

**DESIGN AND ENERGY PERFORMANCE OF
STUDENT BUILDING INTEGRATED WITH
PHASE CHANGE MATERIAL IN
ASTANA, KAZAKHSTAN**

(Capstone Project II)

Bachelor of Engineering

(Civil Engineering)



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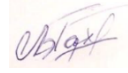
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2017

Declaration

We hereby declare that this report entitled “Design and Energy performance of student building integrated with Phase Change Material in 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

In this report, student resident building integrated with Phase Change Material (PCM) is designed by “EASY Engineering” Construction Company. The project is designated for Astana city’s residents. The proposed location of the 10-storey building is at the intersection of Turan avenue and 31st street, on the left coast side of the city. This paper contains architectural design of the project, structural analysis, geotechnical design and evaluation of energy performance of the building due to application of PCM.

Architectural design of the project specifies non-structural materials, ceilings, floor plan, partitions, stairs and parking details. These material characteristics are further integrated in structural analysis. Moreover, safety issues regarding resident building was also examined based on local codes. The technical drawings of the project are delivered using AutoCAD software and rendered by LUMION Pro 6.

Essential part of the design is extensive structural analysis, in which parallel manual and software estimations of loads, frames, and structural members were accomplished. Building frame made from reinforced concrete is analyzed by SAP2000 software and checked by hand. Structural members including columns, beams, and slabs were designed following Eurocode. Design check for each structural member was also provided. Besides this, detailed reinforcement of structure was made based on reinforcement ratio examined under various load combinations. Software calculation results for loads, slabs, columns and beams were verified by hand calculation. 3D frame of the building was also analyzed with application of ETABS and SAFE software. As a result of structural analysis, detailed technical drawings for structural members were provided.

In geotechnical design part, pile foundation was selected as a suitable basement for student resident building in Astana. Bearing capacity of the single pile was calculated employing static equations and checked using results of cone penetration test extracted from technical report. Based on the results of static analysis of bearing capacity and applied structural load, pile grouping was performed for four major load cases on external, internal and corner columns. Pile group design with pile cap was verified checking lateral resistance and resistance against uplift force. Settlement of pile group was also calculated manually and settlement of single pile under dynamic loading was analyzed in Plaxis 2D software. In addition, sheet pile design, foundation construction process and monitoring, pile quality check after installation was considered

in geotechnical design. Detailed reinforcement of pile foundation along with pile cap design is provided in technical drawings.

This project also covers an energy performance estimation of the student dormitory building. The energy due to the use of Phase Changing Material (PCM) with 10 mm thickness was monitored and estimated. The simulation process was conducted by the Design Builder modeling software, which works in couple with the Energy Plus program, where the temperature change and energy saving was calculated.

Eventually, “EASY Engineering” Construction Company provided construction management for the designed project. It includes project delivery type, cost estimate, project scheduling, work breakdown structure and risk assessment. It was shown that the project is financially and economically feasibility in terms of payback period of PCM and reasonable rent cost of the student dormitory.

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1 INTRODUCTION

1.1 Project Description

“EASY Engineering” Construction Company is aimed to design the student resident building in Astana, Kazakhstan. 10-story student residential building located at the intersection of the Turanavenue and 31st street will be constructed following the design requirements in Eurocodes and local construction regulations. For safe and durable design, pile foundation will be used, while energy performance of the building will be enhanced integrating phase change materials (PCM). The structure type is expected to be reinforced concrete. Architectural design of building considers both owner’s conditions: PCM integrations and the students use.

1.2 Scope of the Project

The scope of the project is to provide final architectural, structural, geotechnical design and energy performance of the building. The design process is based on extensive use of software and code provisions. Final capstone report is supplemented with technical drawings. Appendixes of the report include supportive information to the design.

Team Members

Membership for this project team is restricted to the group of five students, and each role of members is distributed as follows:

1. Yerniyaz Abildin – Structural design team leader;
2. Yerkezhan Madenova – Geotechnical design and group team leader;
3. Bakhytbek Makhin – Architectural design team leader;
4. Gulzhan Negmetova – Construction management team leader;
5. Nadira Yekpyndynova – Energy performance analysis team leader.

Each leader is responsible for own parts, and assigns goals for his/her parts for the whole team, so the main objectives will be achieved by contribution of each team member.

ARCHITECTURAL DESIGN



2 ARCHITECTURAL DESIGN

2.1 Architectural design statement

2.1.1 Introduction of the building

Main purpose of the architectural part of the project is to provide reasonable design of the building to implement PCM (phase change material) panels to the walls. The building provides accommodation for 180 students. Also, building includes multifunctional rooms, kitchens, cafeteria and other functional rooms.

2.1.2 Site location

Location of the student dormitory is shown in figures 2.1.1 and 2.1.2. It is constructed at the intersection “Turan Avenue” and “31st Street”. The area of the territory is 104 m x 56 m. This student dormitory is located near the campus of Nazarbayev University and Nazarbayev Intellectual School, therefore, actuality of the dormitory in this section of the city is beyond doubt. Also, it is not far from the EXPO City which has an official launch in July 2017.

The highlight of the project is phase change material (PCM). They are installed in the walls and absorb the energy and thereby maintain thermal comfort inside building.



Figure 2.1.1 The location of the building in the streets cross-section (Google Maps)



Figure 2.1.2 The location of the building (Google Maps)

2.2 General properties and classification of the building

General building properties:

- Building type: residential building for students (dormitory)
- Total site area: $104 \times 56 = 5\,824 \text{ m}^2$
- Height of the building: 39 m
- Number of stories: 10
- Structure type: reinforced concrete

Classifications of the building (SNIP RK):

- Classification of residential building: Class III
- Fire safety: Class F 1.2 – dormitory
- The degree of fire resistance of building: I

2.2.1 Allocation of the areas

The plan of the floors from 1-9 includes fire exit, elevator hall, kitchen, common room and living rooms. The plan of the ground floor includes fire exit, elevator hall, canteen, common room, pumping room, laundry, minimarket and toilets.

The floor plans are shown in detail in the technical drawing (sheet 1 and 2) and in figures below.

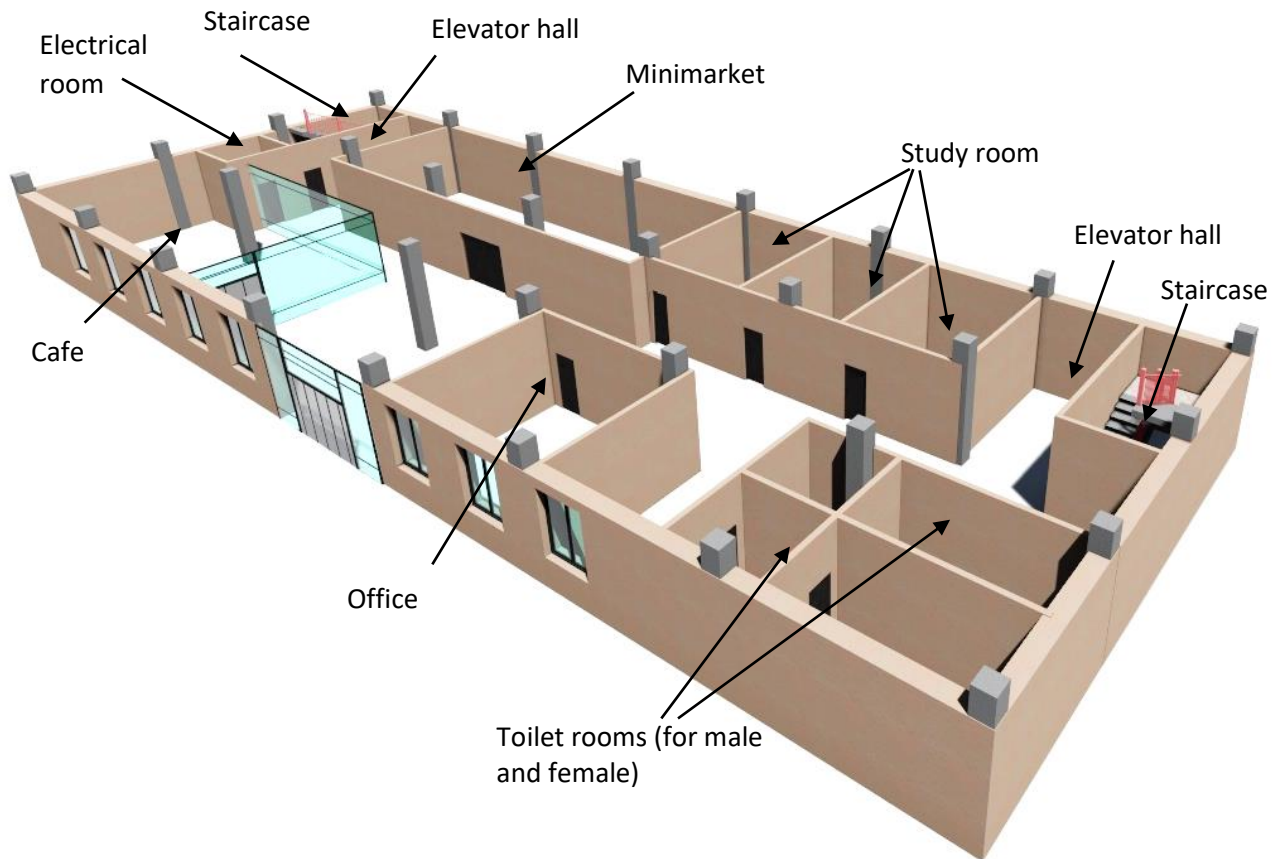


Figure 2.2.1 Rendered ground floor plan (AutoCAD – Lumion 6)

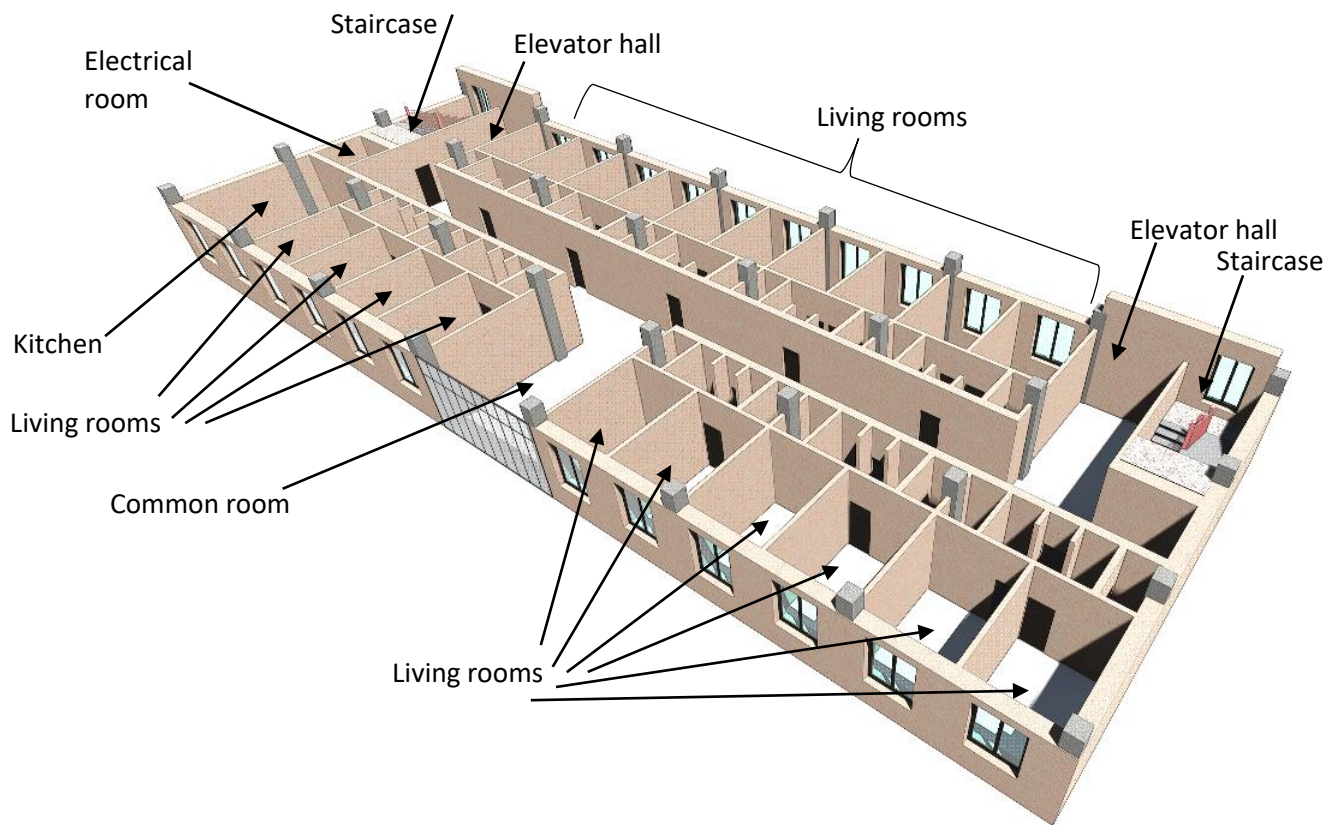


Figure 2.2.2 Rendered typical floor plan (AutoCAD – Lumion 6)

2.3 Non-structural material

Non-structural materials were selected that are appropriate to the aesthetic and functional properties of the dormitory.

