stairs)	0.72	1.8

- Open Studio inputs

Weather data:

The first step is to choose weather data from the database on the Energy Plus website. The only weather data available from Kazakhstan is SEMIPALATINSK weather data. Since the climate of Astana and Semipalatinsk with warm summers and very cold winters are quite similar, there is no problem in choosing this weather data file.

In SEMIPALATINSK weather data file summer design day is July 21st and winter design day is January 21st; maximum dry bulb temperature is 33 degrees Celsius, and minimum one is -32.3 degrees Celsius. Figure 4.3 represents average outdoor air dry bulb temperature in Semipalatinsk.

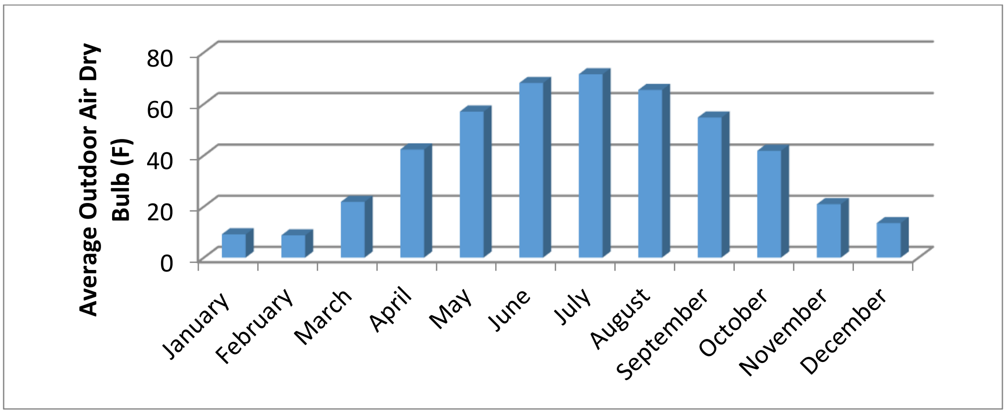


Figure 4.3 Average Outdoor Air Dry Bulb Temperature

Climate Zone:

According to ASHRAE Handbook Fundamentals 2009, annual heating degree days HDD65 in Astana is equal to 10291. Consequently, based on the information from Figure 4.4, it can be concluded that Astana is in the 7th Climate Zone.

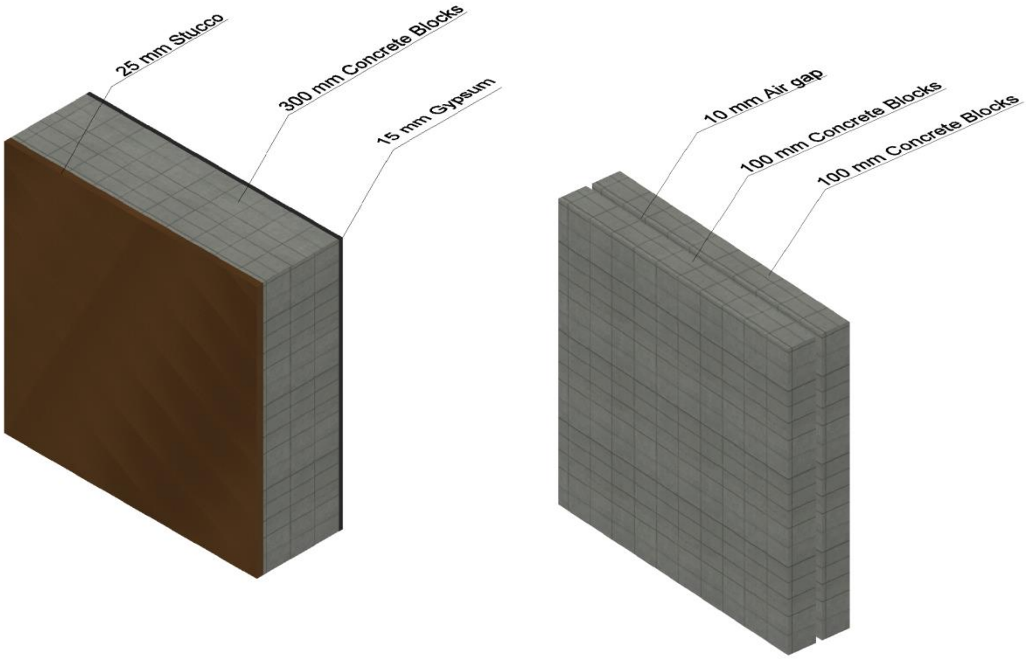
Zone Number	Zone Name	Thermal Criteria (I-P Units)	Thermal Criteria (SI Units)
1A and 1B	Very Hot –Humid (1A) Dry (1B)	9000 < CDD50°F	5000 < CDD10°C
2A and 2B	Hot-Humid (2A) Dry (2B)	6300 < CDD50°F ≤ 9000	3500 < CDD10°C ≤ 5000
3A and 3B	Warm – Humid (3A) Dry (3B)	4500 < CDD50°F ≤ 6300	2500 < CDD10°C < 3500
3C	Warm – Marine (3C)	CDD50°F ≤ 4500 AND HDD65°F ≤ 3600	CDD10°C ≤ 2500 AND HDD18°C ≤ 2000
4A and 4B	Mixed-Humid (4A) Dry (4B)	CDD50°F ≤ 4500 AND 3600 < HDD65°F ≤ 5400	CDD10°C ≤ 2500 AND HDD18°C ≤ 3000
4C	Mixed – Marine (4C)	3600 < HDD65°F ≤ 5400	2000 < HDD18°C ≤ 3000
5A, 5B, and 5C	Cool-Humid (5A) Dry (5B) Marine (5C)	5400 < HDD65°F ≤ 7200	3000 < HDD18°C ≤ 4000
6A and 6B	Cold – Humid (6A) Dry (6B)	7200 < HDD65°F ≤ 9000	4000 < HDD18°C ≤ 5000
7	Very Cold	9000 < HDD65°F ≤ 12600	5000 < HDD18°C ≤ 7000
8	Subarctic	12600 < HDD65°F	7000 < HDD18°C

Figure 4.4 International Climate Zone Definitions (American Society of Heating, Refrigerating, and Air-Conditioning Engineers)

Constructions:

This step includes defining each construction such as walls, floors, ceilings and roof. Based on the Climate Zone, OpenStudio proposes default layers of each construction. All the values were set to default as for Midrise Apartment Construction Set with only exterior and interior walls material changing, so different mixes can be compared and analyzed.

As it is shown in Figure 4.5, Exterior wall consists of 15mm gypsum layer, 300mm concrete blocks, and 25mm stucco. While, interior wall has 10mm air gap between two layers of 100mm of concrete blocks.



a) b)

Figure 4.5 a) Exterior wall layers; b) Interior wall layers

Table 4.3 and 4.4 shows material properties used in simulation. Concrete properties used are obtained in laboratory, while properties of stucco and gypsum are taken from BCL library of Energy Plus.

Table 4.3 Concrete properties used in simulation

Mixture	**Density (kg/m3)**	**Thermal Conductivity (W/(m*K))**
Mix1 (yellow)	1277	0.3187
Mix2 (brown)	1622	0.2932
Mix3 (grey)	1535	0.2797
Mix4(3 sands)	1464	0.181
Mix5 (Normal concrete)	2299	1.5

Table 4.4 Wall layer material properties

Material	**Density (kg/m3)**	**Thermal Conductivity (W/(m*K))**	**Specific Heat (J/(kg*K))**
Stucco	1858	0.6918	837
Gypsum	784.9	0.16	830

Loads:

In this part there are plenty things to adjust such as number of people in building, lighting power, electric, gas and water use definitions. In people load the number of residents per floor was entered – 45. Other parameters were set as defaults as for midrise apartment in Open Studio database.

Schedules:

Since the main purpose of this energy analysis is focusing on heating and cooling loads, in the Schedules tab cooling and heating set points were set to 24 and 21 degrees Celsius respectively. Other definitions are set to default values.

Thermal Zones (HVAC):

All spaces are set to be air conditioned; therefore ideal air loads are turned on for each space.

Results:

Figure 4.6 illustrates heating loads per month for each case. Visually, it can be easily noticed that in case when aerated concrete is used heating loads are significantly lower in cold periods comparing to the case when normal concrete (mix #5) is used. Since winter design day is in January, it is reasonable to compare values of this month. Heating load of the whole building when normal concrete is used equals 1421.53 MBtu, whenever the same indicator for the lowest value (mix #4) is equal to 746.89 MBtu, which results in 47% efficiency of mix #4 comparing to mix #5 during January.

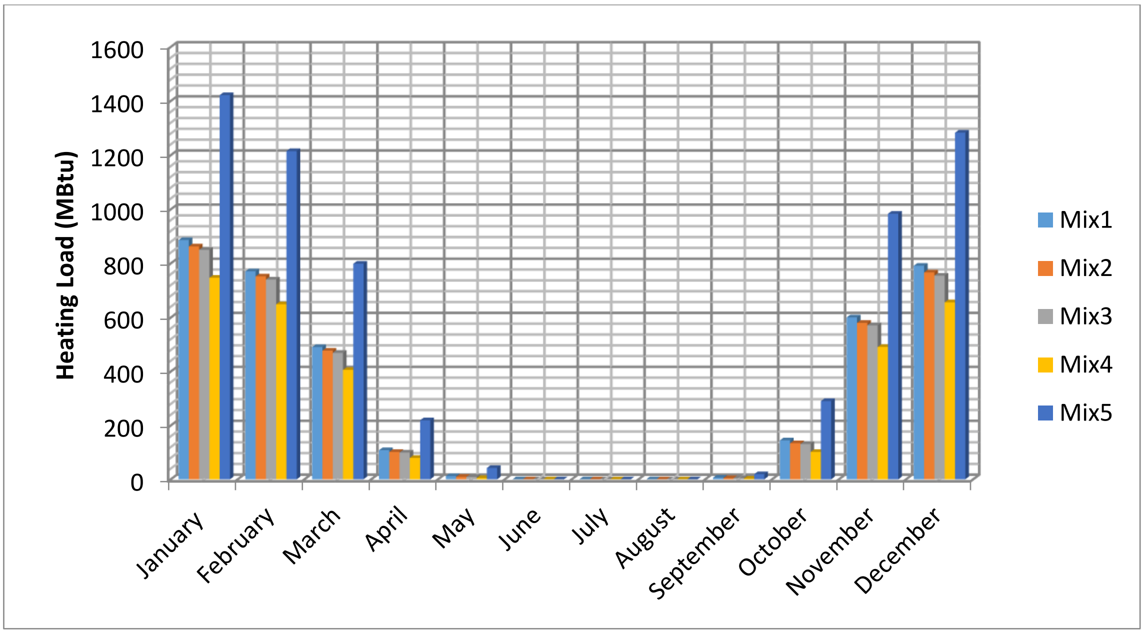


Figure 4.6 Heating load per each month

Figure 4.7 shows cooling loads per month for each case. Similarly, it can be seen that cooling loads are much higher when mix #5 is used. Since summer design day is in July, it is reasonable to compare values of this month. Cooling load of the whole building when normal concrete is used equals 470.27 MBtu, whenever the same indicator for the lowest value (mix #4) is equal to 372.84 MBtu, which results in 21% efficiency of mix #4 comparing to mix #5 during July.

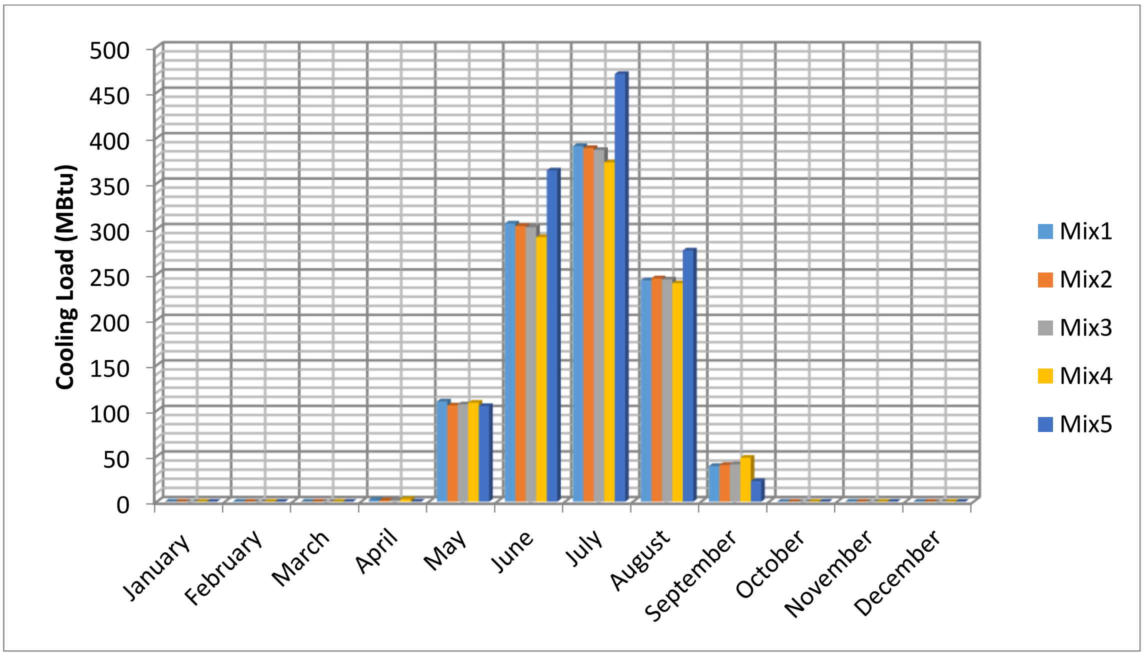


Figure 4.7 Cooling load per each month

Table 4.5. shows the summary of 5 simulations for each concrete mix. Detailed results of each case can be found in Appendix D. Since, total source energy to total site energy ratio is the same for all 5 cases, any of those values can be compared. It can be seen from the Figure 4.8 that total site energy is linearly proportional to thermal conductivity of a wall material. Analyzing total site energy results, using aerated concrete instead of normal concrete results in 23%, 24%, 25%, and 30% energy saving for 4 mixes respectively; this makes mix #4 the most suitable one.

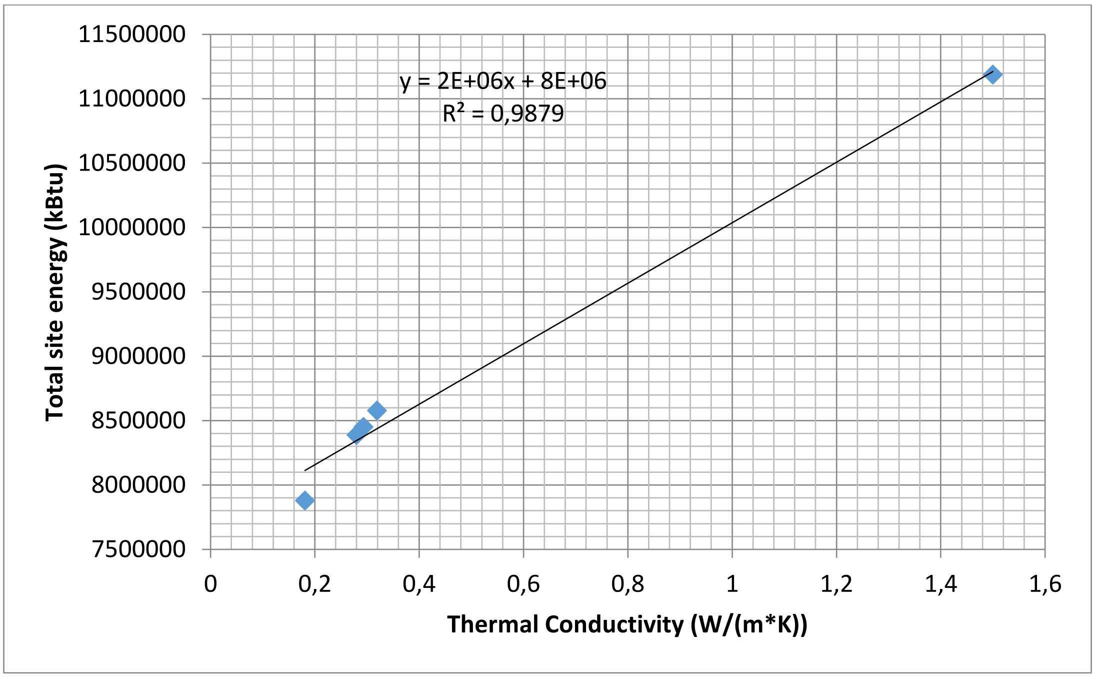


Figure 4.8 Relationship between Thermal Conductivity and Total Site Energy

Table 4.5 Summary of simulations

Mixture	Density (kg/m3)	Thermal Conductivity (W/(m*K))	Total site energy (kBtu)	Total site energy per total building area (kBtu/ft2)	Total source energy (kBtu)	Total site energy per total building area (kBtu/ft2)	Max heating load (Mbtu)	Max cooling load (Mbtu)
Mix1 (yellow)	1277.00	0.3187	8577820.8	35.8	26562204.6	110.9	886.08	390.91
Mix2 (brown)	1622.00	0.2932	8449771.1	35.3	26115679	109	862.6	388.8
Mix3 (grey)	1535.00	0.2797	8388219.5	35	25899576.7	108.1	849.97	386.65
Mix4 (3sands)	1464.00	0.181	7878388.3	32.9	24105377.4	100.6	746.89	372.84
Mix5 (NC)	2299.00	1.5	11187141.9	46.7	35614539.3	148.7	1421.53	470.27

Site energy stands for the energy amount brought into the building to maintain desired conditions. In simple words, it is the energy amount shown on a utility bill. While source energy is the amount of energy consumed to produce and transport the energy to the building.

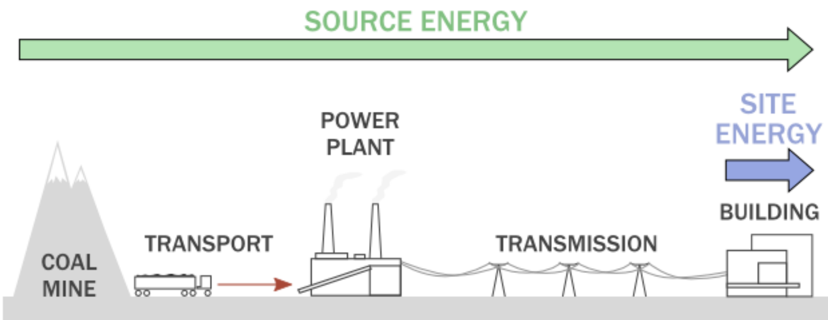


Figure 4.9 Schematic definition of site and source energy

4.2 Heat Loss Calculation

The way to prove the efficiency of using aerated concrete by hand calculation is obtaining heat loss through the walls of the building. Considering the same building geometry analyzed above, heat loss per degree day and heat loss for entire heating season will be compared between all 5 cases. Since, this calculation is very simplified and walls of the whole building are made of the same material, calculations for just only one floor can be made.

According to ASHRAE Handbook Fundamentals 2009, annual heating degree days HDD65 in Astana is equal to 10291.

$$A_{total} = Floor\ Height * Perimeter\ of\ the\ Building - Area\ ofFenestration$$

$$A_{total} = 3.4m * 300m - 181.25m^2 = 838.75\ m^2 = 9020.16ft^2$$

l, Thickness of #1 mix concrete layer = 300mm = 11.811 in
λ, Thermal conductivity of mix #1 concrete = 0.3187 W/m*K = 2.212 Btu*in/hr*ft^2*F

Using the eq(2.5.2.):

$$R = \frac{11.811\ in}{2.212\ Btu * in/hr * ft^2 * F} = 5.340\ hr * ft^2 * F/Btu$$

Now, substituting all the values in to eq(2.5.1.):

$$Q = \frac{9020.16\ ft^2 * 1°F}{5.340\ hr * ft^2 * F/Btu} * 24\ {}^{hr}/_{day} = 40543.12\ \frac{Btu}{degree\ day}$$

To calculate the annual heat loss, the heat loss per degree day is multiplied by the annual degree days in Astana:

$$Q = 30486.56 \frac{Btu}{degree\ day} * 10291\ degree\ days = 417.23\ MMBtu$$

The same principle was used to calculate heat loss for other cases. Results are tabulated in Table 4.6.

Table 4.6 Summary of Heat Loss Calculation

Mixture	Thermal Conductivity (W/(m*K))	Thermal Conductivity (Btu*in/(hr*F*ft2))	Thermal Resistance (hr*ft2*F/Btu)	Heat Loss (Btu/degree day)	Annual Heat Loss (Mbtu)
Mix1 (yellow)	0.3187	2.212	5.340	40543.12	417.23
Mix2 (brown)	0.2932	2.035	5.804	37299.16	383.85
Mix3 (grey)	0.2797	1.941	6.084	35581.77	366.17
Mix4 (3 sands)	0.181	1.256	9.402	23025.74	236.96
Mix5 (NC)	1.5	10.410	1.134	190821.06	1963.74

Based on Figure 4.10, which shows Annual Heat Loss comparison, it can be concluded that using aerated concrete is much more efficient, since heat loss in case of normal concrete is 5-10 times higher.

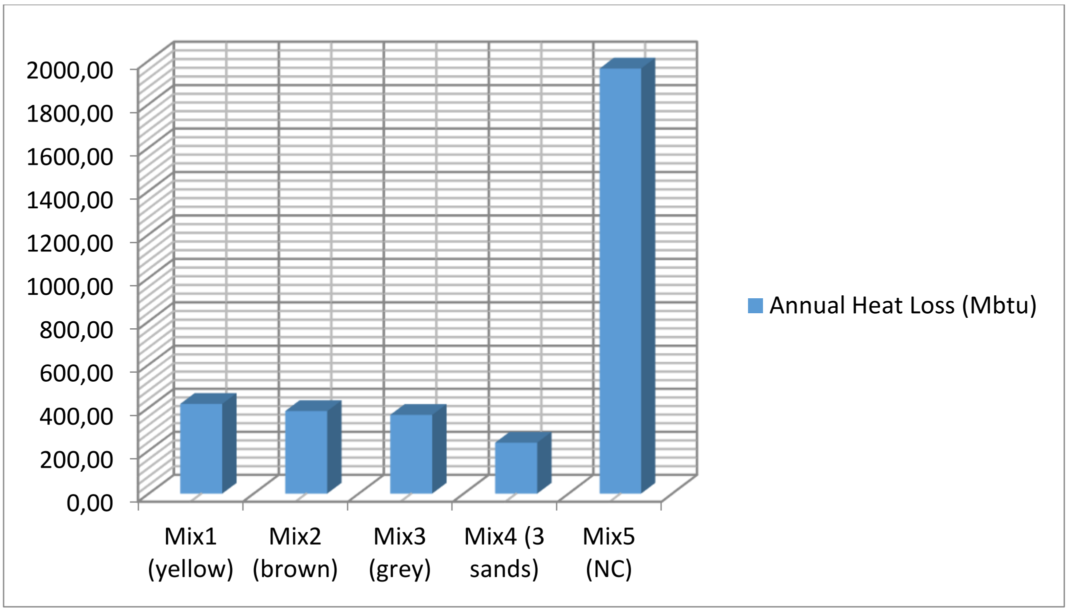


Figure 4.10 Annual Heat Loss Comparison

4.3 Heat Transfer of Construction Wall

Based on the theory, simple calculation is done for one wall, and the feasibility of choosing aerated concrete as the material for exterior walls is evaluated (Figure 4.11).

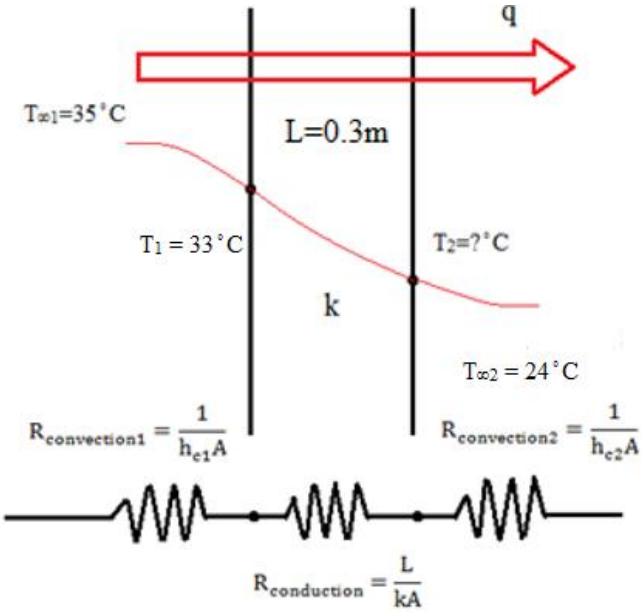


Figure 4.11 The overview of the study model

In this study, the side wall of one typical room is analyzed. All layers except concrete blocks are neglected. For easier calculation, fenestration is also ignored. The wall has dimensions of 5m length and 3.4m height with thickness of 0.3m. The outside temperature is taken as 35 ˚C and the air conditioner has to maintain inside room temperature of 24 ˚C. The outer surface temperature is assumed to be 33˚C. The wall materials are selected as normal concrete and aerated concrete with thermal conductivity values listed in Table 4.7.

Table 4.7 Thermal Conductivity values for wall material

Mixture	**Thermal Conductivity (W/(m*K))**
Mix1 (yellow)	0.3187
Mix2 (brown)	0.2932
Mix3 (grey)	0.2797
Mix4(3 sands)	0.181
Mix5 (NC)	1.5

The heat transferred by air conditioner for all cases is obtained as follows:

The heat transfer:

$$q_{tr} = h_{c1} * A * (T_{\infty 1} - T_1) = \frac{k*A}{L}(T_1 - T_2) = h_{c2} * A * (T_2 - T_{\infty 2})$$

Mean film temperature outside is:

$$T_{f1} = \frac{T_1 + T_{\infty 1}}{2} = \frac{33 + 35}{2} = 34^{\circ}C = 307K$$

According to Holman (1997), air properties at T=307K are:

$k_{306.5}$ = 0.0267706 W/m*K

$\nu_{306.5}$ = 16.3998*10^{-6} m^2/s

$\beta_{306.5}$ = 0.003257 1/K

$Pr_{306.5}$ = 0.70646

Therefore, Grashof number for the air outside the wall can be calculated:

$$Gr_1 = \frac{g * \beta * (T_{\infty 1} - T_1) * L^3}{\nu^2} = \frac{9.8 * 0.003257 * (35 - 33) * 5^3}{(16.3998 * 10^{-6})^2} = 2.9672 * 10^{10}$$

Consequently, Rayleigh number can be calculated:

$$\mathrm{Ra}_1 = \mathrm{Gr}_1 * \mathrm{Pr}_1 = 2.9672 * 10^{10} * 0.70657 = 2.09622 * 10^{10}$$

Since, $10^9 < \mathrm{Ra}_1 < 10^{12}$,

$$\overline{Nu_1} = C * (Ra_1)^a = 0.13 * (2.09622 * 10^{10})^{1/3} = 358.445$$

The convective heat transfer coefficient is calculated as follows:

$$\overline{h_{c1}} = \frac{Nu_1 * k_a}{L} = \frac{358.445 * 0.0267706}{5} = 1.919 \frac{W}{m^2 * K}$$

Finally, the heat transferred to the wall through convection:

$$q_{tr} = \overline{h_{c1}} * A(T_{\infty 1} - T_1) = 1.919 * 5 * 3.4 * (35 - 33) = 65.25\, W$$

Mix #1

The conductive heat transfer the wall is:

$$q_{tr} = k * A * \frac{(T_1 - T_2)}{L}$$

Therefore,

$$T_2 = T_1 - \frac{q_{tr} * L}{k * A} = 32 - \frac{65.25 * 0.3}{0.3187 * 5 * 3} = 29.39^{\circ}C$$

$$T_{f2} = \frac{T_2 + T_{\infty 2}}{2} = \frac{29.39 + 24}{2} = 26.7^{\circ}C = 299.7K$$

According to Holman (1997), air properties at T=299.7K are:

$K_{299.7}$ = 0.026217 W/m*K

$\nu_{299.7}$ = 15.6589*10^{-6} m^2/s

$\beta_{299.7}$ = 0.003337 1/K

$Pr_{299.7}$= 0.70807

Therefore, Grashof number for the air inside the wall can be calculated:

$$Gr_2 = \frac{g * \beta * (T_2 - T_{\infty 2}) * L^3}{\nu^2} = \frac{9.8 * 0.003337 * (29.39 - 24) * 3.0^3}{(15.6589 * 10^{-6})^2} = 8.98 * 10^{10}$$

Consequently, Rayleigh number can be calculated:

$$Ra_2 = Gr_2 * Pr_2 = 8.98 * 10^{10} * 0.70807 = 6.358 * 10^{10}$$

Since, $10^9 < \mathrm{Ra}_2 < 10^{12}$,

$$\overline{Nu_2} = C * (Ra_2)^a = 0.13 * (6.358 * 10^{10})^{1/3} = 518.87$$

The convective heat transfer coefficient is calculated as follows:

$$\overline{h_{c2}} = \frac{Nu_2 * k_a}{L} = \frac{518.87 * 0.026217}{3} = 2.721 \frac{W}{m^2 * K}$$

The heat transferred inside the wall:

$$q_{conv2} = \overline{h_{c2}} * A(T_2 - T_{\infty 2}) = 2.721 * 5 * 3.4 * (29.39 - 24) = 249.15\, W$$

In order to maintain the net heat flow inside the wall same as heat flow transferred to the wall from outside, the air conditioner should transfer:

$$\Delta q = q_{conv2} - q_{tr} = 249.15 - 65.25 = 183.9\,W$$

Following the calculations for mix #1 calculation procedure, only changing thermal conductivity of the wall material, heat transfer by air conditioner was estimated for all 5 mix concrete blocks using Excel Spreadsheet. The detailed table can be found in Appendix D.

Table 4.8 summarizes the heat transfer calculations. It can be seen that using mix #4 with the lowest thermal conductivity value is the most efficient option, which has heat transferred through the wall nearly 12 times less than the wall with normal concrete.

Table 4.8 Heat Transfer Calculation Results

Composition	**Thermal conductivity* (W/(m*K))**	**Δq (W)**
Mix #1 (yellow)	0.3187	183.9
Mix #2 (brown)	0.2932	164.73
Mix #3 (grey)	0.2797	153.47
Mix #4 (3sands)	0.181	31.28
Mix #5 (NC)	1.5	371.68

Figures 4.12 and 4.13 illustrate the heat transfer process in mix #4 concrete wall and normal concrete wall.

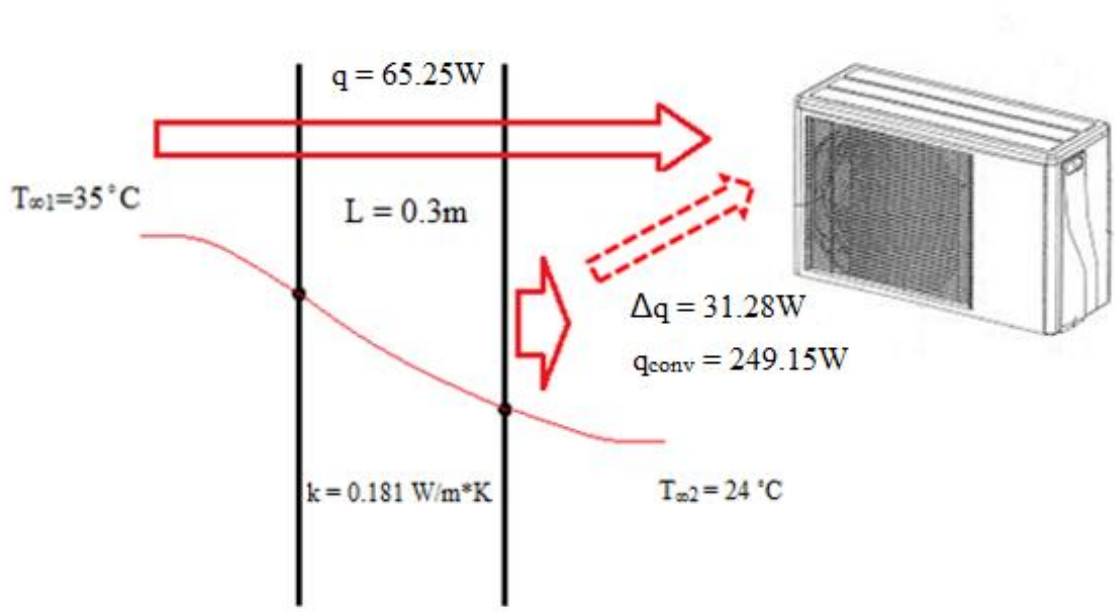


Figure 4.12 Heat Transfer of Mix #4 concrete wall

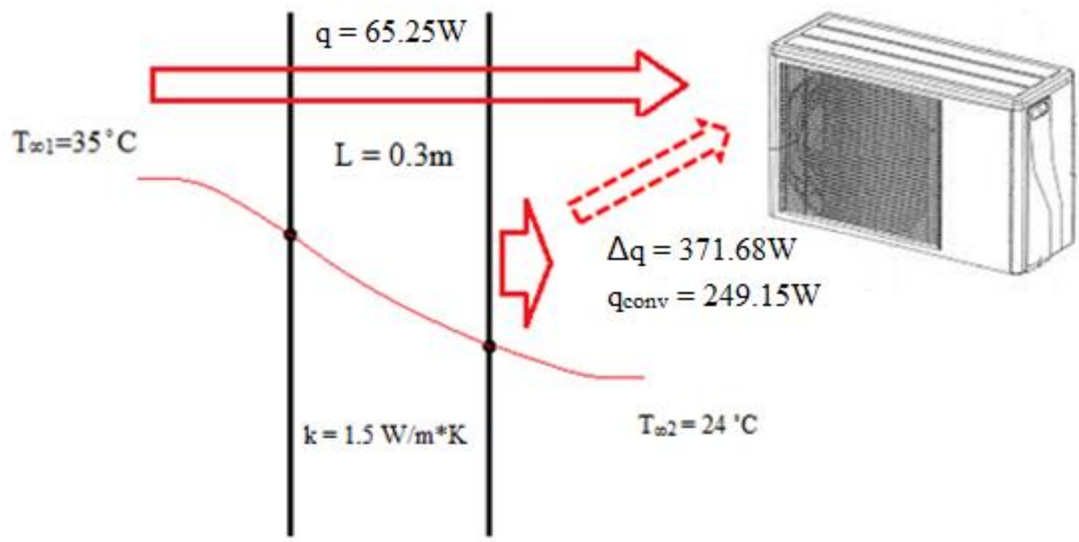


Figure 4.13 Heat Transfer of Mix #5 normal concrete wall

To maintain room temperature at 24 ˚C, the air conditioner has to remove 31.28 W of heat through 5m X 3.4m mix #4 concrete wall. While, when mix #5 normal concrete is used as a wall material, the insulation is not good, and the heat transferred by air conditioner equals 371.68W.

5 Preliminary Structural Member Design

Structural design includes preliminary design of the building members such as slabs, beams, and columns. The design procedure is in accordance with the instructions provided previously in section 2.3. and Eurocodes.

5.1 Design of Slab and Beam

Preliminary design of the slab and beam are depended on the span length which is also the distance between the columns. Following includes the slab and beam dimension calculations for the longest span, L, existing in the building, 5300 mm.

5.1.1 Slab Thickness

According to Eurocode slab thickness is equal to the $\frac{L/2}{20}$.

Slab thickness = $\frac{5300/2}{20}$ = 132.5 mm which can be approximated to 150 mm.

5.1.2 Beam Section

According to steps Eurocode provided:

Depth = 8% of L +cover = 0.08*5300 +50 = 474 mm which can be approximated to 450 mm.

Width = 0.4~0.6 of beam depth = 0.6*450 = 270 mm which can be approximated to 300 mm.

5.1.3 Column Section

In purpose of performing further calculation column section was decided to be 600 x 600 mm. This number can be changed depending on its bearing capacity after detailed structural analysis of the structure in SAP2000 software.

5.2 Load Calculation

In order to obtain specific dimensions of the building members it is needed to know the internal forces occurring in them because of the external loadings. Therefore, calculation of values of loadings acting on the building is essential. In the preliminary design of the proposed building only vertical loads were considered. To ease the calculation procedure, the building was separated to the sections provided in Figure 5.1 below.

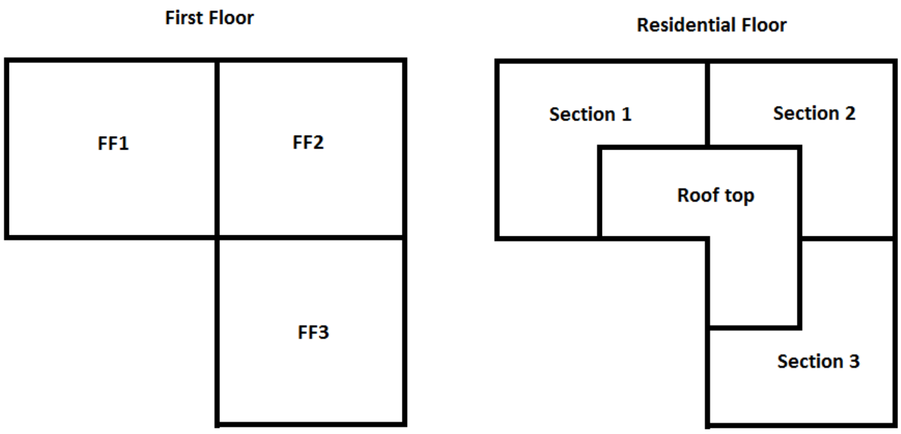


Figure 5.1. Building Sections

5.2.1 Dead Load Calculation

Dead load of the building is calculated by using the unit weights of particular materials from Tables 2.4, 2.5 and 2.6. The values also demonstrated in Table 5.1. Table 5.1 below presents dead load calculation for the whole structure.

Table 5.1. Dead Load Calculation

	Material	Unit weight, N/m^2	Unit weight, N/m^3	Area, m^2	Height, m	Number	Weight, N
Floor/ceiling	Insulated laminate flooring	73.61		2781		33	6755410.53
	Self-leveling floor	36.96		2781		33	3391930.08
	Cement/sand screed	981		2781		33	90029313
	EPS Geofoam	16.67		2781		33	1529855.91
Column	RC		25	1178,865 m^3			29 471.625
Beam	RC		25	1330,965 m^3			33 274,125
Slab	RC		25	3746,25 m^3			93 656,25
Interior walls	AC	490.5		28 605,79			14 031 140

Exterior walls	AC	1471.5		16 247,83			23 908 679,74
Facade	Limestone	1255.7		6450			8099265
Mineral wool		88.3		6450			569535
Total							148 471 531.2

The total volume of RC for beams, slabs and columns from the table are calculated by hand and illustrated in Appendix E. Furthermore, the volume of AC for interior and exterior walls, limestone for façade and mineral wools are received from AutoCad. The table indicates that the total Dead Load of the building is 148 471kN**.**

5.2.2 Live Load Calculation

Live load calculation was performed by following stages:

1. First Floor
2. Sections 1 and 3
3. Section 2

5.2.2.1 First Floor

Live load calculation for first floor presented in Tables 5.2-5.4 below.