2.3.1 Ceiling

In the multifunctional and office areas and in kitchens “the gypsum plasterboard false ceiling system” is installed. This type has following advantages:

- This ceiling has uniform appearance
- Ventilation can be installed
- Easy to change level by creating bulk heads

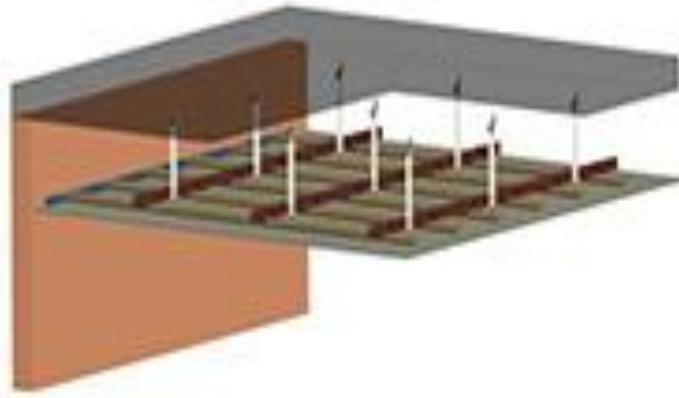


Figure 2.3.1 The gypsum plasterboard false ceiling system (Gyproc n.d.).

“Ecophon ceiling tiles false ceiling system” is perfect ceiling for the living rooms due to its outstanding acoustic performance. Its noise reduction coefficient (NRC): 0.9 – 1 which means that 90 – 100% of the noise is reduced. In addition, this system has different edge designs and it can be easily installed between the false ceiling and soffit (Gyproc n.d.).

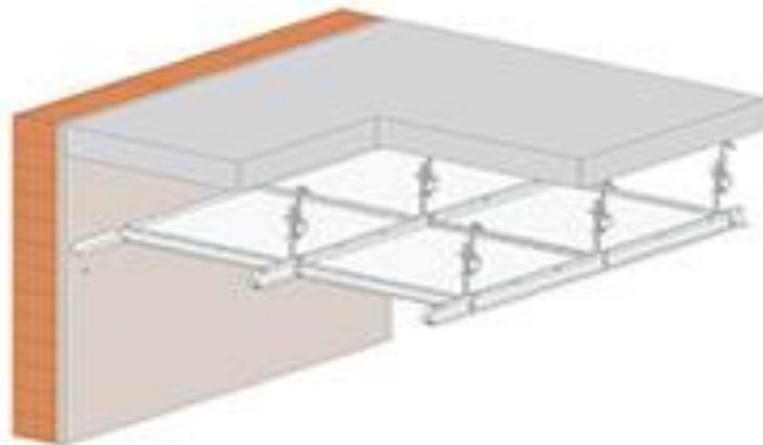


Figure 2.3.2 Ecophon ceiling tiles false ceiling system (Gyproc n.d.).

2.3.2 Floor

Laminate flooring was chosen because it is cheap, strong, durable and appropriate material for common areas with large amount of people. Therefore, it is best solution for the halls, atriums, corridors and multifunctional rooms. Also, it is easier to install this material rather than other flooring materials.

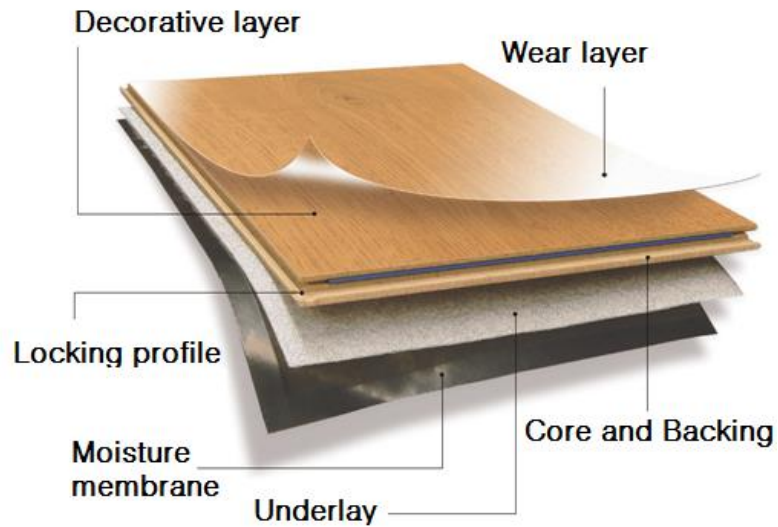


Figure 2.3.3 Laminate flooring (Prestige floors n.d.)

Ceramic tile flooring is chosen for the most waterproofing required rooms, in the bathrooms.

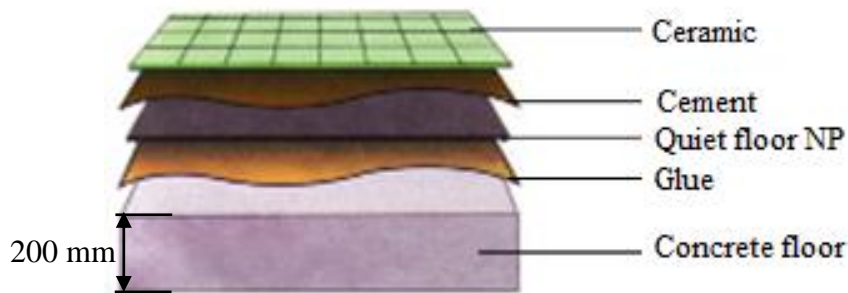


Figure 2.3.4 Ceramic tile (Acoustical surfaces n.d.)

2.3.3 Partitions

Since the building has the Type I, partitions must be framed by metal studs. Galvanized steel sheet metal with 0.45 – 1.37 mm thickness are used to make runner channels and the light gauge steel studs. (Allen 2011). In the picture below the structure of the interior wall is described in detail where the board and masonry wall are spaced away from each other by metal furring strips. The thickness of the masonry is 150 mm.

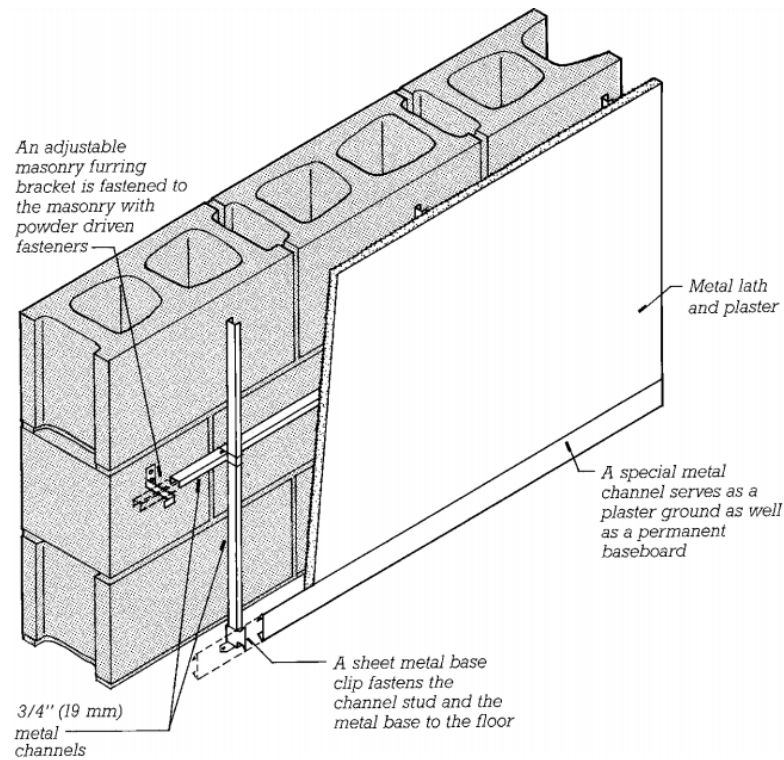


Figure 2.3.5 Partitions (Allen 2011)

2.4 Parking

Several types of parking exist:

- Above-ground parking
- Open parking
- Parking with ramps
- Mechanized parking

The selected type of parking for our project is open parking.

An open parking is the parking that has not external walls. In addition, parking is considered as an open parking if the structure is open (no external walls) at least on two opposite sides with greatest length. (IBC 2.02-05-2000)



Figure 2.4.1 Site layout from Turan avenue's perspective (AutoCAD – Lumion 6)



Figure 2.4.2 Parking (AutoCAD – Lumion 6)

There is two-way traffic flow for vehicles with parking places, which provides comfortable driving and parking in the territory. In addition, the detailed descriptions of the site with parking and traffic flow are illustrated in appendix with technical drawings (page 8: “Site Layout”). There are 2 entrances to the territory for vehicles and 1 for pedestrians from the west side of the building (Turan avenue).

Dimensions:

Minimal dimensions of the parking place (IBC 2.02-05-2000):
Length - 5m and width - 2.3m (3.5m for disabled).

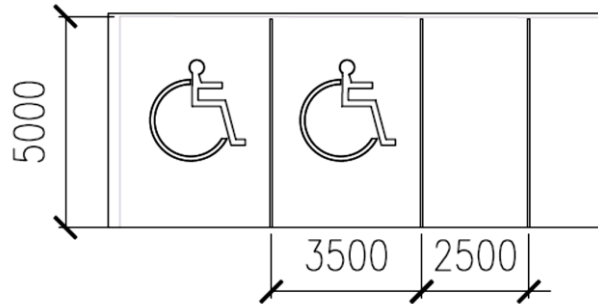


Figure 2.4.3 Dimension of the parking places

Figure 2.4.1 shows that the minimal dimensions were addressed.

2.5 Safety issues

Building includes according to SNIP RK:

- 1) smoke protection;
 - 2) automatic fire extinguishing;
 - 3) elevators for fire departments (fire elevators);
 - 4) automatic fire alarm system;
 - 5) automatic fire alarm and evacuation;
 - 6) means of individual and collective protection and rescue of people;
 - 7) the space-planning and technical solutions to ensure timely evacuation of people and protect them from the hazards of fire;
 - 8) regulation of fire resistance and fire risk constructions and finishing materials;
 - 9) the devices, limiting the spread of fire and smoke (fire barriers, fire compartments, and others.)
- Check formatting

Maximum allowable height of the building and area of the floor is determined according to the table 2.5.1 Since the height and area of the building with the first degree of fire resistance are 39 m and 630 m² (42m x 15m) the design meets the standards.

Table 2.5.1 Maximum allowable height of the building and area of the floor (SNIP RK)

Degree of fire resistance	The class of constructive fire danger building	The maximum height, m	Maximum allowable area of the floor, m ²
I	C0	75	2500
II	C0	50	2500
	C1	28	2200
III	C0	28	1800
	C1	15	1800
IV	C0, C1	5	800
		3	1200
	C2, C3	5	500
		3	900
V	Not regulated	5	500
		3	800

The maximum distance to the nearest evacuative exit is defined according to the table 2.4.2. For the first degree of fire resistance the minimum distance must be not less than 40 m. All floors have 2 fire escapes. The length of the building is 42 m. Therefore, at the location between stairwells the distance is equal to 21 m that is less than 40 m.

Table 2.5.2 Maximum distance to the evacuative exit (SNIP RK 3.02-06-2011)

Degree of fire resistance	The class of constructive fire danger building	Maximum distance from the room/apartment to the evacuative exit, m	
		at the location between the stairwells or external entrances	the dead-end corridor or the gallery
I, II	C0	40	25
II	C1	30	20
III	C0	30	20
	C1	25	15
IV	C0	25	15
	C1, C2, C3	20	10
V	Not regulated	20	10

2.6 Stairs

The width of the stairs intended for the evacuation of people, including, situated in the staircase, are estimated. They are at least not less than the width of emergency exit (door), but also not less than (SNIP RK):

- 1) 1.35 m - class F1.1 and buildings with the number of people of buildings located on any floor other than the first, more than 200 people;
- 2) 0.7 m - for the stairs leading to single workplaces;

3) 0.9 m - for all other cases.

The intermediate platforms in forward march must be at least 1 m.

The widths of the stairs of the dormitory (class F1.2) is 1.2 m that is more than 0.9 m while for the intermediate platforms this dimension is equal to 1.24 m that is more than 1 m.

3 STRUCTURAL DESIGN

3.1 Design loads

The following part of the report will provide information on the computation of design loads that will further be used in the processes of structural analysis and member detailing. Design loads calculated include dead, live, wind and snow loads. The calculations are performed according to the provisions of Eurocodes 0 and 1: Basis of structural design and Actions on structures (2002) with the reference to external sources whenever necessary.

3.1.1 Dead load

Dead load is represented by self-weight of construction works, materials and non-structural elements. In detail dead load can include:

- Walls, partitions, ceiling and flooring;
- Fixed services such as lift equipment, piping, cable trunking etc.;
- Insulation and finishes.

Dead loads applied to this specific structure are presented in the table below. It summarizes the total weight on each level of the building, including roof, attic, typical and ground floors. Detailed information on the distribution and takedown of these loads are provided in the analysis chapter of this report. An illustration of the applied load is also given in Appendix B-1.

Table 3.1.1 Dead load summary

Item	Item element	Volume, m3	Density, kg/m3	Unit weight, kN/m3	Weight, kg	Item quantity	Load, kN
floor finishes	insulation	12.49	105		1311.051		25.72
	concrete screed	624.31	120		74917.2		1469.88
	ceramic tiles	247.03		0.77			190.22
	linoleum	273.93		0.05			13.70
slab		132.91	25				3322.80
ceiling		624.31		0.5			312.16
wall blocks		124.78	600		74865		734.43
external finishes	insulation	37.95	145		5502.75		107.96
	plastering	5.69		18			102.47
	PCM	3.80	860		3263.7		64.03
internal walls		170.82	600		102493.44		2010.92
stairs		5.04		25			251.82
hoistway					1550	2	30.41
columns		0.68		25		32	540.00
beams		7.03		25		12	2107.50
Total on typical floor							11284.01
Total on roof							3103.82
Total on attic							7775.13
Total on ground floor							12722.64

3.1.2 Live load

Live loads on buildings are present from the occupancy and use and may include normal use by people, furniture and movable objects, decorations. Design of live loads is represented by uniformly distributed load on slab. The value of the live load is dependent on the category of the area that is subjected to live load. Live load calculations are presented in the table 3.1.2.

Table 3.1.2 Live load summary

Typical floor			
	q, kN/m2	Area, m2	Load, kN
Bedrooms, kitchens, etc (Category A)	2	624.31	1248.62
Ground floor			
	q, kN/m2	Area, m2	Load, kN
Offices (Category B)	3	101.088	303.264
Cafeteria, common zones (Categories C1, C3)	5	455.83	2279.15
Sports room (Category C4)	5	67.392	336.96
Total			2919.374

Roof			
	q, kN/m ²	Area, m ²	Load, kN
Roof live load (Category H)	0.5	624.31	312.155

3.1.3 Wind load

A procedure for the determination of wind loads on structure is defined by Eurocode 1: Part 4 – Wind actions and include the following steps:

1. Identification of wind velocity and wind pressure

a. Identify basic wind velocity

Basic wind velocity is provided as a multiplication of fundamental wind velocity to directional and season factors. Reference fundamental basic wind velocity for Astana is given as approximately 26 m/s, this value is a result of a meteorological data summary over the last 30 years (Meteoblue, 2014). This value was increased to 40 m/s for conservative design and then used in further calculations.

According to EN 1991-1-4 the directional factor is taken as 1.0 and season factor recommended by the code is also equal to 1.0. Thus, calculated basic wind velocity is 40m/s.

b. Calculate mean wind speed

Mean wind speed at a height z is defined as a function of basic wind velocity, roughness and orography factors. Provided value of orography factor is 1.0, while the roughness factor depends on the elevation of the building and computed from:

$$c_r(z) = 0.19 * \left(\frac{z_0}{z_{0,II}}\right)^{0.07} * \ln \frac{z}{z_0} \text{ for } z_{min} \leq z \leq z_{max} \quad (3.1-1)$$

with $z_{0,II} = 0.05$, minimum elevation of 5 meters and $z_0 = 0.3$ for terrain category III. Maximum z provided as 200 meters. Substituting, roughness factor for 36 meters, the highest elevation of the dormitory, is 0.95.

c. Peak velocity pressure

Peak velocity pressure at elevation z , considers mean and short-term fluctuations in velocity is then determined for the highest elevation of the building:

$$q_p(z) = [1 + 7l_v(z)] * \frac{1}{2} * \rho * v_m^2(z), \quad (3.1-2)$$

where $l_v(z)$ – turbulence intensity given as $\frac{1}{c_0(z) \cdot \ln(\frac{z}{z_0})}$, (3.1-

3)

ρ – air density, recommended value 1.25 kg/m³, and v_m is a mean wind speed.

Identified peak velocity pressure is equal to 2.21 kN/m². Summary of velocity pressures calculated for different level are provided at the end of this section.

2. Computation of wind forces

Wind action on the structure of the dormitory is defined as a sectorial sum of internal and external wind forces. However, the code advises for the design in the most unfavourable conditions. In the case of this building, the effects of external wind forces are taken into account only, since the addition of the internal wind forces leads to the reduction of total wind force.

Total wind forces then will be defined as a multiplication of peak velocity pressure, external wind force and structural coefficients to the reference area. Advised value of structural factor is 1.0 as provided in the clause 6.2 of EN 1991-1-4. External wind pressure coefficients are obtained from the table 7.1 of EN 1991-1-4 and given as 0.8 for windward wall and -0.57 for leeward wall.

Final pressure envelope on the building is dependent on its dimensions, height and length. Since the height of the dormitory is smaller than its length (36 meters compared to 43.6) the pressure envelope has a shape of a uniformly distributed load along the wall, with the value equal to the peak velocity pressure at the highest point of the building, i.e. 2.21 kN/m². Therefore, the total wind forces on each level of the building are calculated with relation to this value.

A summary of wind loads on the building frames, large and small ones are provided in the table below.

Table 3.1.3 Wind forces calculation summary

Level	Elevation, z	Roughness factor, cr(z)	Mean velocity, vm(z)	Peak velocity pressure, N/m2	External wind pressure, N/m2			Reference area A, m2 (small frame)	Reference area A, m2 (large frame)	Total wind force, kN (small frame)	Total wind force, kN (large frame)
					windward wall	leeward wall	total				
Roof	36	0.95	37.90	2210.53	1768.42	-1260.00	3028.42	13.2	11	39.98	33.31
9	33.8	0.94	37.40	2169.74	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
8	30.5	0.91	36.59	2103.96	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
7	27.2	0.89	35.68	2031.62	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
6	23.9	0.87	34.66	1951.14	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
5	20.6	0.84	33.48	1860.30	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
4	17.3	0.80	32.10	1755.78	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
3	14	0.76	30.42	1632.27	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
2	10.7	0.71	28.30	1480.46	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
1	7.4	0.63	25.38	1281.41	1768.42	-1260.00	3028.42	19.8	16.5	59.96	49.97
Ground	4.1	0.56	22.27	1081.48	1768.42	-1260.00	3028.42	21.6	18	65.41	54.51

3.1.5 Snow load

The snow loads should be considered as variable, fixed actions. In accordance with Eurocode 1, snow loads defined as Normal or Exceptional conditions, which have difference in design considerations. In this part, snow load acting on roof will be considered, while snow load acting on ground will be analyzed in geotechnical part. Therefore, snow loads for persistent and transient design situations for our roof will be determined as:

$$s = \mu_i C_e C_t S_k \quad (3.1-1)$$

for which the coefficients are defined by local codes and weather data. The summary of the used values is provided in the table below.

Table 3.1.4 Snow load calculation

definition	sign	value
the snow load shape coefficient	μ_1	0,8
the exposure coefficient	C_e	0,8
the thermal coefficient	C_t	1
the characteristic value of snow load on tile ground	S_k (kN/m ²)	1
Snow Load	s (kN/m²)	0,64

3.1.6 Load combinations

The building should be designed to resist the following load combinations:

1. $DD + LL + LL_r + W$
2. $1.35DD$
3. $1.35DD + 1.5LL + 1.5LL_r$
4. $1.35DD + 1.5LL + 1.5LL_r + 0.9W$
5. $1.35DD + 1.5LL + 0.9W + 0.75S$
6. $1.35DD + 1.05LL + 1.05LL_r + 1.5W$
7. $1.35DD + 1.05LL + 1.5W + 0.75S$
8. $1.35DD + 1.05LL + 0.9W + 1.5S$
9. $1.35DD + 1.5LL + 1.5LL_r + 0.9W_r$
10. $1.35DD + 1.5LL + 0.9W_r + 0.75S$
11. $1.35DD + 1.05LL + 1.05LL_r + 1.5W_r$
12. $1.35DD + 1.05LL + 1.5W_r + 0.75S$
13. $1.35DD + 1.05LL + 0.9W_r + 1.5S$

3.2 Structural analysis

As the initial design of the structure was prepared, it should be analysed in order to investigate whether it can sustain applied loads. Since the proposed structure falls into category of the indeterminate ones, a number of approximations may be necessary to simplify the process of hand calculation. Results and modifications obtained at this stage may then be applied to identify a detailed design of the structure.

To perform an analysis using calculated values of loads it is necessary to transform them and prepare a load takedown sequence. The frame system proposed defines the distribution process of loads from slab to beams and columns. Since the slab is suggested to be a two-way spanning system, the applied loads (dead and live) are transformed onto triangular and trapezoidal line loads on beams. As an example, the peak values of dead loads are provided in the table below. They are then used in the hand analysis and in preparation of model for software analysis.

Table 3.2.1 Dead load distribution

Level	Total dead load on slab, kN	Load on slab, kN/m ²	Load on interior beam w , kN/m	Load on edge beam w , kN/m	Main beam self - weight, kN/m	Minor beam self - weight, kN/m
roof	3103.82	4.67	23.352	11.676	3.75	3
attic	5227.63	7.87	39.331	19.666	3.75	3
typical floors	8636.51	12.99	64.979	32.49	3.75	3
ground floor	9751.14	14.67	73.365	36.683	3.75	3

A detailed process of load takedown is described in hand script from Appendix B.

3.2.1 Frame analysis under dead load

Since the design and dimensioning of members will be performed via the software SAP 2000 it is essentially important to verify that the analysis methods applied there are correct and can produce reliable results. To ensure that, an approximate analysis of the frame internal forces should be performed by hand. Two main load patterns are considered in this report. The initial one is analysis under vertical loading (dead load) and, in the next section of this report, lateral loads (wind load) are considered.

The proposed structure of the building is defined as structurally indeterminate. Currently, approximate methods for hand analysis of such structures exist. In this report to investigate internal axial, shear and moment values under dead load the combination of several approximate methods are used. As the structure of the building is not symmetrical, instead it has large 7 span frame along minor axes and short three span frame along major axis, both of them were considered. In addition, due to the difference in the span lengths, the load shape on beams is different in each axis direction and produces different internal forces.

Initially, to identify the internal forces, the assumptions to transform the frame into statically determinate one will be used. It is assumed that a point of zero moment occurs at a distance of approximately $0.1L$ from the support (see Figure 3.2.1).

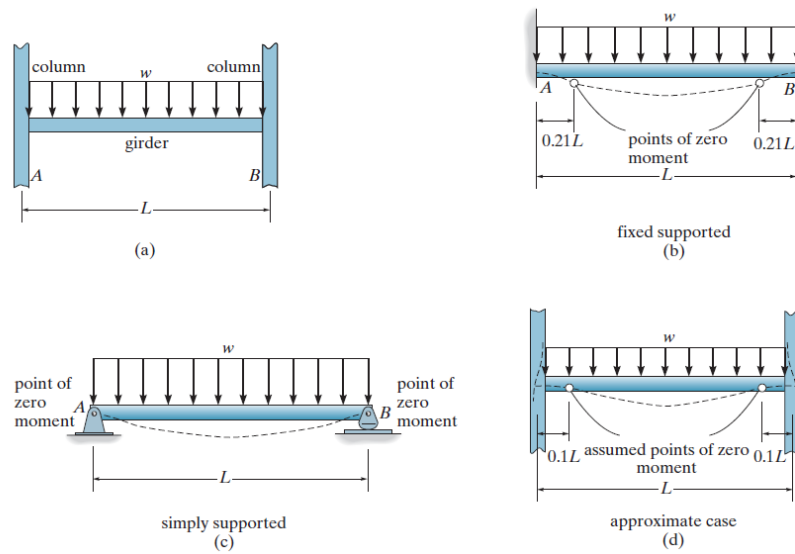


Figure 3.2.1 Assumptions for approximate analysis (Adopted from Hibbeler, 2012).