Table 5.2. Live load for FF1

	Area, m^2	Load per unit area, kN/m^2	Load, kN
Market	102	4.5	459
Hair shop	52	3.5	182
Service room	26.5	2.5	66.25
Technical room	13.54	2.5	33.85
Stairs	13.54	3	40.62
Lift	11.9	3	35.7
WC	79.45	4	317.8
Corridor	601.07	4	2404.28

Table 5.3. Live load for FF2

	Area, m^2	Load per unit area, kN/m^2	Load. kN
Canteen	94.09	3.5	329.315
Technical room	27.08	2.5	67.7
Security room	55	1.75	96.25
Security station	25	2	50
Food court	357.41	2.5	893.525
Corridor	290.49	4	1161.96
Stairs	27.08	3	81.24
Lift	23.85	3	71.55

Table 5.4. Live load for FF3

	Area, m^2	Load per unit area, kN/m^2	Load, kN
Medical room	64	3.5	224
Service room	26.5	2.5	66.25
Technical room	13.54	2.5	33.85
Multifunctional room	70	3.5	245
Office	40	3.5	140
Pharmacy	34	2.5	85
Stairs	13.54	3	40.62
Corridor	510.57	4	2042.28
Fire control room	36.5	2	73
Lift	11.9	3	35.7
WC	79.45	4	317.8

5.2.2.2 Sections 1 and 3

Table 5.5. Live load for Sections 1 and 3

Occupancy or Use	Area, m^2	Live Load per area, kN/m^2	Live Load, kN
Room	393.925	1.5	590.8875
Kitchen	28.9	2	57.8
Stair	13.54	1	13.54
Corridor	207.8	4	831.2
Laundry	23.32	4	93.28

5.2.2.3 Section 2

Table 5.6. Live load for Section 2

Occupancy or Use	Area, m^2	Live Load per area, kN/m^2	Live Load, kN
Room	393.925	1.5	590.8875
Kitchen	28.9	2	57.8
Stair	13.54	2	27.08
Corridor	207.8	4	831.2
Laundry	23.32	4	93.28

5.2.3 Snow Load Calculation

Section 2

Snow load was calculated according to the EN.1991.1.3.2003. For the Block A, equation 2.3.1 was used for the transient design situation.

C_e = 0.8 (Table 2.17), because Astana has windswept topography.

μ_i = 0.8 (Table 2.18), as angle of pitch of roof is less than 30^o.

C_t = 1, as roof has low thermal transmittance.

S_k = 1.8 kPa, as Astana is included in the region #3 by the weight of snow cover (Prof.Chinwi Report, section 2), and from SNiP RK 2.01.07-85 Load and Impact, characteristic value of snow on the ground for Astana city was taken.

By solving equation 2.3.1, value for snow load on the roof of Block A was obtained.

$$S = 0.8 * 0.8 * 1 * 1.8 = 1.152\ kN/m^2$$

Sections 1, 3, and rooftop

For Blocks B, C and D equations 2.3.5 and 2.3.6 were used.

μ_1 = 0.8, assuming the angle of the pitch of the second roof is less than 30^o

$\mu_2 = \mu_s + \mu_w$

Where:

$\mu_s = 0$, for $\alpha \leq 15^o$ and,

$\mu_w = (b_1 + b_2)/2h \leq \gamma h/S_k$

where:

b_1 – width of the taller tower

b_2 – width of the lower tower

h – height between taller and lower towers

Section 3,

b_1 = 30.6 m

b_2 =30.6 m

h = 14.9 m

μ_w = (30.6 + 30.6)/(2*14.9) = 2.05

Now by using equation 2.3.1

$$S = 2.05 * 0.8 * 1 * 1.8 = 2.952\ kN/m^2$$

Section 1

b_1 = 30.6 m

b_2 =30.6 m

h = 25.1 m

μ_w = (30.6 + 30.6)/(2*25.1) = 1.22

Now by using equation 2.3.1

$$S = 1.22 * 0.8 * 1 * 1.8 = 1.755\ kN/m^2$$

Rooftop,

b_1 = 15.6 m

b_2 =15 m

h = 42.1 m

μ_w = (15.6 + 15)/(2*42.1) = 0.363

Now by using equation 2.3.1

$$S = 0.363 * 0.8 * 1 * 1.8 = 0.523\ kN/m^2$$

5.2.4 Wind Load Calculation

According to the report on engineering and geological survey done by LLP "Karagandy GIIZ and Co*" (State License No. 001137 issued by the Committee on Construction) on 16.06.2015, for the chosen location wind are principally in south-west and north-east directions. Wind velocity possible to happen once in five years equals 31 m/s; once in ten years equals 33 m/s; once in hundred year equals 40 m/s. These values are further used in calculation of basic wind velocity (see Figure 5.2).

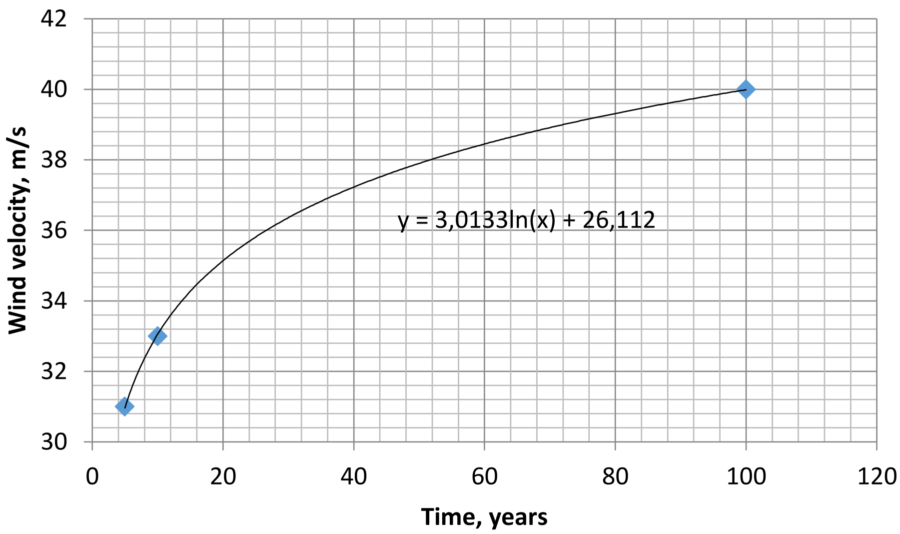


Figure 5.2 Wind velocity vs time

Basic wind velocity from figure above:

Vb = 3.0133 * ln(50) + 26.112 = 37.9 m/s ≈ 38 m/s.

Orography factor, $c_o = 1$;

Terrain category – II: $z_0 = 0.05$ m, $z_{min} = 2$ m, $z_{0,II} = 0.05$ m, terrain factor, $k_r = 0.19$;

Air density, $\rho = 1.25$ kg/m^3;

Reference heights, z_e, for the face of the building subjected to wind in both south-west and north-east directions are equal to 30 m, 39.1 m and 49.3 m. The values obtained using following guideline provided in EN 1991-1-4:2005 (Figure 5.3).

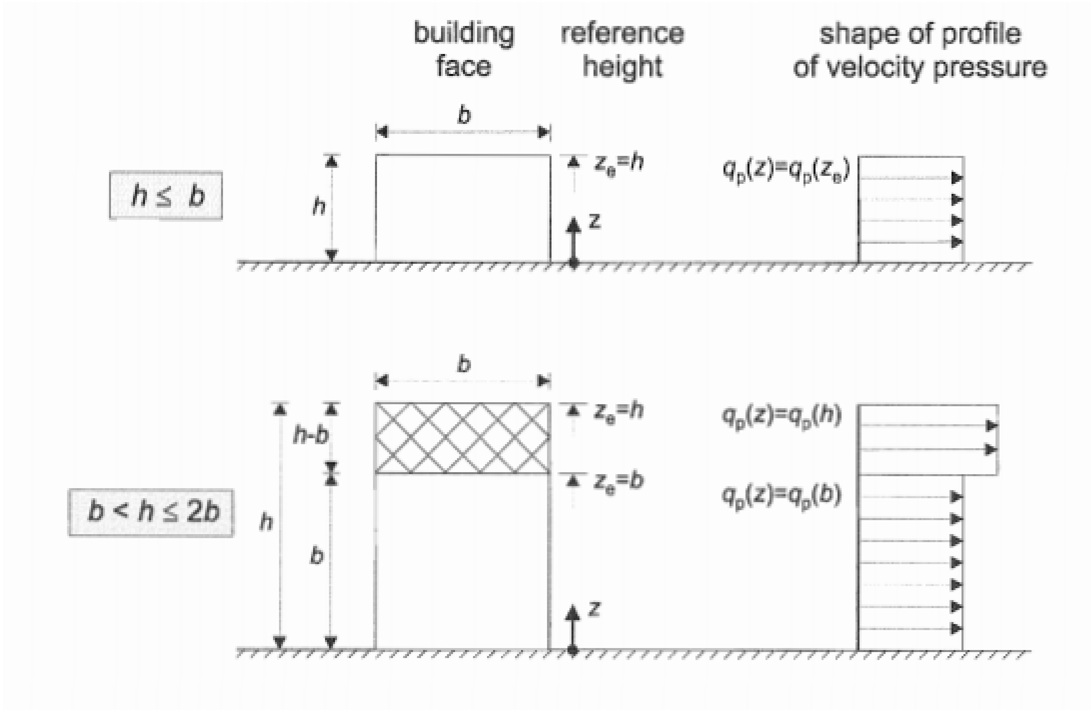


Figure 5.3 Reference height, Ze, depending on h and b, and corresponding velocity pressure profile

Turbulence factor, $k_l = 1$;

Following Table 5.7 summarizes the calculations of wind pressure on surfaces based on the equations presented in literature review part.

Table 5.7 Turbulence intensity and roughness factor values.

Reference heights, z_e, m		30	39.1	49.3
Turbulence Intencity, $I_v(z)$		0.156	0.15	0.145
Roughness factor, $c_r(z)$		1.215	1.266	1.31
Mean wind vellocity, v_m, m/s		46.19	48.10	49.77
Peak velocity pressure, q_p, kN/m^2		2.79	2.965	3.12
Wind pressure, w_e, kN/m^2	in south-west direction	0.782	0.83	0.873
	in north-east direction	0.838	0.889	0.936

5.3 Structural Analysis

5.3.1 Building Model in SAP2000

3D model of the building was built in SAP 2000 software. Following figure presents the building model.

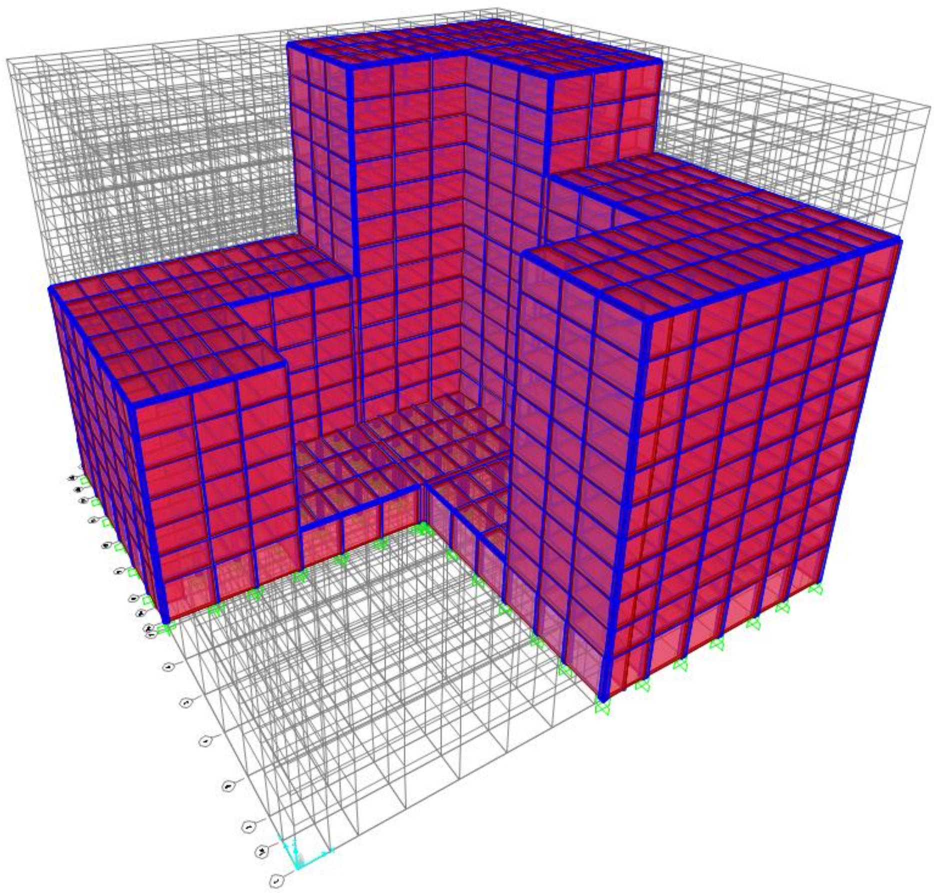


Figure 5.4 3D model of the building

Furthermore, dead load, live load, snow load and wind actions were added in the model to perform further analysis. Assigned loads can be seen in figures below.

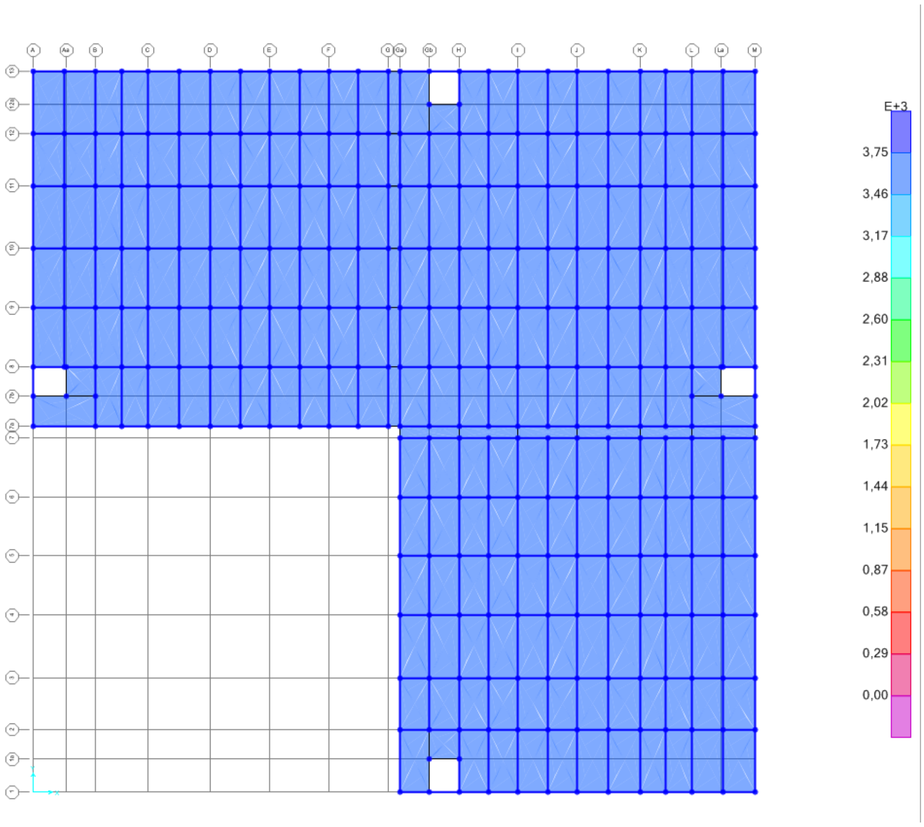


Figure 5.5 Dead loads (values on legend are in kN/m)

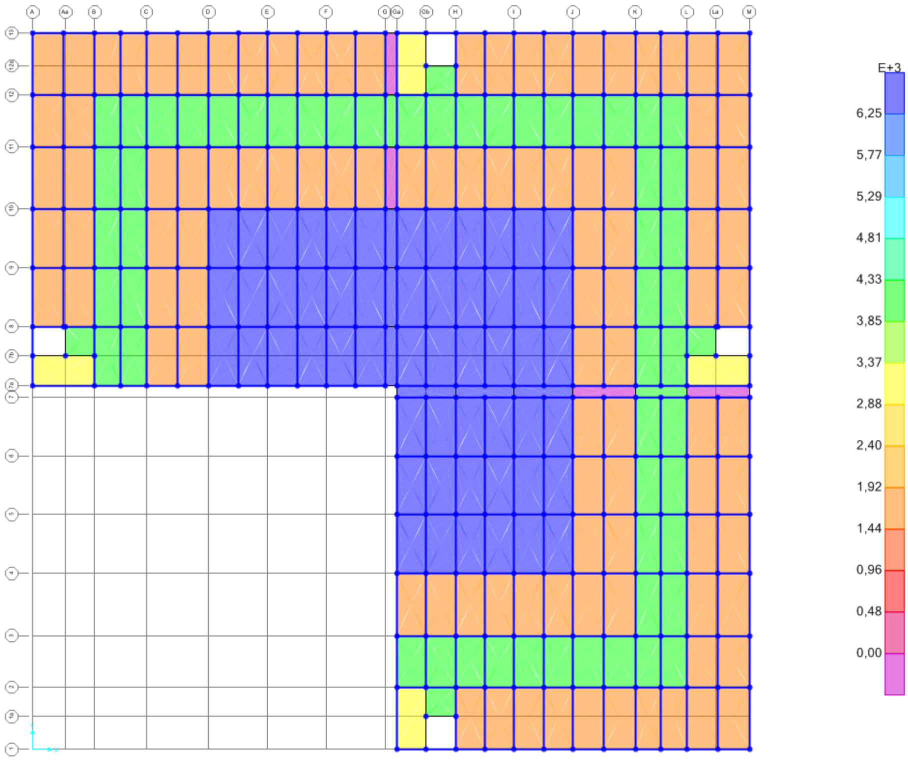


Figure 5.6 Live Loads (values on legend are in kN/m)

The frame on axis 11 and XZ plane is considered as critical one cause analysis results show that maximum values of shear force, bending moment and axial force appeared in members of this frame. Internal forces occurred in the frame can be seen in figures below.

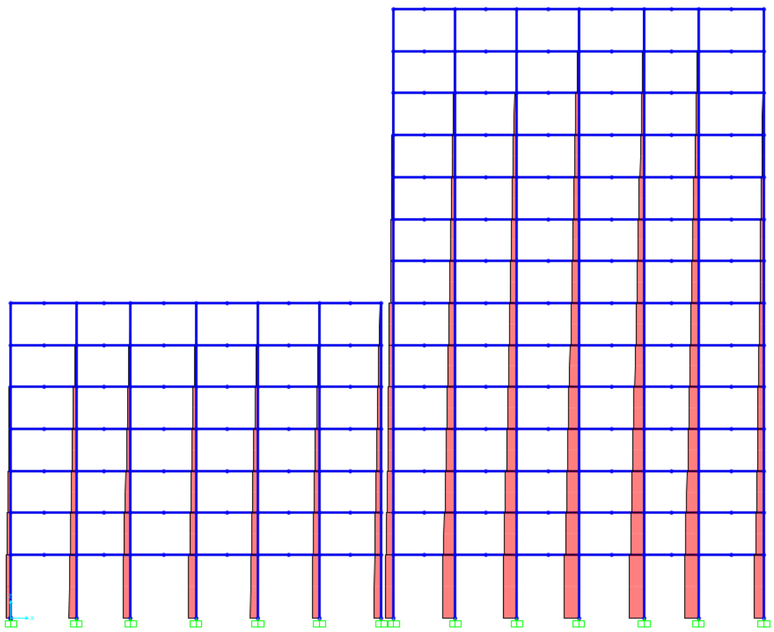


Figure 5.7 Axial forces in axis 11

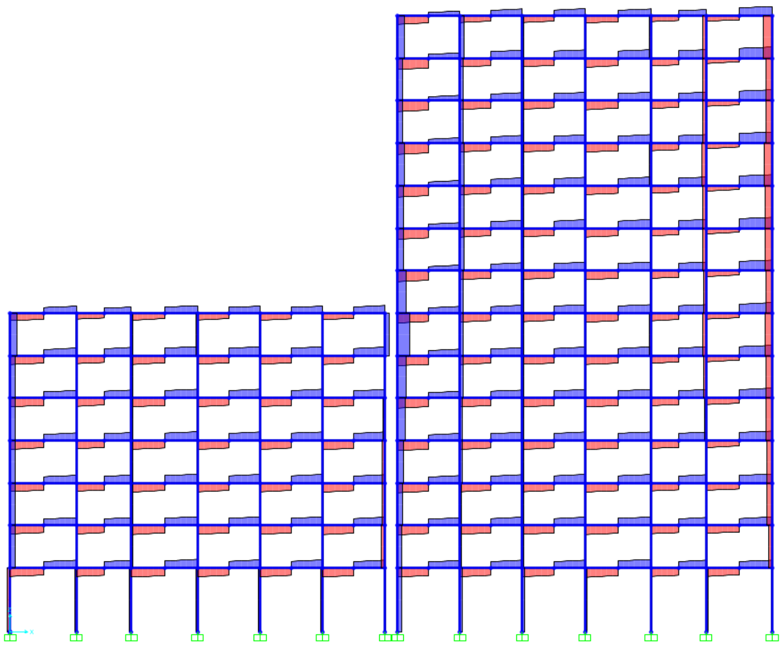
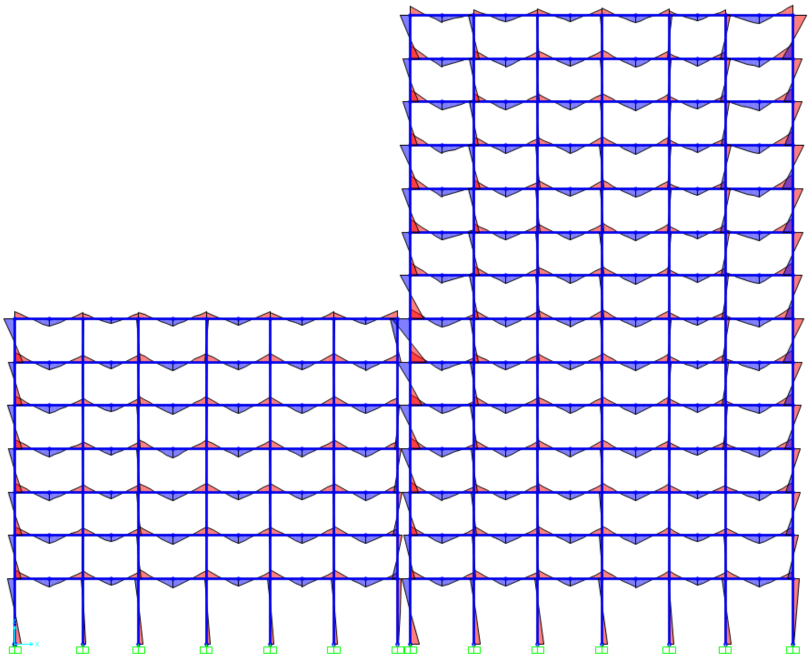


Figure 5.8 Shear forces in axis 11



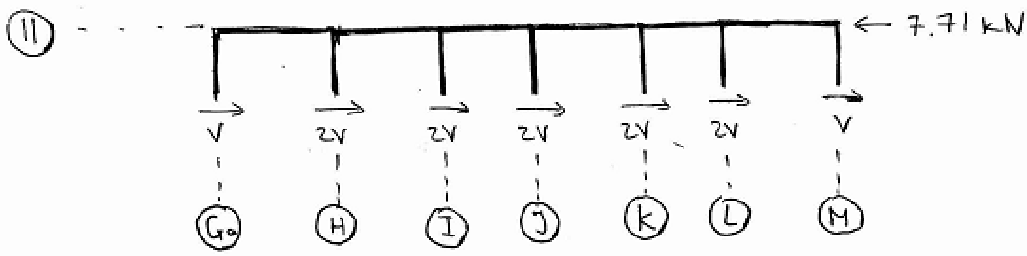
. Figure 5.9 Bending moments in axis 11

5.3.2 Hand Calculations

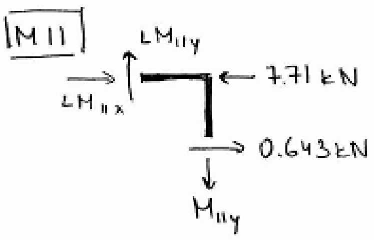
5.3.2.1 Analysis under wind load

The portal frame method is used to perform the hand calculations that are presented below. They were made previously only for the part of the frame at the top. The direction of wind load is taken as in north-east.

Wind Load effect



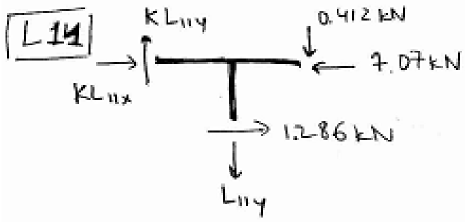
$$V = \frac{7.71}{12} = 0.643 \text{ kN} \qquad 2V = 1.286 \text{ kN}$$



$$LM_{11x} = 7.71 - 0.643 = 7.07 \text{ kN}$$

$$LM_{11y} = \frac{0.643 \times 3.4}{5.3} = 0.412 \text{ kN}$$

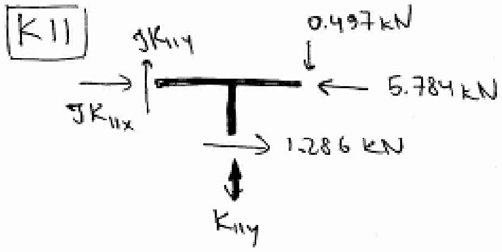
$$M_{11y} = 0.412 \text{ kN}$$



$$KL_{11x} = 7.07 - 1.286 = 5.784 \text{ kN}$$

$$KL_{11y} = \frac{1.286 \cdot 3.4 - 0.412 \cdot 5.3}{4.4} = 0.497 \text{ kN}$$

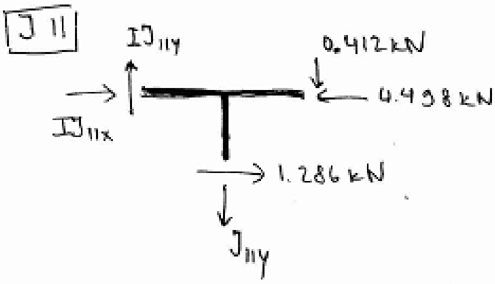
$$L_{11y} = 0.497 - 0.412 = 0.085 \text{ kN}$$



$$JK_{11x} = 5.784 - 1.286 = 4.498 \text{ kN}$$

$$JK_{11y} = \frac{1.286 \cdot 3.4 - 0.497 \cdot 4.4}{5.3} = 0.412 \text{ kN}$$

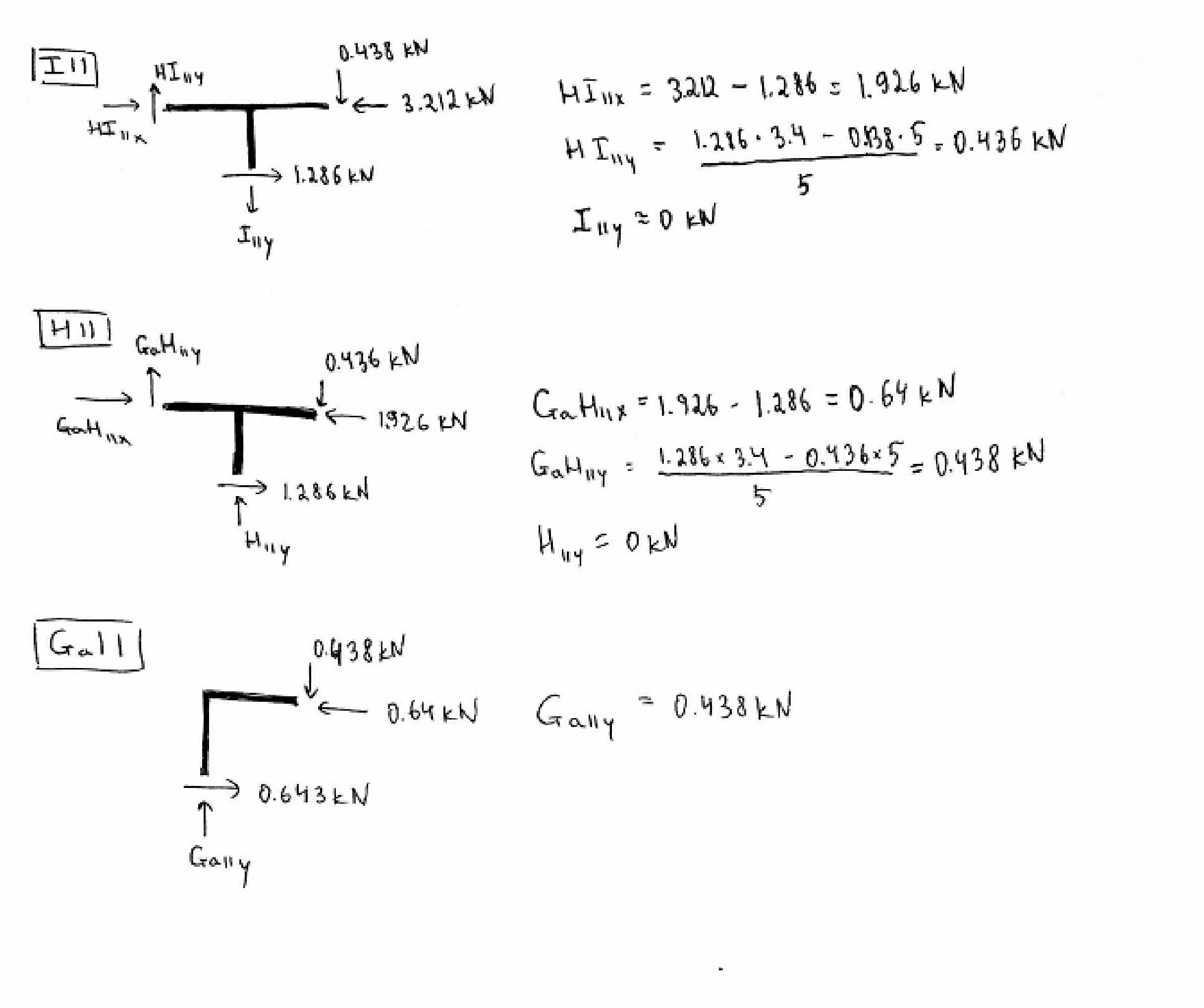
$$K_{11y} = 0.497 - 0.412 = 0.085 \text{ kN}$$



$$IJ_{11x} = 4.498 - 1.286 = 3.212 \text{ kN}$$

$$IJ_{11y} = \frac{-0.412 \cdot 5.3 + 1.286 \cdot 3.4}{5} = 0.438 \text{ kN}$$

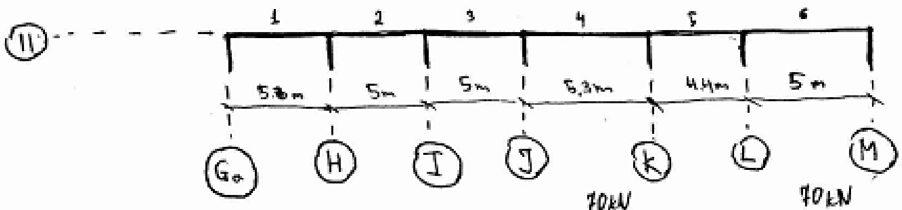
$$J_{11y} = 0.438 - 0.412 = 0.026 \text{ kN}$$

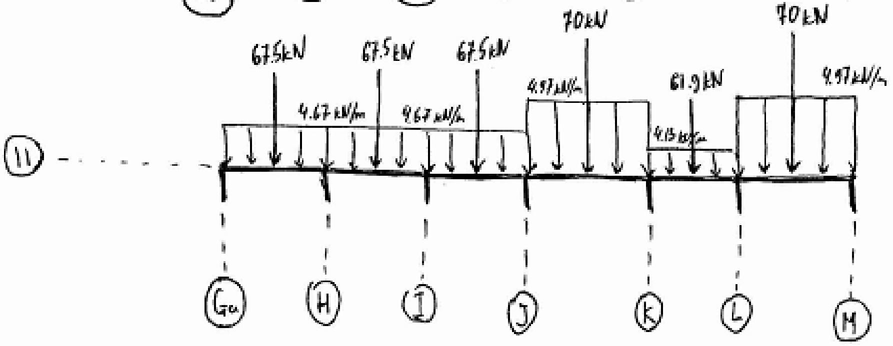


5.3.2.2 Analysis under dead load

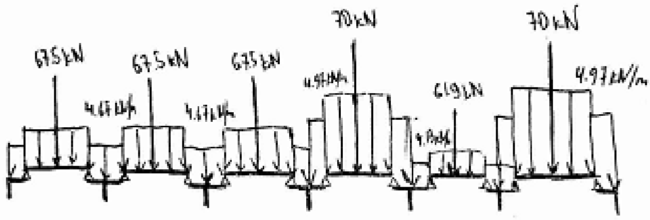
Approximate analysis method, which was described earlier, is used in following calculations of internal forces due to dead load.

Dead Load effect:

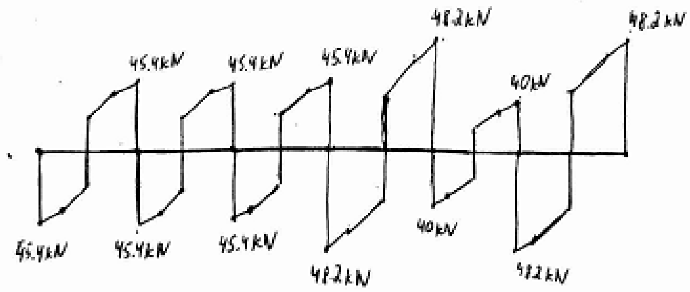




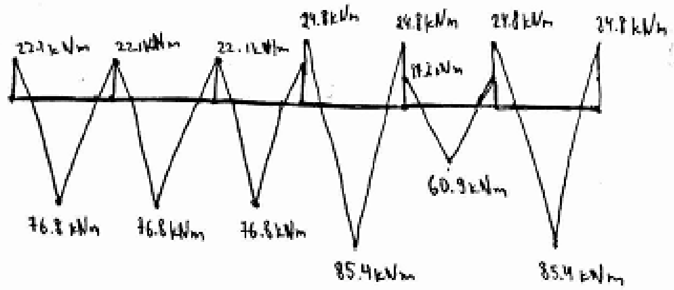
Approximate model:



Shear forces appeared:



Bending moments appeared:



5.3.3 Result comparison

5.3.3.1 Internal forces due to wind actions

Following table presents internal force values obtained by hand calculations and SAP2000 analysis. Discrepancies in the values can be seen from the table. They are mostly due to the fact that wind load applied to the joint M11 in hand calculation possibly different from the wind load applied in SAP2000 model, which is calculated by software itself. Furthermore, the distribution of the shear force in columns in hand calculations, which is the basis for these calculations, differs from the real life distribution that was tried to be achieved by the software.

Table 5.8 Comparison of the internal forces due to wind actions

Member		Axial Force, kN		Shear Force, kN	
		Hand	SAP2000	Hand	SAP2000
Beam	GaH_{11}	0.64	0.568	0.438	0.189
	HI_{11}	1.926	0.752	0.436	0.5
	IJ_{11}	3.212	2.767	0.438	0.358
	JK_{11}	4.498	4.307	0.412	0.749
	KL_{11}	5.784	3.01	0.497	0.391
	LM_{11}	7.07	2.831	0.412	0.19
Column	Ga_{11}	0.438	3.75	0.643	0.19
	H_{11}	0	1.132	1.286	0.591
	I_{11}	0	0.991	1.286	1.478
	J_{11}	0.026	1.201	1.286	2.154
	K_{11}	0.085	0.806	1.286	2.187
	L_{11}	0.085	0.938	1.286	0.575
	M_{11}	0.412	3.508	1.286	0.28

5.3.3.2 Internal forces due to wind actions

In this case values obtained by hand calculations and SAP2000 analysis are relatively close to each other. Firstly, because of dead load applied in both calculation methods are likely to be the same. Also, differences appeared possibly due to the load transfer from slab to beams assumption used in analysis methods.

Table 5.9 Comparison of internal forces due to dead load

Member	Shear force max value, kN		Bending moment max value, kNm	
	Hand	SAP2000	Hand	SAP2000
GaH_{11}	45.4	44.3	76.84	52.175
HI_{11}	45.4	38.9	76.84	44.68
IJ_{11}	45.4	40.3	76.84	47.5
JK_{11}	48.17	41.2	85.37	49.23
KL_{11}	40	34.79	60.87	35.9
LM_{11}	48.2	48.2	85.37	59.76

The group decided to rely on results obtained from analysis of SAP2000 model as assumptions used in the software less rough that those used in hand calculations and more close to real-life case.

5.4 Structural Member Design

5.4.1 Beams and columns

Reinforcement of beams and columns of the building are obtained using Design/Check of Structure command of the software and shown in figures below.

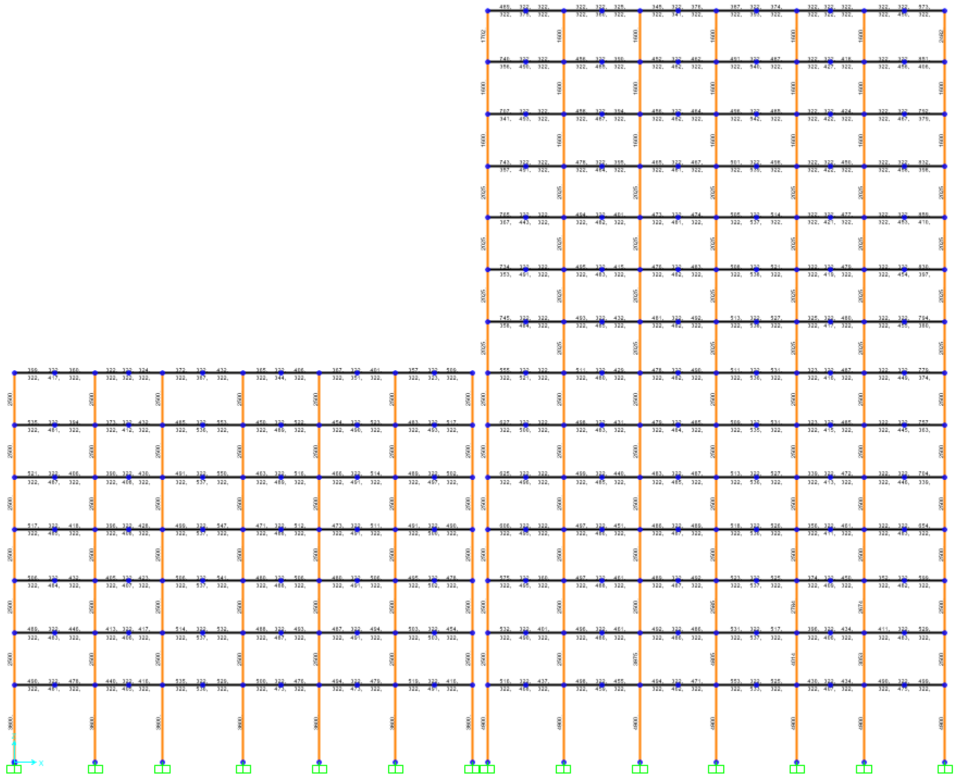


Figure 5.10 Reinforcement details

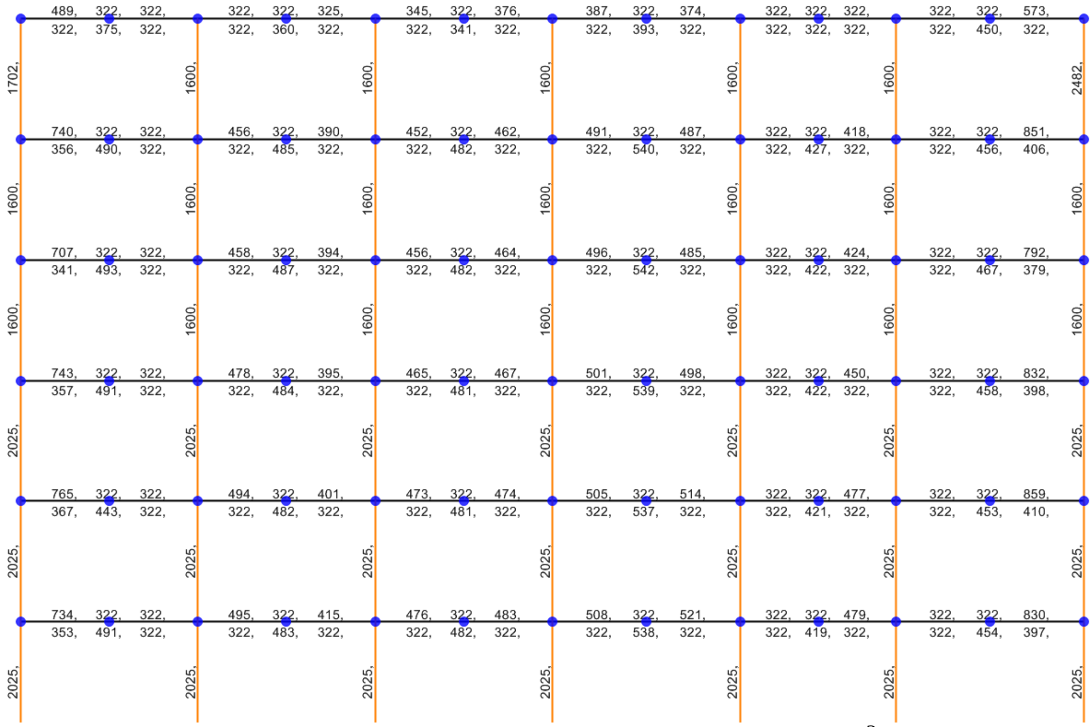


Figure 5.11 Zoomed reinforcement details (all values in mm^2)

Design of the members in SAP 2000 was performed by doing number of iterations changing the section dimensions of the beams and columns. There are final dimensions of the member sections:

- Beams: 300 mm x 450 mm
- Columns of first floor under sections 1 and 3: 600 mm x 600 mm
- Columns of first floor under section 2: 700 mm x 700 mm
- Columns of first floor under rooftop: 500 mm x 500 mm
- Columns between floors 2 and 7: 500 mm x 500 mm
- Columns between floors 8 and 11: 450 mm x 450 mm
- Columns between floors 12 and 14: 400 mm x 400 mm

For more detailed features, such as rebar arrangement in member sections, please refer to technical drawings.