Then, each girder can be considered as simply supported upon $0.8L$ with two cantilevers supporting its ends. In practice, the real position of the inflection point depends on the column's rigidity. It is important to note that the axial forces in the beam are assumed to be zero (Hibbeler, 2012). From these assumptions, the analysis can be then carried out. From trial and error method it was identified that the actual inflection points for this structure lie in the range of 0.22 to $0.3L$.

To identify the distribution of moments within other members of the frame, the theory of joint stiffness from the displacement method of analysis was used (Hibbeler, 2012). The detailed procedure and results are presented in the Appendix B in the end of this report. General trend is that there is no significant difference between hand and software calculation results.

3.2.2 Frame analysis under wind load

To perform manual analysis under lateral loading, several methods are available at hand. One of the most convenient methods is portal method. It utilizes several assumptions to identify internal forces under lateral loads in frames. Those assumptions state that a point of zero moment is assumed to be at the centre of each girder (beam) and at the centre of each column (Figure 3.2.2); and internal columns carry twice the shear of end columns.

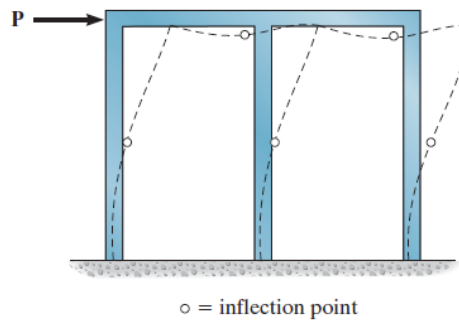


Figure 3.2.2 Portal method assumptions illustrated (Adopted from Hibbeler, 2012).

Results obtained for hand analysis of small frame are compared to SAP2000 output and provided at the Appendix B. Small (short) frame is considered here, since it is assumed to be more sensitive to the application of lateral loads. It is explained by its tall and slender shape.

3.3 Frame design in SAP 2000

Three types of models were generated in SAP2000, 2D models for small and large frame and full 3D model. The materials used are concrete of strength C50/60 and reinforcement are high strength grade 500 bars. Before the simulations, the section properties were adjusted to allow the design for the ultimate states. Since Eurocode does not define exact values for the reduction of second moment inertia values for cracked section, the reference to another widely used code was done. From ACI 318-08 the beam section modifiers were set to 0.35 and 0,7 for columns.

3.3.1 2D frame design

Two types of frames are designed in SAP2000 in accordance with parameter of building shown in table below.

Table 3.3.1 Frame dimensioning

Elevation Level	Elevation height, m	Storey height, m
Attic	37,7	2,2
9	35,5	3,3
8	32,2	3,3
7	28,9	3,3

6	25,6	3,3
5	22,3	3,3
4	19	3,3
3	15,7	3,3
2	12,4	3,3
1	9,1	3,3
Ground	5,8	3,6
Basement	2,2	2,2

- Initial section of long frame beams – 500x300 mm
- Initial section of short frame beams – 500x300 mm
- Initial section of all columns – 500x500 mm

The beam span for short frame is 5 meters, with 6 meters for long frame. With these specifications, the frames were designed in SAP (see Figure 3.3.1)

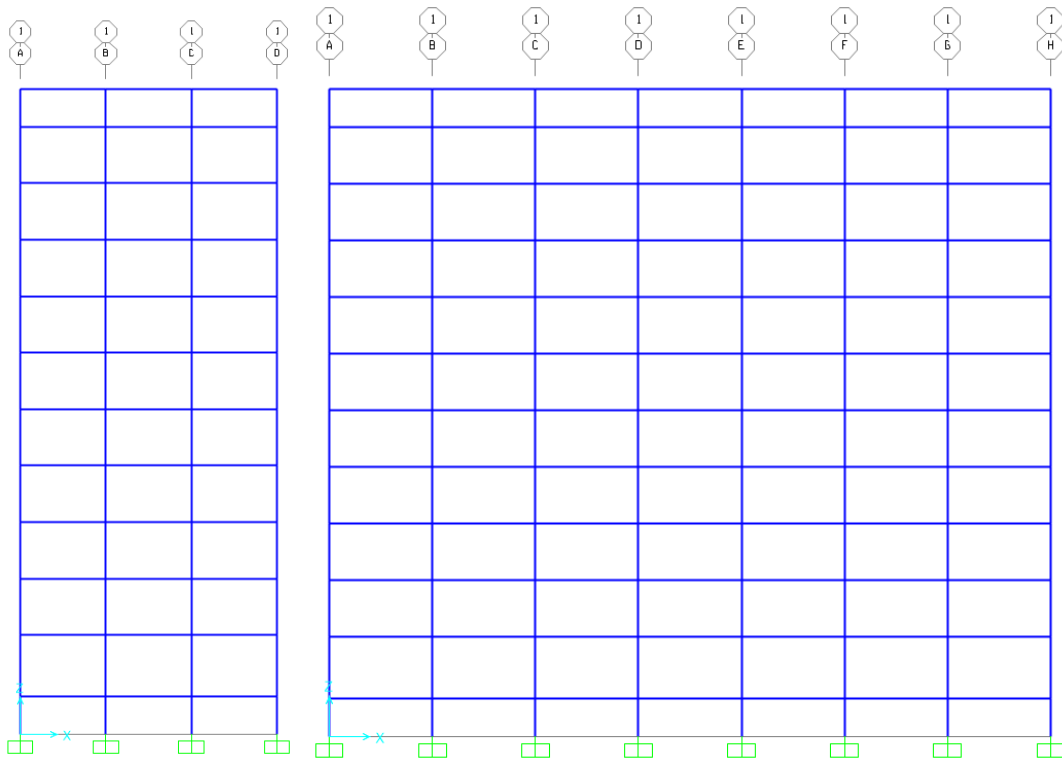


Figure 3.3.1 Short and long frames.

The loads were assigned according to the load takedown pattern defined earlier. Exact shape of dead load and wind load are provided in the Appendix B.

3.3.2 3D frame design and analysis

In order to proceed for beam and column detailing, 3D model of dormitory is designed. Load combinations are generated by program. Isometric and top view of structure can be seen in the following figures.

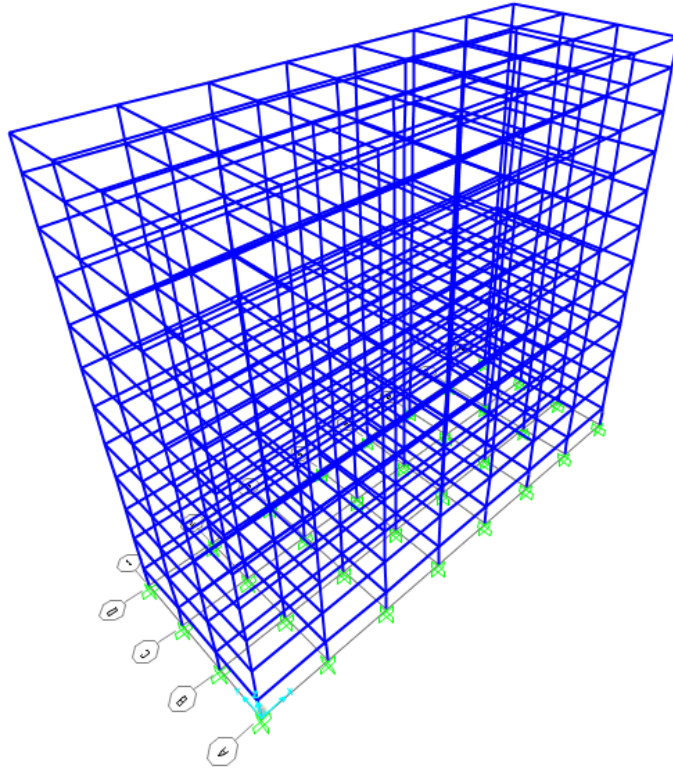


Figure 3.3.2 Isometric view of 3D frame

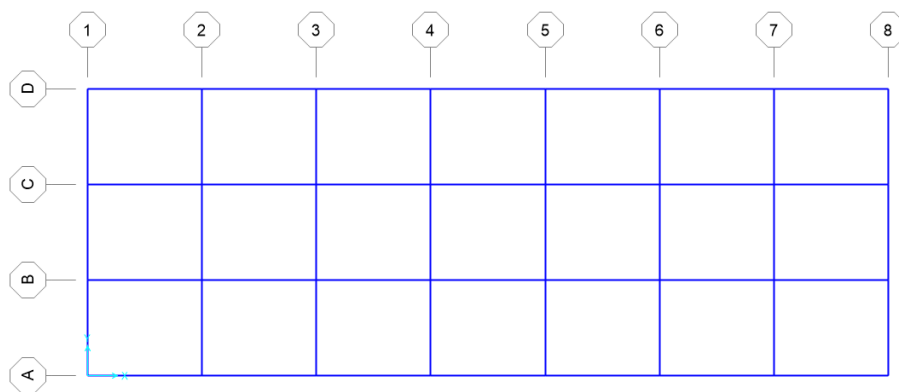


Figure 3.3.3 Top view of 3D frame

Short frame moments of 3D model were compared with 2D model frames to get accurate design assignment in modelling and simulation reinforcement details.

In order to find reinforcement ratio of columns and beams, 3D frame analysis for member design is executed under load combinations. The dimension used at the beginning show that the failure in some members take place, while the others have uneconomical sections (Figure 3.3.4).

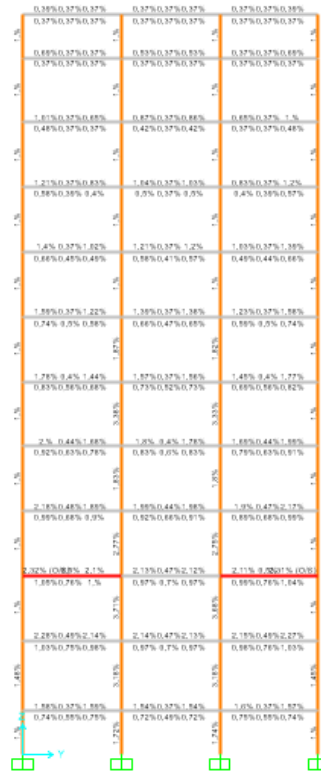


Figure 3.3.4 Failure in short frame

By the trial and error method, the changes to the sections of beams and columns were made. It was noticed that the column sections may be safely reduced in upper floors, and some increase in section of major axis beams may be necessary. By the adjustment of minor beams, it was observed that the structure behaves as for the one-way spanning system: the significant reduction on the sizes of beams along minor axes does not affect the stability of the structure. The only change that occurs is the increase in the reinforcement percentage of the major beams. It was then concluded that the minor beams carry mainly the function of adding stiffness in the longer direction, while the majority of the loads are transferred by major beams.

The updates in frame dimensioning are summarized in the Table 3.3.5 below.

Figure 3.3.5 Finalized frame dimensions

Beams		
Changes	Old section	New section
Long frame beams	500x300	400x300
Short frame beams at 2-3 level of axis 4-5	500x300	550x300
Other short frame beams of all section	500x300	500x300
Column		
Changes	Old section	New section
Columns of 0 and 1 floor	500x500	500x500
Columns of 2-4 floors	500x500	450x450
Columns at 5-11 floors	500x500	400x400

Finally, updated 3D frame was checked in SAP. As a result, all members have passed concrete design requirements set.

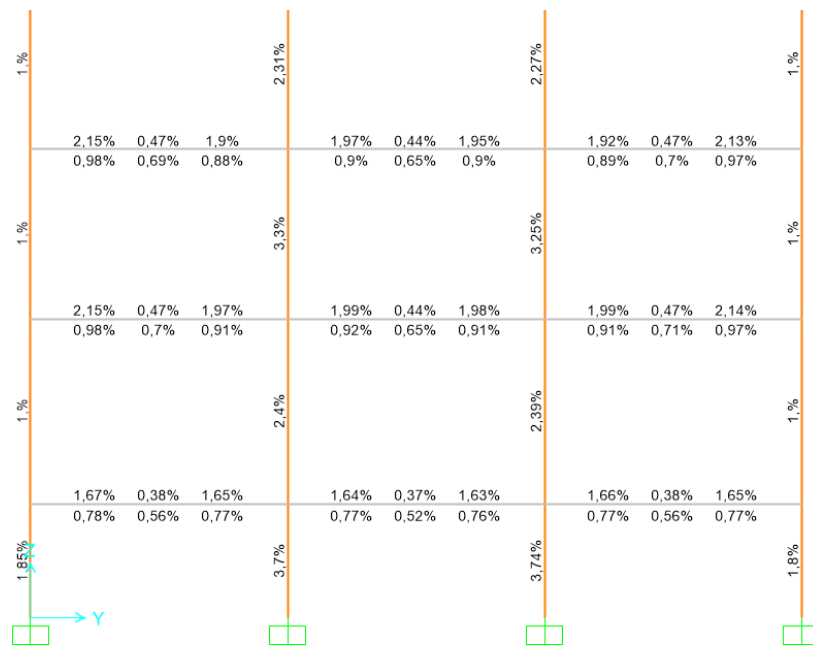


Figure 3.3.6 Example of reinforcement percentage columns and beams for short frame

Additionally, 3D design of structure is developed in ETABS. All design features are assigned as in SAP2000, and the difference in moment diagrams and reinforcement ratio of structural elements for both software can be neglected. Summary data for short frame (frame 5) and long (axis C) frames can be observed in Appendices B.

3.4 Structural detailing

As all the important modifications to the model were applied, the percentage of steel required in each member of the frame can be produced. As these values are obtained, they can be verified by hand calculation procedures defined from Eurocode 2. Then, the detailing of beams and columns is performed. All manual calculations and detailing results are provided in the Appendices B.

3.4.1 Slab design

Considering the slab, the design was performed on the basis of hand calculations with the reference to the relevant codes and annexes. In details, besides EN 1992-1 a reference to British Standard 8110 was done.

This section will describe the procedure for design of one of the slab panels of typical floor. The slab system applied in the building is two-way spanning slab. These types of slabs can be designed by the use of coefficient method, produced from yield line analysis (Bhatt, MacGinley and Choo, 2014). In this method, the moments at centre of the span and its supports are derived from the combination of loads applied, slab panel span and coefficients defined at BS 8110. Shear forces at the supports are also derived by pre-defined coefficients. A full version of both tables can be found in the Appendix B. This method is applicable if two conditions are satisfied:

- characteristic dead and imposed loads are approximately the same in the adjacent spans as in the panel considered;
- the spans of the adjacent panels in the direction perpendicular to the line of the common support are approximately same as that of the panel considered in that direction.

The dormitory frame meets those conditions. The design procedure for the interior panel of typical floor:

1. Identify design values

Trial slab thickness for the typical floor is taken as 200 mm. The effective depth of the inner layer (for conservative design), assuming 25 mm cover and 12 mm bars in both directions:

$$d = 200 - 25 - 12 - \frac{12}{2} = 157 \text{ mm}$$

Design ultimate loads on slab:

$$w = 1.35w_d + 1.5w_l = 1.35 * 13.110 + 1.5 * 2 = 20.699 \text{ kN/m}^2$$

The table below summarizes the coefficients for the moment and shear force calculation at the interior panel for the ratio of long to short span 1.2 (6 to 5 meters).

Panel type	Short span coefficient		Long span coefficient		Short span coefficient		Long span coefficient	
	Cont. edge negative M	Center positive M	Cont. edge negative M	Center positive M	Continuous edge V	Discontinuous edge V	Continuous edge V	Discontinuous edge V
interior	0.042	0.032	0.032	0.024	0.39		0.33	

Design moments are defined as $m_s = \beta_s w l_x^2$ and shear forces: $v_s = \beta_s w l_x$.

2. Minimum steel areas

Minimum area of steel reinforcement is provided by section 9 of EN 1992-1, and should be defined as larger of $0.26 \frac{f_{ctm}}{f_{yk}} * bd$ or $0.0013bd$. Where b – one-meter-wide strip used for design of slab reinforcement, d – effective depth of slab to outer layer of steel, 169 mm. Calculated minimum area of steel is 360 mm²/m; H10 bars at 215 provide 365mm²/m.

3. Design at supports

Moment over short span: $m_{sx} = -0.042 * 20.699 * 25 = 21.734 \frac{\text{kNm}}{\text{m}}$;

$$k = \frac{m_{sx}}{bd^2 f_{ck}} = 21.734 * \frac{10^6}{1000 * 157^2 * 50} = 0.01234;$$

$$\frac{z}{d} = 0.5 * \left(1 + \left(1 - 3 * \frac{k}{1} \right)^{\frac{1}{2}} \right) = 0.741;$$

Required steel area: $A_s = \frac{m_{sx}}{f_{yk} / 1.15 * 0.741 * d} = 432 \text{ mm}^2/\text{m}$. Provide H10 at 180 mm, total of 436mm²/m.

Similar procedure is applied for the longer span, with the respective moment coefficient. The required steel is 328mm²/m, which falls below the limit of minimum area of steel, therefore H10 at 215 mm are provided at longer span supports.

The same process is applied to identify centre moments and required steel area. In the case of interior panel for typical floor, required mid-span steel in both directions is lower than the allowable minimum, therefore H10 bars at 215 mm are provided in both directions.

4. Shear resistance

Shear forces at the edges are computed using the provided coefficients, while the shear resistance of the member is defined by the rules described at the section 6 of EN 1992-1.

Design shear resistance is given as $V_{rd,c} = [C_{rd,c}k(100\rho_1f_{ck})^{\frac{1}{3}}]b_wd$ with a minimum of $V_{rd,c\ min} = 0.035k^{3/2}f_{ck}^{1/2}b_wd$. Computed shear capacity is 301.3 kN, which is far larger than shear forces of 34 and 40 kN at continuous supports. Therefore, no shear reinforcement needed.

5. Reinforcement detailing

Simplified rules for the curtailment of steel for both negative and positive moment steel reinforcement exist (Bhatt, MacGinley and Choo, 2014). They state that 50% of provided support top steel should be extended to the distance 0.3L from the face of support. However, the steel is not curtailed as the area of steel reinforcement would fall below minimum requirement if the bars were cut off. Thus, the full 100% of support top steel extends on 1.5 and 1.8 meters from the face of support for the short and long spans respectively.

Similar procedure is applied to all types of panels at all levels. The summary of results and structural detailing is provided at the Appendix B.

3.4.2 Beam design

Structural detailing of the beams is provided in the tables of Appendix B. To identify the members of the main frame, the reference to the page of Appendix B may be done.

3.4.3 Column design

Structural detailing of the columns is provided in the tables of Appendix B. To identify the members of the main frame, the reference to the page of Appendix B may be done.

3.5 Serviceability design

While the design is performed for the most critical load combination cases, it is important to verify that the building satisfies requirements for the serviceability during its design life. To ensure that, a control of deflections of flexural members and crack control should be performed. The following sections will provide information on the procedures used.

3.5.1 Cracking

The code regulations on serviceability states, chapter 7, specify that the simplified mean of crack control can be used. The measures include the limitation of the maximum bar spacing and bar diameter to limit the maximum crack widths. However, the code also specifies that for the slabs of depth lower or equal to 200 mm, no specific measures of crack control are required.

Depending on the exposure conditions, the max crack width allowed is 0.3 mm, with bar spacing and diameters dependent on the stress level developed in the steel.

3.5.2 Deflections

The application of the limiting span-to-depth ratio allows to omit the explicit deflection calculations. The limiting value of span-to depth ratios can be calculated from the equations provided in the Figure below.

$$\frac{l}{d} = K \left[11 + 1,5\sqrt{f_{ck}} \frac{\rho_0}{\rho} + 3,2\sqrt{f_{ck}} \left(\frac{\rho_0}{\rho} - 1 \right)^{3/2} \right] \quad \text{if } \rho \leq \rho_0 \quad (7.16.a)$$

$$\frac{l}{d} = K \left[11 + 1,5\sqrt{f_{ck}} \frac{\rho_0}{\rho - \rho'} + \frac{1}{12} \sqrt{f_{ck}} \sqrt{\frac{\rho'}{\rho_0}} \right] \quad \text{if } \rho > \rho_0 \quad (7.16.b)$$

where:

l/d is the limit span/depth

K is the factor to take into account the different structural systems

ρ_0 is the reference reinforcement ratio = $10^{-3} \sqrt{f_{ck}}$ [ACI]

ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ' is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)

f_{ck} is in MPa units

Figure 3.5.1 Limiting l/d calculation

The calculated value is then compared to the actual and design is reconsidered where necessary.

In this section of the report the deflections of the slabs will be checked. The detailed table can be found in the Appendix. The actual values of l/d for all slab panels at all levels pass below the limiting value. Therefore, no excessive deflections will occur.

4 GEOTECHNICAL DESIGN

This section of the project deals with the geotechnical design of the student residential building located along Turan avenue in Astana. The geotechnical design plays a vital role to ensure the safe and durable design of the residential building. That is why meticulous design of foundation is required to avoid any unfavorable situations. Firstly, soil characteristics were presented and unpleasant conditions were identified that required design check. Then, bearing capacity of the single pile is determined and necessary pile grouping was accomplished taking into account imposed load by superstructure. Moreover, pile design was checked against lateral loading and uplift force. Calculation of pile group settlement and sheet pile was also provided. Eventually, check of settlement of single pile and sheet pile design is supplemented using Plaxis 2D software.

4.1 Site description

Based on previous studies, Astana city is classified as non-seismic region and there is no danger for the construction industry. The geological feature of the construction site was assumed to be the same as in the territory of the Nazarbayev University, since the chosen area is located nearby the Nazarbayev University zone. Geological investigation results obtained from the technical report of “KaragandaGIIZ and K” firm reveals the soil structure presented in Table 4.1.1.

Table 4.1.1 Geological strata of the site (Technical report by “KaragandaGIIZ and K” firm, 2015)

<i>Fill</i>	Presented by of loam, gruss and rubble soils; its layer capacity is between 0.3 m and 3.0 m.
<i>Saturated sandy loam</i>	Revealed at depth 0.3-3.0 m; lenses of sands and a layer of sandy loam was found; can be described as dark, brown, and gray loams.
<i>Medium dense sand</i>	Was found at depths 4.6-5.5 m. Layer capacity is 0.8-2.4 m; brown saturated sand.
<i>Coarse sand</i>	Revealed at depths 5.3-6.3 m; layer capacity is 1.1-3.2m; saturated, gray and brown.
<i>Gravelly sand</i>	Revealed at depths 5.3-8.5 m; layer capacity is 1.8-7.1 m; saturated.
<i>Gravel</i>	Found at depths 10.3-11.3 m; layer capacity is 0.4-1.9 m; saturated.

On the basis of local construction codes, borings and cone penetration tests were implemented to identify the level of groundwater and soil characteristics. Survey results indicate the water table at depth 1.5 – 2.3 m with fluctuations between 1m and 1.5 m during the seasonal change period. In addition, groundwater is characterized as a

slightly acid, and contaminated with sulfate-sodium. Due to local severe climate and high water table, the ground on the site is susceptible to the freeze actions. Under local construction code named SNiP of RK 2.04-01-2010, average freeze depth in Astana varies for each soil type being 185 cm for clay, 241 cm for loam and sand, and 273 cm for gravel (Technical report by “KaragandaGIIZ and K” LLP, 2015). These factors should be considered while choosing foundation type and during design procedure.