Anchorage design of beams was done following instructions provided in paragraph 8.4 of EN 1991-1-1:2004.

4.4 m Long Beams

Beam 1 End rebar : $\phi = 14\,mm \quad S = 350\,mm$

$$f_{bd} = 2.25 \times 0.7 \times 1 \times 1.73 = 2.73\,MPa$$

$$l_{brd} = \frac{14}{4} \cdot \frac{500}{2.73} = 641\,mm$$

$$c_d = \min\left(\frac{86}{2}\,;\,43\right) = 43\,mm > 3\phi = 42\,mm$$

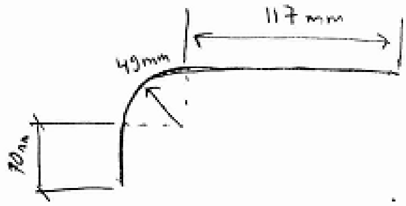
$$\alpha_1 = 0.7 \quad \alpha_2 = 1 - \frac{0.15\,(43 - 14)}{14} = 0.7$$

$$\alpha_3 = 1 - 0.1\,\frac{\frac{2\pi \cdot 10^2}{4} \cdot \frac{641}{350} - 0.25\,\frac{\pi \cdot 14^2}{4}}{\frac{\pi \cdot 14^2}{4}} = 0.84$$

$$\alpha_4 = \alpha_5 = 1$$

$$l_{bd} = 0.7^2 \times 0.84 \times 1 \times 1 \times 641\,mm = 264\,mm$$

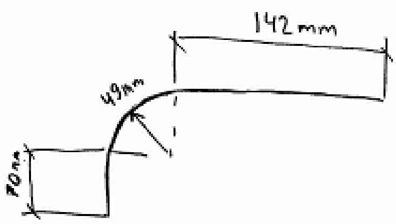
$$5\phi = 5 \cdot 14 = 70\,mm \quad r = 3.5\,\phi = 3.5 \cdot 14 = 49\,mm$$



Middle rebar : $K = 0.05 \quad \alpha_3 = 1 - 0.05 \times 1.6 = 0.92$

$$l_{bd} = 0.7^2 \times 0.92 \times 1 \times 1 \times 641 = 289\,mm$$

$$5\phi = 70\,mm \quad r = 49\,mm$$



5m Long Beams

Beam 1 End rebar: $\phi = 12mm$ $S = 450mm$

$l_{brd} = \frac{12}{4} \cdot \frac{500}{275} = 549.45mm$

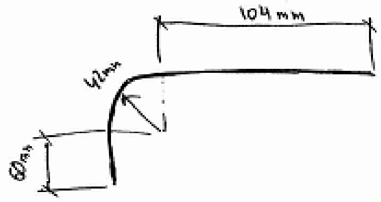
$C_d = \min\left(\frac{88}{2} ; 44\right) = 44mm > 3\phi = 36mm$

$\alpha_1 = 0.7$ $\alpha_2 = 1 - 0.15 \cdot \frac{44-12}{12} = 0.6$

$\alpha_3 = 1 - 0.1 \times 1.446 = 0.855$

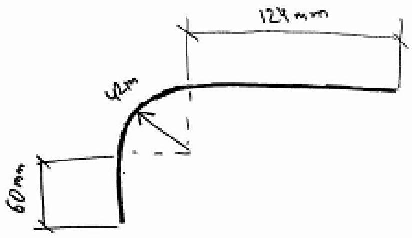
$\alpha_4 = \alpha_5 = 1$

$l_{bd} = 0.7 \times 0.6 \times 0.855 \times 1 \times 1 \times 549.45 = 230mm$



Middle rebar: $\alpha_3 = 1 - 0.05 \times 1.446 = 0.928$

$l_{bd} = 0.7 \times 0.6 \times 0.928 \times 1 \times 1 \times 549.45 = 250mm$



Beam 2

End rebar: $\phi = 14mm$ $S = 390mm$ $l_{brd} = \frac{14}{4} \cdot \frac{500}{273} = 641mm$

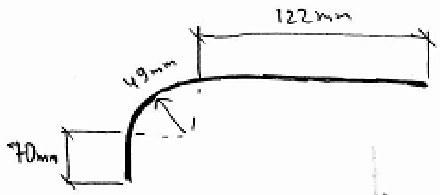
$C_d = 43mm > 42mm$

$\alpha_1 = 0.7$ $\alpha_2 = 1 - 0.15 \cdot \frac{43-14}{14} = 0.7$

$\alpha_3 = 1 - 0.1 \times 1.43 = 0.857$

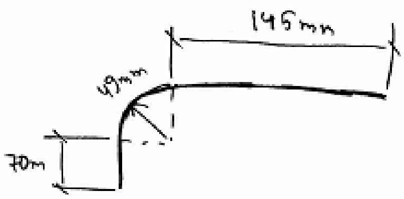
$\alpha_5 = \alpha_4 = 1$

$l_{bd} = 0.7 \times 0.7 \times 0.857 \times 1 \times 1 \times 641 = 269mm$



Middle rebar: $\alpha_3 = 1 - 0.05 \times 1.43 = 0.93$

$l_{bd} = 0.7 \times 0.7 \times 0.93 \times 1 \times 1 \times 641 = 292mm$



Beam 3

End rebar $\phi = 14mm \quad S = 300\,mm$

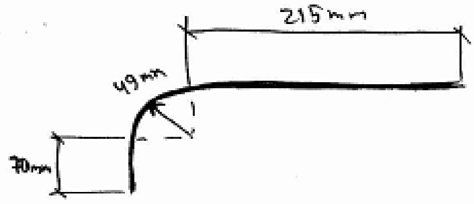
$$l_{brd} = \frac{14}{4} \cdot \frac{500}{2.73} = 641\,mm$$

$$C_d = 18\,mm$$

$$\alpha_1 = 1 \quad \alpha_2 = 1 - \frac{0.15(18-14)}{14} = 0.7$$

$$\alpha_3 = 1 - 0.1 \cdot 1.93 = 0.807$$

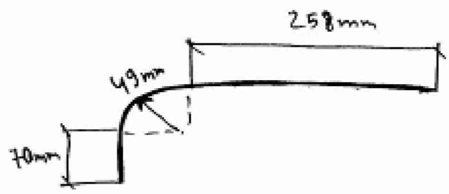
$$l_{bd} = 1 \times 0.7 \times 0.807 \times 1 \times 1 \times 641 = 362\,mm$$



Middle rebar

$$\alpha_3 = 1 - 0.05 \cdot 1.93 = 0.904$$

$$l_{bd} = 1 \times 0.7 \times 0.904 \times 1 \times 1 \times 641 = 405\,mm$$



Beams 4 & 5

End rebar $\phi = 16mm \quad S = 350\,mm$ $\quad l_{brd} = \frac{16}{4} \cdot \frac{500}{2.73} = 733\,mm$

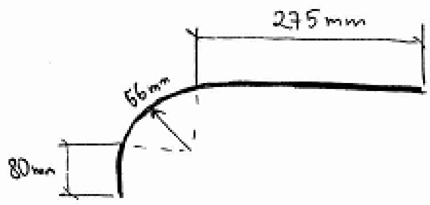
$$a = 84\,mm \quad C_d = 42mm < 48mm$$

$$\alpha_1 = 1.0 \quad \alpha_2 = 1 - 0.15 \times \frac{42-16}{16} = 0.7$$

$$\alpha_3 = 1 - 0.1 \times 1.36 = 0.864$$

$$\alpha_4 = \alpha_5 = 1$$

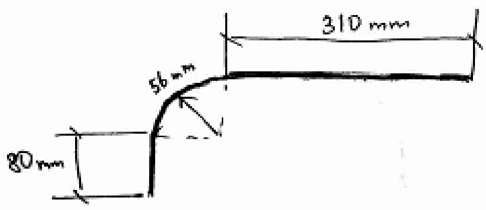
$$l_{bd} = 1 \times 0.7 \times 0.864 \times 733 = 443\,mm$$



Middle rebar

$$\alpha_3 = 1 - 0.05 \times 1.36 = 0.932$$

$$l_{bd} = 1 \times 0.7 \times 0.932 \times 1 \times 1 \times 733 = 478\,mm$$



5.3m Long Beams

Beams 1 & 3

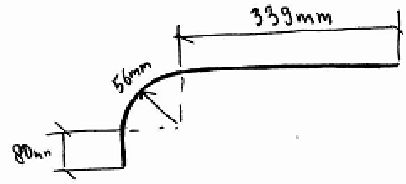
End rebar: $\phi = 16\,mm$ $S = 290\,mm$ $\quad l_{brd} = \frac{16}{4} \times \frac{500}{2.73} = 732.6\,mm$

$a = 67\,mm$ $\quad c_d = 33.5 < 48\,mm$

$\alpha_1 = 1.0$ $\quad \alpha_2 = 1 - 0.15 \times \frac{33.5 - 16}{16} = 0.836$

$\alpha_3 = 1 - 0.1 \times 1.73 = 0.827$ $\quad \alpha_4 = \alpha_5 = 1$

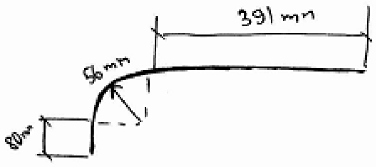
$l_{bd} = 1 \times 0.836 \times 0.827 \times 1 \times 1 \times 732.6 = 507\,mm$



Middle rebar:

$\alpha_3 = 1 - 0.05 \times 1.73 = 0.9135$

$l_{bd} = 1 \times 0.836 \times 0.9135 \times 1 \times 1 \times 732.6 = 559\,mm$



Beam 2

End rebar: $\phi = 14\,mm$ $\quad S = 350\,mm$

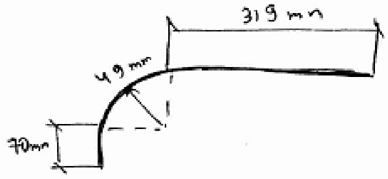
$a = 69\,mm$ $\quad c_d = 34.5\,mm$ $\quad l_{brd} = \frac{14}{4} \times \frac{500}{2.73} = 641\,mm$

$\alpha_1 = 1.0$ $\quad \alpha_2 = 1 - 0.15 \times \frac{34.5 - 14}{14} = 0.78$

$\alpha_3 = 1 - 0.1 \times 0.69 = 0.931$

$\alpha_4 = \alpha_5 = 1.0$

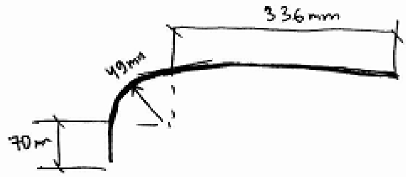
$l_{bd} = 1 \times 0.78 \times 0.931 \times 1 \times 1 \times 641 = 466\,mm$



Middle rebar

$\alpha_3 = 1 - 0.05 \times 0.69 = 0.966$

$l_{bd} = 1 \times 0.78 \times 0.966 \times 1 \times 1 \times 641 = 483\,mm$



Beam 4

End rebar: $\phi = 12mm$ $\quad S = 650mm$ $\quad l_{brd} = \frac{12}{4} \times \frac{500}{273} = 550\ mm$

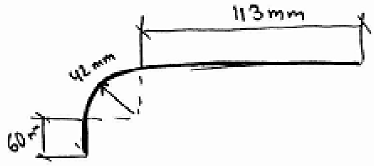
$a = 113\ mm$ $\quad C_d = 44mm > 36\ mm$

$\alpha_1 = 0.7$ $\quad \alpha_2 = 1 - 0.15 \times \frac{44 - 12}{12} = 0.7$

$\alpha_3 = 1 - 0.1 \times 1.14 = 0.886$

$\alpha_4 = \alpha_5 = 1$

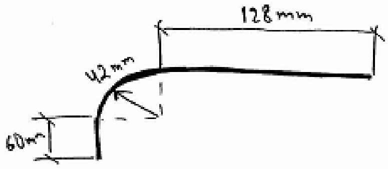
$l_{bd} = 0.7^2 \times 0.886 \times 1 \times 1 \times 550 = 239mm$



Middle rebar:

$\alpha_3 = 1 - 0.05 \times 1.14 = 0.943$

$l_{bd} = 0.7^2 \times 0.943 \times 1 \times 1 \times 550 = 254mm$



Beams 5 & 6

End rebar: $\phi = 12mm$ $\quad S = 450mm$

$l_{brd} = \frac{12}{4} \times \frac{500}{273} = 550mm$

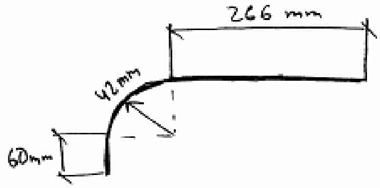
$a = 50.5mm$ $\quad C_d = 25.3mm < 36mm$

$\alpha_1 = 1.0$ $\quad \alpha_2 = 1 - 0.15 \cdot \frac{25.3 - 12}{12} = 0.834$

$\alpha_3 = 1 - 0.1 \times 1.45 = 0.855$

$\alpha_4 = \alpha_5 = 1$

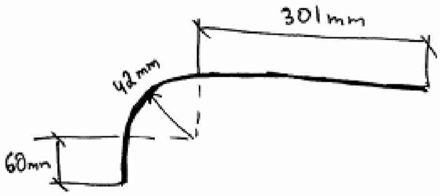
$l_{bd} = 1 \times 0.834 \times 0.855 \times 1 \times 1 \times 550 = 392.2mm$



Middle rebar:

$\alpha_3 = 1 - 0.05 \times 1.45 = 0.93$

$l_{bd} = 1 \times 0.834 \times 0.93 \times 550 = 427\ mm$



Following the deflection check procedure, basic and actual $^{l}/_{d}$ values were calculated for all beam types. Results are tabulated in TableX. It can be seen from the table that in all 13 cases, actual $^{l}/_{d}$ is lower than basic one, which implies that deflection check is completed successfully and deflection is not controlled in design of beam. More detailed information can be found in Appendix B.

Table 5.10 Deflection check for beams

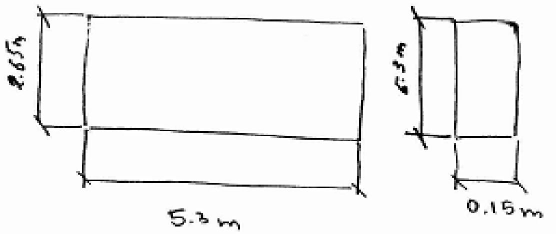
Beam #	Beam length	Beam type	l/d actual	l/d basic
1	4.4m	internal span	11	35.05
		end span	11	30.37
2	5m	internal span	12.5	52.56
		end span	12.5	45.55
3	5m	internal span	12.5	52.56
		end span	12.5	45.55
4	5m	internal span	12.5	35.05
		end span	12.5	30.37
5	5m	internal span	12.5	35.05
		end span	12.5	30.37
6	5m	internal span	12.5	27.75
		end span	12.5	24.05
7	5m	internal span	12.5	35.05
		end span	12.5	30.37
8	5.3m	internal span	13.25	35.89
		end span	13.25	31.11
9	5.3m	internal span	13.25	35.89
		end span	13.25	31.11
10	5.3m	internal span	13.25	35.89
		end span	13.25	31.11
11	5.3m	internal span	13.25	52.56
		end span	13.25	45.55
12	5.3m	internal span	13.25	28.8
		end span	13.25	24.96
13	5.3m	internal span	13.25	28.8
		end span	13.25	24.96

5.4.2 Slabs

Slab design was performed by using loads generated in SAP2000 model. Below, determination of reinforcement for the most frequent slabs in the building is presented.

The design was performed with help of guidance provided by Moss and Brooker (2005).

Slab design



$c = 20\ mm$

$\phi = 10\ mm$

$d = 130\ mm$

$$w = 1.35 \times 3.75 + 1.5 \times 1.75 = 7.69\ kN/m^2$$

$$M = \frac{7.69 \times 5.3^2 \times 2.65}{8} = 71.55\ kNm$$

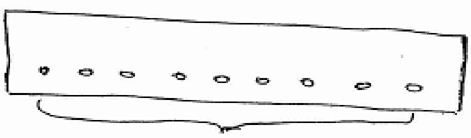
$$K = \frac{71.55 \times 10^6}{2 \cdot 2650 \times 130^2 \times 25} = \frac{0.064}{2} < 0.151 = K' \quad \delta = 0.8$$

$$\frac{z}{d} = 0.95 \quad z = 0.95 \cdot 130 = 123.5\ mm.$$

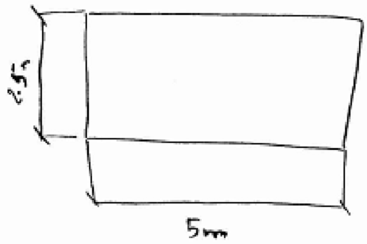
$$A_s = \frac{71.55 \times 10^6 \times 1.15}{500 \times 123.5} = 1332.5\ mm^2$$

$A_{min} = 519\ mm^2 \quad A_{max} = 15900\ mm^2$

$n = 17 \quad S = 312\ mm$



$17 - \phi 10mm$ @ 312mm



$C = 20\,mm$

$\phi = 10\,mm$

$d = 130\,mm$

$w = 1.35 \times 3.75 + 1.5 \times 1.75 = 7.69\,kN/m^2$

$$M = \frac{7.69 \cdot 2.5^2 \cdot 5}{8} = 30\,kNm$$

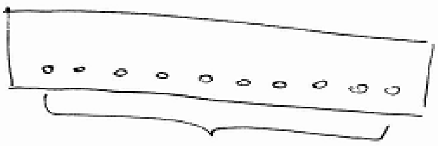
$$K = \frac{30 \times 10^6}{5000 \cdot 130^2 \cdot 25} = 0.03 < 0.151 = K' \quad \delta = 0.8$$

$$\frac{z}{d} = 0.95 \quad z = 0.95 \cdot 130 = 123.5\,mm$$

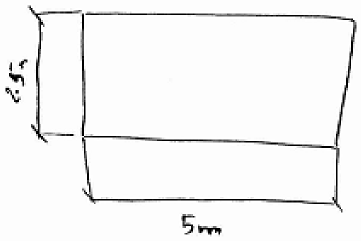
$$A_s = \frac{30 \times 10^6 \times 1.15}{500 \cdot 123.5} = 558.7\,mm^2$$

$A_{s\,min} = \underline{845\,mm^2}$ $\quad A_{s\,max} = 30\,000\,mm^2$

$n = 11 \quad s = 455\,mm$



$11 - \phi\,10mm @ 455\,mm$



$C = 20\ mm$

$\phi = 10\ mm$

$d = 130\ mm$

$w = 1.35 \times 3.75 + 1.5 \times 1.75 = 7.69\ kN/m^2$

$$M = \frac{7.69 \cdot 2.5^2 \cdot 5}{8} = 30\ kNm$$

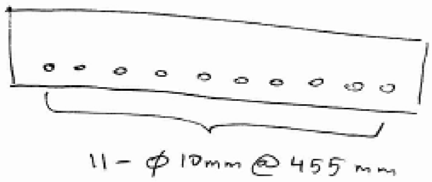
$$k = \frac{30 \times 10^6}{5000 \cdot 130^2 \cdot 25} = 0.03 < 0.151 = K' \qquad \delta = 0.8$$

$$\frac{z}{d} = 0.95 \qquad z = 0.95 \cdot 130 = 123.5\ mm$$

$$A_s = \frac{30 \times 10^6 \times 1.15}{500 \cdot 123.5} = 558.7\ mm^2$$

$A_{smin} = \underline{\underline{845\ mm^2}}$ $\qquad A_{smax} = 30\,000\ mm^2$

$n = 11 \qquad s = 455\ mm$



Following the deflection check procedure, basic and actual l/d values were calculated for all slab types. Results are tabulated in TableX. It can be seen from the table that for all slab types, actual l/d is lower than basic one, which implies that deflection check is completed successfully and deflection is not controlled in design of slab. More detailed information can be found in Appendix B.

Table 5.11 Deflection check for slabs

Slab type	l/d actual	l/d basic
5300x2650	20.38462	62.3287
4400x2200	16.92308	63.7325
5000x2500	19.23077	77.9282

6 Geotechnical design

6.1 Design axial compression load

Following the pile design approach described in section 2.4.9, the design load and pile resistance capacity should be estimated. As design loads, the critical column's axial compression load will be taken, and the piles will be designed for critical case. The design building consists of 4 sections (roof top is Section 4) that differ by heights, and these sections are assumed to be connected with expansion joints. Therefore, the foundation of the sections will also be designed separately. The Figure 6.1 below shows the building sections. Figure 6.2 illustrates column positions with their axial compression loads. The building sections are differed by colors, and positions of the critical columns are shown, as well.

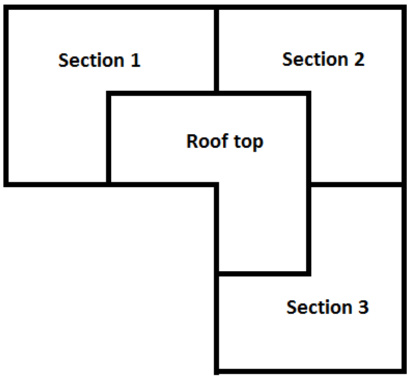


Figure 6.1 Building sections

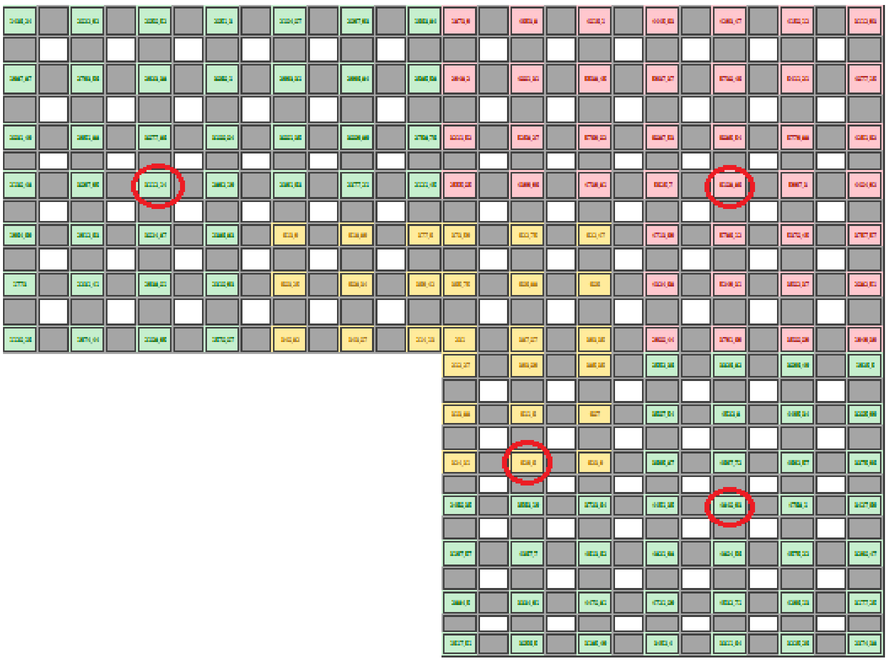


Figure 6.2 Positions of critical columns

6.2 Pile compressive design resistance

Based on the literature review, deep foundations, specifically, precast concrete piles were selected to be the most suitable. Piles will be installed by driving technique. As for soil conditions, Ospanova (2015) states that the soil depth before alluvial deposits is approximately 17-19 m. The calculations were carried out based on cone penetration test (CPT) results conducted by LLP "KaragandaGIIZ and Co" (State License №001137 issued by Construction Committee). Cone penetration data is shown in Table 6.1.

Table 6.1 Cone penetration test data

#	Soil Layer	Thickness, h (m)	Depth, h (m)	Tip Resistance (kN/m^2)	Skin Friction (kN/m^2)
1	Backfill	1,65	1,65	2800	122
2	Loam	2,9	4,55	1500	38
3	Medium sand	0,9	5,45	11800	87
4	Coarse sand	0,85	6,3	18500	140
5	Sand and gravel	2,2	8,5	18500	140
6	Gravel	2,8	11,3	19100	85
7	Loam	2,8	14,1	-	-

The calculations were done taking into account different pile sections (D = 0.3, 0.4, 0.5 m). Factor of safety was taken as 2.5 for all estimations. For all pile widths, calculated Q_p was higher than the limiting value. Therefore, it was calculated by equation $Q_p = A_p * q_t$ (N^*=231.0 for φ'=38°). Detailed calculation for both Q_p and Q_s is given in Appendix C. Tables 6.2-6.4 below represent the resistance of single pile based on CPT.

Table 6.2 Resistance of single pile (D = 0.3 m)

Pile length, L (m)	6	8	9	10	11	12
Qp (kN)	812,1	812,1	812,1	812,1	812,1	812,1
Qs (kN)	649,22	900,97	1026,97	1152,97	1155,22	1231,72
Qu (kN)	1461,4	1713,1	1839,1	1965,1	1967,4	2043,9
Qall (kN)	584,5	685,2	735,6	786,0	786,9	817,5

Table 6.3 Resistance of single pile (D = 0.4 m)

Pile length, L (m)	6	7	8	9	10	11
Qp (kN)	1443,8	1443,8	1443,8	1443,8	1443,8	1443,8
Qs (kN)	914,40	1106,37	1286,02	1411,29	1512,27	1614,27
Qu (kN)	2358,2	2550,2	2729,8	2855,1	2956,1	3058,1
Qall (kN)	943,3	1020,1	1091,9	1142,0	1182,4	1223,2

Table 6.4 Resistance of single pile (D = 0.5 m)

Pile length, L (m)	6	8	9	10	11	12
Qp (kN)	2256,0	2256,0	2256,0	2256,0	2256,0	2256,0
Qs (kN)	1207,15	1686,79	1797,14	1934,84	2056,81	2172,41
Qu (kN)	3463,1	3942,7	4053,1	4190,8	4312,8	4428,4
Qall (kN)	1385,2	1577,1	1621,2	1676,3	1725,1	1771,3

General pile parameters for D=0.3 m, D=0.4 m, and D=0.5 m are shown in Tables 6.5-6.7.

- For D=0.3 m

Table 6.5 General parameters (D=0.3 m)

Width, b (m)	0,3
Area, Aq (m²)	0,09
Perimeter, p (m)	1,2

- For D=0.4 m

Table 6.6 General parameters (D=0.4 m)

Width, b (m)	0,4
Area, Aq (m²)	0,16
Perimeter, p (m)	1,6

- For D=0.5 m

Table 6.7 General parameters (D=0.5 m)

Width, b (m)	0,5
Area, Aq (m²)	0,25
Perimeter, p (m)	2

6.3 Group piles

- for D=0.3m

Assuming 4 piles in a group, by taking $n_1 = n_2 = 2$,

$$d = 3D = 3 * 0.3 = 0.9\ m$$

$$L_g = (2-1)0.9 + 2\left(\frac{0.3}{2}\right) = 1.2\ m$$

$$B_g = (2-1)0.9 + 2\left(\frac{0.3}{2}\right) = 1.2\ m$$

$$\eta = \frac{2(2+2-2)0.9 + 4 * 0.3}{4 * 0.3 * 2 * 2} = 1$$

$$=> \ Q_{g(u)} = \Sigma Q_u$$

Assuming 2 piles in a group, by taking $n_1 = 2$ and $n_2 = 1$,

$d = 3D = 3 * 0.3 = 0.9\ m$

$L_g = (2-1)0.9 + 2\left(\frac{0.3}{2}\right) = 1.2\ m$

$B_g = (1-1)0.9 + 2\left(\frac{0.3}{2}\right) = 0.3\ m$

$\eta = \frac{2(2+1-2)0.9 + 4 * 0.3}{4 * 0.3 * 2 * 1} = 1.25$

$=> \ Q_{g(u)} = 1.25 * \Sigma Q_u$

- for D=0.4 m

Assuming 4 piles in a group, by taking $n_1 = n_2 = 2$,

$d = 3D = 3 * 0.4 = 1.2\ m$

$L_g = (2-1)1.2 + 2\left(\frac{0.4}{2}\right) = 1.6\ m$

$B_g = (2-1)1.2 + 2\left(\frac{0.4}{2}\right) = 1.6\ m$

$\eta = \frac{2(2+2-2)1.2 + 4 * 0.4}{4 * 0.4 * 2 * 2} = 1$

$=> \ Q_{g(u)} = \Sigma Q_u$

- for D=0.5 m

Assuming 4 piles in a group, by taking $n_1 = n_2 = 2$,

$d = 3D = 3 * 0.5 = 1.5\ m$

$L_g = (2-1)1.5 + 2\left(\frac{0.5}{2}\right) = 2.0\ m$

$B_g = (2-1)1.5 + 2\left(\frac{0.5}{2}\right) = 2.0\ m$

$\eta = \frac{2(2+2-2)1.5 + 4 * 0.5}{4 * 0.5 * 2 * 2} = 1$

$=> \ Q_{g(u)} = \Sigma Q_u$

6.4 Pile cap design

Following eq(2.4.15), the vertical load on a pile cap can be calculated (Table 6.8). Design loads are maximum axial loads, obtained from SAP2000 software.

Table 6.8 Design loads and calculated vertical loads on pile caps

Sections	Design load, kN	Weight, kN	Vertical load on a pile cap, kN
1	3497,2	195,8	3693,1
2	6335,1	327,6	6662,7
3	5067,9	327,6	5395,5
4	905,9	94,6	1000,5

Next, suitable pile types can be chosen for each block, based on the given loads on a pile cap. Assuming 4 piles in a pile group and η=1 efficiency, 4 times the resistance of one pile will be equal to group capacity. Table 6.9 illustrates the piles for each section.

Table 6.9 Pile types by blocks

Sections	Vertical load on a pile cap, kN	Pile type	Pile group capacity, kN
1	3693,1	C60.40	3773,1
2	6662,7	C110.50	6900,4
3	5395,5	C60.50	5541,0
4	1000,5	C60.30	2338,2

Vertical load on a pile was estimated by eq(2.4.16) considering 2 cases: when axial load is maximum, and, when horizontal load is maximum. Moment arm and moment of inertia for 3 pile widths are shown in Table 6.10, and calculations of vertical load on a pile are given Tables 6.11 and 6.12.

Table 6.10 Moment of Inertia for piles

D =	0,3		
Lg =	1,2	Bg =	1,2
x =	0,6	Ixx =	1,44
y =	0,6	Iyy =	1,44

D =	0,4		
Lg =	1,6	Bg =	1,6
x =	0,8	Ixx =	2,56
y =	0,8	Iyy =	2,56

D =	0,5		
Lg =	2	Bg =	2
x =	1	Ixx =	4
y =	1	Iyy =	4

Table 6.11 Vertical load on a pile (case 1)

#	Vertical load on a pile cap, kN	x-x direction		y-y direction		Vertical load on a pile, kN	Horizontal load on a pile, kN
		Horizontal force, kN	Moment, kNm	Horizontal force, kN	Moment, kNm		
1	3693,1	26,75	61,93	8,03	18,58	3770,5	6,98
2	6662,7	32,21	73,84	9,66	22,15	6736,6	8,41
3	5395,5	33,92	86,74	10,18	26,02	5482,3	8,85
4	1000,5	35,98	68,71	10,79	20,61	1115,0	9,39

Table 6.12 Vertical load on a pile (case 2)

#	Vertical load on a pile cap, kN	x-x direction		y-y direction		Vertical load on a pile, kN	Horizontal load on a pile, kN
		Horizontal force, kN	Moment, kNm	Horizontal force, kN	Moment, kNm		
1	3564,3	28,49	58,41	8,55	17,52	3661,6	7,44
2	6550,5	47,15	109,20	14,15	32,76	6732,5	12,31
3	5395,5	33,92	86,74	10,18	26,02	5540,1	8,85
4	1000,5	35,98	68,71	10,79	20,61	1115,0	9,39

The largest vertical load of 2 cases will be considered during further calculations, which is case 1. Table 6.13 illustrate the column sizes and pile cap dimensions under each section. Here, pile cap cover = 1.5D, L = 2*pile cap cover + d (=3D).

Table 6.13 Column and Pile cap dimensions

Sections	Column section, mm	Pile cap cover	Pile cap dimensions, mm
1	600x600	600	2400x2400x800
2	700x700	750	3000x3000x900
3	700x700	750	3000x3000x900
4	500x500	450	1800x1800x600

To continue the calculations, moments about 1-1 and 2-2 axis need to be found (Figure 6.3). Values are shown in Table 6.14. It can be observed that the moments about both axis are the same, as the pile cap is square shaped. Similarly, reactions of all 4 piles are same. Dead load is a combination of loads of pile cap, slab portion above pile cap, and backfill.

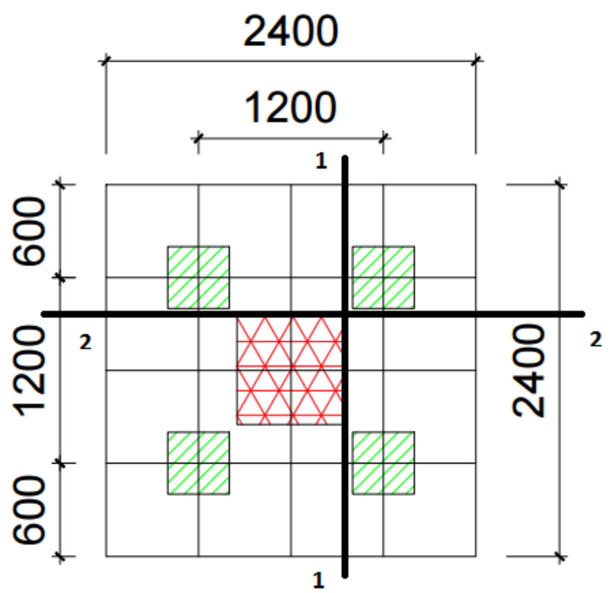


Figure 6.3 Pile cap for Section 1 with axis shown

Table 6.14 Moments about 1-1 and 2-2 axis

Sections	Dead load, kN/m²	Moment due to dead load, kNm	Reaction force, kN	Moment due to reaction, kNm	Combined moment, kNm
1	47,60	46,3	942,6	565,6	611,8
2	50,96	101,1	1684,1	1347,3	1448,4
3	50,96	110,1	1370,6	1233,5	1343,6
4	40,88	15,5	278,7	111,5	127,0

Next step is reinforcement design of pile caps. The cover to reinforcement depends on the concentration of sulfates, which is 0.75 according to ground investigations. Thus, minimum cover on binding = 50 mm and minimum cover elsewhere = 90 mm (assume 90 mm everywhere). C25/30 and G500 are used for design. Before proceeding onwards, K should be checked (eq(2.4.18)). The values of K presented in Table 6.15 are assumed to be allowable, as they are ≤ 0.156.

Table 6.15 K values

Sections	Combined moment, kNm	d_x, mm	K
1	611,8	802	0,0132
2	1448,4	800	0,0251
3	1343,6	800	0,0233
4	127,0	804	0,0036

Calculated z values were all higher than 0.95d, therefore the latter was chosen for further calculations. Table 6.16 shows the A_{st} required for pile cap design. Table 6.17 contains the summary of reinforcement design for pile caps. Illustrations of reinforcement design are shown in technical drawings.

Table 6.16 Ast calculated

Sections	K	z, mm		Ast, mm²
		eq(2.4.20)	0,95d	
1	0,0132	790,0	761,9	1846,1
2	0,0251	777,0	760,0	7301,9
3	0,0233	778,7	760,0	6773,4
4	0,0036	800,7	763,8	382,4

Table 6.17 Summary of reinforcement design

Sections	Reinforcement bar diameter, mm	Reinforcement amount in one direction	Spacing of bars, mm	Reinforcement cover, mm
1	16	10	230,7	90
2	20	24	102,6	90
3	20	22	114,3	90
4	12	4	528,0	90

Punching shear stress need to be considered, unless the spacing of piles is less than 3D. In the given case, spacing of piles = 3D, therefore, punching shear stress check should be conducted. Shear stress values should not exceed 5 N/mm^2 or $0.8(f_{cu})^{0.5}$ = 4.38 N/mm^2 (Table 6.18). Calculations are assumed to be acceptable, as the values are within the required range.

Table 6.18 Shear stress values

Sections	Column punching shear stress, N/mm²	Punching shear stress at perimeter of pile, N/mm²	Pile punching shear stress, N/mm²
1	2,24	0,84	0,38
2	3,01	1,08	0,51
3	2,64	0,87	0,41
4	1,11	0,49	0,23

Pile cap for the columns at the edge of sections were designed following the same procedure, but for 2 piles in a cap. For 2 pile group, $Q_{g(u)} = 1.25 * \Sigma Q_u$. Summary of results are shown below in Table 6.19.

Table 6.19 Summary of calculation for edge columns

#	Rebar diameter, mm	N in x direction	N in y direction	Spacing of bars, mm	Pile cap dimensions, mm	Pile type	Column section, mm	Pile cap cover
1	16	5	1	389,0	1800x900x600	C80.30	600x600	450
2	16	8	1	215,4	1800x900x600	C120.30	700x700	450
3	16	6	1	308,0	1800x900x600	C110.30	600x600	450

6.5 Settlement estimation

Following the literature review elastic settlement of the group piles was calculated for each section (Figure 6.4). The largest settlement of group piles, which is 18.733 mm, is observed under Section 3. Section 2 has the largest elastic settlement of pile and settlement of pile from tip load, whereas settlement of pile due to load along pile shaft is the highest for Section 3, thus resulting in largest total settlement. Differential settlement between sections is 13.589 mm with slope of 0.65 ‰.