Mechanical-physical properties of the soil taken from the technical report of the “KaragandaGIIZ and K” firm and which was used for the foundation design procedure are given in Table 4.1.2. It should be noted the material partial factors are not applied to ground property values in below Table. Also, layer thickness of each soil represents the average value of the layer thickness from Table 4.1.2. Soil profile of the site produced from geological conditions can be seen in Figure 4.1.1

Table 4.1.2 Mechanical-physical properties of soil (Technical report by “KaragandaGIIZ and K” firm, 2015)

Soil layer	Layer thickness, h (m)	Undrained cohesion, c_u (kN/m ²)	Effective frictional angle, φ'	Modulus of Deformation, E (MPa)	Density, ρ (g/cm ³)	Saturated unit weight, γ_{sat} (kN/m ³)
Fill	1.3	-	-	-	1.87	18.3
Sandy loam	4.3	18	22	6.0	1.97	19.3
Medium dense sand	1.6	-	35	17	1.92	18.8
Coarse/gravelly sand	6.3	-	38	21	2.00	19.6
Gravel	1.2	-	-	23	2.05	20.1

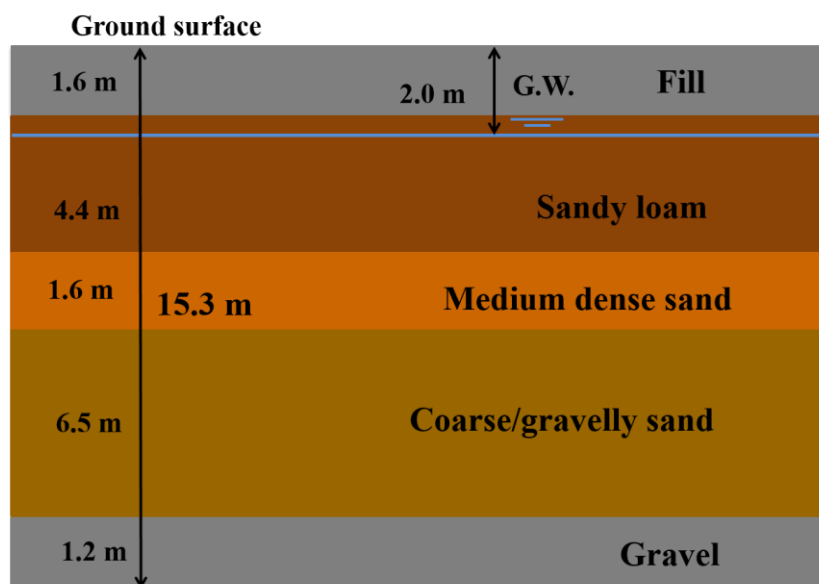


Figure 4.1.1 Soil profile of the site

4.2 Foundation design

The foundation is an essential part of any construction process. It transmits superstructure loads to more reliable ground layer so that whole structure can be supported safely. When considering foundation type, two basic concerns should be satisfied. Particularly, total settlement of the foundation should be in range, and bearing capacity of the foundation must be enough to withstand the vertical and lateral loads. Considering existing soil conditions and transmittable load of a structure, the applicability of shallow and deep foundations was evaluated. Taking into account close water table to the ground surface and heavy load of the 10-storey building, which is sandy loam, shallow foundation does not properly suit, since the imposed loads should be transferred to safer ground part. In addition, due to the weak upper layer, failure in terms of differential settlement can occur in case of shallow foundation. However, using deep foundation can overcome these unfavorable conditions. Deep foundation can withstand lateral loading and distribute the structural loads vertically to the greater depth below the ground surface. Although the deep foundation is not cost beneficial choice comparing to shallow foundation, it is preferable over shallow foundation and should be designed for the proposed project to ensure long standing of the residential building on problematic soil.

It should be noted that 2.5 m of soil will be excavated to locate pile cap and an underground floor. Thus, soil profile becomes as in Figure 4.2.1. Pile driving starts in loam and the water table is on ground surface level, which requires to consider temporary sheet pile design during the construction process.

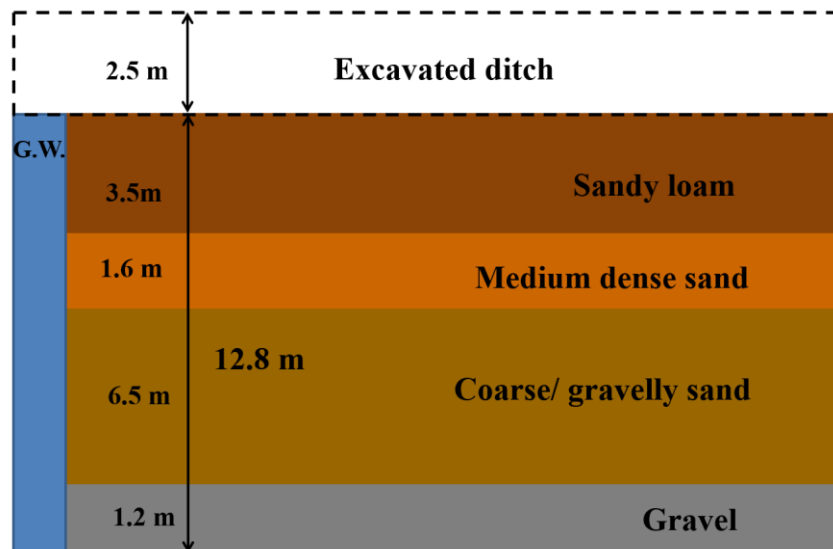


Figure 4.2.1 Soil profile after excavation

4.2.1 Deep foundation selection

Deep foundations feasible for the building of student dormitory are piles and drilled-shaft foundation. The main difference between these types is in the construction process: precast piles are fabricated first and then driven to the ground, while the drilled-shaft foundation is cast in-place (Das, 2007 & Coduto, 2014). To make a decision on one of these, the following factors should be distinguished. First, end-bearing capacity and frictional resistance of the piles are greater than drilled-shaft. Second, the sophisticated construction process of drilled-shaft requires skilled workers and effective supervision. In turn, these needs may face with high salary requirements, shortage of qualified supervisors and depend on the human factor (ibid). In addition, prefabricated piles can be damaged slightly when transporting to the site, whereas long travel distance and bad weather may cause a permanent damage on concrete mixture during delivery for drilled-shaft foundation (Tomlinson & Woodward, 2014). Finally, pile foundation is a common type of foundation employed in Astana. From the above evaluation, it can be inferred that pile foundation is a right choice for the student dormitory building.

Furthermore, to choose the pile type, pile material, durability, load carrying capacity, cost and local availability were examined. Considering the contaminated groundwater and above factors, the decision on pile type was made on behalf of precast concrete pile because it is corrosion resistant, durable, widely available in the local market and affordable in terms of cost. Even if the piles made from steel have a greater carrying capacity with comparison to concrete piles, they were exempted from options due to their susceptibility to corrosion and high cost. Timber piles was also not chosen because of their low bearing capacity. Thus, precast concrete pile will be used designed for the project. The length of the pile will be identified during the design and calculation procedure. Regarding cross-section of the pile, square shape is chosen, since it is widely used and fabricated one in Astana city.

4.2.2 Design load actions

Pile foundation should be designed to withstand the axial loads (vertical superstructure loads) and lateral loads generated from winds and earthquake. Axial loads consist of compression and tension forces applied to the foundation. Compressive loads are opposed by toe (Q_p -point) bearing resistance and shaft resistance (Q_s). Skin friction (Q_s) along the side of the pile and pile weight (W_p) resists tensile loads. Lateral

loads act horizontally to the pile and generate shear force and moment. Design load actions are illustrated in Figure 4.2.2.

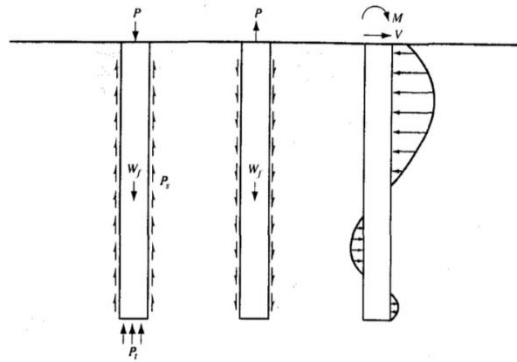


Figure 4.2.2 Load Transfer Mechanism: 1) Compressive Loads; 2) Tensile Loads; 3) Lateral Loads (Coduto, 2014)

As it was mentioned, the water table is close to the ground surface and soil is susceptible to freeze actins due to severe climate and soil conditions. The hydrostatic pattern of the groundwater during seasonal change presents risk of failure for the foundation. Therefore, it is necessary to check the uplift resistance of the pile foundation. Considering heavy loads by the superstructure, necessity of grouping of piles can be seen. Therefore, axial capacity of a pile group based on single pile design and pile cap will be considered in further sections.

4.2.3 Design calculation methods

According to Eurocode 7, there are four methods to estimate the load-carrying capacity of the piles: use of static penetration test (SPT) and cone penetration test (CPT) results, use static equations for bearing capacity calculation, pile load tests, and dynamic method (Bond and Harris, 2008). For the 10-storey dormitory building, static analysis using bearing capacity equations will be employed because of limitations in carrying out of on-site pile load tests. However, it is possible to check the bearing capacity of the pile using CPT results from the report on geological investigation in Astana.

4.3 Bearing capacity of pile group under vertical loads

Because of heavy structural loads, one pile is not enough to carry the applied load. Therefore, the necessity of grouping piles is obvious. Difference in stress transferred to the soil by the single pile and pile group can be seen in Figure 4.3.1. Because of overlapping of the stress transferred from pile group, average load carrying-capacity of the piles in the group may be less than the individual piles in many cases

(Das, 2007 and Tomlinson & Woodward, 2014). Therefore, piles in the group should be spaced and designed efficiently to maximize its ultimate bearing capacity.

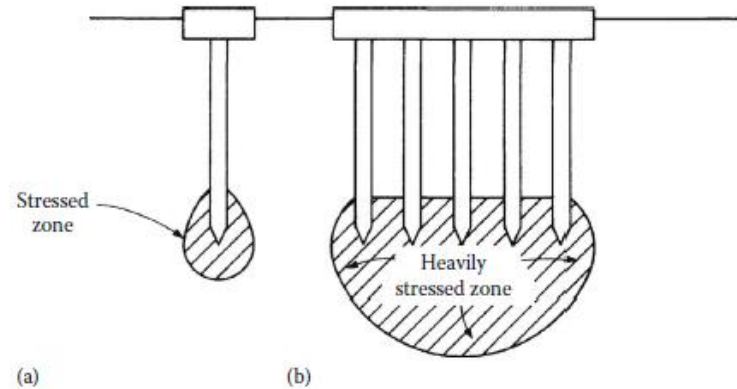


Figure 4.3.1 Comparison of stressed zones below single pile (a) & pile group (b) (Tomlinson & Woodward, 2014: pp.240)

Calculation of load carrying-capacity of the pile group requires to estimate the carrying-capacity of the single pile. The pile length and the axial pile capacity of the single pile depend on the load transfer mechanism of the pile. There are three types of piles as point-bearing piles, frictional piles and compaction piles (Figure 4.3.2). In point-bearing piles, ultimate load capacity of the pile completely depends on bearing capacity of the bedrock or hard stratum. In case of absence of rock or rock like soil, the load is carried by skin friction along the side of the pile. Compaction piles are related to the pile installation method and used to properly compact the soil (Das, 2007).

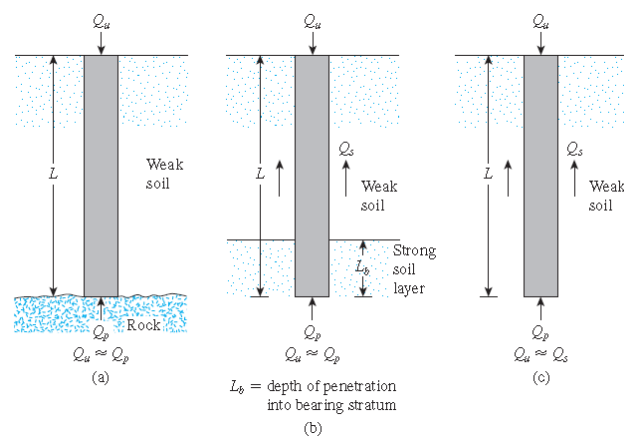


Figure 4.3.2 (a) and (b) Point bearing piles; (c) Friction pile (Das, 2007: pp.547)

According to soil profile in Figure 4.1.1, sandy loam overlays medium to coarse sand. From this soil pattern, it is more likely that piles derive their bearing capacity from both skin friction and end bearing capacity of medium dense sand. The ultimate bearing capacity of a pile can be expressed as:

$$Q_{ult} = Q_s + Q_p \quad (4.1)$$

Where Q_s is shaft resistance along pile length and Q_p is point-bearing capacity of the pile. Further, components of ultimate pile capacity will be computed separately using several static equations to compare and average them.