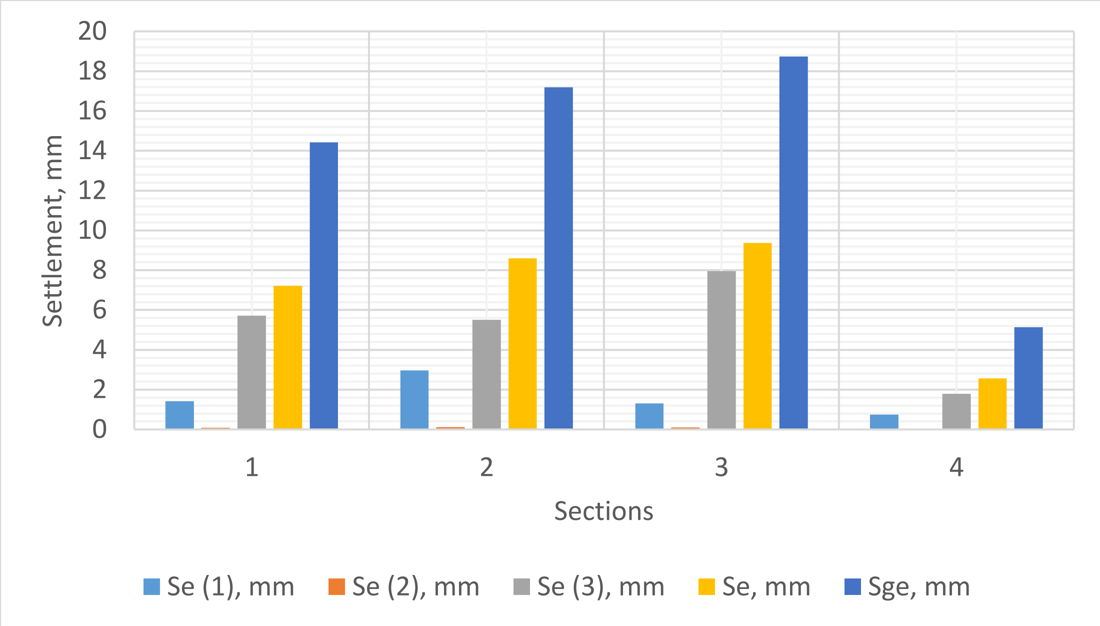


Figure 6.4 Elastic settlement of the sections

Table 6.20 represents allowable maximum settlements from different sources, and the minimum value is 2.5 cm. The largest group settlement of the design building is 18.7 mm = 1.9 cm < 2.5 cm. Estimated settlement is less than the allowable maximum settlement, thus design is acceptable.

Table 6.20 Allowable maximum settlement

Sections	Terzaghi and Peck (1967)	L/H	Polshin and Tokar (1967)
1	2,5 cm	0,6	6,5 cm
2	2,5 cm	0,8	6,5 cm
3	2,5 cm	1,2	6,5 cm
4	2,5 cm	3,5	5,5 cm

Differential settlement within sections are shown in Table 6.21. Slope of differential settlement between the maximum and minimum loads of whole building is estimated to be 0.81 ‰. Maximum allowable differential settlement is 2.2‰, thus the settlement is acceptable.

Table 6.21 Differential settlements of each section

Sections	ΔSe (1), mm	ΔSe (2), mm	ΔSe (3), mm	ΔSe, mm	ΔSge, mm	Differential settlement slope, ‰
1	0,75	0,05	3,03	3,83	7,66	0,26
2	1,68	0,06	3,13	4,87	9,74	0,39
3	0,68	0,05	4,13	4,87	9,73	0,39
4	0,48	0,02	1,16	1,67	3,33	0,24

6.6 Plaxis simulation results

Pile installation was simulated on Plaxis software, in order to observe the soil behavior. Figures 6.5 and 6.6 below illustrate the simulation outputs. As it can be observed, the total displacement is 0.0155 mm, which is too small and can not be seen from the Figure 6.7. Therefore, other further results were taken with different scale. Figures 6.8 and 6.9 represent total displacements in y and x directions, respectively. Major vertical displacements are at the tip and top of the pile. In addition, displacement of the soil also took place. In case of x direction, no big displacements were observed in soil layers, and major displacements are at the tip and top of the pile. Stress accumulation increases with depth (Figures 6.10-11), max effective stress = 1.191 kN/m^2. Figure 6.12 represents the vertical displacement and dynamic time relationship during pile driving.

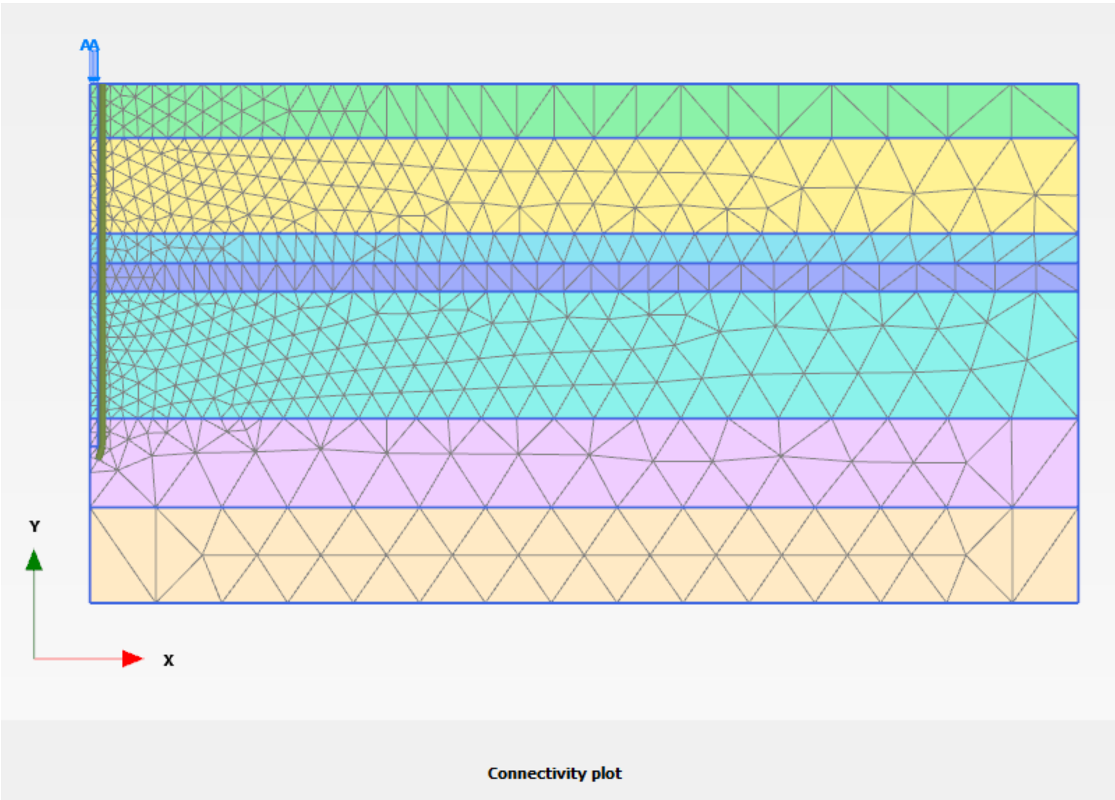


Figure 6.5 Generated mesh

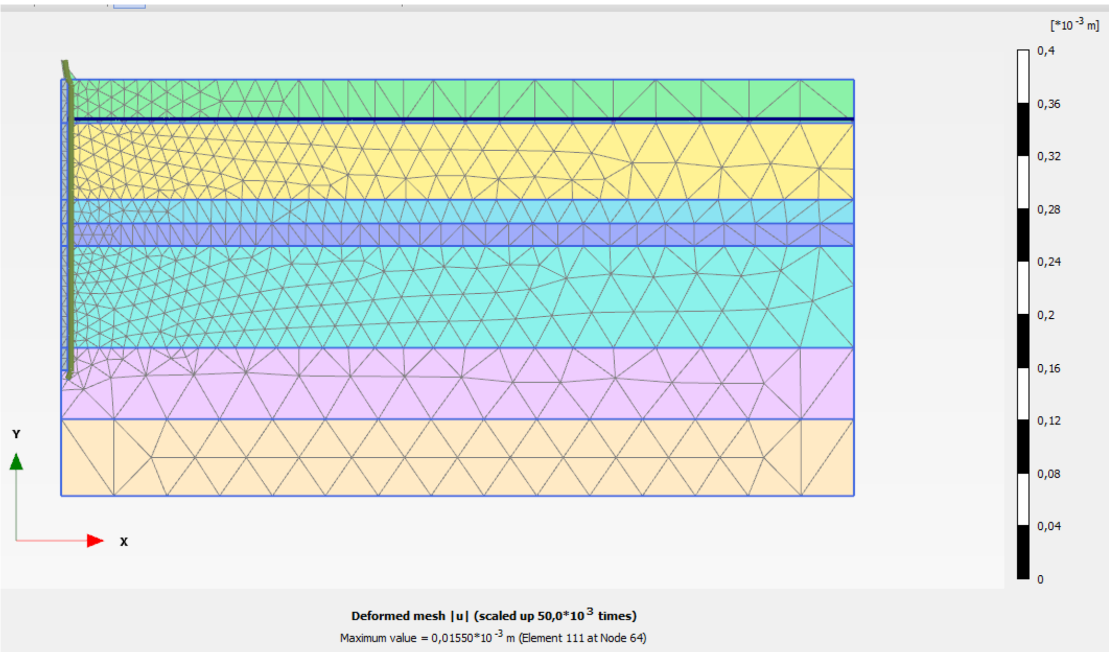


Figure 6.6 Deformed mesh

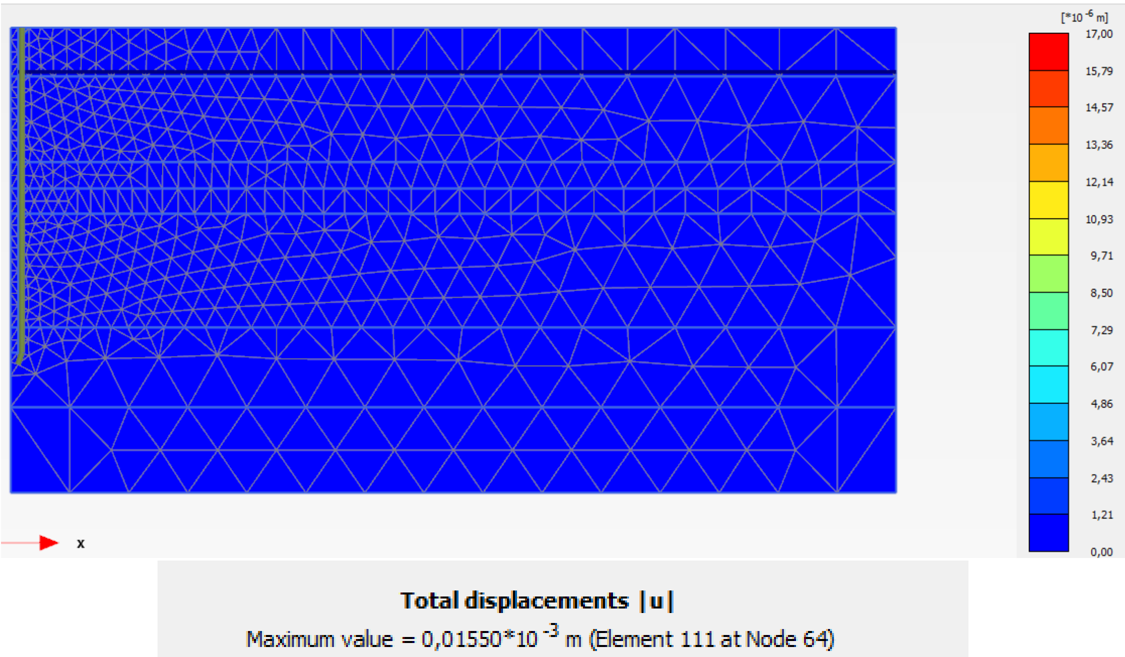
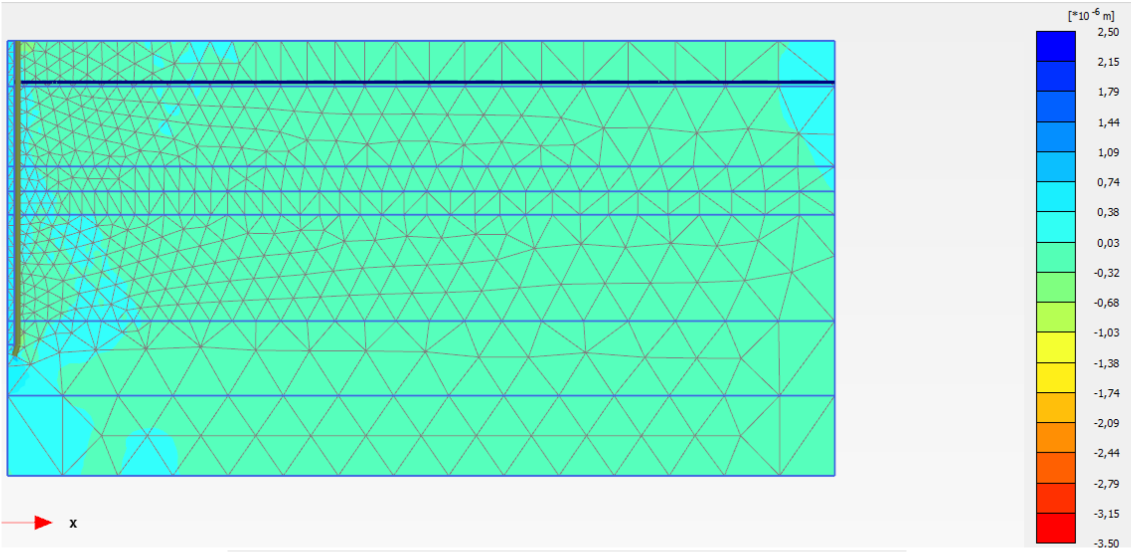


Figure 6.7 Total Displacements

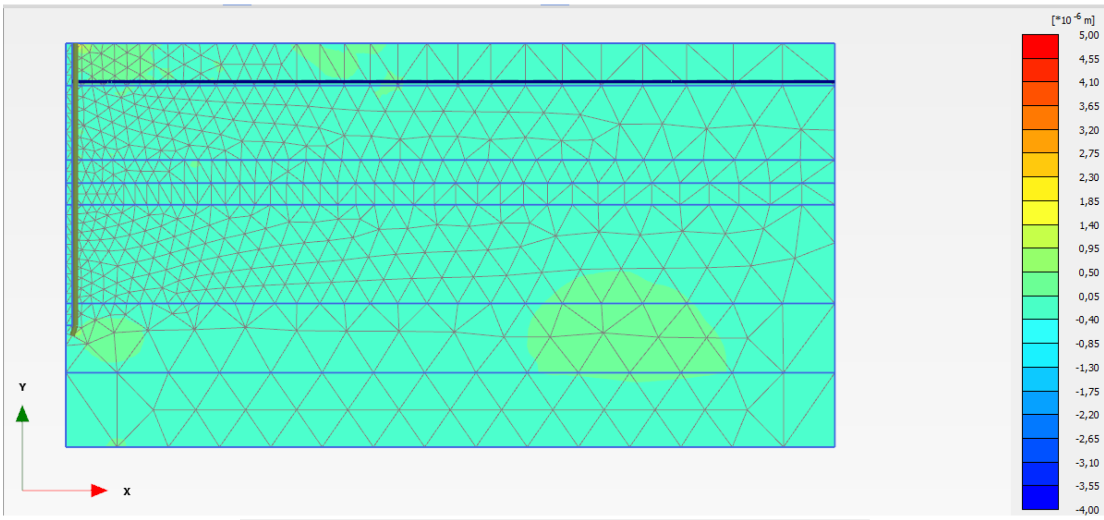


Total displacements u_y

Maximum value = $0{,}01477 \cdot 10^{-3}$ m (Element 111 at Node 64)

Minimum value = $-1{,}034 \cdot 10^{-6}$ m (Element 111 at Node 69)

Figure 6.8 Total vertical displacements



Total displacements u_x

Maximum value = $4{,}643 \cdot 10^{-6}$ m (Element 111 at Node 67)

Minimum value = $-4{,}694 \cdot 10^{-6}$ m (Element 111 at Node 64)

Figure 6.9 Total horizontal displacements

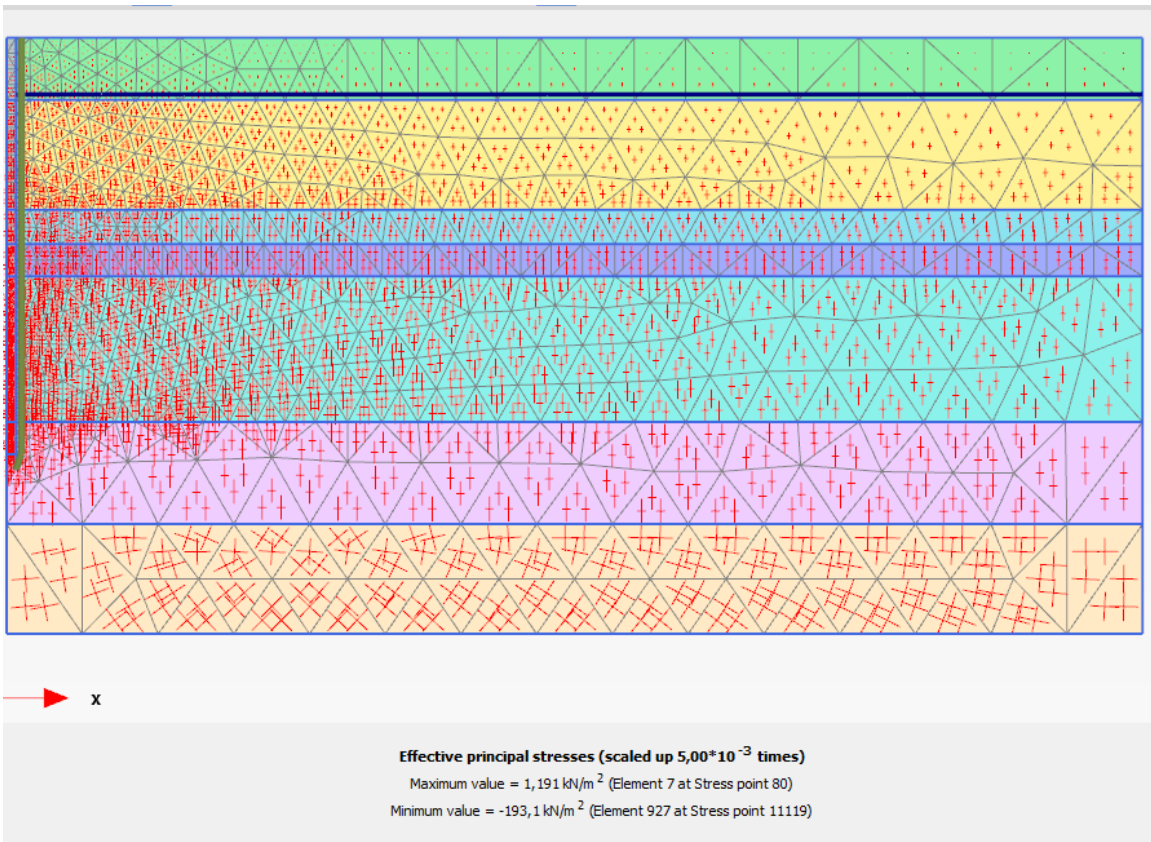


Figure 6.10 Effective principal stresses

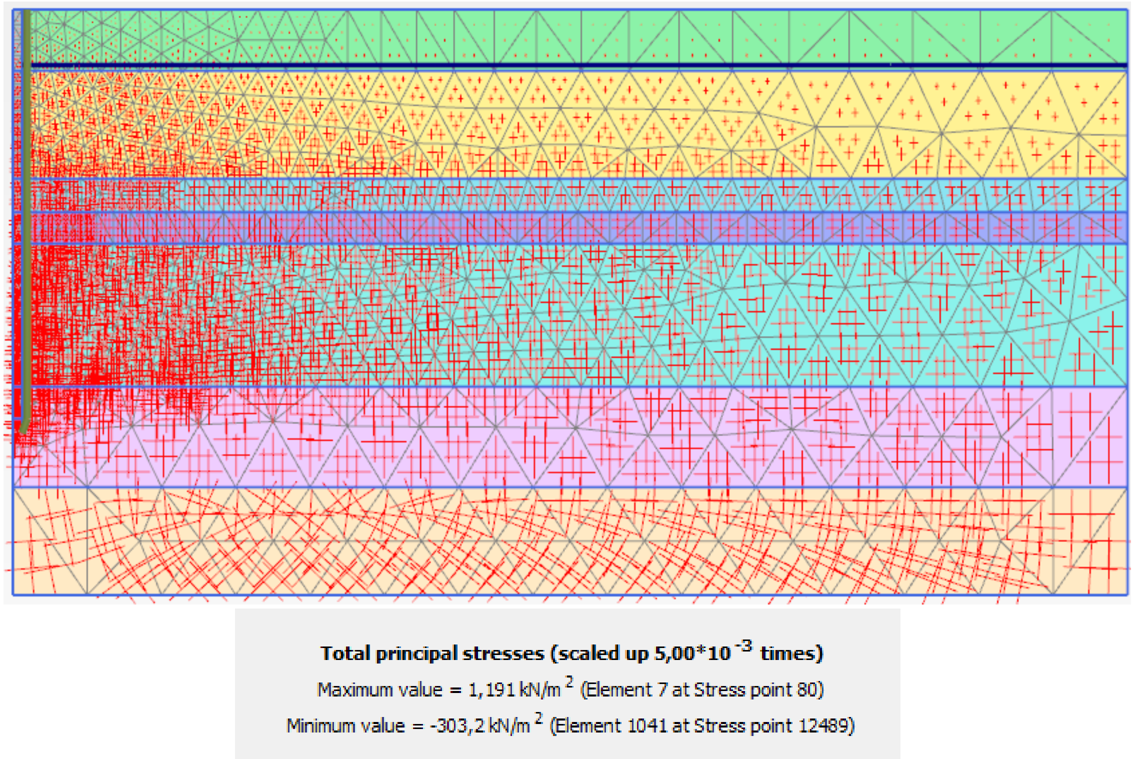


Figure 6.11 Total principal stresses

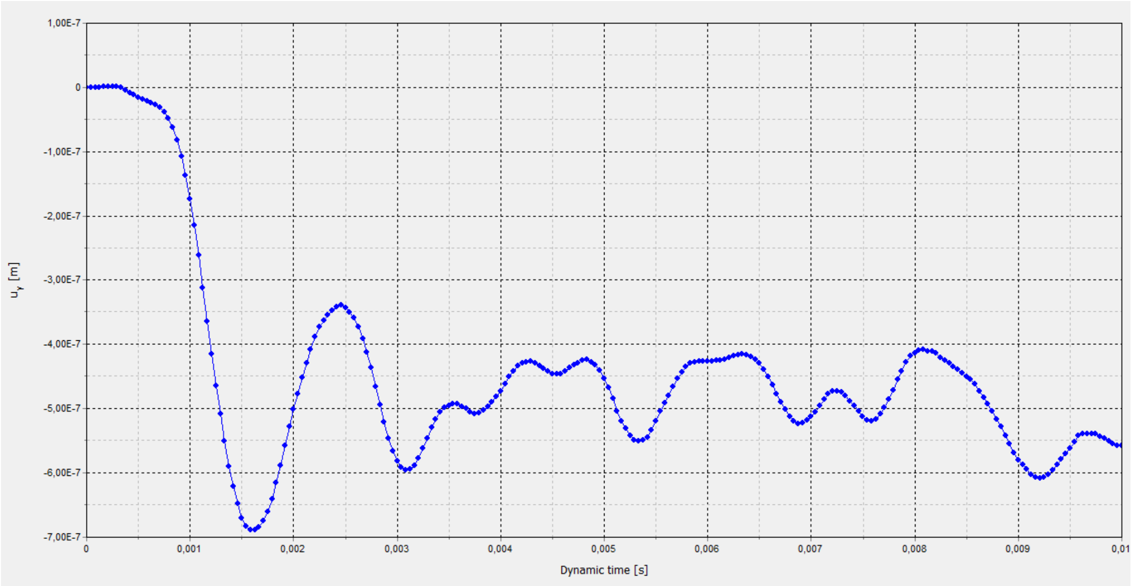


Figure 6.12 Vertical displacement vs dynamic time

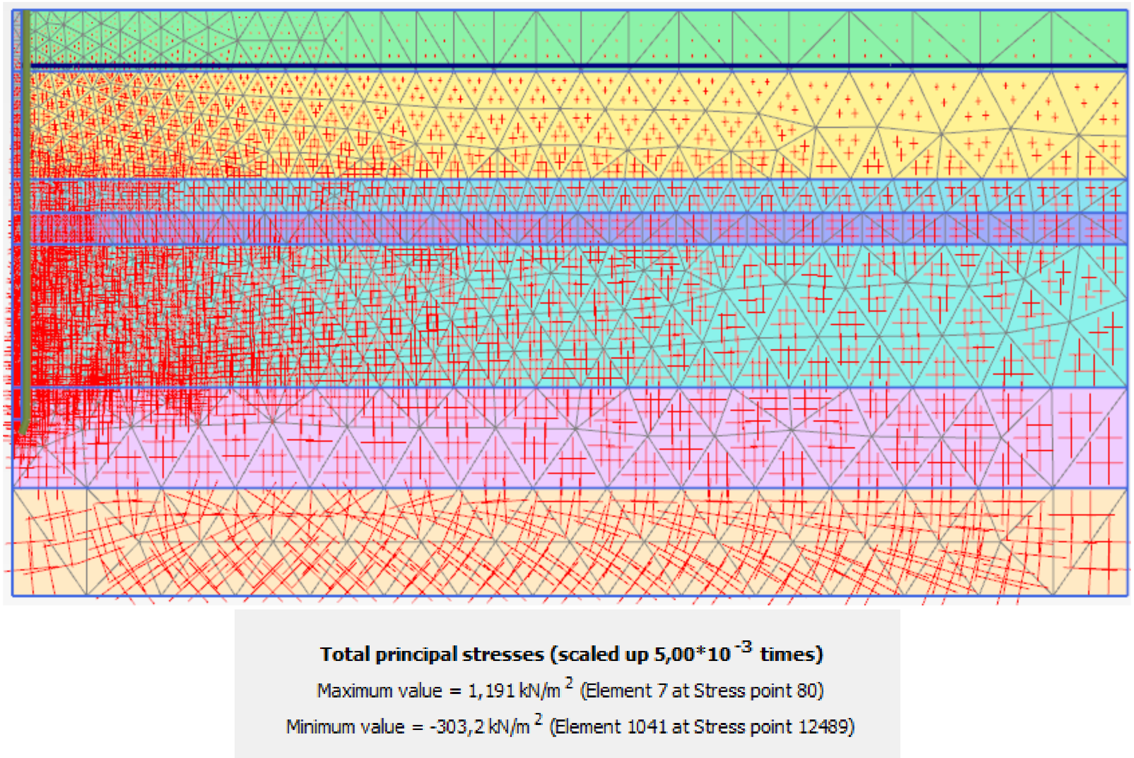


Figure 6.13 Total principal stresses

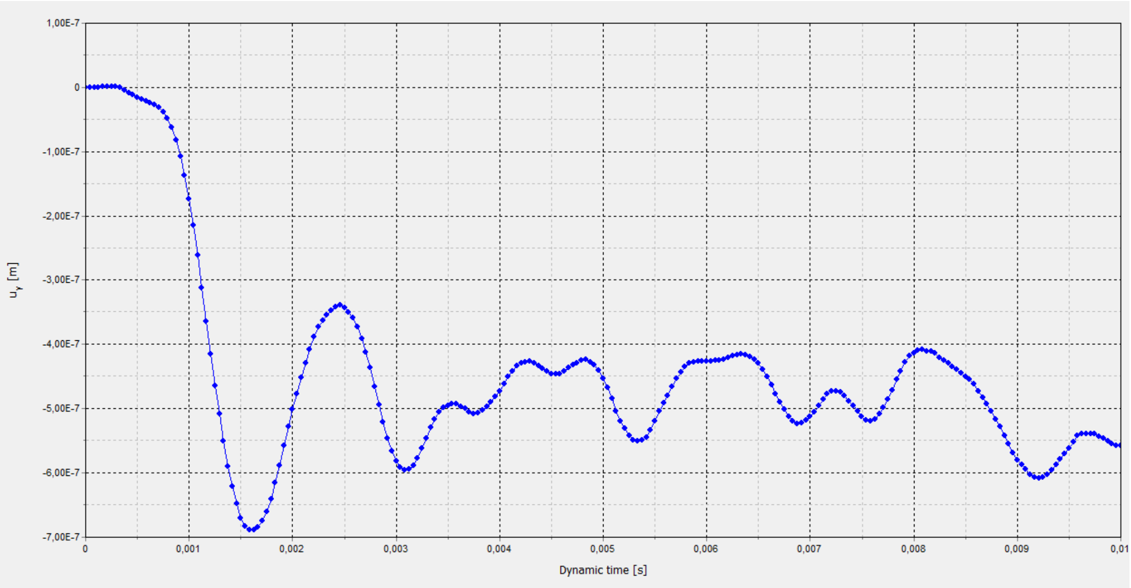


Figure 6.14 Vertical displacement vs dynamic time

7 Construction Management

7.1 Construction planning

Project Management (PM) is assemble of disciplines, which is principally used to initiate, plan, and control and deliver resources and budget of a project to accomplish particular success and correspond to special achievement criteria. From 1950 numerous organizations began to intentionally use Project Management (PM) Tools and methods for complex projects not only in engineering sphere, but also applied science, economics, as well as in sport. Every composite project apply Project Management Tools, including Gantt chart, WBS (Work Breakdown Structure), Cost Estimation and Resource allocation, etc., to complete the project within the certain time and cost limitation. Further sections of the project include the Gantt chart, WBS and Cost estimation of residence building at Nazarbayev University (Capstone-II).

Gantt chart is one of the most approved methods of Project Management which illustrates the diagram of project activities or tasks against time. Briefly, it's a graphical representation of project schedule. The Gantt chart of Capstone-II project is described in Figure 7.1 below.

Essentially, the Gantt chart is used to complete project job activities in proper time. Therefore, generally, it is constructed before project start. Consequently, DC Group decided to develop preliminary Gantt chart for construction of the residential building. Figure 7.2 illustrates the Gantt chart of residential building in Nazarbayev University by DC Group.

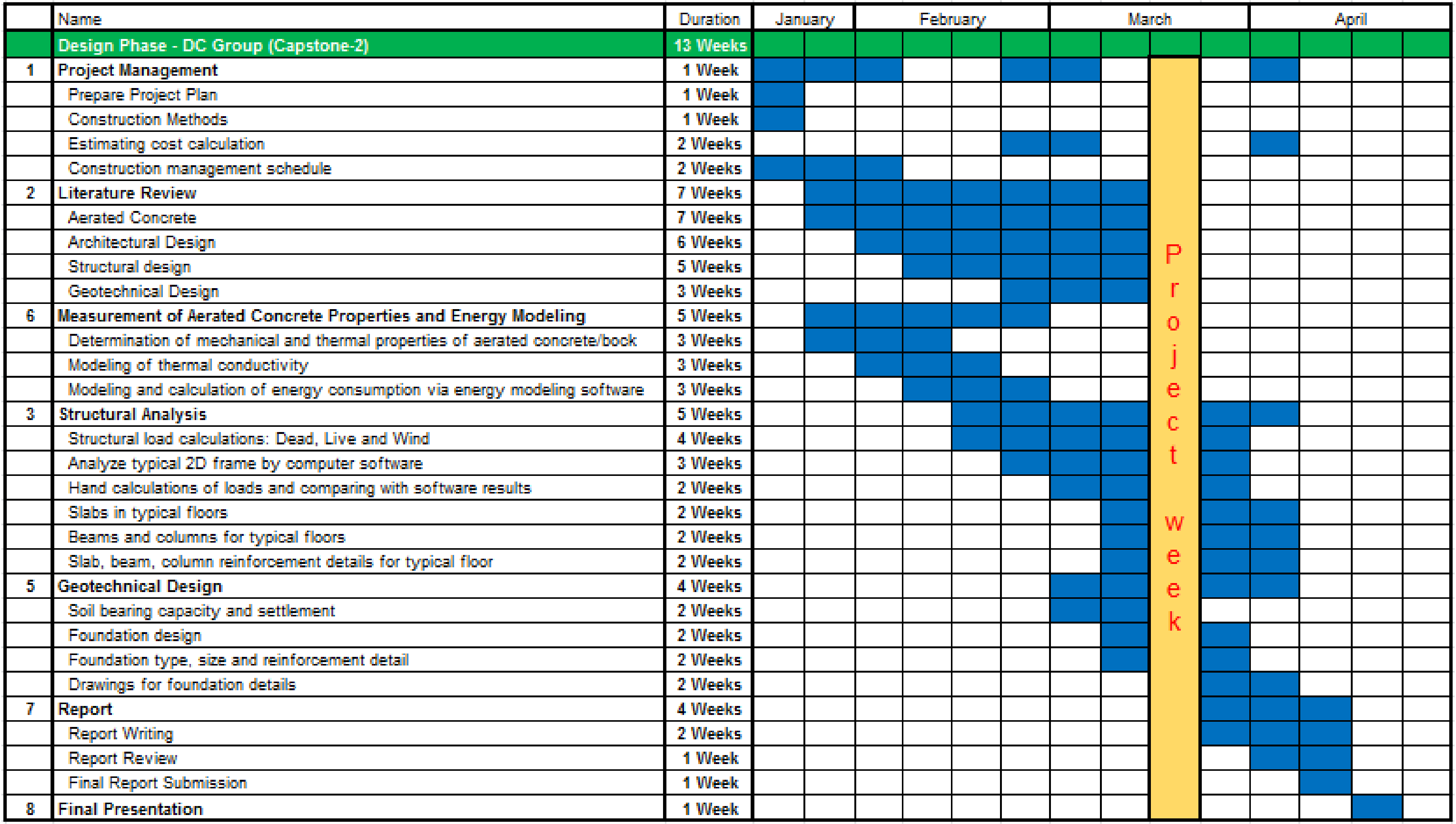


Figure 7.1. The Gantt chart of Capstone-II

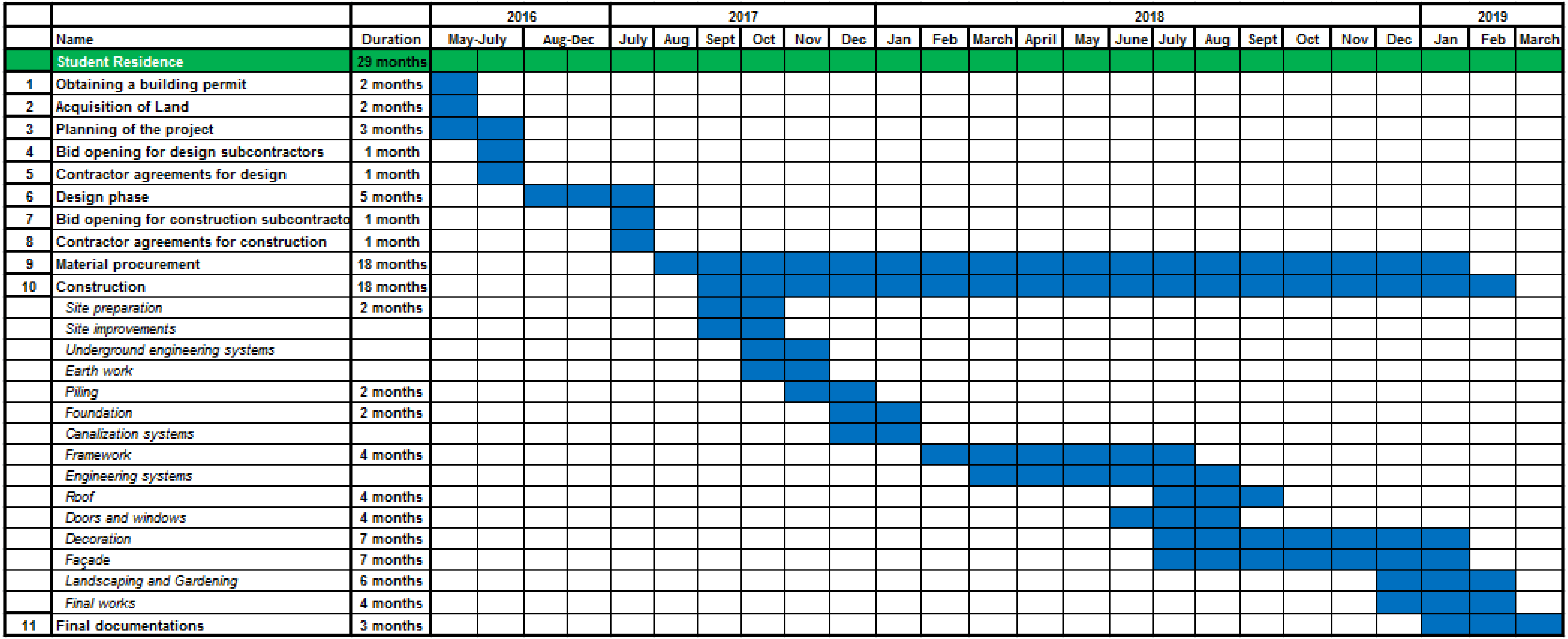


Figure 7.2. The Gantt chart of construction of residential building in Nazarbayev University by DC Group

The Gantt chart in Figure 7.2 illustrates that the duration of the construction phase of the residence building in Nazarbayev University is approximately 18 months. The data was taken from BI Group company estimations. The Figure 7.3 estimates the minimum and maximum durations of constructed buildings by the company in the last two years. Therefore, for the duration of 14 story residential building (18 months) DC Group calculated the mean value of 15 and 21 months.

Proposal of BI Group for construction duration

Number of stores	units.	Min.	Max.	Proposal		
				for 2020	for 2021	на 2022 (Under ideal conditions)
under 9	мес.	13	17	13	12	10
10	мес.	13	17	13	12	10
11	мес.	14	19	14	13	11
12	мес.	14	20	14	13	12
13	мес.	[illegible]	[illegible]	15	14	12
14	мес.	15	21	15	14	12
15	мес.	[illegible]	21	16	15	13
16	мес.	18	22	18	17	14
17	мес.	18	24	18	17	15
18	мес.	18	24	18	17	15
19	мес.	19	25	19	18	16
20	мес.	19	27	19	18	16
21	мес.	20	29	20	19	17
22	мес.	21	30	21	20	18

Figure 7.3 Proposal of BI Group for construction duration

A Work Breakdown Structure (WBS) is a separation of project activities into small part with the purpose of perfect completion. In Capstone-II DC Group constructed the WBS and divided work allocations among team members. For a complex project to be properly controlled and described it's required to construct its work breakdown structure. Numerous WBS could be built with different structure and management styles by aiming the same project completion. Therefore, WBS has considerable influence on the process of successfully project management and completion. WBS of every project has to be designed before the start of construction, usually in a stage of design (Globerson, 1994). In majority situations WBS likewise assist to identify the relationships between employees. Figure 7.4 below illustrates the WBS of DC Group for Capstone-II.