4.3.1 Point bearing capacity of the single pile (Q_p) in sand: static analysis

Since the pile toe is considered to be terminated in the sand, methods to compute base resistance especially in sand will be considered. The general formula to find point bearing capacity of the pile is given by equation:

$$Q_p = A_p \sigma_p \quad (4.2.1)$$

Where A_p = area of the pile base, σ_p = unit point resistance. Numerous methods are available to calculate the end-bearing capacity of the pile in sand. Three of them were examined and summarized in Table 4.3.1. Equations of total stress, pore water pressure, and effective stress were also provided in Table below, since each method requires estimation of the effective stress.

Table 4.3.1 Q_p calculation methods (Taken from Das, 2007, Ch.11)

Method	Equation
Meyerhof's Method	$Q_p = A_p \sigma_p = A_p \sigma' N_q^* \quad (4.2.1 - a) \leq A_p \sigma_t \quad (4.2.1-b)$ <p>Where: $\sigma_t = 0.5 p_a N_q^* \tan \varphi'$ A_p = area of pile tip σ_p = unit point resistance σ' = effective vertical stress at the level of the pile tip N_q^* = bearing capacity factor (is obtained from Table 1 in Appendix C-1) σ_t = limiting point resistance p_a = atmospheric pressure = 100 kN/m² φ' = effective soil angle of the bearing stratum</p>
Vesic's Method	$Q_p = A_p \sigma_p = A_p \overline{\sigma'_0} N_{\sigma}^* \quad (4.2.1-c)$ <p>Where: $\overline{\sigma'_0}$ = mean effective normal ground stress at the level of the pile point (tip) = $\left(\frac{1+2K_0}{3}\right) \sigma'$ $K_0 = 1 - \sin \varphi'$ = earth pressure coefficient at rest N_{σ}^* = bearing capacity factor = $f(I_{rr})$ (is obtained from Table 2 in Appendix C-1) I_{rr} = reduced rigidity index for the soil = $\frac{I_r}{1+I_r \Delta} \quad (4.2.1-d)$ I_r = rigidity index = $\frac{E_s}{2(1+\mu_s)\sigma' \tan \varphi'} \quad (4.2.1-e)$ E_s = modulus of elasticity of soil (21MPa for coarse sand) $\Delta = 0.005 \left(1 - \frac{\varphi' - 25}{20}\right) \frac{q'}{p_a} \quad (4.2.1-f)$</p>

	$\mu_s = \text{Poisson's ratio of sand} = 0.1 + 0.3 \left(\frac{\varphi' - 25}{20} \right)$ for $25^\circ \leq \varphi' \leq 45^\circ$ Since $\varphi' = 38^\circ$ for sand, $\mu_s = 0.295$
Coyle & Castello's Method	$Q_p = A_p \sigma_p = A_p \sigma' N_\sigma^*$ (4.2.1-g) N_σ^* = bearing capacity factor (is obtained from Figure 1 in Appendix C-1)
Total in-situ stress (σ)	$\sigma = H\gamma_w - (H_A - H)\gamma_{sat}$ (4.2.1-h) γ_w – unit weight of water, γ_{sat} – saturated unit weight of the soil, H-height of the water table from the top of the soil column, H_A - distance between point A and the water table.
Pore water pressure (u)	$u = H_A \gamma_w$ (4.2.1-i)
Effective stress (σ')	$\sigma' = \sigma - u$ (4.2.1-j)

The point-bearing estimation was made in an Excel spreadsheet as can be seen in Table 4.3.2. The calculation expanded using different values of pile cross-section and length to obtain their optimal values. It was observed that prefabricated piles with cross-section 0.3 m x 0.3 m, 0.35m x 0.35m, and 0.4m x 0.4m are available in Astana. However, piles can also be made by order. Taking this into account, piles with square cross-section and breadth of 0.4 m and 0.5 m will be used to calculate the base and the frictional resistances of the pile.

Table 4.3.2 Calculation of point bearing capacity of the single pile

Reference	In-situ stress			Meyerhof's method			Vesic's method					Coyle & Castello's method			Base resistance
	4.2.1-h	4.2.1-i	4.2.1-j	Table 1	4.2.1-a	4.2.1-b	4.2.1-f	4.2.1-e	4.2.1-d	Table 2	4.2.1-c	Fig.1	Table 2	4.2.1-g	$(Q_{p1}+Q_{p3})/2$
Pile Length, L (m)	σ	u	σ'	N^*_{σ}	Q_p	Q_{p1}	Δ	I_r	I_{rr}	N^*_{σ}	Q_{p2}	L/D	N^*_{σ}	Q_{p3}	Average Q_p
6	115.27	58.86	56.41	231	2084.914	1443.82	0.000987	183.97	155.70	137.79	733.18	15	76	685.95	1064.88
7	134.87	68.67	66.2	231	2446.752	1443.82	0.001159	156.77	132.67	126.94	792.72	17.5	78.5	831.47	1137.64
8	154.47	78.48	75.99	231	2808.59	1443.82	0.00133	136.57	115.58	118.90	852.26	20	79.5	966.59	1205.20
9	174.07	88.29	85.78	231	3170.43	1443.82	0.001501	120.98	102.39	112.68	911.79	22.5	78	1070.53	1257.18
10	193.67	98.1	95.57	231	3532.27	1443.82	0.001672	108.59	91.90	106.72	962.05	25	78.5	1200.36	1322.09
11.5	223.07	112.82	110.26	231	4075.02	1443.82	0.001929	94.13	79.66	99.40	1033.75	28.8	77	1358.34	1401.08
Pile width = 0.5 m ($A_b=0.25 \text{ m}^2$)															
Reference	In-situ stress			Meyerhof's method			Vesic's method					Coyle & Castello's method			Base resistance
	4.2.1-h	4.2.1-i	4.2.1-j	Table 1	4.2.1-a	4.2.1-b	4.2.1-f	4.2.1-e	4.2.1-d	Table 2	4.2.1-c	Fig.1	Table 2	4.2.1-g	$(Q_{p1}+Q_{p3})/2$
Pile Length, L (m)	σ	u	σ'	N^*_{σ}	Q_p	Q_{p1}	Δ	I_r	I_{rr}	N^*_{σ}	Q_{p2}	L/D	N^*_{σ}	Q_{p3}	Average Q_p
6	115.27	58.86	56.41	231	3257.68	2255.96	0.000987	183.97	155.70	137.79	1145.60	12	74	1043.59	1649.77
7	134.87	68.67	66.2	231	3823.05	2255.96	0.001159	156.77	132.67	126.94	1238.63	14	75	1241.25	1748.61
8	154.47	78.48	75.99	231	4388.42	2255.96	0.00133	136.57	115.58	118.90	1331.65	16	77	1462.81	1859.38
9	174.07	88.29	85.78	231	4953.80	2255.96	0.001501	120.98	102.39	112.68	1424.68	18	78.5	1683.43	1969.70
10	193.67	98.1	95.57	231	5519.17	2255.96	0.001672	108.59	91.90	106.72	1503.20	20	79.5	1899.45	2077.71
11.5	223.07	112.82	110.26	231	6367.23	2255.96	0.001929	94.13	79.66	99.40	1615.23	23	78	2149.97	2202.97

Note on calculation of base resistance:

- According to study, Vesic's method is less accurate and it can contribute to unreasonable reduction of point-bearing capacity of a pile, therefore, it is a less preferable method with comparison to Meyerhof's and Coyle and Castello's methods. That is why base resistance was averaged using latter two methods.

4.3.2 Skin friction (Q_s) of the single pile: static analysis

According to soil profile, the pile is embedded to two layers of the soil: loam and sand. Hence the frictional resistance should be calculated for each layer separately. For conservative estimation, the loam will be treated as a clay, while two layers of the sand will be combined and characteristics of the thick layer will be taken. The frictional shaft resistance of the pile can be found employing the equation (4.2.2):

$$Q_s = \sum p \Delta L f \quad (4.2.2)$$

Where p presents perimeters of the pile section, ΔL is incremental pile length over which p and f taken to be constant, f is unit friction resistance at any depth z . The Coyle and Castello's method is used to calculate the f in the sand, while several methods are available for clay as shown in Table 4.3.3 Table 4.3.4 demonstrates skin friction calculation in Excel spreadsheet using equations from Table 4.3.3

Table 4.3.3 Skin friction calculation methods (taken from Das, 2007, Ch.11)

Method	Equations
Sand	
Coyle & Castello's method	$Q_s = pLK\overline{\sigma'_o} \tan \delta' \quad (4.2.2-a)$ $K = \text{effective earth pressure coefficient (Figure 2 in Appendix C-1)}$ $\overline{\sigma'_o} = \text{mean effective overburden pressure}$ $\delta' = \text{soil-pipe friction angle} = 0.8\phi'$
Clay	
λ – Method	$Q_s = pL\lambda(\overline{\sigma'_o} + 2c_u) \quad (4.2.2-b)$ $\overline{\sigma'_o} = \text{mean effective stress for the entire embedment length}$ $c_u = \text{mean undrained shear strength}$ $\lambda \text{ depends on embedment length (Table 3 in Appendix C-1)}$
α – Method	$Q_s = pL\alpha c_u \quad (4.2.2-c)$ $\text{For } \alpha \text{ use Table 4 in Appendix C-1.}$ $\overline{\sigma'_o} = \text{average vertical effective stress, } C \approx 0.4 \text{ to } 0.5 \text{ for bored piles and } \geq 0.5 \text{ for driven piles}$
β – Method	$Q_s = pLf\beta\sigma'_o \quad (4.2.2-d)$ $\sigma'_o = \text{effective vertical stress}$ $\beta = K \tan \phi'_R \quad (4.2.2-e)$ $\phi'_R = \text{drained friction angle of remolded clay } K = 1 - \sin \phi'_R \text{ (for normally consolidated clays)}$

Table 4.3.4 Calculation of skin friction of the single pile

Shaft resistance in clay:			Pile width (B) = 0.4 m ($A_b=0.16 \text{ m}^2$)							
Reference	Soil parameters			λ method		α method		β method		Frictional resistance
	p = 4B	Table 4.1.2	4.2.1-j	Table 4	4.2.2-b	Table 5	4.2.2-c	4.2.2-e	Qs3	$(Qs1+Qs2)/2$
Pile length (m)	p (m)	c_u	σ_o ave	λ	Qs1	α	Qs2	β	Qs3	Qs ave
3.5	1.6	18	16.61	0.382	112.54	0.936	94.35	0.2887	26.85	103.44
Shaft resistance in sand:			Pile width = 0.5 m ($A_b=0.25 \text{ m}^2$)							
3.5	2	18	16.61	0.382	140.67	0.936	117.94	0.2887	33.56	129.30
Shaft resistance in sand:			$Q_s=(K*\sigma_o \text{ ave}*\tan(\delta'))L*p$ (4.2.3-a)							
Pile length (m)	Pile width (m)	p (m)	σ_o ave	L/B	K	$\delta'=0.8\phi'$	$\tan(\delta')$	Qs	Total Qs=Qs(clay)+Qs(sand)	
6	0.4	1.6	62.59	23.75	1.7	28.8	0.55	561.51	664.96	
7.5	0.5	2	69.9275	22	1.8	28.8	0.55	1037.96	1167.26	

Notes on calculation of frictional resistance:

- When calculating shaft resistance in clay, result of β method does not included into estimation of average Qs because of notable difference value with comparison to the other two methods;
- Frictional resistance in sand was calculated considering the effect of pile installation and critical pile depth. According to Meyerhof, when the friction angle of the sand below the pile tip is 38° , soil-pile friction angle is 36° as shown in Figure 4.3.3. This is due to the increased density of soil surrounding the pile because of pile driving and vibration effects.

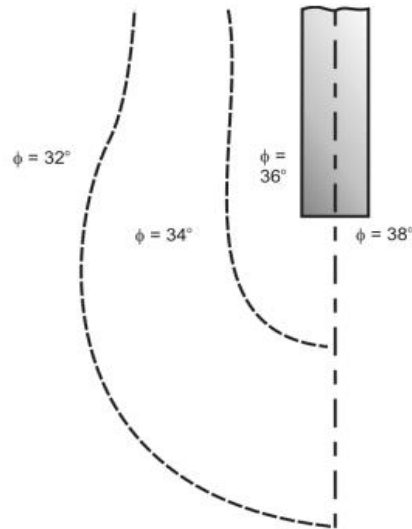


Figure 4.3.3 Friction angle contours of driven pile in sand (Meyerhof, 1963)

- Estimate of frictional resistance in sand included critical depth (L'), since unit skin friction increases till this depth approximately linearly, and after that remains constant (see Figure 4.3.4). In a more conservative way, $L' \approx 15D = 15 \cdot 0.4$ (0.5) = 6.0 m (7.5 m) (Tomlinson, and Boorman, 2001). Since the frictional resistance of the sand remains constant after 6 m (7.5 m) depth, further calculation was not made.