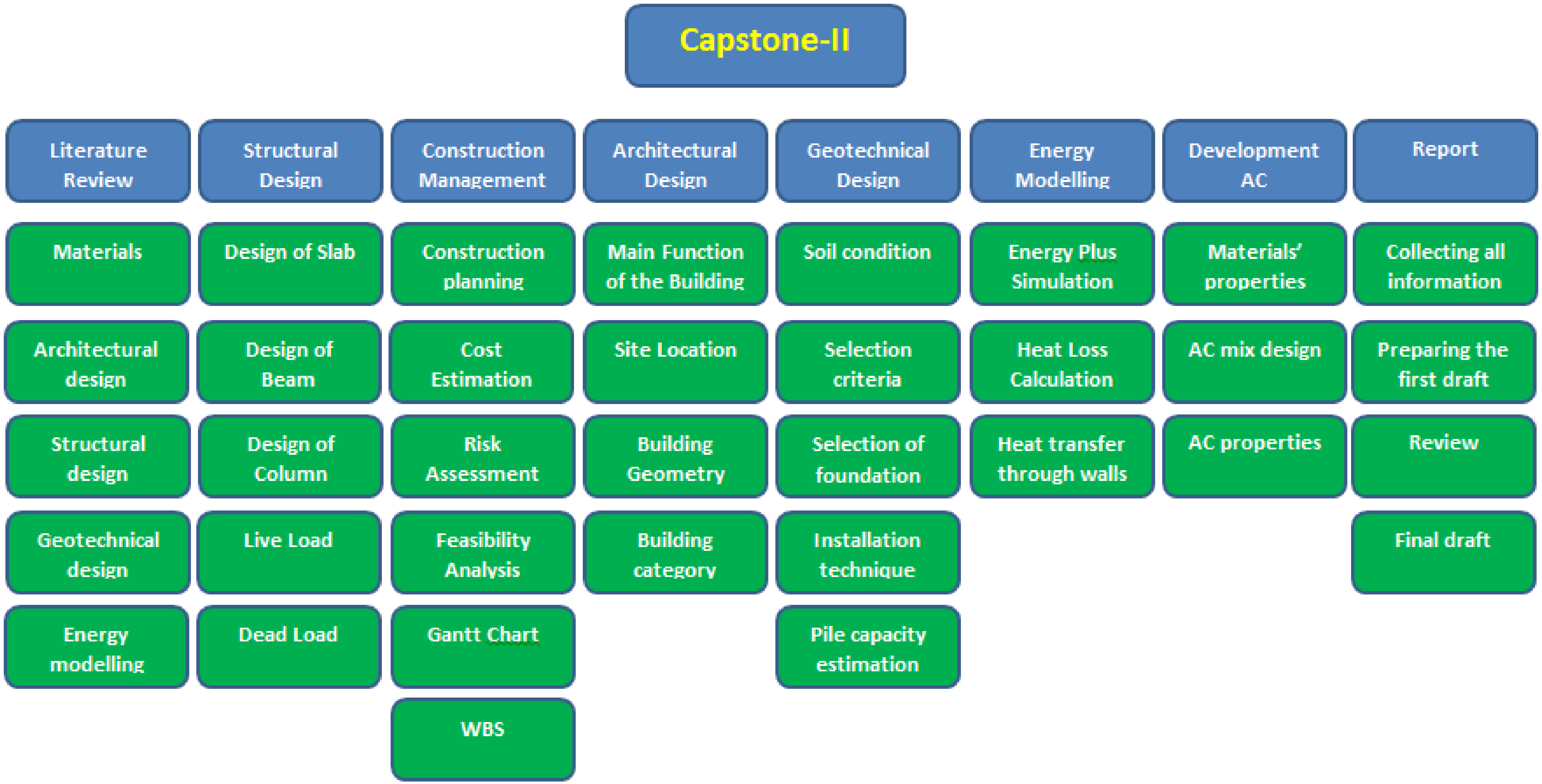


Figure 7.4 The WBS of Capstone-II

7.5

7.2 Cost Estimation.

During the design phase of Capstone-II all expenses are approximately predicted and upgraded in order to secure budget dividend of owner (Sears et al, 2008). Furthermore, cost estimation of the project in the phase of design ensures the correct convention between owner and contractors (Peurifoy and Oberlender, 2008). The following parts of the report describe the preliminary cost assessment procedures, total cost and material estimations of residence.

The primary tool that applied to calculate the total cost of construction of the building is Microsoft Project Software (MS Project). By providing the resource allocations in terms of materials and employees and durations of every activity the total cost of the residence is automatically estimated as shown in Table 7.1. As software indicates, the total estimation of the building is 2 709 540 995,00 KZT. The table also presents the distribution of each materials and labors to individual activities.

The cost calculations for materials to be used in entire residence are assessed as shown in Table 7.2. The labor costs of the project are calculated by installing the display of the software as 20 working days in months and 8 hours per day. Furthermore, the software also estimates the techniques that are utilized in construction site. On the other hand, the individual types of materials and labors are also described in Table 7.3 below.

Table 7.1 Cost estimation of residence building in Nazarbayev University by using MS Project

Name	Duration	Start	Finish	Resource Names	Cost
Student residence building	29 months	01.08.16	30.02.19	Foreman; Project Manager; Section Chief; Accountant; Financier; HR; Lawyer; Safety; Security; AC; Canalization systems; RC; Canteen devices; Decoration Materials; Doors; Earthwork machines; Electrical devices; Engineering networks; Façade Workers;	T2 709 540 995,00
Design Phase	5 months	01.08.16	30.12.16	Financier; Foreman; HR; Lawyer; Project Manager; Section Chief	T1 128 000,00
Earth works	2 months	01.09.17	30.10.17	Earthwork machines; Foundation Workers; Pile driver; Piles; RC; Rebar; Canalization systems; Engineering networks; Palification Workers; Safety	T808 839 200,00
Foundation	2 months	01.12.17	30.01.18	Foundation Workers; Palification Workers; Piles	T59 168 000,00
Framework	4 months	01.02.18	30.05.18	Frame Workers; AC; RC; Rebar	T677 400 000,00
Filling window apertures	4 months	01.04.18	30.07.18	Window and Door Installers	T3 320 000,00
Roof	4 months	01.03.18	30.16.18	Roof Workers; Roof materials	T350 571 895,00
Decoration works	7 months	01.06.18	30.12.18		T632 966 400,00
Facade	7 months	01.07.18	30.01.19	Façade Workers; Limestone for Façade	T91 147 500,00
Finishing works	4 months	01.11.18	30.02.19	Gardening materials; Road construction	T85 000 000,00

Table 7.2 Cost estimation of individual materials

№	Material Type	Unit of measurement	Amount	Cost for unit (tenge)	Overall for material type
1	Pile	piece	588	14 000	8 232 000
2	NC	m3	6 256,00	16 000	100 096 000
3	Limestone	m2	6450	8 550	55 147 500
4	Rebar	kg	156 000	130	20 280 000
5	AC	m3	5686,74	9 800	55 730 000
6	Window	piece	374	48 000	17 952 000
7	Door	piece	548	32 000	17 536 000
8	Engineering network	-	1	170 000 000	170 000 000
9	Electrical devices	-	1	105 000 000	105 000 000
10	Gardening materials	-	1	10 000 000	10 000 000
11	Lifting systems	-	1	50 000 000	50 000 000
12	Sanitary wares	-	1	45 000 000	45 000 000
13	Roof Materials	-	1	150 000 000	150 000 000
14	Canalisation systems	-	1	60 000 000	60 000 000
	Overall			665 128 480	819 973 500

Table 7.3 Cost distribution between individual activities of residence construction in Nazarbayev University

Resource Name	Type	Initials	Units	Std. Rate
Project Manager	Work	P	1	T5 000,00/hr
Section Chief	Work	S	1	T2 700,00/hr
Foreman	Work	F	2	T2 000,00/hr
Master	Work	M	3	T1 500,00/hr
Palification Workers	Work	P	40	T1 000,00/hr
Foundation Workers	Work	F	40	T1 000,00/hr
Frame Workers	Work	F	50	T1 000,00/hr
Window and Door Installers	Work	W	20	T1 000,00/hr
Roof Workers	Work	R	30	T1 000,00/hr
Façade Workers	Work	F	50	T1 000,00/hr

HR	Work	H	1	T1 500,00/hr
Lawer	Work	L	1	T2 300,00/hr
Security	Work	S	4	T2 800,00/hr
Accountant	Work	A	1	T2 000,00/hr
Financier	Work	F	1	T2 500,00/hr
Safety	Work	S	2	T2 400,00/hr
Piles	Material	P		T8 064 000,00
Limestone for Façade	Material	L		T55 147 500,00
Pile driver	Work	P	2	T19 000,00/hr
Earthwork machines	Work	E	2	T10 000,00/hr
Cranes	Work	C	2	T15 000,00/hr
Bulldozer	Work	B	2	T13 000,00/hr
Trucks	Work	T	3	T15 000,00/hr
Loader	Work	L	2	T15 000,00/hr
Backhoe	Work	B	2	T15 000,00/hr
Tracked Excavator	Work	T	2	T16 000,00/hr
Concrete Mixer Trucks	Work	C	3	T15 000,00/hr
Mortar Mixer	Work	M	2	T14 000,00/hr
Rebar	Material	R		T20 280 000,00
Windows	Material	W		T17 952 000,00
Doors	Material	D		T17 536 000,00
Engineering networks	Material	E		T170 000 000,00
Electrical devices	Material	E		T105 000 000,00
Gardening materials	Material	G		T10 000 000,00
Lifting Systems	Material	L		T50 000 000,00
Sanitary wares	Material	S		T45 000 000,00
Roof materials	Material	R		T150 000 000,00
Canalization systems	Material	C		T60 000 000,00
Road construction	Material	R		T5 000 000,00

Inside decoration of the residence is calculated applying average local assessments of finishing works by BI Group Company. The data is taken from BI Group according to the statistic that the company builds approximately 40% of construction in Astana (BI, 2016). Company's estimation for inside decoration is illustrated in Table 7.4.

Table 7.4 Average cost estimation of BI Group for decoration works in 2016

№	Type of work	Cost estimation of work, tenge							Cost for 1 м^2
		1 floor	2 floor	3 floor	4 floor	5 floor	6 floor	Overall:	
1	Arrangement of partitions of drywall	1 496 362	1 450 899	1 450 899	1 450 899	1 450 899	1 450 899	**8 750 858**	25 344
2	Decorating the walls (roughing)	2 826 461	3 419 977	3 419 977	3 419 977	3 419 977	3 419 977	**19 926 346**	
3	Wall decoration (finishing)	1 579 493	2 176 349	2 176 349	2 176 349	2 176 349	2 176 349	**12 461 238**	
4	Ceiling Finishes (roughing)	748 181	932 721	932 721	932 721	932 721	932 721	**5 411 786**	
5	Ceiling Finish (finishing)	415 656	518 178	518 178	518 178	518 178	518 178	**3 006 548**	
6	Arrangement of floors (finishing)	1 205 402	1 658 171	1 658 171	1 658 171	1 658 171	1 658 171	**9 496 256**	
7	Installation of doors	41 566	207 271	207 271	207 271	207 271	207 271	**1 077 922**	
	Overall:	**8 313 120**	**10 363 567**	**10 363 567**	**10 363 567**	**10 363 567**	**10 363 567**	**60 130 953**	

As shown in the table above, inside decoration of a building for 1m^2 costs approximately 25 344 Tenge. Consequently, for the residence building in Astana by DC Group it could be estimated as:

Overall Cost of inside decoration = Overall area of the building * Cost for 1m^2 =

= 24 975 m^2 * 25 344 Tenge/m^2 = 632 966 400 KZT.

The 12% tax for Kazakhstan is installed in MS Project software, consequently the program automatically calculated the tax.

Since the final cost estimation of entire project is developed before construction of the residence, this report of Capstone-II indicates the approximate calculations of cost analysis. As mentioned before, cost estimation of residential building assessed approximately and upgraded to safety. However, the budget (2 788 355 695,00 KZT) calculated by DC Group compared with other similar Astana city buildings and checked by the BI Group Company (the leading construction company in Kazakhstan) project managers to ensure and prevent the over and underestimation.

7.3 Risk Assessment

Construction complex projects suggest high dangers connected with any system inside the design, in this way hazard management is extensive reaching way to recognize, investigate, moderate or dispose those dangers to achieve successful goals. Initially, risk management aims to investigate and prevent the serious consequences of activities (Banaitiene & Banaitis, 2012). Therefore, the primary purpose of risk management is to prevent hazards in construction areas. On the other way, risk management is fundamental tool for owners to decrease project cost, time and quality negative influences. Cost and time are directly affected quality of a building, and thus there are several representations of Risk Assessments of a project. One of the generally utilized assessments is Risk Severity Matrix (Figure 7.5).

Likelihood	5	PS3	PS1	E1	F3	C1
	4	PS2	PS6	PS7	PS4	C6
	3	E4	E5	C5	F5	C2
	2	PS9	F2	PS8	C4	E2
	1	PS5	F4	C3	E3	F1
		1	2	3	4	5
		Impact				

Figure 7.5 Risk Severity Matrix

Risk severity matrix produces the representations of risks according to likelihood and impact. The vertical lanes indicate the probability, while horizontal lanes define outcomes of risks. The red zones are the risky regions, by changing to orange, yellow and green, which mean less likelihood and have negligible impact of risks on final result (Larson, 2011). Description of likelihood and impact numbers of Risk severity matrix implies:

Impact: 1- negligible; 2-minor; 3-moderate; 4-major; 5-severe

Likelihood: 1- rare; 2-unlikely; 3-possible; 4-likely; 5-almost certain

Moreover, risk matrix severity allows calculating the Risk value. The main formula for risk value assessment could be stated as:

$$\text{Impact x Probability} = \text{Risk Value} \qquad \text{(Eq. 5.3.1)}$$

Four primary risk classes of complex construction projects are demonstrated in Tables D1, D2, D3, D4 in Appendix D. Risk classes include construction, design, PM and environmental categories. The tables indicate that human health and structural and geotechnical construction error risks have more Risk values, 25 and 15 respectively.

Therefore, despite safety for human health takes complementary time during construction, it deserves significant attention. Green regions of Risk severity matrix has negligible impact and fewer tendencies to occur, consequently takes insignificant attention.

Finally, effective Risk management plan should be constructed before every complex project and maintained during entire construction life. Errors in financial costs and delaying of duration of a project is prevented by Risk management plan, and quality as well controlled by management tools.

7.4 Feasibility Analysis

Feasibility analysis is applied to estimate the advantages and disadvantages of the project and demonstrates the ways of activities which bring the project to successful completion. The constituent elements of feasibility analysis depend on type and scope of a project. Actually, efficient feasibility study consists of 3 steps: project evaluation using selected capabilities of feasibility analysis, summary of evaluation results and recommendations. The Figure 7.6 demonstrates workflow of feasibility analysis steps and its content.

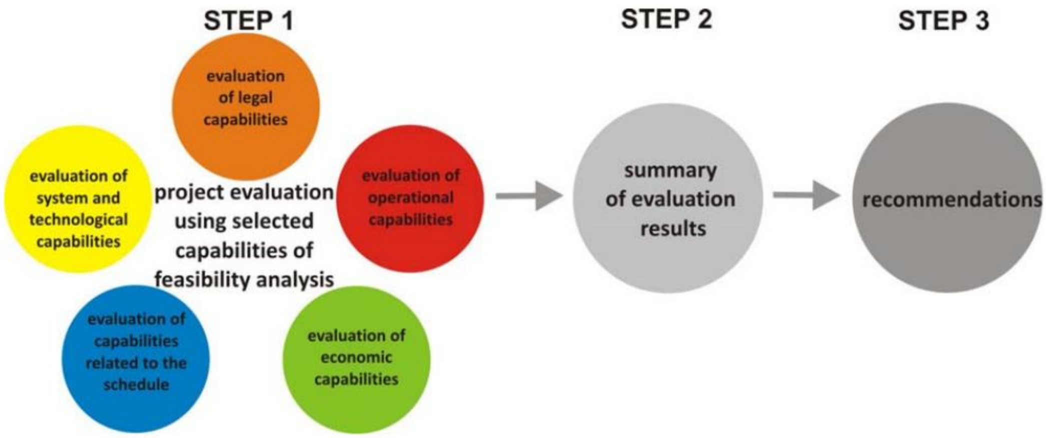


Figure 7.6 Workflow in feasibility analysis

As shown in Figure 7.6, selected capabilities of feasibility analysis a project give sufficiently reasons to summarize and evaluate the project. Separated evaluation capabilities from the figure above initiate to construct the Feasibility analysis matrix as shown in Table 7.5. The table is constructed to choose the material type of the building. According to the advantages and disadvantages of three candidate materials, investigating new mixture design of AC for residence building in Nazarbayev University is the most feasible selection.

Table 7.5 Feasibility analysis matrix

Feasibility criteria	W	Candidate1: NC	Candidate2: AC that already used in industry	Candidate3: AC with new mixture design
Operational feasibility	30%	Difficult to build on site, because of heavy weight of blocks Score: 70	Easy to produce out site and deliver to use to construct the walls, because of light weight Score: 100	Easy to produce out site and deliver to use to construct the walls, because of light weight Score: 100
Technical feasibility	15%	Do not need laboratory equipment to investigate the new design Score: 100	Do not need laboratory to investigate the new design Score: 100	Needs laboratory equipment and materials to test new mixture designs Score: 85
Economic feasibility	30%	Cheaper than AC, however, heavier. Thus, Dead Load of a building will be high and material usage respectively is more. Consequently, expensive. More	Lighter than NC, thus the Dead Load is less than AC. However, more expensive than RC. Heat transfers less than NC. Saves energy	Lighter than RC and cheaper than AC that already used in industry. The cost is less than NC and AC (used). Heat transfers less than
		energy transfers than AC, more energy consumption	inside buildings	NC. Saves energy inside buildings

		Score: 70	Score: 80	Score: 100
Schedule feasibility	5%	Do not required time schedule to investigate new designs Score: 100	Do not required time schedule to investigate new designs Score: 100	Required some time (months, years) to design new mixture and test them for application Score: 90
Risk feasibility	20	No risks Score: 100	No risks Score: 100	Risky, because of possibility of fail Score: 90
Ranking	100%	82%	94%	95,75%

As illustrated in the Table 7.6 the average costs for AC and NC are approximately 9 800 and 2 051.28 KZT respectively for each cubic meter in Astana. The table indicates that constructing the building with AC is expensive for 44 065 265,92 KZT than using the NC.

Table 7.6 AC and NC costs

	Amount (m^3)	**Cost per m^3**	**Total cost (KZT)**
AC	5686,78	9800	55730444
NC	5686,78	2051,28	11665178,08
Difference			44065265,92

During energy simulation shown in Section 4, total site energy consumed by the building in one year was obtained. Knowing average cost of electricity of 1kWh in Astana, it can be calculated that using mix#4 concrete as a material for exterior wall instead of mix#5 results in 14 545 499,38 KZT saving each year. (see Table 7.7)

Table 7.7 AC and NC energy consumption differences

Mixture	Total site energy (kBtu)	Total site energy (kWh)	Cost per kWh (KZT)	Annual Energy Cost (KZT)
Mix4 (3sands)	7878388,3	2308927,69	15	34633915,35
Mix5 (NC)	11187141,9	3278627,649	15	49179414,73
Difference				**-14545499,38**

Consequently, despite that using AC is more expensive than silicate brick, the spent money could be paid off in 3,03 years:

Pay off = 44065265,92/14545499,38 = 3,03 years

Nazarbayev University has 6 students residential buildings for approximately 2 500 students, 416 students for each building. However, each existing residential building is estimated only for 308 students. Thus, almost 26% of students need more residential places in the territory of the University. The building which will be constructed by DC Group is estimated to the capacity of 385 residents. The number of students without dormitory places will be decreased to 12%.

8 Conclusion

In this study, the design of student residence building for master students in Nazarbayev University with the capacity of 385 people is proposed. Besides, to solve the problem, the world faces nowadays, which is the huge energy consumption by buildings (30% in Kazakhstan), using aerated concrete blocks as exterior wall material is suggested in order to provide proper insulation, so the heat loss will be not significant. To prove the energy efficiency, simple energy simulation using Energy Plus software was done as well as heat loss and heat transfer through the wall calculations. Both modelling and hand calculations showed that substituting normal concrete blocks with aerated concrete ones lead to significant energy savings. Moreover, laboratory works, where aerated concrete blocks were casted and tested for mechanical and thermal properties. As a result suitable mixture design was obtained and its properties were used in energy modeling and structural design of the building.

In terms of architectural design, firstly, the exact location was chosen basing on such factors as noise, view and access to academic building and parking area. Then, the building was oriented in a way the sun touches almost every side of the building. Since the designed dormitory is located on the campus of one of the most prestigious universities in country, where different conferences and forums are often held, to provide aesthetically pleasant and unusual view of 14 story dormitory, a unique shape building comprised of 4 L-shape towers with different heights is proposed.

Regarding structural design of the building, due to the fact that Kazakhstan officially accepted Eurocode, deep review of the code was done and structural member design was conducted according to the guidelines provided in the code. Furthermore, SAP2000 software was used to build the model of the building and get necessary internal forces and stresses appeared in structural members of the building.

By reviewing a lot of literature and considering soil condition of Nazarbayev University area, it was decided to use precast concrete piles as a foundation type, since the dormitory is considered as a high rise building and requires high load capacity. Foundation was designed separately for each section, in order to avoid cracks due to settlement. Design was checked for punching shear stress and maximum allowable settlement, and it was considered that design is acceptable.

Moreover, cost estimation was done including all possible expenses for construction materials, human resources, and necessary equipment. The total cost of construction of such student residence building was estimated to be 2 788 355 695,00 KZT. Additionally, using AC blocks instead of normal concrete makes the construction more expensive for 44 065 265,92 KZT. However, Energy Plus software analysis showed that by using AC Block 14 545 499,38 KZT can be saved each year and additional expenses will be paid off in 3.03 years.

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10 Appendix A

10.1 Particle size distribution of sand from Karasar (yellow)

Table 10.1. Sieve analysis of yellow sand 1

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve+ agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	562,0	4,0	0,4%	0,4%	99,6%
#16	1,18	497,0	508,0	11,0	1,1%	1,4%	98,6%
#30	0,6	465,0	568,0	103,0	9,9%	11,4%	88,6%
#50	0,3	427,0	1120,0	693,0	66,9%	78,3%	21,7%
#100	0,15	430,0	643,0	213,0	20,6%	98,8%	1,2%
#200	0,075	418,0	429,0	11,0	1,1%	99,9%	0,1%
Pan	0	402,0	403,0	1,0	0,1%	100,0%	0,0%
Total Retained Mass(g) =				1 036,0			
Original Sample Mass(g) =				1 036			
Percent Loss(%) =				0,0%			
Fineness Modulus (FM) =					1,90		

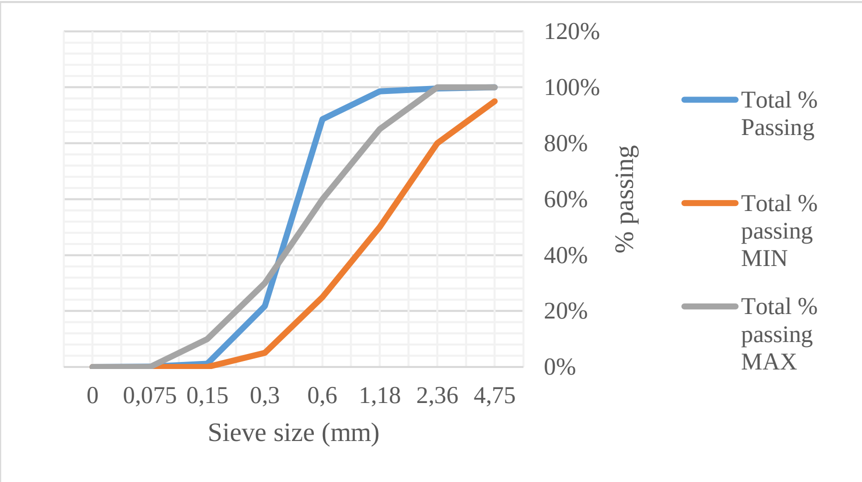


Figure 10.1. Particle size distribution of yellow sand 1

Table 10.2. Sieve analysis of yellow sand 2

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve+ agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing

#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	561,0	3,0	0,3%	0,3%	99,7%
#16	1,18	497,0	508,0	11,0	1,0%	1,2%	98,8%
#30	0,6	465,0	579,0	114,0	9,9%	11,1%	88,9%
#50	0,3	427,0	1208,0	781,0	68,0%	79,2%	20,8%
#100	0,15	430,0	656,0	226,0	19,7%	98,9%	1,1%
#200	0,075	418,0	430,0	12,0	1,0%	99,9%	0,1%
Pan	0	402,0	403,0	1,0	0,1%	100,0%	0,0%
Total Retained Mass(g) =				1 148,0			
Original Sample Mass(g) =				1 148,0			
Percent Loss(%) =				0,0%			

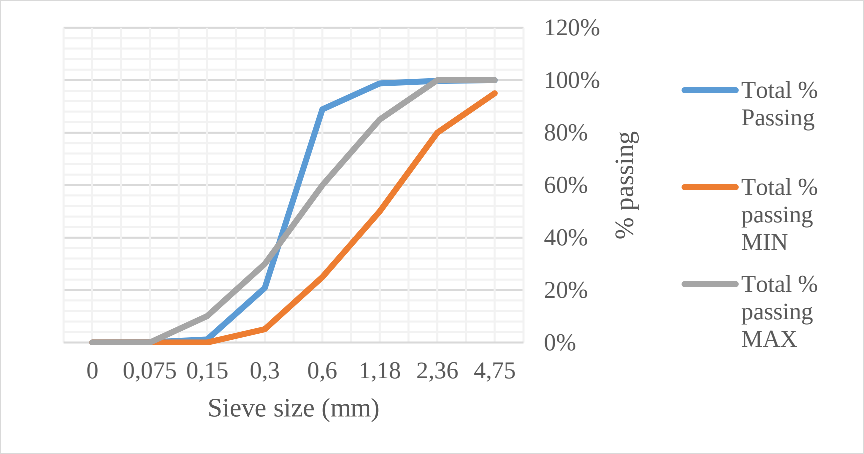


Figure 10.2. Particle size distribution of yellow sand 2

Table 10.3. Sieve analysis of yellow sand 3

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve+ agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	562,0	4,0	0,3%	0,3%	99,7%
#16	1,18	497,0	511,0	14,0	1,1%	1,4%	98,6%
#30	0,6	465,0	598,0	133,0	10,0%	11,4%	88,6%
#50	0,3	427,0	1360,0	933,0	70,3%	81,6%	18,4%
#100	0,15	430,0	659,0	229,0	17,2%	98,9%	1,1%
#200	0,075	418,0	432,0	14,0	1,1%	99,9%	0,1%
Pan	0	402,0	403,0	1,0	0,1%	100,0%	0,0%
Total Retained Mass(g) =				1 328,0			

Original Sample Mass(g) =	1 328,0
Percent Loss(%) =	0,0%
Fineness Modulus (FM) =	1,94

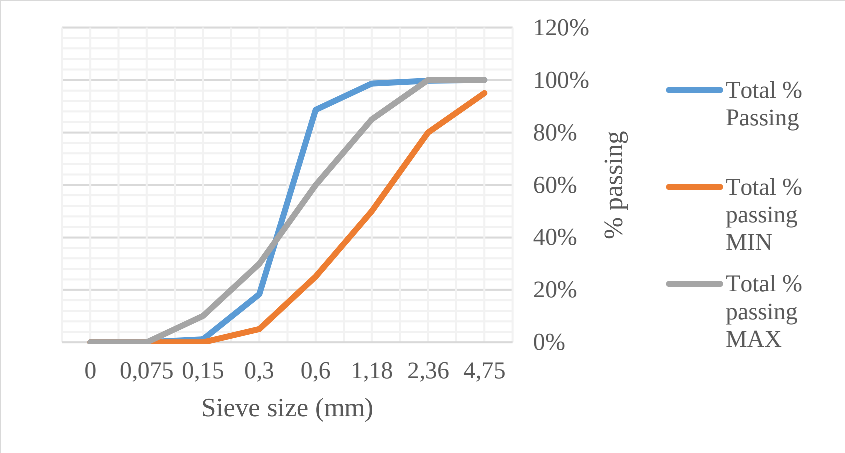


Figure 10.3. Particle size distribution of yellow sand 3

10.2 Particle size distribution of sand from Red Flag (brown)

Table 10.4. Sieve analysis of brown sand 1

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve+ agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	634,0	76,0	8,3%	8,3%	91,7%
#16	1,18	497,0	550,0	53,0	5,8%	14,1%	85,9%
#30	0,6	465,0	500,0	35,0	3,8%	17,9%	82,1%
#50	0,3	427,0	450,0	23,0	2,5%	20,4%	79,6%
#100	0,15	430,0	759,0	329,0	35,9%	56,3%	43,7%
#200	0,075	418,0	764,0	346,0	37,7%	94,0%	6,0%
Pan	0	402,0	457,0	55,0	6,0%	100,0%	0,0%
Total Retained Mass(g) =				917,0			
Original Sample Mass(g) =				917			
Percent Lost(%) =				0,0%			
Fineness Modulus (FM) =					1,17		

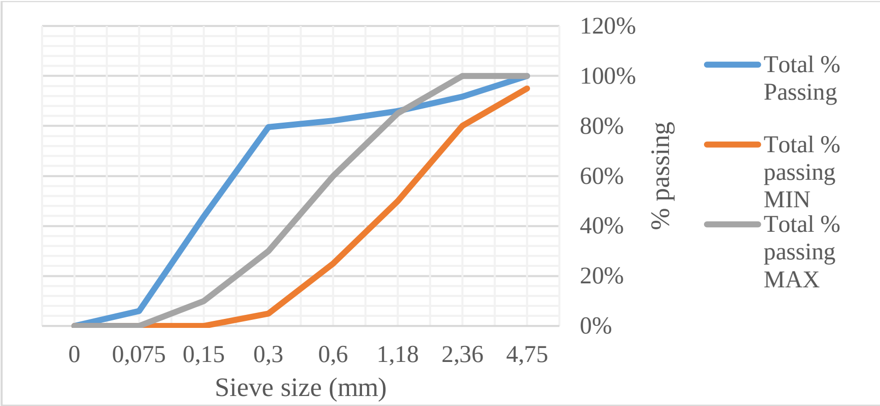


Figure 10.4. Particle size distribution of brown sand 1

Table 10.5. Sieve analysis of brown sand 2

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve+ agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	639,0	81,0	7,9%	7,9%	92,1%
#16	1,18	497,0	545,0	48,0	4,7%	12,5%	87,5%
#30	0,6	465,0	504,0	39,0	3,8%	16,3%	83,7%
#50	0,3	427,0	456,0	29,0	2,8%	19,1%	80,9%
#100	0,15	430,0	814,0	384,0	37,2%	56,4%	43,6%
#200	0,075	418,0	811,0	393,0	38,1%	94,5%	5,5%
Pan	0	402,0	459,0	57,0	5,5%	100,0%	0,0%
Total Retained Mass(g) =				1 031,0			
Original Sample Mass(g) =				1 031,0			
Percent Loss(%) =				0,0%			
Fineness Modulus (FM) =					1,12		

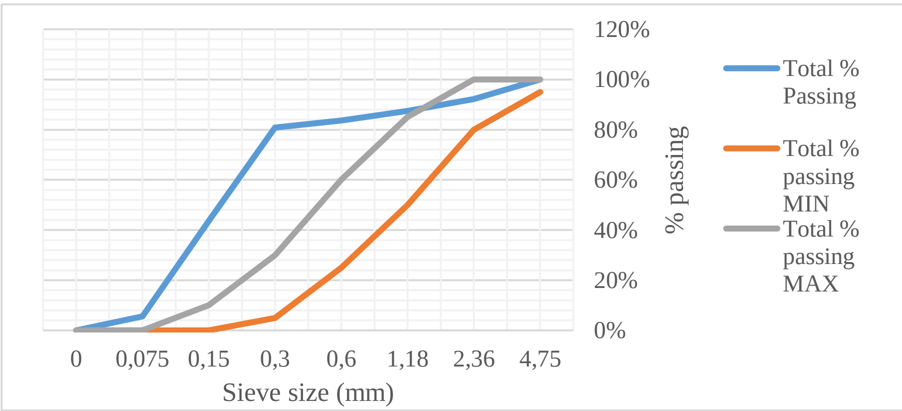


Figure 10.5. Particle size distribution of brown sand 2

Table 10.6. Sieve analysis of brown sand 3

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve+ agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	597,0	39,0	5,2%	5,2%	94,8%
#16	1,18	497,0	516,0	19,0	2,5%	7,7%	92,3%
#30	0,6	465,0	485,0	20,0	2,7%	10,4%	89,6%
#50	0,3	427,0	444,0	17,0	2,3%	12,6%	87,4%
#100	0,15	430,0	634,0	204,0	27,1%	39,7%	60,3%
#200	0,075	418,0	803,0	385,0	51,1%	90,8%	9,2%
Pan	0	402,0	471,0	69,0	9,2%	100,0%	0,0%
Total Retained Mass(g) =				753,0			
Original Sample Mass(g) =				753,0			
Percent Loss(%) =				0,0%			
Fineness Modulus (FM) =					0,76		

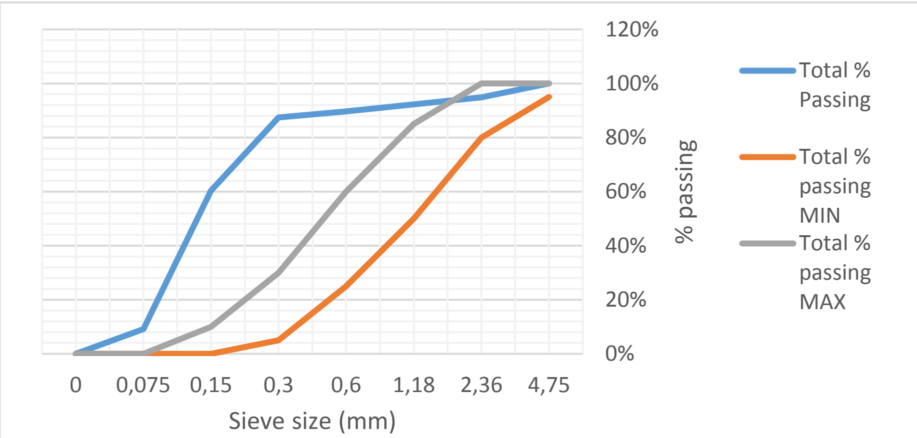


Figure 10.6. Particle size distribution of brown sand 3

10.3 Particle size distribution of sand from Korgalzhyn (grey)

Table 10.7. Sieve analysis of grey sand 1

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve + agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	684,0	126,0	12,3%	12,3%	87,7%

#16	1,18	497,0	773,0	276,0	27,0%	39,3%	60,7%
#30	0,6	465,0	843,0	378,0	37,0%	76,3%	23,7%
#50	0,3	427,0	610,0	183,0	17,9%	94,2%	5,8%
#100	0,15	430,0	477,0	47,0	4,6%	98,8%	1,2%
#200	0,075	418,0	428,0	10,0	1,0%	99,8%	0,2%
Pan	0	402,0	404,0	2,0	0,2%	100,0%	0,0%
Total Retained Mass(g) =				1 022,0			
Original Sample Mass(g) =				1 022			
Percent Lost(%) =				0,0%			
Fineness Modulus (FM) =					3,21		

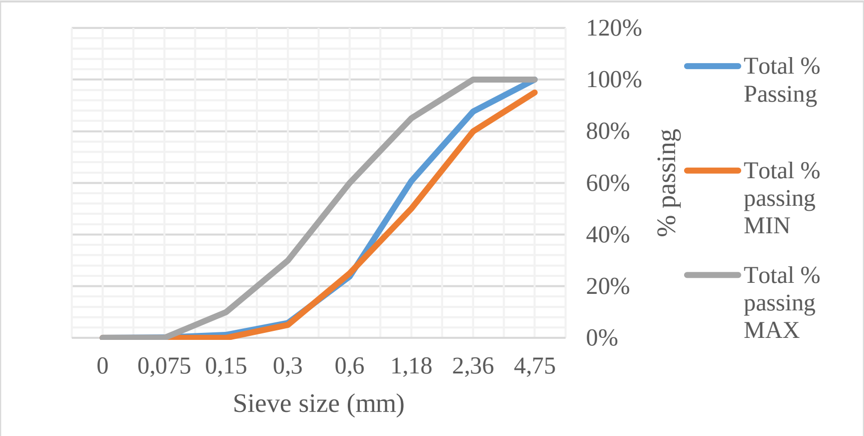


Figure 10.7. Particle size distribution of grey sand 1

Table 10.8. Sieve analysis of grey sand 2

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve + agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	680,0	122,0	11,9%	11,9%	88,1%
#16	1,18	497,0	764,0	267,0	26,1%	38,0%	62,0%
#30	0,6	465,0	830,0	365,0	35,6%	73,6%	26,4%
#50	0,3	427,0	620,0	193,0	18,8%	92,5%	7,5%
#100	0,15	430,0	491,0	61,0	6,0%	98,4%	1,6%
#200	0,075	418,0	432,0	14,0	1,4%	99,8%	0,2%
Pan	0	402,0	404,0	2,0	0,2%	100,0%	0,0%
Total Retained Mass(g) =				1 024,0			
Original Sample Mass(g) =				1 024,0			

Percent Lost(%) =	0,0%
Fineness Modulus (FM) =	3,14

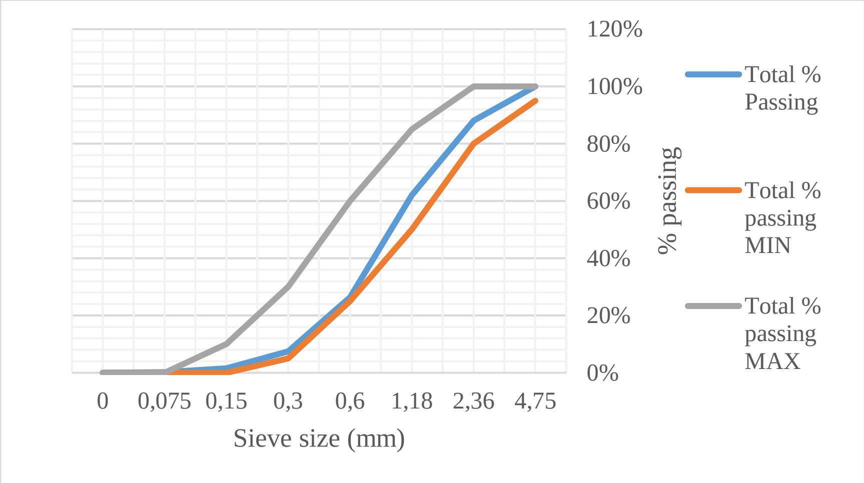


Figure 10.8. Particle size distribution of grey sand 2

Table 10.9. Sieve analysis of grey sand 3

Sieve size	Sieve size(mm)	Mass Sieve(g)	Mass Sieve + agg. (g)	Mass Retained (g)	% Retained on Sieve	Total % Retained	Total % Passing
#4	4,75	476,0	476,0	0,0	0,0%	0,0%	100,0%
#8	2,36	558,0	722,0	164,0	13,0%	13,0%	87,0%
#16	1,18	497,0	839,0	342,0	27,2%	40,2%	59,8%
#30	0,6	465,0	927,0	462,0	36,7%	76,9%	23,1%
#50	0,3	427,0	642,0	215,0	17,1%	94,0%	6,0%
#100	0,15	430,0	491,0	61,0	4,8%	98,8%	1,2%
#200	0,075	418,0	430,0	12,0	1,0%	99,8%	0,2%
Pan	0	402,0	405,0	3,0	0,2%	100,0%	0,0%
Total Retained Mass(g) =				1 259,0			
Original Sample Mass(g) =				1 258,0			
Percent Lost(%) =				-0,1%			
Fineness Modulus (FM) =					3,23		

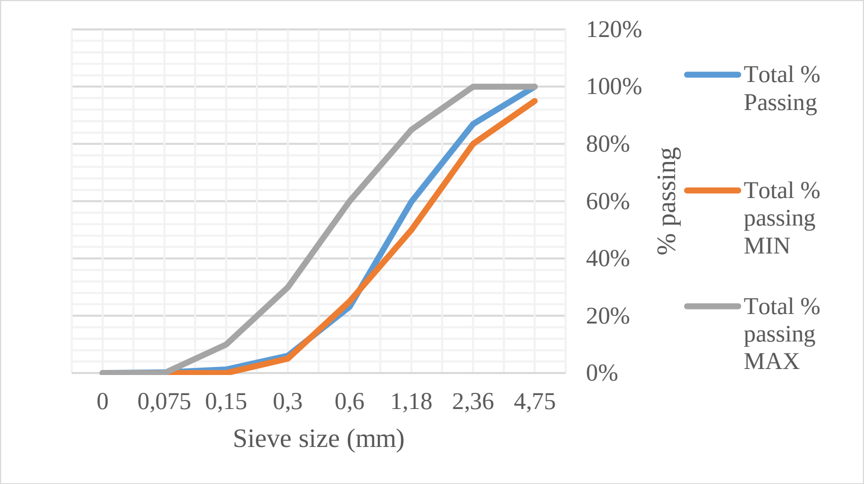


Figure 10.9. Particle size distribution of grey sand 3

10.4 Final mixture proportions

Table 10.10 Mixture proportions for yellow sand (w/c = 0.69) (mix 1)

Ingredients	Percentages	Volume, m^3	Mass, kg	80% filling	1/4 of 80%	Moisture adjusted, kg
Sand	42,00%	5,31E-03	14,922	11,937	2,984	2,994
PC	9,23%	1,17E-03	3,662	2,930	0,732	0,732
Lime	9,23%	1,17E-03	2,566	2,053	0,513	0,513
Gypsum	2,05%	2,59E-04	0,601	0,481	0,120	0,120
Water	37,50%	4,74E-03	4,741	3,793	0,948	0,917
Aluminum powder	0,06%	7,59E-06	0,020	0,016	0,004	0,004

Table 10.11 Mixture proportions for brown sand (w/c = 0.69) (mix 2)

Ingredients	Percentages	Volume, m^3	Mass, kg	80% filling	1/4 of 80%	Moisture adjusted, kg
Sand	42,00%	5,31E-03	12,320	9,856	2,464	2,617
PC	9,23%	1,17E-03	3,662	2,930	0,732	0,732
Lime	9,23%	1,17E-03	2,566	2,053	0,513	0,513
Gypsum	2,05%	2,59E-04	0,601	0,481	0,120	0,120
Water	37,50%	4,74E-03	4,741	3,793	0,948	0,894
Aluminum powder	0,06%	7,59E-06	0,020	0,016	0,004	0,004

Table 10.12 Mixture proportions for grey sand (w/c = 0.69) (mix 3)

Ingredients	Percentages	Volume, m^3	Mass, kg	80% filling	1/4 of 80%	Moisture adjusted, kg
Sand	42,00%	5,31E-03	13,328	10,663	2,666	2,728
PC	9,23%	1,17E-03	3,662	2,930	0,732	0,732
Lime	9,23%	1,17E-03	2,566	2,053	0,513	0,513
Gypsum	2,05%	2,59E-04	0,601	0,481	0,120	0,120
Water	37,50%	4,74E-03	4,741	3,793	0,948	0,871

Aluminum powder	0,06%	7,59E-06	0,020	0,016	0,004	0,004

Table 10.13 Mixture proportions for sand mix (w/c = 0.69) (mix 4)

Ingredients	Percentages	Volume, m³	Mass, kg	80% filling	1/4 of 80%	Moisture adjusted, kg
Sand	42,00%	5,31E-03	13,541	10,833	2,708	2,717
PC	9,23%	1,17E-03	3,662	2,930	0,732	0,732
Lime	9,23%	1,17E-03	2,566	2,053	0,513	0,513
Gypsum	2,05%	2,59E-04	0,601	0,481	0,120	0,120
Water	37,50%	4,74E-03	4,741	3,793	0,948	0,920
Aluminum powder	0,06%	7,59E-06	0,020	0,016	0,004	0,004

Table 10.14 Mixture proportions for normal concrete (w/c = 0.69) (mix 5)

Ingredients	1	m^3	0,0135648	m^3
Fine aggregate	749,9	kg	10,173	kg
PC	436,2	kg	5,917	kg
Coarse aggregate	868,4	kg	11,779	kg
Air	0,2	kg	0,003	kg
Water	301,0	kg	4,082	kg

10.5 Strength test results

Table 10.15 Compressive strength test results for mix 1

Mixture	Sample	Maximum load (kN)			Compressive strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
1	1	7,600	11,065	16,154	0,760	1,106	1,615
	2	8,116	10,104	12,845	0,812	1,010	1,284
	3	8,210	8,818	8,855	0,821	0,882	0,886
	average	7,975	9,996	12,618	0,798	0,999	1,262

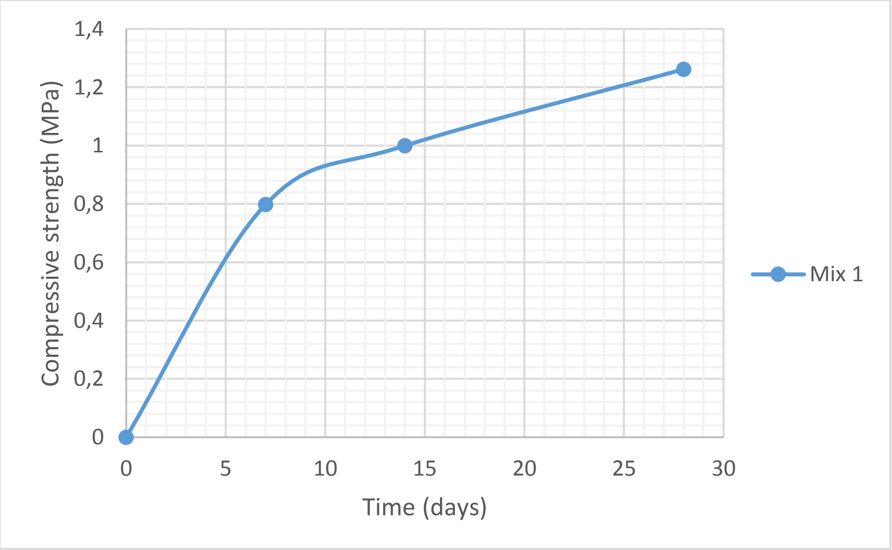


Figure 10.10 Compressive strength test results for mix 1

Table 10.16 Flexural strength test results for mix 1

Mixture	Sample	Maximum load (kN)			Flexural strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
1	1	0,311	0,365	0,413	1,166	1,369	1,549
	2	0,317	0,371	0,405	1,189	1,391	1,519
	average	0,314	0,368	0,409	1,178	1,380	1,534

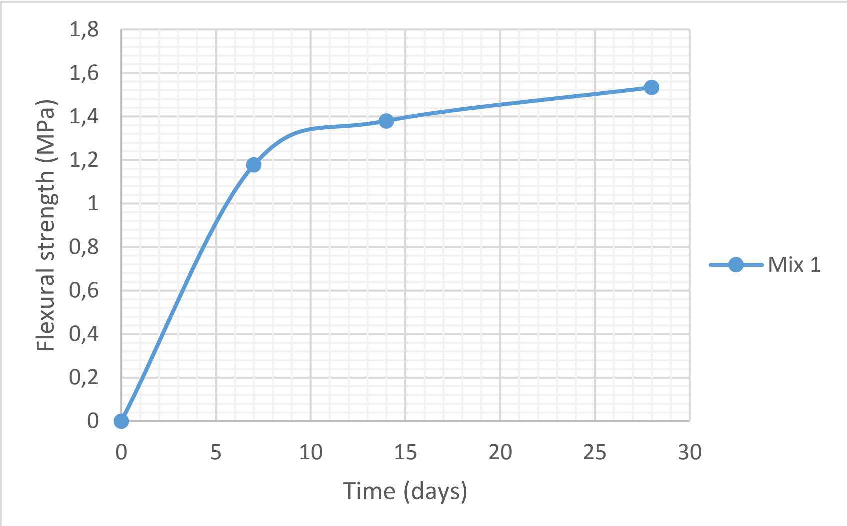


Figure 10.11 Flexural strength test results for mix 1

Table 10.17 Compressive strength test results for mix 2

Mixture	Sample	Maximum load (kN)			Compressive strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
2	1	16,098	22,073	26,757	1,610	2,207	2,676
	2	21,281	24,245	28,217	2,128	2,424	2,822
	3	23,919	23,015	28,629	2,392	2,301	2,863
	average	20,433	23,111	27,868	2,043	2,311	2,787

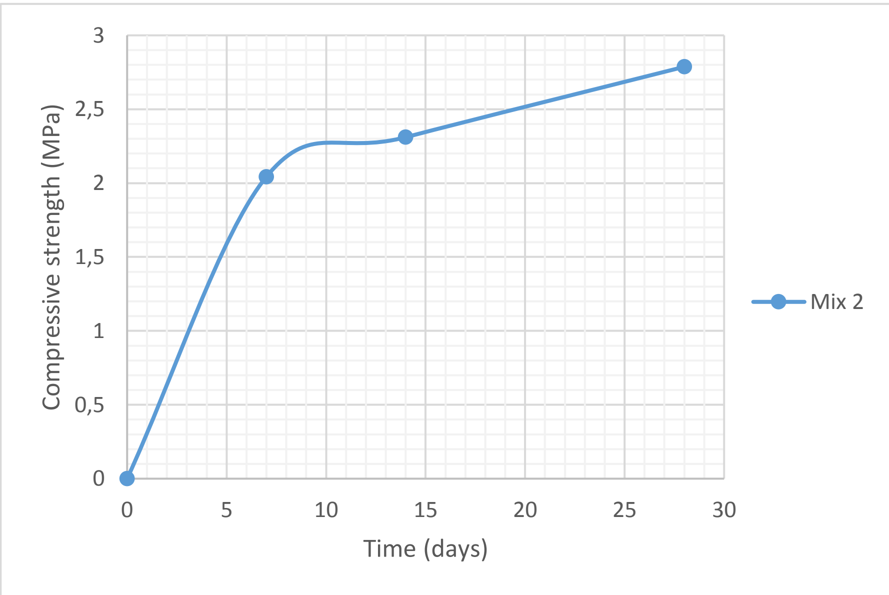


Figure 10.12 Compressive strength test results for mix 2

Table 10.18 Flexural strength test results for mix 2

Mixture	Sample	Maximum load (kN)			Flexural strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
2	1	0,489	0,553	0,607	1,834	2,074	2,276
	2	0,462	0,516	0,532	1,733	1,935	1,995
	average	0,476	0,535	0,570	1,783	2,004	2,136

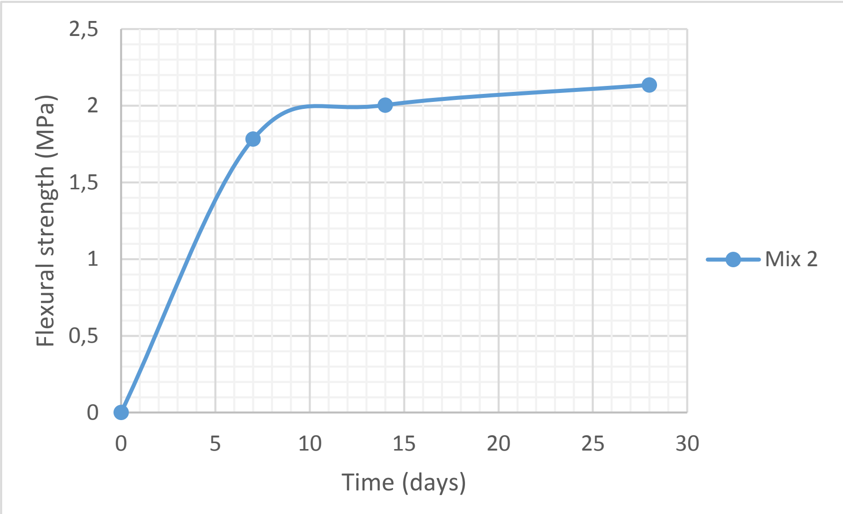


Figure 10.13 Flexural strength test results for mix 2

Table 10.19 Compressive strength test results for mix 3

Mixture	Sample	Maximum load (kN)			Compressive strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
3	1	21,537	21,206	24,142	2,154	2,121	2,414
	2	19,515	24,142	26,298	1,952	2,414	2,63
	3	21,094	20,498	25,361	2,109	2,050	2,536
	average	20,715	21,949	25,267	2,072	2,195	2,527

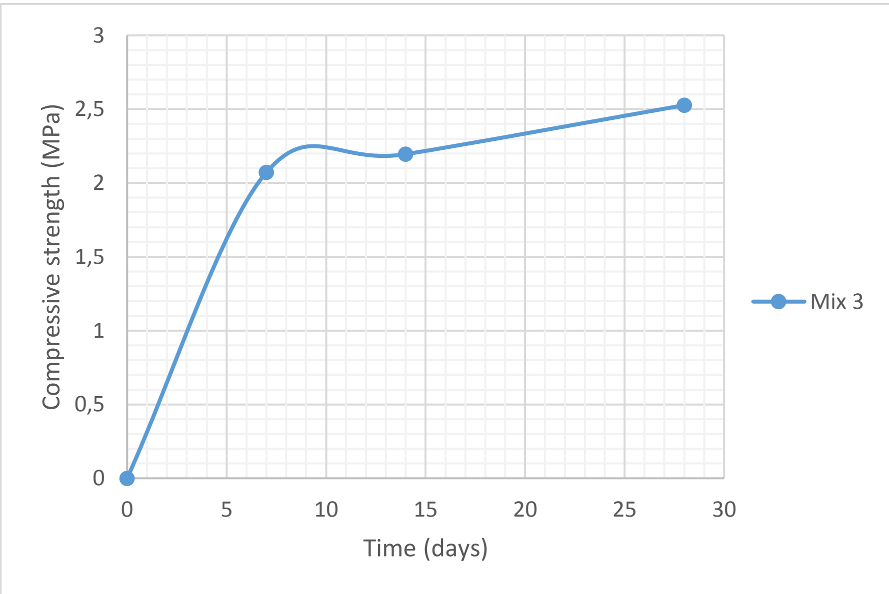


Figure 10.14 Compressive strength test results for mix 3

Table 10.20 Flexural strength test results for mix 3

Mixture	Sample	Maximum load (kN)			Flexural strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
3	1	0,521	0,566	0,577	1,954	2,123	2,164
	2	0,477	0,505	0,580	1,789	1,894	2,175
	average	0,499	0,536	0,579	1,871	2,008	2,169

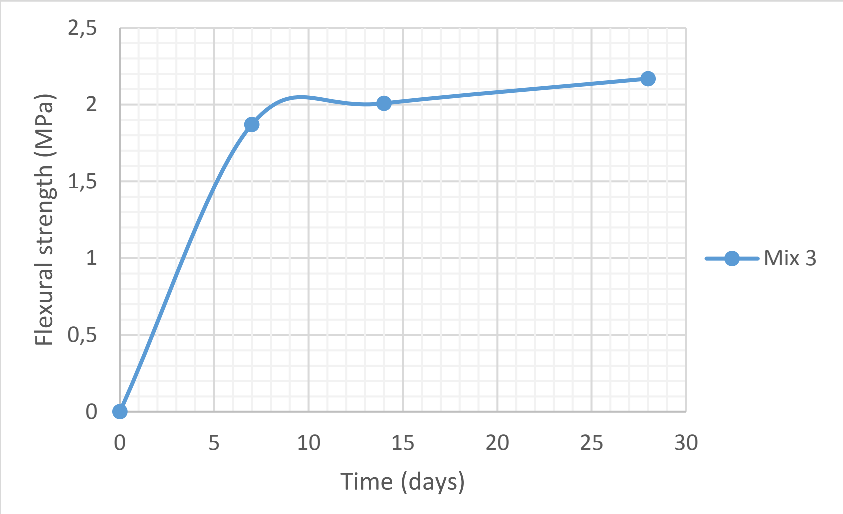


Figure 10.15 Flexural strength test results for mix 3

Table 10.21 Compressive strength test results for mix 4

Mixture	Sample	Maximum load (kN)			Compressive strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
4	1	11,764	15,427	19,174	1,176	1,543	1,917
	2	12,407	15,231	17,617	1,241	1,523	1,762
	3	12,798	16,760	20,013	1,280	1,676	2,001
	average	12,323	15,806	18,935	1,232	1,581	1,893

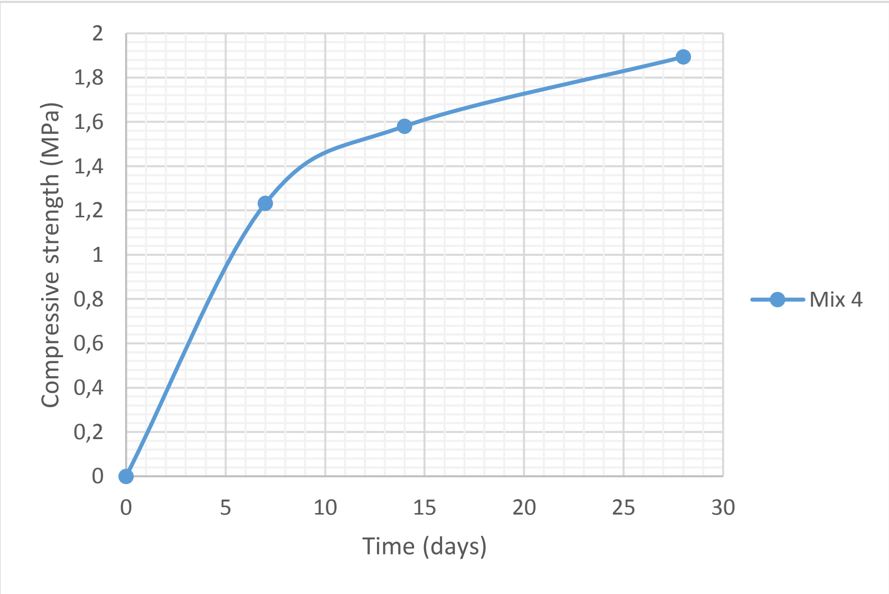


Figure 10.16 Compressive strength test results for mix 4

Table 10.22 Flexural strength test results for mix 4

Mixture	Sample	Maximum load (kN)			Flexural strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
4	1	0,383	0,485	0,543	1,436	1,819	2,036
	2	0,378	0,450	0,500	1,418	1,688	1,875
	average	0,381	0,468	0,522	1,427	1,753	1,956

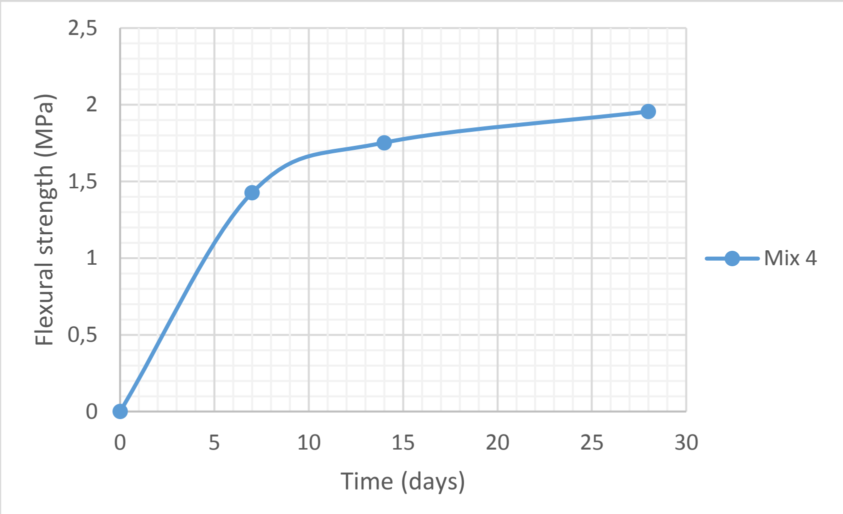


Figure 10.17 Flexural strength test results for mix 4

Table 10.23 Compressive strength test results for mix 5

Mixture	Sample	Maximum load (kN)			Compressive strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
5	1	151,171	255,048	255,039	15,117	25,505	25,504
	2	236,306	255,235	255,029	23,631	25,523	25,503
	3	239,344	254,833	255,178	23,934	25,483	25,518
	average	208,940	255,039	255,082	20,894	25,504	25,508

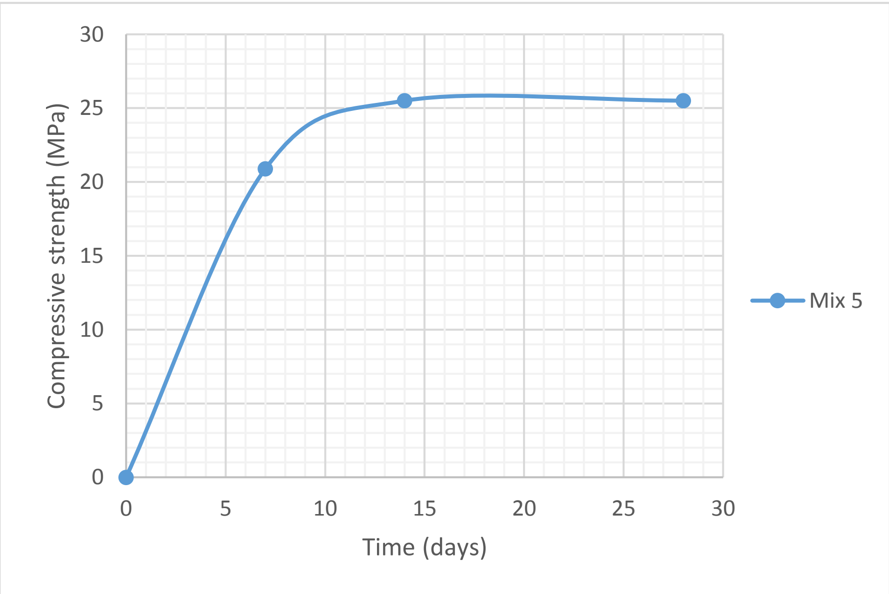


Figure 10.18 Compressive strength test results for mix 5

Table 10.24 Flexural strength test results for mix 5

Mixture	Sample	Maximum load (kN)			Flexural strength (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
5	1	1,999	2,065	2,445	7,496	7,744	9,169
	2	1,911	2,094	2,720	7,166	7,853	10,200
	average	1,955	2,080	2,583	7,331	7,798	9,684

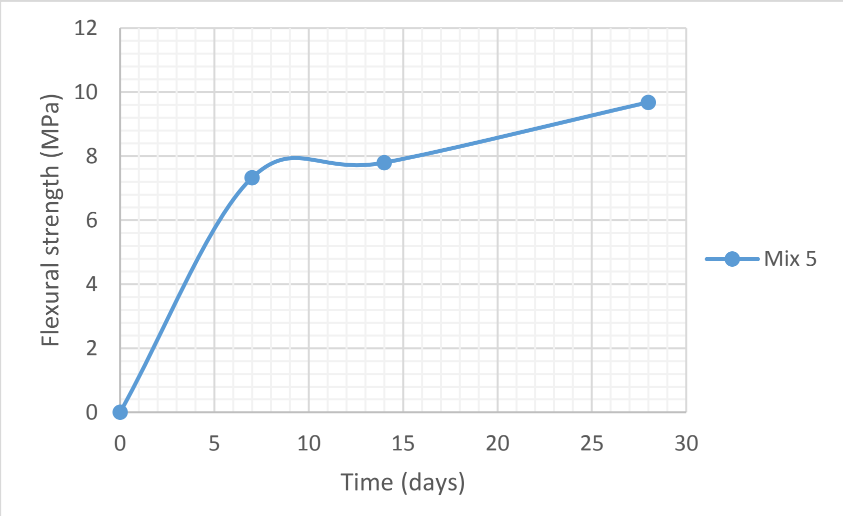


Figure 10.19 Flexural strength test results for mix 5

10.6 Porosity test results

Table 10.25 Porosity test results (test 1)

Mix	Sample	Oven-Dry Mass, g	Saturated Mass (immersion), g	Saturated Mass (boiling), g	Immersed Mass, g	Absorption after immersion, %	Absorption after immersion and boiling, %	Bulk density (dry), kg/m3	Bulk density (after immersion), kg/m3	Bulk density (after immersion and boiling), kg/m3	Volume of voids, %	Mean volume of voids, %
1	1	684	896	1031	319	30,99	50,73	961	1258	1448	48,74	49,75
	2	782	1032	1201	390	31,97	53,58	964	1273	1481	51,66	
	3	612	800	911	299	30,72	48,86	1000	1307	1489	48,86	
2	1	855	1080	1134	429	26,32	32,63	1213	1532	1609	39,57	41,47
	2	941	1185	1296	498	25,93	37,73	1179	1485	1624	44,49	
	3	454	578	619	210	27,31	36,34	1110	1413	1513	40,34	
3	1	954	1180	1256	500	23,69	31,66	1262	1561	1661	39,95	40,50
	2	1050	1300	1387	566	23,81	32,10	1279	1583	1689	41,05	
4	1	756	1014	1147	358	34,13	51,72	958	1285	1454	49,56	49,56
5	1	1375	1489	1480	796	8,29	7,64	2010	2177	2164	15,35	15,17
	2	1379	1495	1485	775	8,41	7,69	1942	2106	2092	14,93	
	3	1523	1644	1637	888	7,94	7,49	2033	2195	2186	15,22	

Table 10.26 Porosity test results (test 2)

Mix	Sample	Oven-Dry Mass, g	Saturated Mass (immersion), g	Saturated Mass (boiling), g	Immersed Mass, g	Absorption after immersion, %	Absorption after immersion and boiling, %	Bulk density (dry), kg/m3	Bulk density (after immersion), kg/m3	Bulk density (after immersion and boiling), kg/m3	Volume of voids, %	Mean volume of voids, %
1	1	684	883	1052	432	29,09	53,80	1103	1424	1697	59,35	60,25
	2	782	1033	1229	519	32,10	57,16	1101	1455	1731	62,96	
	3	612	777	934	383	26,96	52,61	1111	1410	1695	58,44	
2	1	855	1053	1168	553	23,16	36,61	1390	1712	1899	50,89	52,51
	2	941	1153	1318	625	22,53	40,06	1358	1664	1902	54,40	
	3	454	559	630	293	23,13	38,77	1347	1659	1869	52,23	
3	1	954	1164	1290	633	22,01	35,22	1452	1772	1963	51,14	53,86
	2	1050	1284	1411	773	22,29	34,38	1646	2013	2212	56,58	
4	1	756	985	1190	558	30,29	57,41	1196	1559	1883	68,67	68,67
5	1	1375	1480	1482	952	7,64	7,78	2594	2792	2796	20,19	19,68
	2	1379	1486	1489	918	7,76	7,98	2415	2602	2608	19,26	
	3	1523	1635	1639	1047	7,35	7,62	2573	2762	2769	19,59	

11 Appendix B

4.4.1.2 Minimum cover, c_{min}

(1)P Minimum concrete cover, c_{min}, shall be provided in order to ensure:
- the safe transmission of bond forces (see also Sections 7 and 8)
- the protection of the steel against corrosion (durability)
- an adequate fire resistance (see EN 1992-1-2)

(2)P The greater value for c_{min} satisfying the requirements for both bond and environmental conditions shall be used.

$$c_{min} = \max \{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\ \text{mm}\} \quad (4$$

where:

$c_{min,b}$ minimum cover due to bond requirement, see 4.4.1.2 (3)
$c_{min,dur}$ minimum cover due to environmental conditions, see 4.4.1.2 (5)
$\Delta c_{dur,\gamma}$ additive safety element, see 4.4.1.2 (6)
$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel, see 4.4.1.2 (7)
$\Delta c_{dur,add}$ reduction of minimum cover for use of additional protection, see 4.4.1.2 (8)

Figure 11.1. Concrete cover 1

(3) In order to transmit bond forces safely and to ensure adequate compaction of the concrete, the minimum cover should not be less than $c_{min,b}$ given in table 4.2.

Table 4.2: Minimum cover, $c_{min,b}$, requirements with regard to bond

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

Note: The values of $c_{min,b}$ for post-tensioned circular and rectangular ducts for bonded tendons, and pre-tensioned tendons for use in a Country may be found in its National Annex. The recommended values for post-tensioned ducts are:

circular ducts: diameter

rectangular ducts: greater of the smaller dimension or half the greater dimension

There is no requirement for more than 80 mm for either circular or rectangular ducts.

The recommended values for pre-tensioned tendon:

1,5 x diameter of strand or plain wire

2,5 x diameter of indented wire.

(4) For prestressing tendons, the minimum cover of the anchorage should be provided in accordance with the appropriate European Technical Approval.

(5) The minimum cover values for reinforcement and prestressing tendons in normal weight concrete taking account of the exposure classes and the structural classes is given by $c_{min,dur}$.

Note: Structural classification and values of $c_{min,dur}$ for use in a Country may be found in its National Annex. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1.

The recommended values of $c_{min,dur}$ are given in Table 4.4N (reinforcing steel) and Table 4.5N (prestressing steel).

Table 4.3N: Recommended structural classification

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Notes to Table 4.3N

1. The strength class and w/c ratio are considered to be related values. A special composition (type of cement, w/c value, fine fillers) with the intent to produce low permeability may be considered.

2. The limit may be reduced by one strength class if air entrainment of more than 4% is applied.

Figure 11.2. Concrete cover 2

Table 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Table 4.5N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for prestressing steel

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	15	20	25	30	35	40
S2	10	15	25	30	35	40	45
S3	10	20	30	35	40	45	50
S4	10	25	35	40	45	50	55
S5	15	30	40	45	50	55	60
S6	20	35	45	50	55	60	65

(6) The concrete cover should be increased by the additive safety element $\Delta c_{dur,\gamma}$.

Note: The value of $\Delta c_{dur,\gamma}$ for use in a Country may be found in its National Annex. The recommended value is 0 mm.

(7) Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta c_{dur,st}$. For such situations the effects on all relevant material properties should be considered, including bond.

Note: The value of $\Delta c_{dur,st}$ for use in a Country may be found in its National Annex. The recommended value, without further specification, is 0 mm.

(8) For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur,add}$.

Note: The value of $\Delta c_{dur,add}$ for use in a Country may be found in its National Annex. The recommended value, without further specification, is 0 mm.

(9) Where in-situ concrete is placed against other concrete elements (precast or in-situ) the minimum concrete cover of the reinforcement to the interface may be reduced to a value corresponding to the requirement for bond (see (3) above) provided that:
- the strength class of concrete is at least C25/30,
- the exposure time of the concrete surface to an outdoor environment is short (< 28 days),
- the interface has been roughened.

(10) For unbonded tendons the cover should be provided in accordance with the European Technical Approval.

(11) For uneven surfaces (e.g. exposed aggregate) the minimum cover should be increased by at least 5 mm.

Figure 11.3. Concrete cover

Table 8.1N: Minimum mandrel diameter to avoid damage to reinforcement

a) for bars and wire

Bar diameter	Minimum mandrel diameter for bends, hooks and loops (see Figure 8.1)
$\phi \leq 16$ mm	4ϕ
$\phi > 16$ mm	7ϕ

b) for welded bent reinforcement and mesh bent after welding

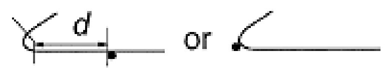
Minimum mandrel diameter	
	
5ϕ	$d \geq 3\phi$: 5ϕ $d < 3\phi$ or welding within the curved zone: 20ϕ
Note: The mandrel size for welding within the curved zone may be reduced to 5ϕ AC1 where the welding is carried out in accordance with AC2 EN ISO 17660. AC2 AC1	

Figure 11.4. Minimum mandrel diameter

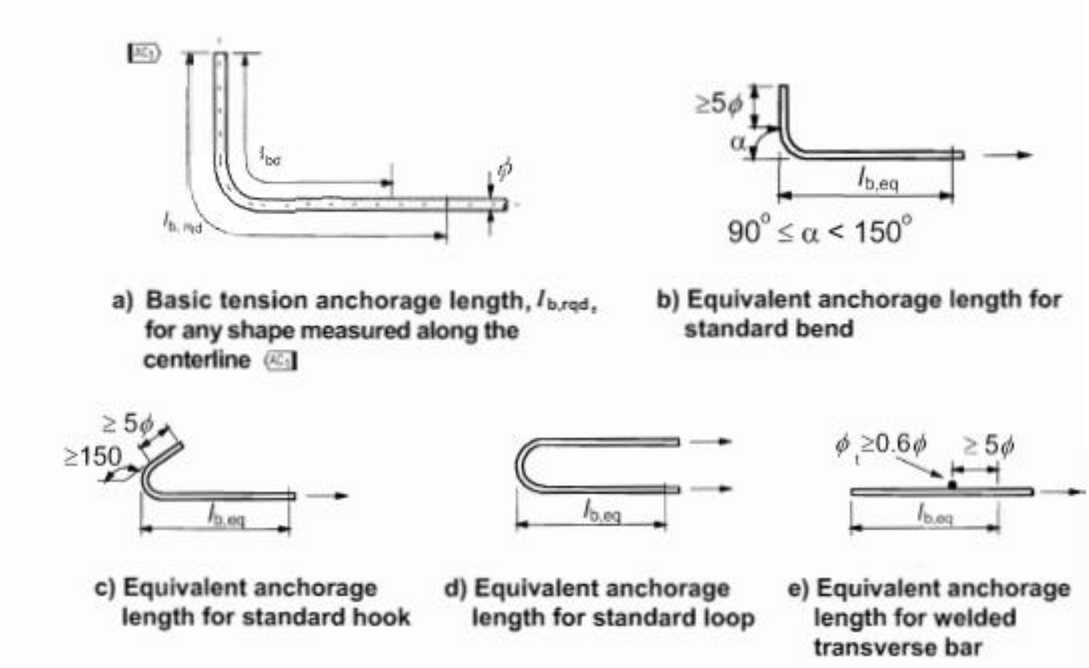


Figure 11.5. Methods of anchorage

6.2.3 Members requiring design shear reinforcement

(1) The design of members with shear reinforcement is based on a truss model (Figure 6.5). Limiting values for the angle θ of the inclined struts in the web are given in 6.2.3 (2).

In Figure 6.5 the following notations are shown:

- α is the angle between shear reinforcement and the beam axis perpendicular to the shear force (measured positive as shown in Figure 6.5)
- θ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force
- F_{td} is the design value of the tensile force in the longitudinal reinforcement
- F_{cd} is the design value of the concrete compression force in the direction of the longitudinal member axis.
- b_w is the minimum width between tension and compression chords
- z is the inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force, the approximate value $z = 0,9d$ may normally be used.

In elements with inclined prestressing tendons, longitudinal reinforcement at the tensile chord ⟦AC1⟧ should be provided to carry the longitudinal tensile force due to shear defined in (7).⟦AC1⟧

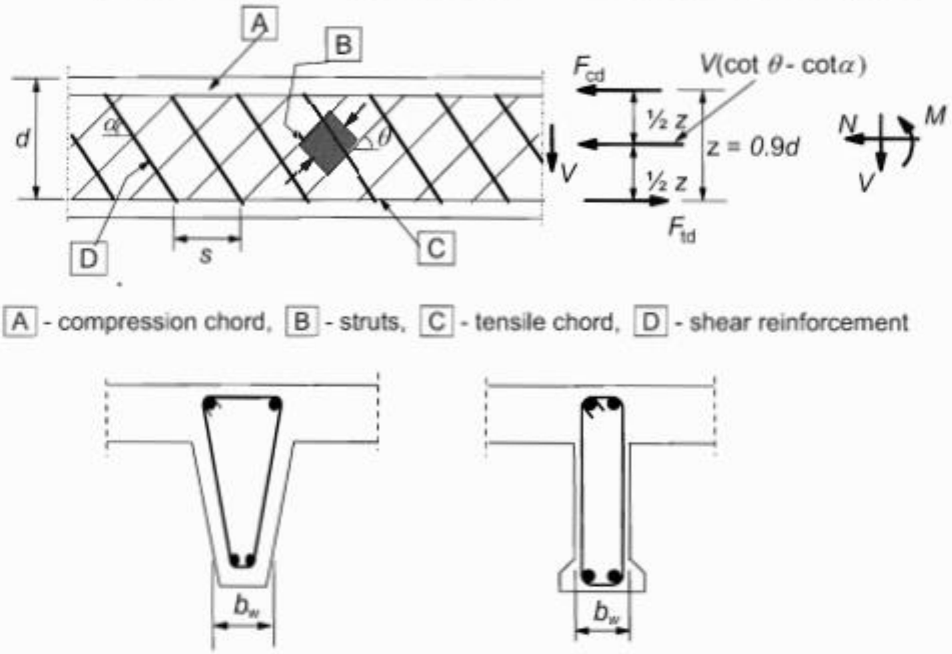


Figure 6.5: Truss model and notation for shear reinforced members

(2) The angle θ should be limited.

Note: The limiting values of $\cot\theta$ for use in a Country may be found in its National Annex. The recommended limits are given in Expression (6.7N).

$$1 \leq \cot\theta \leq 2,5 \qquad (6.7N)$$

Figure 11.6. Shear reinforcement 1

(3) For members with vertical shear reinforcement, the shear resistance, V_{Rd} is the smaller value of:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \qquad (6.8)$$

Note: If Expression (6.10) is used the value of f_{ywd} should be reduced to 0,8 f_{ywk} in Expression (6.8)

and

$$V_{Rd,max} = \alpha_{cw} b_w z \nu_1 f_{cd}/(\cot\theta + \tan\theta) \qquad (6.9)$$

where:

A_{sw} is the cross-sectional area of the shear reinforcement
s is the spacing of the stirrups
f_{ywd} is the design yield strength of the shear reinforcement
ν_1 is a strength reduction factor for concrete cracked in shear
α_{cw} is a coefficient taking account of the state of the stress in the compression chord

Note 1: The value of ν_1 and α_{cw} for use in a Country may be found in its National Annex. The recommended value of ν_1 is ν (see Expression (6.6N)).

Note 2: If the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk}, ν_1 may be taken as:

$\nu_1 = 0,6$ for $f_{ck} \le 60$ MPa (6.10.aN)
$\nu_1 = 0,9 - f_{ck}/200 > 0,5$ for $f_{ck} \ge 60$ MPa (6.10.bN)

Note 3: The recommended value of α_{cw} is as follows:

1 for non-prestressed structures
$(1 + \sigma_{cp}/f_{cd})$ for $0 < \sigma_{cp} \le 0,25 f_{cd}$ (6.11.aN)
1,25 for $0,25 f_{cd} < \sigma_{cp} \le 0,5 f_{cd}$ (6.11.bN)
$2,5 (1 - \sigma_{cp}/f_{cd})$ for $0,5 f_{cd} < \sigma_{cp} < 1,0 f_{cd}$ (6.11.cN)

where:

σ_{cp} is the mean compressive stress, measured positive, in the concrete due to the design axial force. This should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than 0.5d cot θ from the edge of the support.

Note 4: The maximum effective cross-sectional area of the shear reinforcement, $A_{sw,max}$, for $\cot\theta = 1$ is given by:

$$\frac{A_{sw,max} f_{ywd}}{b_w s} \le \frac{1}{2} \alpha_{cw} \nu_1 f_{cd} \qquad (6.12)$$

(4) For members with inclined shear reinforcement, the shear resistance is the smaller value of

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha \qquad (6.13)$$

and

$$V_{Rd,max} = \alpha_{cw} b_w z \nu_1 f_{cd} (\cot\theta + \cot\alpha)/(1 + \cot^2\theta) \qquad (6.14)$$

Note: The maximum effective shear reinforcement, $A_{sw,max}$ for $\cot\theta = 1$ follows from:

$$\frac{A_{sw,max} \; f_{ywd}}{b_w s} \le \frac{\frac{1}{2}\alpha_{cw} \nu f_{cd}}{\sin\alpha} \qquad (6.15)$$

Figure 11.7. Shear reinforcement 2

(5) In regions where there is no discontinuity of V_{Ed} (e.g. for uniformly distributed loading applied at the top) the shear reinforcement in any length increment $l = z(\cot\theta)$ may be calculated using the smallest value of V_{Ed} in the increment.

(6) Where the web contains grouted metal ducts with a diameter $\phi > b_w/8$ the shear resistance $V_{Rd,max}$ should be calculated on the basis of a nominal web thickness given by:

$$b_{w,nom} = b_w - 0{,}5\Sigma\phi \qquad (6.16)$$

where ϕ is the outer diameter of the duct and $\Sigma\phi$ is determined for the most unfavourable level.

For grouted metal ducts with $\phi \le b_w/8$, $b_{w,nom} = b_w$

For non-grouted ducts, grouted plastic ducts and unbonded tendons the nominal web thickness is:

$$b_{w,nom} = b_w - 1{,}2\,\Sigma\phi \qquad (6.17)$$

The value 1,2 in Expression (6.17) is introduced to take account of splitting of the concrete struts due to transverse tension. If adequate transverse reinforcement is provided this value may be reduced to 1,0.

(7) The additional tensile force, ΔF_{td}, in the longitudinal reinforcement due to shear V_{Ed} may be calculated from:

$$\Delta F_{td} = 0{,}5\,V_{Ed}\,(\cot\theta - \cot\alpha) \qquad (6.18)$$

$(M_{Ed}/z) + \Delta F_{td}$ should be taken not greater than $M_{Ed,max}/z$, where $M_{Ed,max}$ is the maximum moment along the beam.

(8) For members with loads applied on the upper side within a distance $0{,}5d \le a_v \le 2{,}0d$ the contribution of this load to the shear force V_{Ed} may be reduced by $\beta = a_v/2d$.
The shear force V_{Ed}, calculated in this way, should satisfy the condition

$$V_{Ed} \le A_{sw} \cdot f_{ywd} \sin\alpha \qquad (6.19)$$

where $A_{sw} \cdot f_{ywd}$ is the resistance of the shear reinforcement crossing the inclined shear crack between the loaded areas (see Figure 6.6). Only the shear reinforcement within the central $0{,}75\,a_v$ should be taken into account. The reduction by β should only be applied for calculating the shear reinforcement. It is only valid provided that the longitudinal reinforcement is fully anchored at the support.

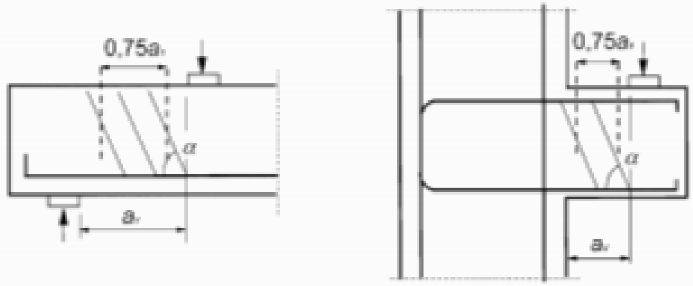


Figure 6.6: Shear reinforcement in short shear spans with direct strut action

Figure 11.8. Shear reinforcement 3

For $a_v < 0,5d$ the value $a_v = 0,5d$ should be used.

The value V_{Ed} calculated without reduction by β, should however always be less than $V_{Rd,max}$, see Expression (6.9).

6.2.4 Shear between web and flanges

(1) The shear strength of the flange may be calculated by considering the flange as a system of compressive struts combined with ties in the form of tensile reinforcement.

(2) A minimum amount of longitudinal reinforcement should be provided, as specified in 9.3.1.

(3) The longitudinal shear stress, v_{Ed}, at the junction between one side of a flange and the web is determined by the change of the normal (longitudinal) force in the part of the flange considered, according to:

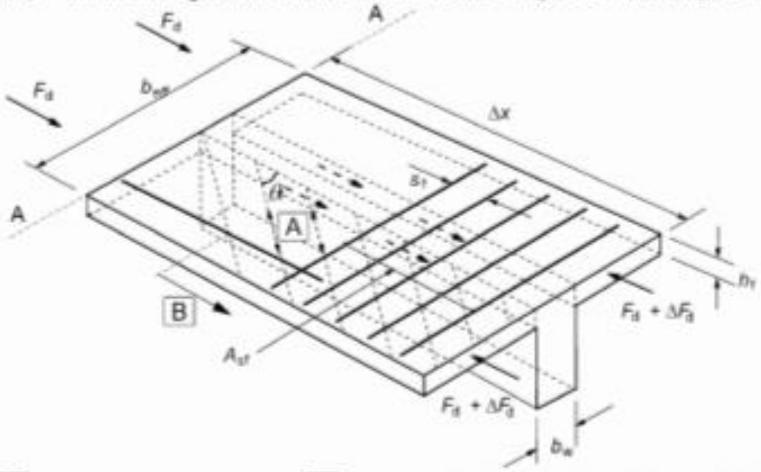
$$v_{Ed} = \Delta F_d/(h_f \cdot \Delta x) \qquad (6.20)$$

where:

h_f is the thickness of flange at the junctions

Δx is the length under consideration, see Figure 6.7

ΔF_d is the change of the normal force in the flange over the length Δx.



A - compressive struts   B - longitudinal bar anchored beyond this projected point (see 6.2.4 (7))

Figure 6.7: Notations for the connection between flange and web

The maximum value that may be assumed for Δx is half the distance between the section where the moment is 0 and the section where the moment is maximum. Where point loads are applied the length Δx should not exceed the distance between point loads.

(4) The transverse reinforcement per unit length A_{sf}/s_f may be determined as follows:

$$(A_{sf}f_{yd}/s_f) \geq v_{Ed} \cdot h_f / \cot\theta_f \qquad (6.21)$$

Figure 11.9. Shear reinforcement 4

To prevent crushing of the compression struts in the flange, the following condition should be satisfied:

$$v_{Ed} \leq \nu f_{cd} \sin\theta_f \cos\theta_f \qquad (6.22)$$

Note: The permitted range of the values for cot θ_f for use in a country may be found in its National Annex. The recommended values in the absence of more rigorous calculation are:

$1{,}0 \leq \cot\theta_f \leq 2{,}0$ for compression flanges ($45° \geq \theta_f \geq 26{,}5°$)

$1{,}0 \leq \cot\theta_f \leq 1{,}25$ for tension flanges ($45° \geq \theta_f \geq 38{,}6°$)

(5) In the case of combined shear between the flange and the web, and transverse bending, the area of steel should be the greater than that given by Expression (6.21) or half that given by Expression (6.21) plus that required for transverse bending.

(6) If v_{Ed} is less than or equal to kf_{ctd} no extra reinforcement above that for flexure is required.

Note: The value of k for use in a Country may be found in its National Annex. The recommended value is 0,4.

(7) Longitudinal tension reinforcement in the flange should be anchored beyond the strut required to transmit the force back to the web at the section where this reinforcement is required (See Section (A - A) of Figure 6.7).

6.2.5 Shear at the interface between concrete cast at different times

(1) In addition to the requirements of 6.2.1- 6.2.4 the shear stress at the interface between concrete cast at different times should also satisfy the following:

$$v_{Edi} \leq v_{Rdi} \qquad (6.23)$$

v_{Edi} is the design value of the shear stress in the interface and is given by:

$$v_{Edi} = \beta V_{Ed} / (z\, b_i) \qquad (6.24)$$

where:

- β is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered
- V_{Ed} is the transverse shear force
- z is the lever arm of composite section
- b_i is the width of the interface (see Figure 6.8)
- v_{Rdi} is the design shear resistance at the interface and is given by:

$$v_{Rdi} = c\, f_{ctd} + \mu\, \sigma_n + \rho\, f_{yd} (\mu \sin\alpha + \cos\alpha) \leq 0{,}5\, \nu\, f_{cd} \qquad (6.25)$$

where:

- c and μ are factors which depend on the roughness of the interface (see (2))
- f_{ctd} is as defined in 3.1.6 (2)P
- σ_n stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0{,}6\, f_{cd}$, and negative for tension. When σ_n is tensile $c\, f_{ctd}$ should be taken as 0.
- ρ $= A_s / A_i$

Figure 11.10. Shear reinforcement 5

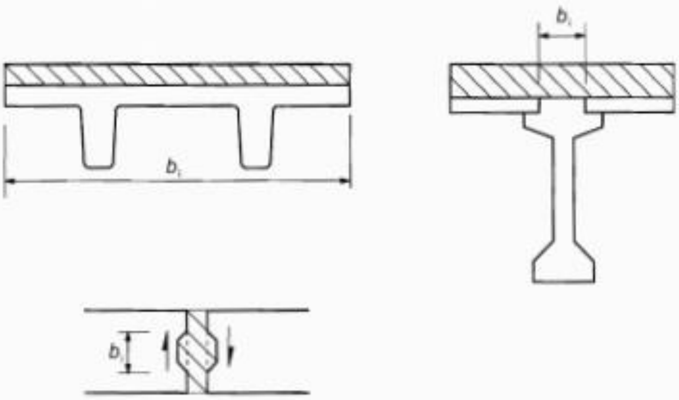
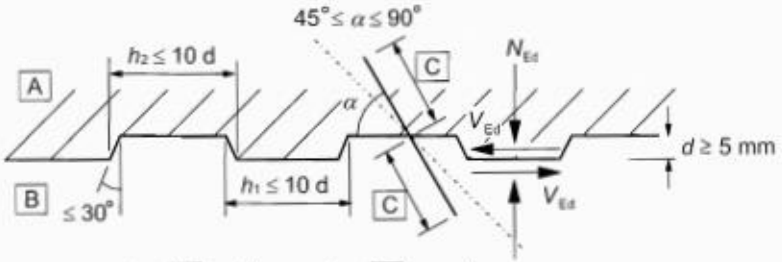


Figure 6.8: Examples of interfaces

A_s is the area of reinforcement crossing the interface, including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface.
A_i is the area of the joint
α is defined in Figure 6.9, and should be limited by $45° \leq \alpha \leq 90°$
ν is a strength reduction factor (see 6.2.2 (6))



A - new concrete, B - old concrete, C - anchorage

Figure 6.9: Indented construction joint

(2) In the absence of more detailed information surfaces may be classified as very smooth, smooth, rough or indented, with the following examples:

- Very smooth: a surface cast against steel, plastic or specially prepared wooden moulds: c = 0,025 to 0,10 and μ = 0,5
- Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: c = 0,20 and μ = 0,6
- Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: c = 0,40 and μ = 0,7
- Indented: a surface with indentations complying with Figure 6.9: c = 0,50 and μ = 0,9

(3) A stepped distribution of the transverse reinforcement may be used, as indicated in Figure 6.10. Where the connection between the two different concretes is ensured by reinforcement

Figure 11.11. Shear Reinforcement 6

(beams with lattice girders), the steel contribution to v_{Rdi} may be taken as the resultant of the forces taken from each of the diagonals provided that $45° \leq \alpha \leq 135°$.

(4) The longitudinal shear resistance of grouted joints between slab or wall elements may be calculated according to 6.2.5 (1). However in cases where the joint can be significantly cracked, c should be taken as 0 for smooth and rough joints and 0,5 for indented joints (see also 10.9.3 (12)).

(5) Under fatigue or dynamic loads, the values for c in 6.2.5 (1) should be halved.

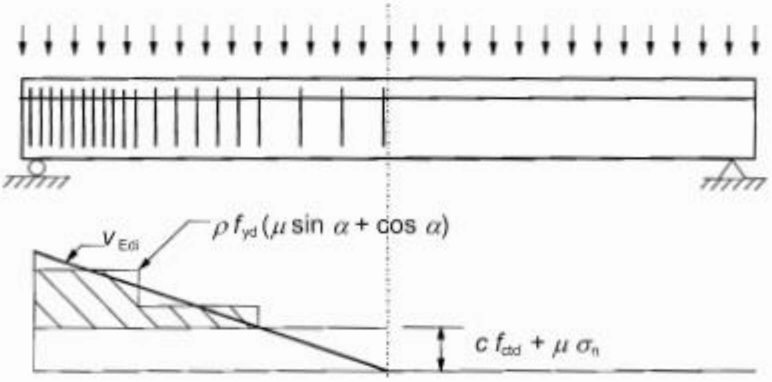


Figure 6.10: Shear diagram representing the required interface reinforcement

Figure 11.12. Shear Reinforcement 7

Table 11.1. Maximum value of k for different f_{ck} and moment redistribution ratio

f_{ck}	$k = M/(bd^2 f_{ck})$				
	$\delta = 1.0$	$\delta = 0.9$	$\delta = 0.8$	$\delta = 0.7$*	Balanced
≤ 50	0.196	0.167	0.136	0.102	*0.247*
55	0.154	0.125	0.093	0.059	*0.227*
60	0.145	0.117	0.087	0.055	*0.213*
70	0.130	0.105	0.078	0.049	*0.191*
80	0.117	0.094	0.070	0.044	*0.173*
90	0.107	0.086	0.064	0.040	*0.159*

(1) The area of longitudinal tension reinforcement should not be taken as less than $A_{s,min}$.

Note 1: See also 7.3 for area of longitudinal tension reinforcement to control cracking.

Note 2: The value of $A_{s,min}$ for beams for use in a Country may be found in its National Annex. The recommended value is given in the following:

$$A_{s,min} = 0{,}26 \frac{f_{ctm}}{f_{yk}} b_t d \qquad \text{but not less than } 0{,}0013 b_t d \qquad (9.1N)$$

Where:

b_t denotes the mean width of the tension zone; for a T-beam with the flange in compression, only the width of the web is taken into account in calculating the value of b_t.

f_{ctm} should be determined with respect to the relevant strength class according to Table 3.1.

Alternatively, for secondary elements, where some risk of brittle failure may be accepted, $A_{s,min}$ may be taken as 1,2 times the area required in ULS verification.

(2) Sections containing less reinforcement than $A_{s,min}$ should be considered as unreinforced (see Section 12).

(3) The cross-sectional area of tension or compression reinforcement should not exceed $A_{s,max}$ outside lap locations.

Note: The value of $A_{s,max}$ for beams for use in a Country may be found in its National Annex. The recommended value is $0{,}04A_c$.

(4) For members prestressed with permanently unbonded tendons or with external prestressing cables, it should be verified that the ultimate bending capacity is larger than the flexural cracking moment. A capacity of 1,15 times the cracking moment is sufficient.

9.2.1.2 Other detailing arrangements

(1) In monolithic construction, even when simple supports have been assumed in design, the section at supports should be designed for a bending moment arising from partial fixity of at least β_1 of the maximum bending moment in the span.

Figure 11.13. Beam design 1

Note 1: The value of β_1 for beams for use in a Country may be found in its National Annex. The recommended value is 0,15.

Note 2: The minimum area of longitudinal reinforcement section defined in 9.2.1.1 (1) applies.

(2) At intermediate supports of continuous beams, the total area of tension reinforcement A_s of a flanged cross-section should be spread over the effective width of flange (see 5.3.2). Part of it may be concentrated over the web width (See Figure 9.1).

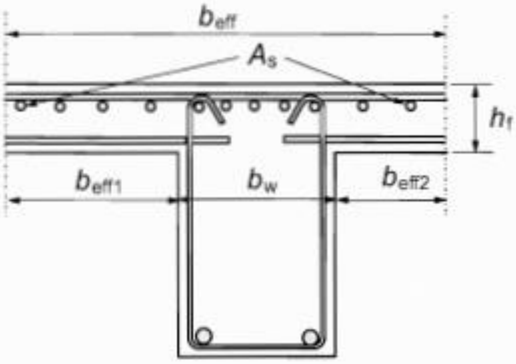


Figure 9.1: Placing of tension reinforcement in flanged cross-section.

(3) Any compression longitudinal reinforcement (diameter ϕ) which is included in the resistance calculation should be held by transverse reinforcement with spacing not greater than 15ϕ.

9.2.1.3 Curtailment of longitudinal tension reinforcement

(1) Sufficient reinforcement should be provided at all sections to resist the envelope of the acting tensile force, including the effect of inclined cracks in webs and flanges.

(2) For members with shear reinforcement the additional tensile force, ΔF_{td}, should be calculated according to 6.2.3 (7). For members without shear reinforcement ΔF_{td} may be estimated by shifting the moment curve a distance $a_l = d$ according to 6.2.2 (5). This "shift rule" may also be used as an alternative for members with shear reinforcement, where:

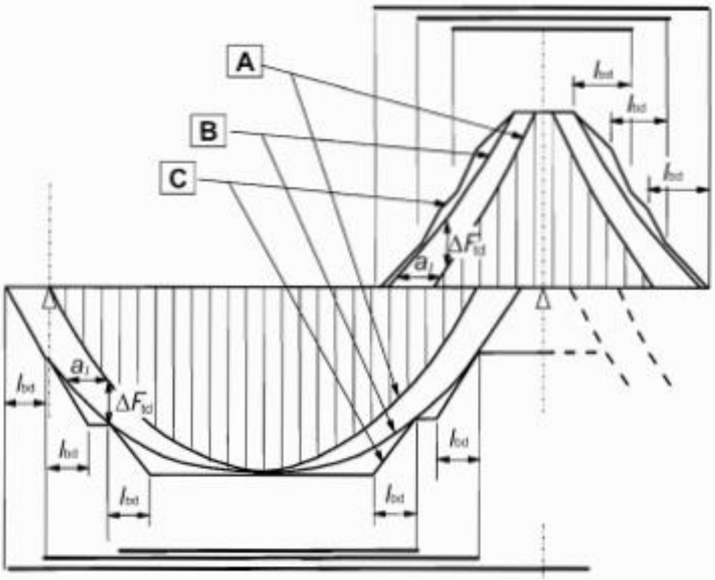
$$a_l = z (\cot \theta - \cot \alpha)/2 \quad \text{(symbols defined in 6.2.3)} \tag{9.2}$$

The additional tensile force is illustrated in Figure 9.2.

(3) The resistance of bars within their anchorage lengths may be taken into account, assuming a linear variation of force, see Figure 9.2. As a conservative simplification this contribution may be ignored.

(4) The anchorage length of a bent-up bar which contributes to the resistance to shear should be not less than 1,3 l_{bd} in the tension zone and 0,7 l_{bd} in the compression zone. It is measured from the point of intersection of the axes of the bent-up bar and the longitudinal reinforcement.

Figure 11.14. Beam design 2



A - Envelope of $M_{Ed}/z + N_{Ed}$ B - acting tensile force F_s C - resisting tensile force F_{Rs}

Figure 9.2: Illustration of the curtailment of longitudinal reinforcement, taking into account the effect of inclined cracks and the resistance of reinforcement within anchorage lengths

9.2.1.4 Anchorage of bottom reinforcement at an end supports

(1) The area of bottom reinforcement provided at end supports with little or no end fixity assumed in design, should be at least β_2 of the area of steel provided in the span.

Note: The value of β_2 for beams for use in a Country may be found in its National Annex. The recommended value is 0,25.

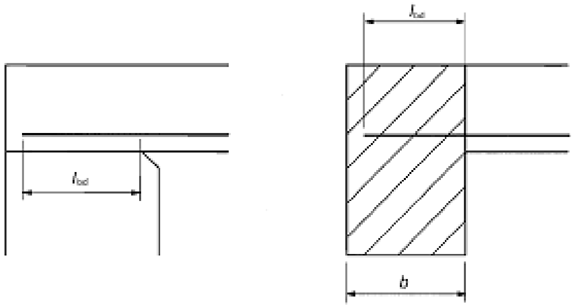
(2) The tensile force to be anchored may be determined according to 6.2.3 (7) (memberswith shear reinforcement) including the contribution of the axial force if any, or according to the shift rule:

$$F_{Ed} = |V_{Ed}| \cdot a_l / z + N_{Ed} \qquad (9.3)$$

where N_{Ed} is the axial force, to be added to or subtracted from the tensile force; a_l see 9.2.1.3 (2).

(3) The anchorage length is l_{bd} according to 8.4.4, measured from the line of contact between beam and support. Transverse pressure may be taken into account for direct support. See Figure 9.3.

Figure 11.15. Beam design 3



a) Direct support: Beam supported by wall or column **b) Indirect support:** Beam intersecting another supporting beam

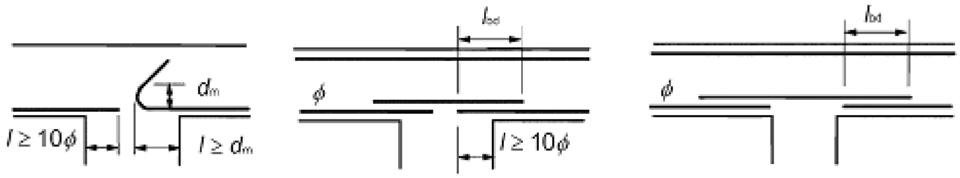
Figure 9.3: Anchorage of bottom reinforcement at end supports

9.2.1.5 Anchorage of bottom reinforcement at intermediate supports

(1) The area of reinforcement given in 9.2.1.4 (1) applies.

(2) The anchorage length should not be less than 10ϕ (for straight bars) or not less than the diameter of the mandrel (for hooks and bends with bar diameters at least equal to 16 mm) or twice the diameter of the mandrel (in other cases) (see Figure 9.4 (a)). These minimum values are normally valid but a more refined analysis may be carried out in accordance with 6.6.

(3) The reinforcement required to resist possible positive moments (e.g. settlement of the support, explosion, etc.) should be specified in contract documents. This reinforcement should be continuous which may be achieved by means of lapped bars (see Figure 9.4 (b) or (c)).



a) **b)** **c)**

Figure 9.4: Anchorage at intermediate supports

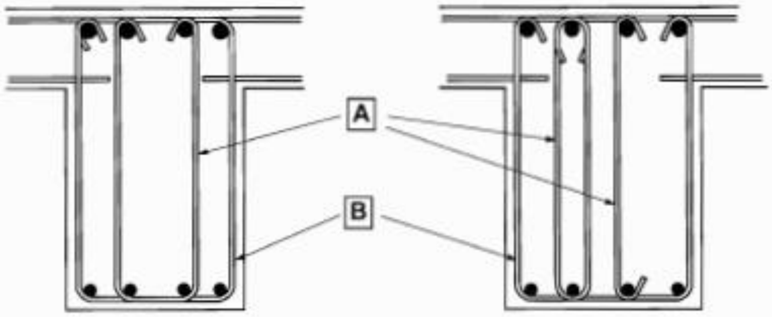
9.2.2 Shear reinforcement

(1) The shear reinforcement should form an angle α of between 45° and 90° to the longitudinal axis of the structural element.

(2) The shear reinforcement may consist of a combination of:
- links enclosing the longitudinal tension reinforcement and the compression zone (see Figure 9.5);
- bent-up bars;

Figure 11.16. Beam design 4

- cages, ladders, etc. which are cast in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones.



A Inner link alternatives   B Enclosing link

Figure 9.5: Examples of shear reinforcement

(3) Links should be effectively anchored. A lap joint on the leg near the surface of the web is permitted provided that the link is not required to resist torsion.

(4) At least β_3 of the necessary shear reinforcement should be in the form of links.

Note: The value of β_3 for use in a Country may be found in its National Annex. The recommended value is 0, 5.

(5) The ratio of shear reinforcement is given by Expression (9.4):

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin\alpha) \tag{9.4}$$

where:

ρ_w is the shear reinforcement ratio
ρ_w should not be less than $\rho_{w,min}$
A_{sw} is the area of shear reinforcement within length s
s is the spacing of the shear reinforcement measured along the longitudinal axis of the member
b_w is the breadth of the web of the member
α is the angle between shear reinforcement and the longitudinal axis (see 9.2.2 (1))

Note: The value of $\rho_{w,min}$ for beams for use in a Country may be found in its National Annex. The recommended value is given Expression (9.5N)

$$\rho_{w,min} = (0{,}08\sqrt{f_{ck}})/f_{yk} \tag{9.5N}$$

(6) The maximum longitudinal spacing between shear assemblies should not exceed $s_{l,max}$.

Note: The value of $s_{l,max}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.6N)

$$s_{l,max} = 0{,}75d\,(1 + \cot\alpha) \tag{9.6N}$$

where α is the inclination of the shear reinforcement to the longitudinal axis of the beam.

Figure 11.17. Beam design 5

(7) The maximum longitudinal spacing of bent-up bars should not exceed $s_{b,max}$:

Note: The value of $s_{b,max}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.7N)

$$s_{b,max} = 0{,}6\, d\, (1 + \cot \alpha) \qquad (9.7N)$$

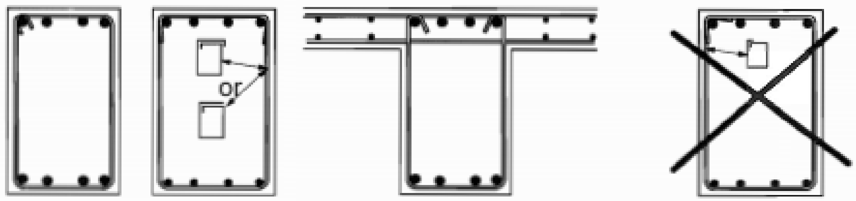
(8) The transverse spacing of the legs in a series of shear links should not exceed $s_{t,max}$:

Note: The value of $s_{t,max}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.8N)

$$s_{t,max} = 0{,}75d \le 600 \text{ mm} \qquad (9.8N)$$

9.2.3 Torsion reinforcement

(1) The torsion links should be closed and be anchored by means of laps or hooked ends, see Figure 9.6, and should form an angle of 90° with the axis of the structural element.



a1) a2) a3)

a) recommended shapes **b) not recommended shape**

Note: The second alternative for a2) (lower sketch) should have a full lap length along the top.

Figure 9.6: Examples of shapes for torsion links

(2) The provisions of 9.2.2 (5) and (6) are generally sufficient to provide the minimum torsion links required.

(3) The longitudinal spacing of the torsion links should not exceed $u / 8$ (see 6.3.2, Figure 6.11, for the notation), or the requirement in 9.2.2 (6) or the lesser dimension of the beam cross-section.

(4) The longitudinal bars should be so arranged that there is at least one bar at each corner, the others being distributed uniformly around the inner periphery of the links, with a spacing not greater than 350 mm.

9.2.4 Surface reinforcement

(1) It may be necessary to provide surface reinforcement either to control cracking or to ensure adequate resistance to spalling of the cover.

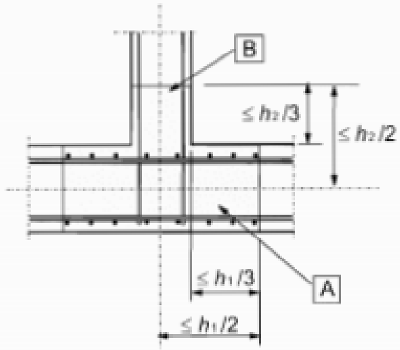
AC2) **Note:** Guidance on surface reinforcements is given in Informative Annex J. (AC2

Figure 11.18. Beam Design 6

9.2.5 Indirect supports

(1) Where a beam is supported by a beam instead of a wall or column, reinforcement should be provided and designed to resist the mutual reaction. This reinforcement is in addition to that required for other reasons. This rule also applies to a slab not supported at the top of a beam.

(2) The supporting reinforcement between two beams should consist of links surrounding the principal reinforcement of the supporting member. Some of these links may be distributed outside the volume of the concrete, which is common to the two beams, (see Figure 9.7).



A supporting beam with height h_1 B supported beam with height h_2 ($h_1 \geq h_2$)

Figure 9.7: Placing of supporting reinforcement in the intersection zone of two beams (plan view)

Figure 11.19. Beam Design 7

9.3 Solid slabs

(1) This section applies to one-way and two-way solid slabs for which b and l_{eff} are not less than $5h$ (see 5.3.1).

9.3.1 Flexural reinforcement

9.3.1.1 General

(1) For the minimum and the maximum steel percentages in the main direction 9.2.1.1 (1) and (3) apply.

Note: In addition to Note 2 of 9.2.1.1 (1), for slabs where the risk of brittle failure is small, $A_{s,min}$ may be taken as 1,2 times the area required in ULS verification.

(2) Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. In areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment.

(3) The spacing of bars should not exceed $s_{max,slabs}$.

Note; The value of $s_{max,slabs}$ for use in a Country may be found in its National Annex. The recommended value is:

- for the principal reinforcement, $3h \leq 400$ mm, where h is the total depth of the slab;

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Figure 11.20. Slab Design 8

- for the secondary reinforcement, $3{,}5h \leq$ 450 mm .

In areas with concentrated loads or areas of maximum moment those provisions become respectively:
- for the principal reinforcement, $2h \leq 250$ mm
- for the secondary reinforcement, $3h \leq 400$ mm.

(4) The rules given in 9.2.1.3 (1) to (3), 9.2.1.4 (1) to (3) and 9.2.1.5 (1) to (2) also apply but with $a_l = d$.

9.3.1.2 Reinforcement in slabs near supports

(1) In simply supported slabs, half the calculated span reinforcement should continue up to the support and be anchored therein in accordance with 8.4.4.

Note: Curtailment and anchorage of reinforcement may be carried out according to 9.2.1.3, 9.2.1.4 and 9.2.1.5.

(2) Where partial fixity occurs along an edge of a slab, but is not taken into account in the analysis, the top reinforcement should be capable of resisting at least 25% of the maximum moment in the adjacent span. This reinforcement should extend at least 0,2 times the length of the adjacent span, measured from the face of the support. It should be continuous across internal supports and anchored at end supports. At an end support the moment to be resisted may be reduced to 15% of the maximum moment in the adjacent span.

9.3.1.3 Corner reinforcement

(1) If the detailing arrangements at a support are such that lifting of the slab at a corner is restrained, suitable reinforcement should be provided.

9.3.1.4 Reinforcement at the free edges

(1) Along a free (unsupported) edge, a slab should normally contain longitudinal and transverse reinforcement, generally arranged as shown in Figure 9.8.

(2) The normal reinforcement provided for a slab may act as edge reinforcement.

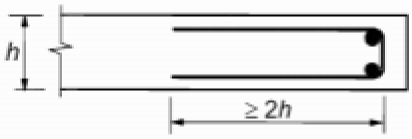


Figure 9.8: Edge reinforcement for a slab

9.3.2 Shear reinforcement

(1) A slab in which shear reinforcement is provided should have a depth of at least 200 mm.

(2) In detailing the shear reinforcement, the minimum value and definition of reinforcement ratio in 9.2.2 apply, unless modified by the following.

(3) In slabs, if $|V_{Ed}| \leq 1/3\ V_{Rd,max}$, (see 6.2), the shear reinforcement may consist entirely of bent-up bars or of shear reinforcement assemblies.

Figure 11.21. Slab Design 9

(4) The maximum longitudinal spacing of successive series of links is given by:

$$s_{max} = 0,75d(1+\cot\alpha) \qquad (9.9)$$

where α is the inclination of the shear reinforcement.

The maximum longitudinal spacing of bent-up bars is given by:

$$s_{max} = d. \qquad (9.10)$$

(5) The maximum transverse spacing of shear reinforcement should not exceed 1,5d.

9.4 Flat slabs

9.4.1 Slab at internal columns

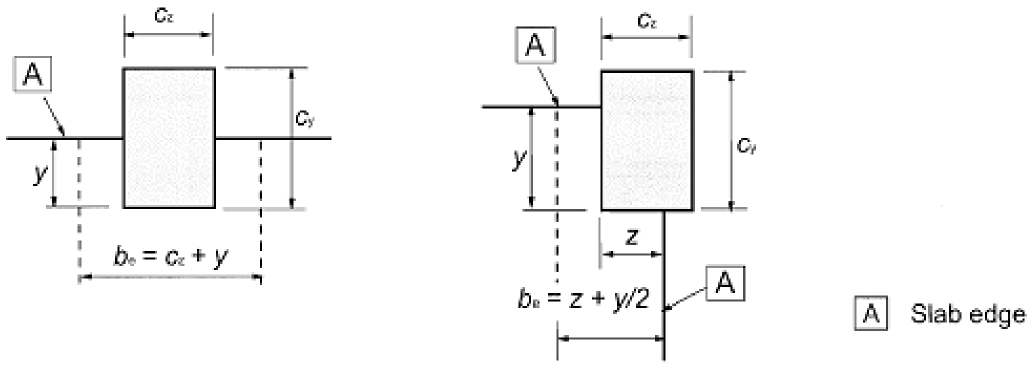
(1) The arrangement of reinforcement in flat slab construction should reflect the behaviour under working conditions. In general this will result in a concentration of reinforcement over the columns.

(2) At internal columns, unless rigorous serviceability calculations are carried out, top reinforcement of area 0,5 A_t should be placed in a width equal to the sum of 0,125 times the panel width on either side of the column. A_t represents the area of reinforcement required to resist the full negative moment from the sum of the two half panels each side of the column.

(3) Bottom reinforcement (≥ 2 bars) in each orthogonal direction should be provided at internal columns and this reinforcement should pass through the column.

9.4.2 Slab at edge and corner columns

(1) Reinforcement perpendicular to a free edge required to transmit bending moments from the slab to an edge or corner column should be placed within the effective width b_e shown in Figure 9.9.



Note: y can be > c_y

Note: z can be > c_z and y can be > c_y

a) Edge column

b) Corner column

Note: y is the distance from the edge of the slab to the innermost face of the column.

Figure 9.9: Effective width, b_e, of a flat slab

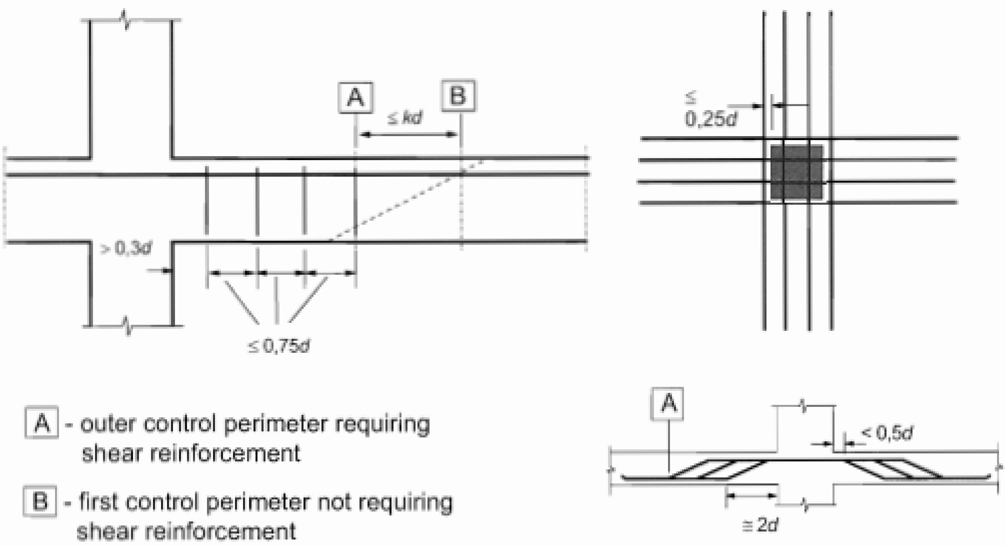
Figure 11.22. Slab Design 10

9.4.3 Punching shear reinforcement

(1) Where punching shear reinforcement is required (see 6.4) it should be placed between the loaded area/column and *kd* inside the control perimeter at which shear reinforcement is no longer required. It should be provided in at least two perimeters of link legs (see Figure 9.10). The spacing of the link leg perimeters should not exceed 0,75*d*.

The spacing of link legs around a perimeter should not exceed 1,5*d* within the first control perimeter (2*d* from loaded area), and should not exceed 2*d* for perimeters outside the first control perimeter where that part of the perimeter is assumed to contribute to the shear capacity (see Figure 6.22).

For bent down bars as arranged in Figure 9.10 b) one perimeter of link legs may be considered sufficient.



a) Spacing of links **b) Spacing of bent-up bars**

Figure 9.10: Punching shear reinforcement

Note: See 6.4.5 (4) for the value of *k*.

(2) Where shear reinforcement is required the area of a link leg (or equivalent), $A_{sw,min}$, is given by Expression (9.11).

AC2 $$A_{sw,min} \cdot (1,5 \cdot \sin\alpha + \cos\alpha)/(s_r \cdot s_t) \geq 0,08 \cdot \frac{\sqrt{f_{ck}}}{f_{yk}} \quad (9.11)$$ AC2

where :

α is the angle between the shear reinforcement and the main steel (i.e. for vertical links α = 90° and sin α = 1)

s_r is the spacing of shear links in the radial direction

s_t is the spacing of shear links in the tangential direction

f_{ck} is in MPa

Figure 11.23. Slab Design 11

The vertical component of only those prestressing tendons passing within a distance of 0.5d of the column may be included in the shear calculation.

(3) Bent-up bars passing through the loaded area or at a distance not exceeding 0,25*d* from this area may be used as punching shear reinforcement (see Figure 9.10 b), top).

(4) The distance between the face of a support, or the circumference of a loaded area, and the nearest shear reinforcement taken into account in the design should not exceed *d*/2. This distance should be taken at the level of the tensile reinforcement. If only a single line of bent-up bars is provided, their slope may be reduced to 30°.

Figure 11.24. Slab Design 12

9.5 Columns

9.5.1 General

(1) This clause deals with columns for which the larger dimension h is not greater than 4 times the smaller dimension b.

9.5.2 Longitudinal reinforcement

(1) Longitudinal bars should have a diameter of not less than ϕ_{min}.

Note: The value of ϕ_{min} for use in a Country may be found in its National Annex. The recommended value is 8 mm.

(2) The total amount of longitudinal reinforcement should not be less than $A_{s,min}$.

Note: The value of $A_{s,min}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.12N)

$$A_{s,min} = \frac{0,10\ N_{Ed}}{f_{yd}} \text{ or } 0,002\ A_c \text{ whichever is the greater} \qquad (9.12N)$$

where:

f_{yd} is the design yield strength of the reinforcement

N_{Ed} is the design axial compression force

(3) The area of longitudinal reinforcement should not exceed $A_{s,max}$.

Note: The value of $A_{s,max}$ for use in a Country may be found in its National Annex. The recommended value is 0,04 A_c outside lap locations unless it can be shown that the integrity of concrete is not affected, and that the full strength is achieved at ULS. This limit should be increased to 0,08 A_c at laps.

(4) For columns having a polygonal cross-section, at least one bar should be placed at each corner. The number of longitudinal bars in a circular column should not be less than four.

9.5.3 Transverse reinforcement

(1) The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm.

(2) The transverse reinforcement should be anchored adequately.

Figure 11.25. Beam Design 13

(3) The spacing of the transverse reinforcement along the column should not exceed $s_{cl,tmax}$

Note: The value of $s_{cl,tmax}$ for use in a Country may be found in its National Annex. The recommended value is the least of the following three distances:
- 20 times the minimum diameter of the longitudinal bars
- the lesser dimension of the column
- 400 mm

(4) The maximum spacing required in (3) should be reduced by a factor 0,6:

(i) in sections within a distance equal to the larger dimension of the column cross-section above or below a beam or slab;

(ii) near lapped joints, if the maximum diameter of the longitudinal bars is greater than 14 mm. A minimum of 3 bars evenly placed in the lap length is required.

(5) Where the direction of the longitudinal bars changes, (e.g. at changes in column size), the spacing of transverse reinforcement should be calculated, taking account of the lateral forces involved. These effects may be ignored if the change of direction is less than or equal to 1 in 12.

(6) Every longitudinal bar or bundle of bars placed in a corner should be held by transverse reinforcement. No bar within a compression zone should be further than 150 mm from a restrained bar.

Figure 11.26. Beam Design 14

Deflection check for beams

4.4m beam interior span

fck	25	Mpa
Beam Length	4400	mm
As	461.8	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.003848
ρ’	0.003848

l/d basic	35.04581
l/d actual	11

5m beam #1 interior span

fck	25	Mpa
Beam Length	5000	mm
As	339.29	mm2
As’	339.29	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.002827
ρ’	0.002827

l/d basic	52.56006
l/d actual	12.5

5m beam #2

interior span

fck	25	Mpa
Beam Length	5000	mm
As	339.29	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.002827
ρ’	0.003848

l/d basic	52.56006
l/d actual	12.5

5m beam #3

interior span

fck	25	Mpa
Beam Length	5000	mm
As	461.8	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.003848
ρ’	0.003848

l/d basic	35.04581
l/d actual	12.5

5m beam #4

interior span

fck	25	Mpa
Beam Length	5000	mm
As	461.8	mm2
As’	603.19	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.003848
ρ’	0.005027

l/d basic	35.04581
l/d actual	12.5

5m beam #5

interior span

fck	25	Mpa
Beam Length	5000	mm
As	600	mm2
As’	600	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.005
ρ’	0.005

l/d basic	27.75
l/d actual	12.5

5m beam #6

interior span

fck	25	Mpa
Beam Length	5000	mm
As	461.8	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.003848
ρ’	0.003848

l/d basic	35.04581
l/d actual	12.5

5.3m beam #1

interior span

fck	25	Mpa
Beam Length	5300	mm
As	452.39	mm2
As’	804.25	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.00377
ρ’	0.006702

l/d basic	35.89392
l/d actual	13.25

5.3m beam #2

interior span

fck	25	Mpa
Beam Length	5300	mm
As	452.39	mm2
As’	615.75	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.00377
ρ’	0.005131

l/d basic	35.89392
l/d actual	13.25

5.3m beam #3

interior span

fck	25	Mpa
Beam Length	5300	mm
As	452.39	mm2
As’	804.25	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.00377
ρ’	0.006702

l/d basic	35.89392
l/d actual	13.25

5.3m beam #4

interior span

fck	25	Mpa
Beam Length	5300	mm
As	339.29	mm2
As’	339.29	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.002827
ρ’	0.002827

l/d basic	52.56006
l/d actual	13.25

5.3m beam #5

interior span

fck	25	Mpa
Beam Length	5300	mm
As	565.49	mm2
As’	565.49	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.004712
ρ’	0.004712

l/d basic	28.79837
l/d actual	13.25

5.3m beam #6

interior span

fck	25	Mpa
Beam Length	5300	mm
As	565.49	mm2
As’	565.49	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.5	

ρ0	0.005
ρ	0.004712
ρ’	0.004712

l/d basic	28.79837
l/d actual	13.25

4.4m beam

end span

fck	25	Mpa
Beam Length	4400	mm
As	461.8	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.003848
ρ’	0.003848

l/d basic	30.37304
l/d actual	11

5m beam #1

end span

fck	25	Mpa
Beam Length	5000	mm
As	339.29	mm2
As’	339.29	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.002827
ρ’	0.002827

l/d basic	45.55205
l/d actual	12.5

5m beam #2 end span

fck	25	Mpa
Beam Length	5000	mm
As	339.29	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.002827
ρ’	0.003848

l/d basic	45.55205
l/d actual	12.5

5m beam #3 end span

fck	25	Mpa
Beam Length	5000	mm
As	461.8	mm2
As’	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.003848
ρ’	0.003848

l/d basic	30.37304
l/d actual	12.5

5m beam #4 end span

fck	25	Mpa

Beam Length	5000	mm
As	461.8	mm2
As'	603.19	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.003848
ρ'	0.005027

l/d basic	30.37304
l/d actual	12.5

5m beam #5 end span

fck	25	Mpa
Beam Length	5000	mm
As	600	mm2
As'	600	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.005
ρ'	0.005

l/d basic	24.05
l/d actual	12.5

5m beam #6 end span

fck	25	Mpa
Beam Length	5000	mm
As	461.8	mm2
As'	461.8	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.003848
ρ'	0.003848

l/d basic	30.37304
l/d actual	12.5

5.3m beam #1 end span

fck	25	Mpa

Beam Length	5300	mm
As	452.39	mm2
As'	804.25	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.00377
ρ'	0.006702

l/d basic	31.10807
l/d actual	13.25

5.3m beam #2 end span

fck	25	Mpa
Beam Length	5300	mm
As	452.39	mm2
As'	615.75	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.00377
ρ'	0.005131

l/d basic	31.10807
l/d actual	13.25

5.3m beam #3 end span

fck	25	Mpa
Beam Length	5300	mm
As	452.39	mm2
As'	804.25	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.00377
ρ'	0.006702

l/d basic	31.10807
l/d actual	13.25

5.3m beam #4 end span

fck	25	Mpa

Beam Length	5300	mm
As	339.29	mm2
As’	339.29	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.002827
ρ’	0.002827

l/d basic	45.55205
l/d actual	13.25

5.3m beam #5 end span

fck	25	Mpa
Beam Length	5300	mm
As	565.49	mm2
As’	565.49	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.004712
ρ’	0.004712

l/d basic	24.95859
l/d actual	13.25

5.3m beam #6 end span

fck	25	Mpa
Beam Length	5300	mm
As	565.49	mm2
As’	565.49	mm2
b (width)	300	mm
d (depth)	400	mm
k	1.3	

ρ0	0.005
ρ	0.004712
ρ’	0.004712

l/d basic	24.95859
l/d actual	13.25

Slab type 5300x2650

fck	25	Mpa
L	2650	mm
As	1332.5	mm2
As’	0	mm2

ρ0	0.005
ρ	0.001934
ρ’	0

l/d basic	62.32872
l/d actual	20.38462

b (width)	5300	mm
d (depth)	130	mm
k	1	

Slab type 4400x2200

fck	25	Mpa
L	2200	mm
As	1090.6	mm2
As'	0	mm2
b (width)	4400	mm
d (depth)	130	mm
k	1	

ρ0	0.005
ρ	0.001907
ρ'	0

l/d basic	63.73248
l/d actual	16.92308

Slab type 5000x2500

fck	25	Mpa
L	2500	mm
As	1090.6	mm2
As'	0	mm2
b (width)	5000	mm
d (depth)	130	mm
k	1	

ρ0	0.005
ρ	0.001678
ρ'	0

l/d basic	77.92817
l/d actual	19.23077

12 Appendix C

Table 12.1 Interpolated values of N_q* Based on Meyerhof's Theory

Soil friction angle, ϕ (deg)	N_q^*
20	12.4
21	13.8
22	15.5
23	17.9
24	21.4
25	26.0
26	29.5
27	34.0
28	39.7
29	46.5
30	56.7
31	68.2
32	81.0
33	96.0
34	115.0
35	143.0
36	168.0
37	194.0
38	231.0
39	276.0
40	346.0
41	420.0
42	525.0
43	650.0
44	780.0
45	930.0

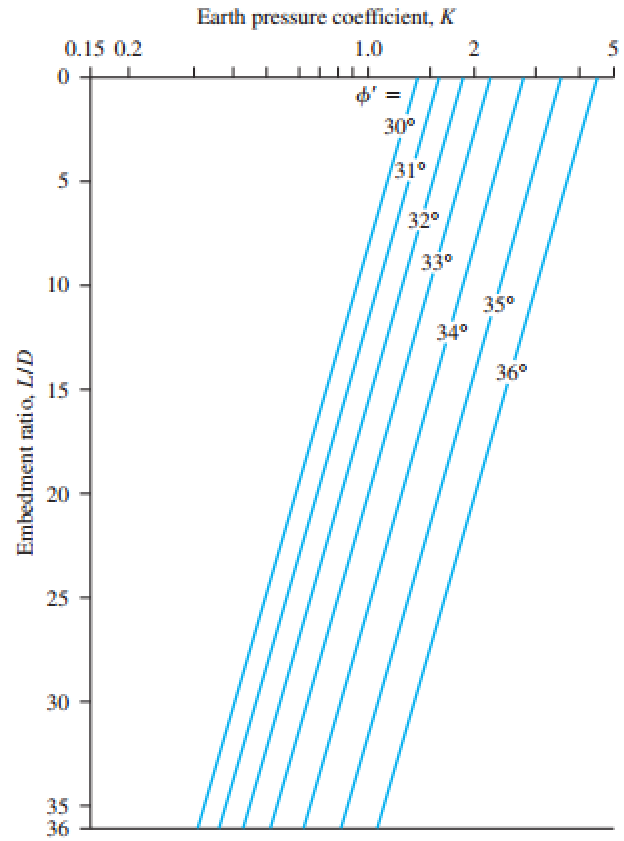


Figure 12.1 Variation of K with L/D

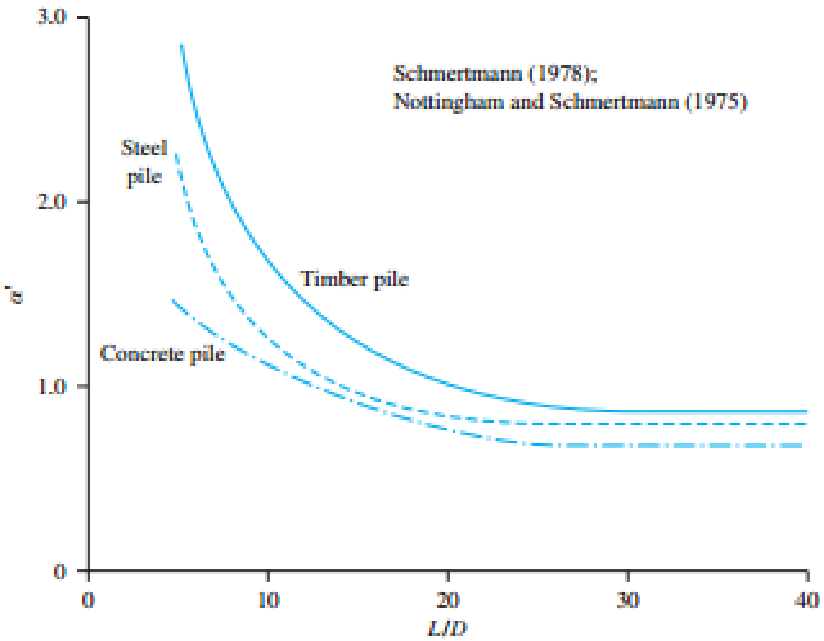


Figure 12.2 Variation of α’ with embedment ratio for pile in sand (electric cone penetrometer)

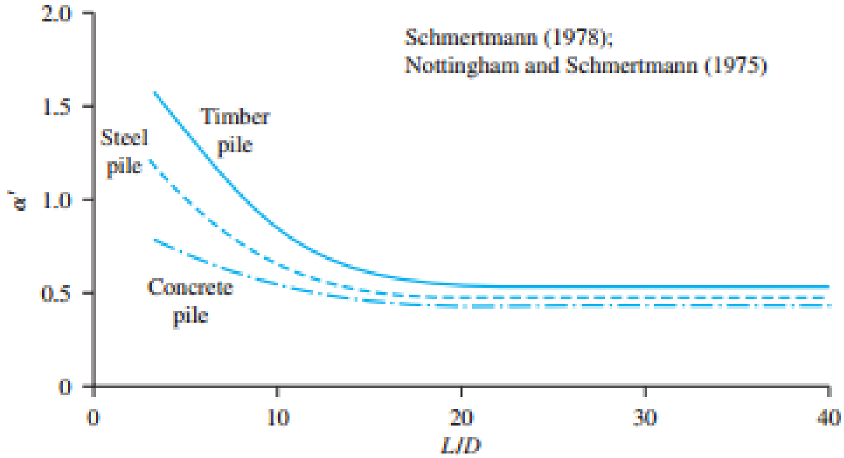


Figure 12.3 Variation of α' with embedment ratio for pile in sand (mechanical cone penetrometer)

Table 12.2 Allowable maximum settlement from different sources

Allowable maximum settlement (cm)	Type of soil	foundation	Reference
2.5	sand	isolated	TERZAGHI and PECK [1967]
5.0	»	continuous	TOMLINSON [1980]
4.0	»	isolated	SKEMPTON and McDONALD [1956]
4.5-6.5	»	continuous	» »
6.5	clay	isolated	» »
6.5-10.0	»	continuous	» »

Isolated foundation = plinths and beams
Continuous foundation = slabs and rafts

Item no.	Kind of building & type of foundation	Average settlement (cm)
1	Buildings with plain brick walls on continuous and separate foundations with wall length L to wall height H (H counted from foundation footing):	
	L/H ≥ 2.5	8 (5.5)
	L/H ≤ 1.5	10 (6.5)
2	Buildings with brick walls, reinforced with reinforced concrete or reinforced brick belts (not depending on ratio of L/H)	15 (10)
3	Frame buildings	10 (6.5)
4	Solid reinforced concrete foundations of blast furnaces, smoke stacks, silos, water towers, etc.	30 (20)

(from POLSHIN and TOKAR, 1967)

Table 12.3 Allowable maximum settlement and differential settlement

Type of settlement		**Maximum permissible settlement during operational phase**		**Distance between cross-sections 1)**
		First 3 years of operation	**First 9 years of operation**	
Settlement in simple profile	moderate settlement	24 cm	40 cm	-
	distortion settlement	17‰	28‰	-
Differences in settlement between adjacent profiles	differential settlement (slope)	2.8‰	4.8‰	25 m
		2.2‰	3.6‰	50 m
		1.4‰	2.3‰	100 m
	warping (variation in distortion settlement per 25 metres)	10‰	17‰	25 m
		7.5‰	12.5‰	50 m
		5‰	8‰	100 m

13 Appendix D

13.1 Detailed results of simulation

Mix1

Table 13.1 Cooling and Heating Loads

Monthly Load Profiles - view table

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average Outdoor Air Dry Bulb (F)	9.1	8.7	21.8	42.3	57.0	68.1	71.5	65.4	54.7	41.8	20.9	13.5
Cooling Load (MBtu)	0.0	0.0	0.0	2.04	110.07	305.74	390.91	243.2	39.23	0.0	0.0	0.0
Heating Load (MBtu)	886.08	770.47	490.58	108.03	11.85	0.0	0.0	0.0	6.02	144.4	600.43	791.16

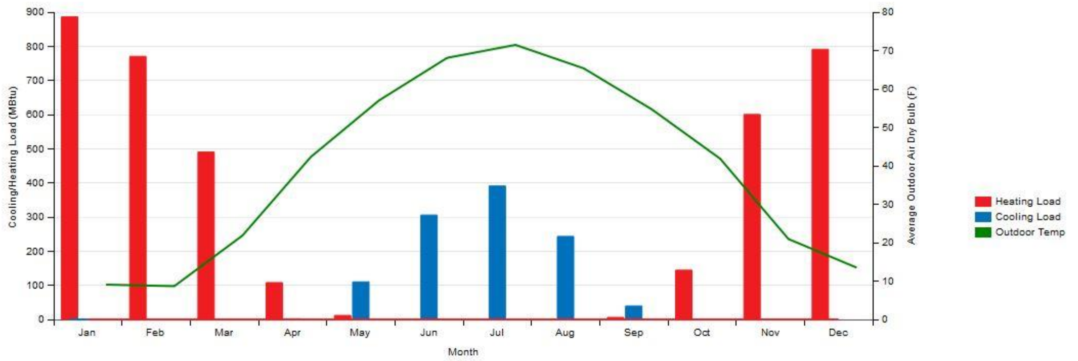


Figure 13.1 Cooling and Heating Loads
13.1

Table 13.2 Site and Source Energy Summary

Site and Source Energy

	Total Energy (kBtu)	Energy Per Total Building Area (kBtu/ft^2)	Energy Per Conditioned Building Area (kBtu/ft^2)
Total Site Energy	8577820.8	35.8	35.8
Net Site Energy	8577820.8	35.8	35.8
Total Source Energy	26562204.6	110.9	110.9
Net Source Energy	26562204.6	110.9	110.9

Mix2

Table 13.3 Cooling and Heating Loads

Monthly Load Profiles - view table

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average Outdoor Air Dry Bulb (F)	9.1	8.7	21.8	42.3	57.0	68.1	71.5	65.4	54.7	41.8	20.9	13.5
Cooling Load (MBtu)	0.0	0.0	0.0	1.58	105.78	302.98	388.8	245.39	40.34	0.0	0.0	0.0
Heating Load (MBtu)	862.6	752.01	477.23	101.53	9.11	0.0	0.0	0.0	4.23	133.84	580.24	766.51

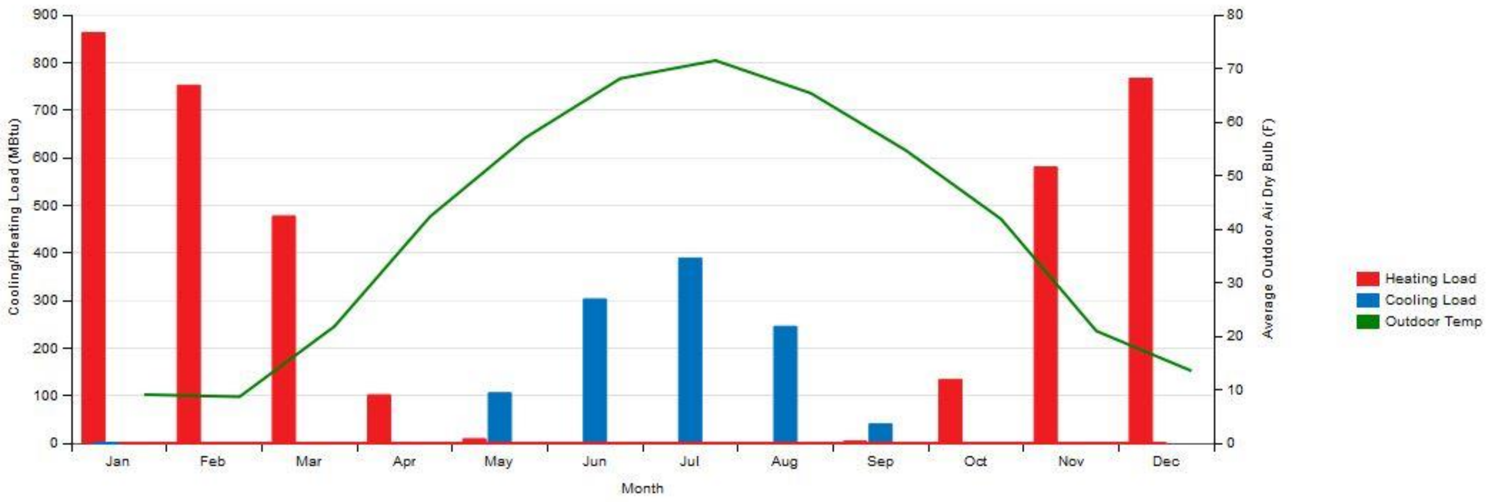


Figure 13.2 Cooling and Heating Loads

Table 13.4 Site and Source Energy Summary

Site and Source Energy

	Total Energy (kBtu)	Energy Per Total Building Area (kBtu/ft^2)	Energy Per Conditioned Building Area (kBtu/ft^2)
Total Site Energy	8449771.1	35.3	35.3
Net Site Energy	8449771.1	35.3	35.3
Total Source Energy	26115679.0	109.0	109.0
Net Source Energy	26115679.0	109.0	109.0

Mix3

Table 13.5Cooling and Heating Loads

Monthly Load Profiles - view table

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average Outdoor Air Dry Bulb (F)	9.1	8.7	21.8	42.3	57.0	68.1	71.5	65.4	54.7	41.8	20.9	13.5
Cooling Load (MBtu)	0.0	0.0	0.0	1.82	106.91	301.71	386.65	244.2	41.11	0.0	0.0	0.0
Heating Load (MBtu)	849.97	740.24	469.13	99.26	8.83	0.0	0.0	0.0	4.26	130.89	571.02	754.61

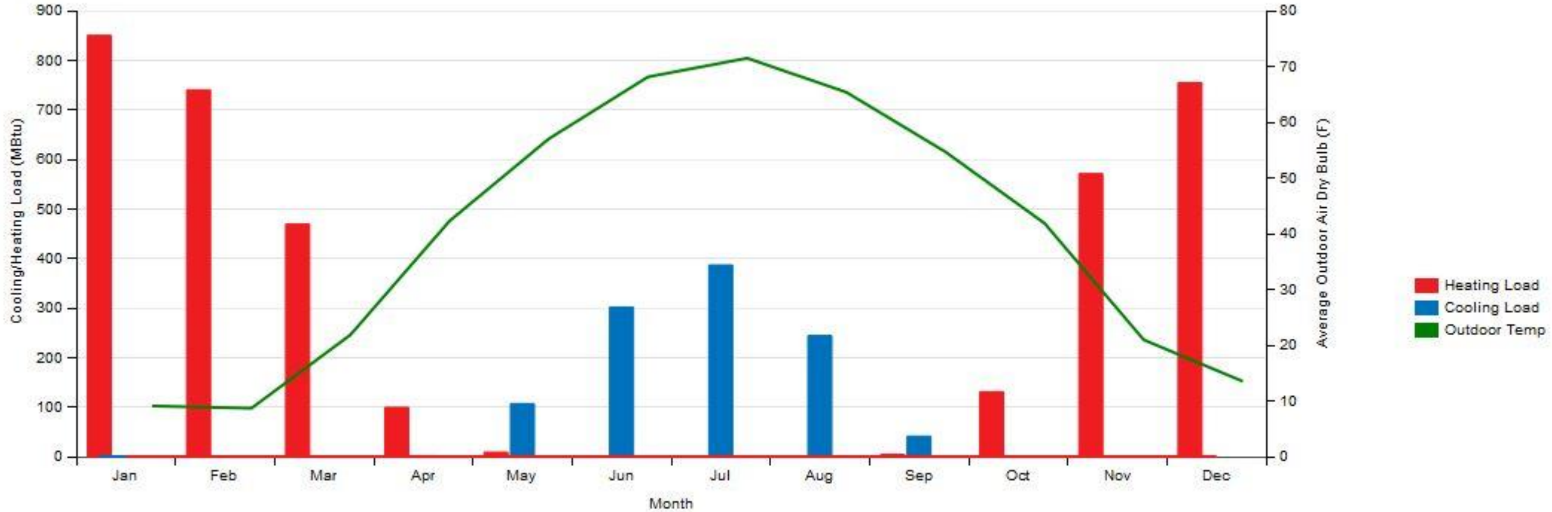


Figure 13.3 Cooling and Heating Loads

Table 13.6 Site and Source Energy Summary

Site and Source Energy

	Total Energy (kBtu)	Energy Per Total Building Area (kBtu/ft^2)	Energy Per Conditioned Building Area (kBtu/ft^2)
Total Site Energy	8388219.5	35.0	35.0
Net Site Energy	8388219.5	35.0	35.0
Total Source Energy	25899576.7	108.1	108.1
Net Source Energy	25899576.7	108.1	108.1

Mix4

Table 13.7 Cooling and Heating Loads

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average Outdoor Air Dry Bulb (F)	9.1	8.7	21.8	42.3	57.0	68.1	71.5	65.4	54.7	41.8	20.9	13.5
Cooling Load (MBtu)	0.0	0.0	0.0	2.96	108.7	290.72	372.84	240.07	48.33	0.0	0.0	0.0
Heating Load (MBtu)	746.89	648.83	405.93	79.5	4.6	0.0	0.0	0.0	2.76	101.46	491.01	656.17

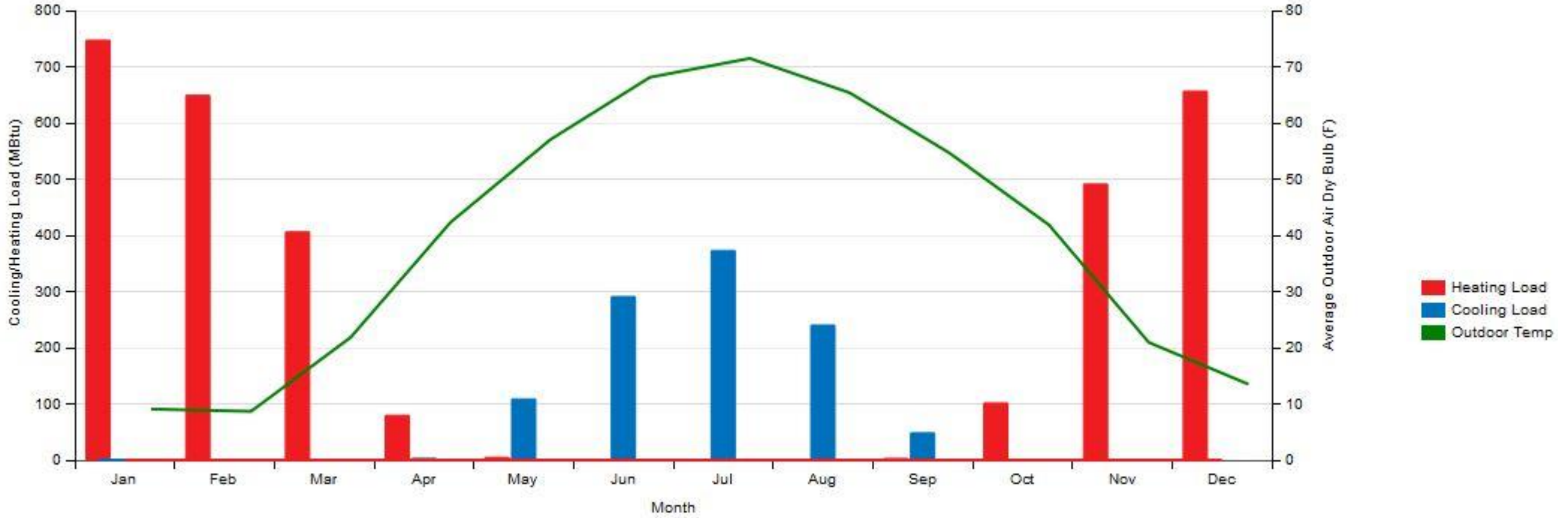


Figure 13.4 Cooling and Heating Loads

Table 13.8 Site and Source Energy Summary

	Total Energy (kBtu)	Energy Per Total Building Area (kBtu/ft^2)	Energy Per Conditioned Building Area (kBtu/ft^2)
Total Site Energy	7878388.3	32.9	32.9
Net Site Energy	7878388.3	32.9	32.9
Total Source Energy	24105377.4	100.6	100.6
Net Source Energy	24105377.4	100.6	100.6

Mix5

Table 13.9 Cooling and Heating Loads

Monthly Load Profiles - view table

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average Outdoor Air Dry Bulb (F)	9.1	8.7	21.8	42.3	57.0	68.1	71.5	65.4	54.7	41.8	20.9	13.5
Cooling Load (MBtu)	0.0	0.0	0.0	0.04	105.11	364.08	470.27	276.09	22.61	0.0	0.0	0.0
Heating Load (MBtu)	1421.53	1214.71	798.71	218.84	42.13	0.0	0.0	0.0	18.87	290.34	983.35	1282.86

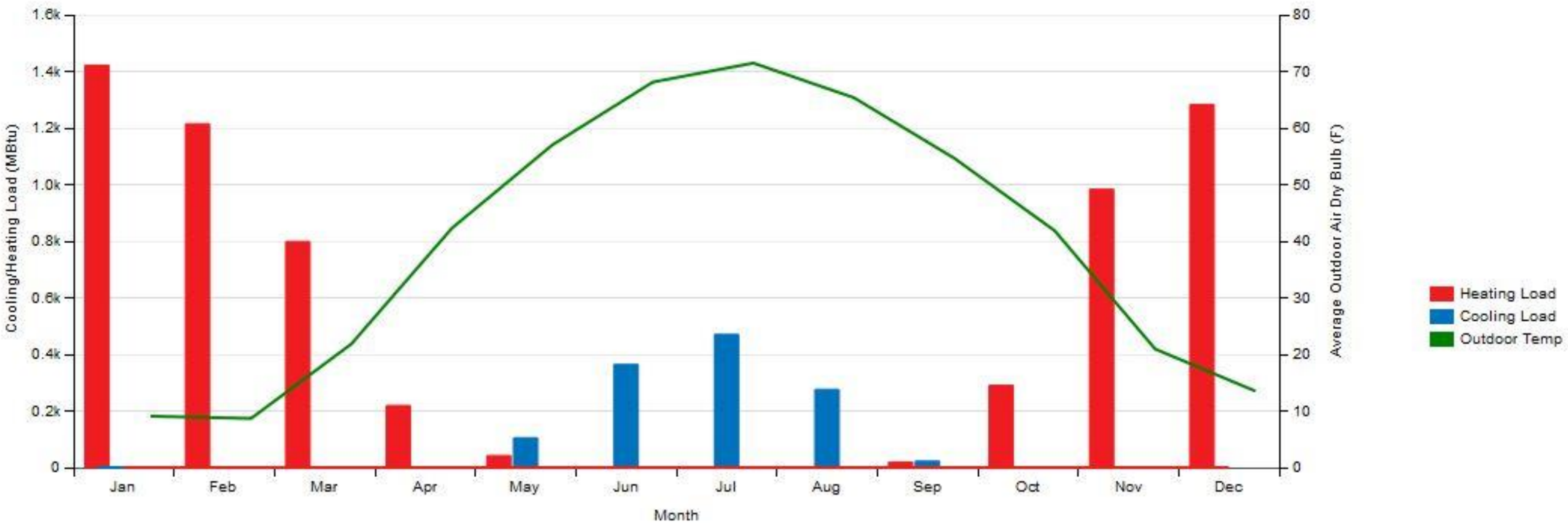


Figure 13.5 Cooling and Heating Loads

Table 13.10 Site and Source Energy Summary

Site and Source Energy

	Total Energy (kBtu)	Energy Per Total Building Area (kBtu/ft^2)	Energy Per Conditioned Building Area (kBtu/ft^2)
Total Site Energy	11187141.9	46.7	46.7
Net Site Energy	11187141.9	46.7	46.7
Total Source Energy	35614539.3	148.7	148.7
Net Source Energy	35614539.3	148.7	148.7

13.2 Detailed results of heat transfer calculation

Table 13.11 Heat transfer summary

Mixture	Thermal conductivity* (W/(m*K))	Inner surface temperature, T_2 (°C)	Mean film temperature inside, T_{f2}(K)	Thermal conductivity, k (W/(m*K)	Viscosity, ν (10^-6 m^2/s)	β (1/K)	Prandtl number	Grashof number (x10^10)	Rayleigh number (x10^10)	Nusselt number	The convective heat transfer coeffcient of the air inside, h_{c2} (W/(m^2*K)	Heat trsnferred by convection inside the wall, q_{conv2} (W)	Δq (W)
Mix #1 (yellow)	0.3187	29.39	299.69	0.026217	15.65892	0.003337	0.70807	8.980	6.358416	518.87	2.721	249.15	183.90
Mix #2 (brown)	0.2932	29.07	299.54	0.026203	15.64938	0.003338	0.70813	8.471	5.998478	508.89	2.667	229.98	164.73
Mix #3 (grey)	0.2797	28.88	299.44	0.026198	15.63337	0.003340	0.70812	8.174	5.787932	502.86	2.635	218.72	153.47
Mix #4	0.181	26.64	298.32	0.026113	15.51955	0.003352	0.70837	4.498	3.186074	412.12	2.152	96.53	31.28
Mix #5 (NC)	1.5	32.23	301.12	0.026325	15.80318	0.003321	0.70775	13.410	9.491149	592.99	3.122	436.93	371.68

14 Appendix E

14.1 Risk categories

Table 14.1. Risk categories and mitigation plan for construction

Construction risks			
Abbr	Risk category	Risk value	Mitigation plan
C1	Safety of human health	25	Techniques of safety, medical insurance
C2	Structural and geptechnical errors	15	Invite to the design team proffessional engineers
C3	Material selection errors	3	Choose appropriate materials and analyze alternatives
C4	Equipment hazards	8	Change and repair equipments regularly
C5	Documentary risks	9	Assign key employees who works with documentations
C6	Technology changes	20	Respond to any amendments

Table 14.2. Risk categories and mitigation plan for design

Design risks			
Abbr	Risk category	Risk value	Mitigation plan
E1	Drawing errors	15	Double check the drawings
E2	Design errors	10	Eliminate errors in design
E3	Conflicting standards	4	Contact with engineers
E4	Uncertanties	3	Create communication systems between empoyees
E5	Controvercies between designs	6	Ivite proffessional engineers

Table 14.3. Risk categories and mitigation plan for project management

Project management risks			
Abbr	Risk category	Risk value	Mitigation plan
PS1	Schedule delay	10	Assign additional labor and machine power
PS2	Financial: inflation and tax	4	Follow financial plan
PS3	Fdelays in delivering materials	5	Improve transportation services and monitor on-time deliverables
PS4	Quality concerns	16	Contact quality control and assurance department
PS5	Misunderstanding between employe	1	Assign key employees who works with documentations
PS6	Productivity	8	Enhance communication between departments, employees
PS7	Actual amount of work	12	Constant control to decrease the gap between baseline and actual activities
PS8	Inaccurant assessments of materials	6	Assign staff responsible for monitoring accurate purchase and delivering of materials
PS9	On-site ethics	2	Improve communication between workers and staff, provide a code of engineering ethics

Table 14.4. Risk categories and mitigation plan for environmental risks

Environmental risks			
Abbr	Risk category	Risk value	Mitigation plan
F1	Hazardous materials	5	Test materials for hazards
F2	Incomplete environmental analyzes	4	Conduct environmental analysis
F3	High seismicity and poor weather conditions	20	Discontinue construction works and restart themat the time of at least acceptable working conditions
F4	Failure to meet green certification requirements	2	At least use ecologically friendly materials and
F5	Availability of resources	12	Allocate available resources and maintain adequate resource management plan

14.1 RC Calculation

RC for columns:

$V = \sum a*b*h*n$, where:

V = required amount of concrete for each floor in m^3

a, b = dimensions of particular column

h = height of a floor

n = required number of particular column for specific floor

1^{st} floor:

$V1 = 0{,}6*0{,}6*(7*4+12+28+12)*h1+0{,}5*0{,}5*(6*3+9)*h1+0{,}7*0{,}7*(7*4+12)*h1 =$
$=0{,}6*0{,}6*(7*4+12+28+12)*5{,}1+0{,}5*0{,}5*(6*3+9)*5{,}1+0{,}7*0{,}7*(7*4+12)*5{,}1 =$
$281{,}265m^3$

2^{nd} floor:

$V2 = 0{,}5*0{,}5*(14*4+12+24+28)*h2 = 0{,}5*0{,}5*(14*4+12+24+28)*3{,}4 =102m^3$

3^{rd} floor:

$V3 = 0{,}5*0{,}5*(14*4+12+24+28)*h3 = 0{,}5*0{,}5*(14*4+12+24+28)*0{,}34 =102m^3$

4^{th} floor:

$V4 = 0{,}5*0{,}5*(14*4+12+24+28)*h4 = 0{,}5*0{,}5*(14*4+12+24+28)*0{,}34 =102m^3$

5^{th} floor:

$V5 = 0{,}5*0{,}5*(14*4+12+24+28)*h5 = 0{,}5*0{,}5*(14*4+12+24+28)*0{,}34 =102m^3$

6^{th} floor:

$V6 = 0,5*0,5*(14*4+12+24+28)*h6 = 0,5*0,5*(14*4+12+24+28)*0,34 =102m^3$

7th floor:

$V7 = 0,5*0,5*(14*4+12+24+28)*h7 = 0,5*0,5*(14*4+12+24+28)*0,34 =102m^3$

8th floor:

$V8 = 0,45*0,45*(14*4+12+12)*h8 = 0,45*0,45*(14*4+12+12)*3,4 = 55,08m^3$

9th floor:

$V9 = 0,45*0,45*(14*4+12+12)*h9 = 0,45*0,45*(14*4+12+12)*3,4 = 55,08m^3$

10th floor:

$V10 = 0,45*0,45*(14*4+12+12)*h10 = 0,45*0,45*(14*4+12+12)*3,4 = 55,08m^3$

11th floor:

$V11 = 0,45*0,45*(14*4+12+12)*h11 = 0,45*0,45*(14*4+12+12)*3,4 = 55,08m^3$

12th floor:

$V12 = 0,4*0,4*(7*4+12)*h12 = 0,4*0,4*(7*4+12)*3,4 = 21,76m^3$

13th floor:

$V13 = 0,4*0,4*(7*4+12)*h13 = 0,4*0,4*(7*4+12)*3,4 = 21,76m^3$

14th floor:

$V14 = 0,4*0,4*(7*4+12)*h13 = 0,4*0,4*(7*4+12)*3,4 = 21,76m^3$

$V = \sum V1\text{-}V14 = 281,265m^3 + 102m^3 + 102m^3 + 102m^3 + 102m^3 + 102m^3 + 102m^3 + 55,08m^3 + 55,08m^3 + 55,08m^3 + 55,08m^3 + 55,08m^3 + 21,76m^3 + 21,76m^3 + 21,76m^3 =$ **$1178,865m^3$ (RC for columns)**

RC for slabs:

$V = A*h$, where:

V = total required concrete for slabs in m^3

A = total area where needed to place slab

h = the thickness of slab

The area of each floor of every block is $675m^2$. Therefore:

$A = 675*2 + 675*8 + 675*12 + 675*15 = 24975m^2$

$V = 24975*0,15 =$ **$3746,25m^3$ (RC for slabs)**

RC for beams:

$V = \sum a*b*h*l$, where:

V = required amount of concrete for each floor in m^3

a, b = dimensions of particular beam

l = the length of a particular beam

n = required number of particular beam for specific floor

1st floor:

V1 = 0,45*0,3*((30 - 0,6*7)*4 + (30 - 0,6*4 - 0,5*3)*3 + (30 - 0,6*7)*4 + (30 – 0,6*4 – 0,5*3)*3)*2 + +0,45*0,3*((30-0,7*7)*4+(30-0,7*4-0,5*3)*3+(30-0,7*7)*4 + (30 – 0,7*4 – 0,5*3)*3) = 98,01 + 47,925 = =145,935m^3

2nd floor:

V2 = 0,45*0,3*((60 - 0,5*14)*4 + (15 - 0,5*4)*10 + (15 - 0,5*3)*4 + (15 – 0,5*4)*3) +0,45*0,3*((15 -0,5*4)*14+(15-0,5*3)*4+(60-0,5*14)*4 + (15 – 0,5*4)*3) = 58,725 + 65,745 = 124,47m^3

3rd floor:

V3 = 0,45*0,3*((60 - 0,5*14)*4 + (15 - 0,5*4)*10 + (15 - 0,5*3)*4 + (15 – 0,5*4)*3) +0,45*0,3*((15 -0,5*4)*14+(15-0,5*3)*4+(60-0,5*14)*4 + (15 – 0,5*4)*3) = 58,725 + 65,745 = 124,47m^3

4th floor:

V4 = 0,45*0,3*((60 - 0,5*14)*4 + (15 - 0,5*4)*10 + (15 - 0,5*3)*4 + (15 – 0,5*4)*3) +0,45*0,3*((15 -0,5*4)*14+(15-0,5*3)*4+(60-0,5*14)*4 + (15 – 0,5*4)*3) = 58,725 + 65,745 = 124,47m^3

5th floor:

V5 = 0,45*0,3*((60 - 0,5*14)*4 + (15 - 0,5*4)*10 + (15 - 0,5*3)*4 + (15 – 0,5*4)*3) +0,45*0,3*((15 -0,5*4)*14+(15-0,5*3)*4+(60-0,5*14)*4 + (15 – 0,5*4)*3) = 58,725 + 65,745 = 124,47m^3

6th floor:

V6 = 0,45*0,3*((60 - 0,5*14)*4 + (15 - 0,5*4)*10 + (15 - 0,5*3)*4 + (15 – 0,5*4)*3) +0,45*0,3*((15 -0,5*4)*14+(15-0,5*3)*4+(60-0,5*14)*4 + (15 – 0,5*4)*3) = 58,725 + 65,745 = 124,47m^3

7th floor:

V7 = 0,45*0,3*((60 - 0,5*14)*4 + (15 - 0,5*4)*10 + (15 - 0,5*3)*4 + (15 – 0,5*4)*3) +0,45*0,3*((15 -0,5*4)*14+(15-0,5*3)*4+(60-0,5*14)*4 + (15 – 0,5*4)*3) = 58,725 + 65,745 = 124,47m^3

8^{th} floor:

V8 = 0,45*0,3*((30 - 0,45*7)*8 + (15 - 0,45*4)*6 + (15 - 0,45*4)*7*2 + (30 – 0,45*6)*4) = 79,38m^3

9^{th} floor:

V9 = 0,45*0,3*((30 - 0,45*7)*8 + (15 - 0,45*4)*6 + (15 - 0,45*4)*7*2 + (30 – 0,45*6)*4) = 79,38m^3

10^{th} floor:

V10 = 0,45*0,3*((30 - 0,45*7)*8 + (15 - 0,45*4)*6 + (15 - 0,45*4)*7*2 + (30 – 0,45*6)*4) = 79,38m^3

11^{th} floor:

V11 = 0,45*0,3*((30 - 0,45*7)*8 + (15 - 0,45*4)*6 + (15 - 0,45*4)*7*2 + (30 – 0,45*6)*4) = 79,38m^3

12^{th} floor:

V12 = 0,45*0,3*((30 - 0,4*7)*4 + (15 - 0,4*4)*3 + (15 - 0,4*4)*7 + (15 – 0,4*3)*4) = 40,23m^3

13^{th} floor:

V13 = 0,45*0,3*((30 - 0,4*7)*4 + (15 - 0,4*4)*3 + (15 - 0,4*4)*7 + (15 – 0,4*3)*4) = 40,23m^3

14^{th} floor:

V14 = 0,45*0,3*((30 - 0,4*7)*4 + (15 - 0,4*4)*3 + (15 - 0,4*4)*7 + (15 – 0,4*3)*4) = 40,23m^3

V = ∑V1-V14 = 145,935m^3 + 124,47m^3 + 124,47m^3 + 124,47m^3 + 124,47m^3 + 124,47m^3 + 124,47m^3 + 79,38m^3 + 79,38m^3 + 79,38m^3 + 79,38m^3 + 79,38m^3 + 40,23m^3 + 40,23m^3 + 40,23m^3 = **1330,965 m^3 (RC for beams)**

Total RC for columns, beams and slabs:

V = 1178,865m^3 + 3746,25m^3 + 1330,965m^3 = 6256,08m^3