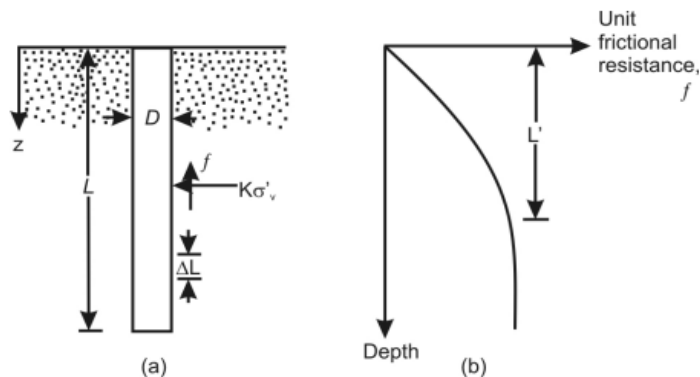


Figure 4.3.4 Unit frictional resistance for piles in sand (Das, 2007: pp.56)

4.3.3 Check of the bearing capacity of the pile in the field using results of CPT

Carrying-capacity of piles found from static equations need to be checked field tests to ensure adequate design of carrying-capacity of piles. In general, two most used in-situ tests to identify bearing capacity of the pile are standard penetration test (SPT) and cone penetration test (CPT). For soil in Astana, CPT was conducted by “KaragandaGIIZ and K” firm. In cone penetration testing in soil in Astana, conical tip with cone base diameter of

35.7 mm and apex angle of 60° was used. Standard pipe with diameter of 36 mm played a role of rod to transmit the pressure to the tip. Diameter and length of sleeve friction are 35.7 mm and 310 mm, respectively. The range of the unit resistance of the soil to the cone is 1 – 30 MPa, whereas the range of measuring of resistivity of sleeve friction is 5 – 500 kPa. Accuracy of the measurement is 5 % (Technical Report by KaragandaGIIZ and K” firm, 2015).

In CPT, strain gauge probe works as following: pressed probe into the ground causes pressure on cone, which transmits to the lower part of the strain gauge, where strain gauge detectors are located. The change in the resistance of the strain gauges is fixed by a measuring device. The force through the widened part of the strain gauge and threaded connection is directly transmitted to the probe body, while the upper part of the dynamometer, where the strain gauges are attached, is not loaded. When the frictional forces act on the sleeve probe, the force through the ledge is transferred to the top of the strain gauge, where the strain gauge detectors are located. At the same time, the change in the resistance of the strain gauges is fixed by the measuring device, and this load is not transferred to the lower part of the strain gauge (ibid). Average CPT results obtained in this way were extracted from technical report on geotechnical investigation by “KaragandaGIIZ and K” firm and given in Table 4.3.5. Entire results of cone penetration testing conducted at different borehole numbers can be found in Appendix C-2.

Table 4.3.5 CPT results (taken from the Technical Report of “KaragandaGIIZ and K” firm)

Soil layer	Cone resistance, MPa	Sleeve friction, kPa
Fill	2.8	122
Loam	1.5	38
Medium dense sand	11.8	87
Coarse sand	18.5	140

Meyerhof (1956) recommended that $q_p \approx q_c$ in sand, where q_p is unit point resistance of the pile, and q_c is cone penetration resistance (Das, 2007). According to pile design, considered in sections 1.3.1-1.3.2, pile with lengths 6 m to 11.5 m is terminated in coarse sand layer. Therefore, cone penetration resistance of coarse sand layer is used to calculate point resistance of the single pile. Table 4.3.6 shows the comparison between point resistance of the pile calculated employing static equations and CPT results.

Table 4.3.6 Point resistance of the pile calculated using static equations and CPT results

pile length	pile width = 0.4 m		pile width = 0.5 m	
	Qp (static equations)	Qp (CPT results)	Qp (static equations)	Qp (CPT results)
6 m	1064.88 kN	$Q_p = A_p q_p$ $0.16 * 18500 = 2960 \text{ kN}$	1649.77 kN	$Q_p = A_p q_p$ $0.25 * 18500 = 4625 \text{ kN}$
7 m	1137.64 kN		1748.61 kN	
8 m	1205.20 kN		1859.38 kN	
9 m	1257.18 kN		1969.70 kN	
10 m	1322.09 kN		2077.71 kN	
11.5 m	1401.08 kN		2202.97 kN	

Based on soil profile, 3.5 m of the pile length is penetrated through loam layer. Treating loam as clay for more conservative estimation, Nottingham and Schmertmann (1975) and Schmertmann's correlation (equation 4.2.3) for skin friction can be used to find the shaft resistance of the pile (Das, 2007).

$$Q_s = \sum \alpha' p \Delta L f_c \quad (4.2.3)$$

Where α' = correlation factor, which can be obtained from Figure 4.3.4, p = perimeter of the pile section, ΔL = incremental pile length, f_c = sleeve friction in clay layer. From Table 4.3.5, sleeve friction is equal to 38 kN/m^2 for loam layer. Based on Figure 4.3.5, $\alpha' = 0.88$ for concrete piles when $f_c/p_a = 0.38$. Taking these values into account, shaft friction of the pile can be calculated using equation 4.2.2. Comparison between shaft resistance in clay found from static equations and CPT results can be seen from Table 4.3.7.

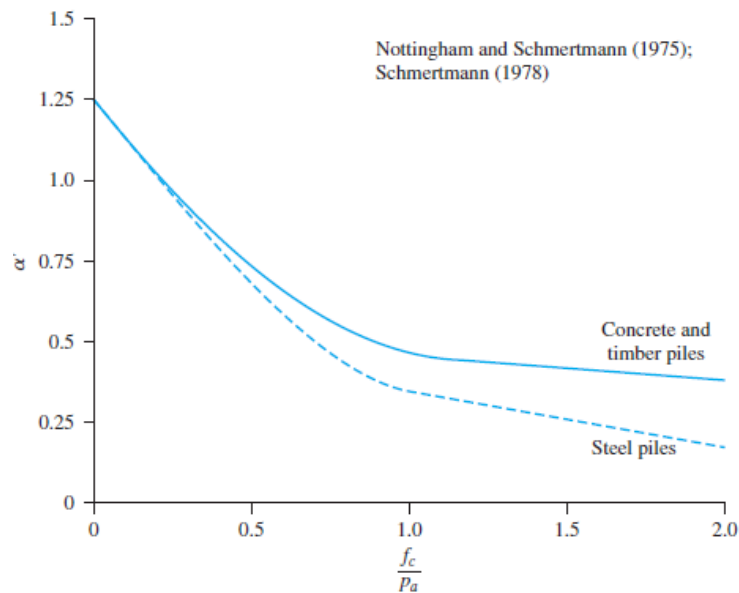


Figure 4.3.5 Variation of α' with f_c/p_a for piles in clay (p_a = atmospheric pressure = 100 kN/m^2) (Das, 2007: pp.579)

Table 4.3.7 Comparison between shaft resistance in clay

Pile shaft resistance in clay			
Pile width	Static equation		CPT results
0.4 m	103.4 kN	<	187.264
0.5 m	129.3 kN	<	234.08

Rest part of the pile length is passed through sand layer. Unit shaft friction of the pile in sand can be calculated using Table 4.3.8, in which ultimate unit shaft friction = $0.012 q_c$ for precast concrete piles.

Table 4.3.8 Relationships between pile shaft and cone resistance in sand

Pile type	Ultimate unit shaft friction
Timber	$0.012 q_c$
Precast concrete	$0.012 q_c$
Precast concrete enlarged base ^a	$0.018 q_c$
Steel displacement	$0.012 q_c$
Open-ended steel tube ^b	$0.008 q_c$
Open-ended steel tube driven into fine to medium sand	$0.0033 q_c$

Source: After Meigh, A.C., *Cone Penetration Testing*, CIRIA-Butterworth, London, UK, 1987.

For conservative estimation, cone resistance of the medium dense sand was considered. Results of CPT based shaft resistance calculation in sand and comparison between shaft resistance obtained from CPT results and static equations is shown in Table 4.3.9.

Table 4.3.9 Comparison between shaft resistance in sand

Pile shaft resistance in sand			
Pile width	Static equation		CPT results
0.4 m	561.5	<	792.96
0.5 m	1038.0	<	1132.8

Following above bearing capacity evaluation based on CPT results, it can be inferred that static assessment of bearing capacity of the pile is safe and can be used for further design procedures.

4.4 Applied load of superstructure

The frame of 10-storey building including underground floor was analyzed with SAP2000 under different load combinations. The most critical loads at each base joints on

pile foundation was inserted in Table 4.4.1. Maximum vertical load by superstructure is 7858.51 kN, which is carried by internal column. Generally, the greatest loads are carried by interior columns, while corner columns take the lowest loads and external columns carry intermediate loads. This can be seen from Table 4.4.1 and Figure 4.4.2 From Table below, critical loads can be divided into four major loads with maximum values of 3416.6 kN, 4393.1 kN, 5691.6 kN and 7858.5 kN. This requires four pile group design for each case. Maximum horizontal load given in Table 4.4.2 was also obtained from computer simulation to check the lateral resistance of the pile.

Table 4.4.1 Critical loads on base pile foundation (obtained from SAP2000 software)

#	Base joints	Shear, Hx	Shear, Hy	Axial, N	Mx	My
Combination: 1.35DD+1.05LL+1.05LL _R +1.5W _r (Combo 11)						
1	1	48.11	177.89	3400.33	344.86	26.29
2	49	-9.15	232.92	5621.55	-382.42	-11.77
3	97	-4.05	232.30	5691.49	-381.83	-7.23
4	145	-2.81	232.41	5683.63	-381.81	-5.40
5	193	-1.12	232.40	5683.65	-381.81	-3.25
6	241	0.21	232.26	5691.59	-381.78	-1.32
7	289	5.21	232.68	5620.42	-382.18	3.32
8	337	-34.30	178.88	3416.60	-345.98	-21.62
Combination: 1.35DD+1.5LL+1.5LL _R +0.9W (Combo 4)						
9	13	95.20	-118.02	4393.09	214.40	69.11
10	61	-1.88	-139.66	7709.25	229.25	1.28
11	109	2.94	-139.90	7858.51	229.24	4.30
12	157	1.67	-139.82	7849.06	229.12	3.09
13	205	1.29	-139.82	7849.07	229.12	2.50
14	253	-0.02	-139.87	7858.46	229.22	1.27
15	301	5.23	-139.61	7710.51	229.18	4.63
16	25	93.83	-118.14	4381.11	214.49	68.05
Combination: 1.35DD+1.5LL+1.5LL _R +0.9W _r (combo 9)						
17	349	-76.11	118.36	4380.55	-214.70	-52.48
18	73	-9.02	139.30	7709.53	-228.96	-9.41

19	121	-2.42	139.42	7857.96	-228.85	-4.27
20	169	-2.09	139.34	7848.48	-228.71	-3.49
21	217	-0.87	139.34	7848.47	-228.72	-2.11
22	265	-0.50	139.45	7858.01	-228.89	-1.31
23	313	5.67	139.37	7708.27	-229.05	3.50
24	361	-77.48	117.81	4392.55	-214.22	-53.54
Combination: 1.35DD+1.05LL+1.05LL _R +1.5W (Combo 6)						
25	37	61.14	-193.66	3416.34	356.25	47.28
26	85	1.09	-249.45	5619.34	393.68	4.80
27	133	3.61	-250.60	5689.90	394.21	6.12
28	181	2.38	-251.62	5681.98	394.82	4.83
29	229	1.57	-251.64	5682.01	394.84	3.83
30	277	0.24	-250.66	5689.94	394.29	2.45
31	325	2.86	-249.73	5620.71	393.98	3.67
32	373	-74.94	-192.70	3400.26	355.18	-51.93

Table 4.4.2 Critical lateral load at base joints (obtained from SAP2000 software)

Combination	Base joint	Hx, kN	Hy, kN	Shear, kN	Axial, kN
Combo 6	37	61.14	-193.66	203.08	3416.34
Combo 6	13	92.12	-196.77	217.27	4230.37
Combo 6	229	1.57	-251.64	251.64	5682.01
Combo 11	61	-11.06	234.40	234.66	7360.78

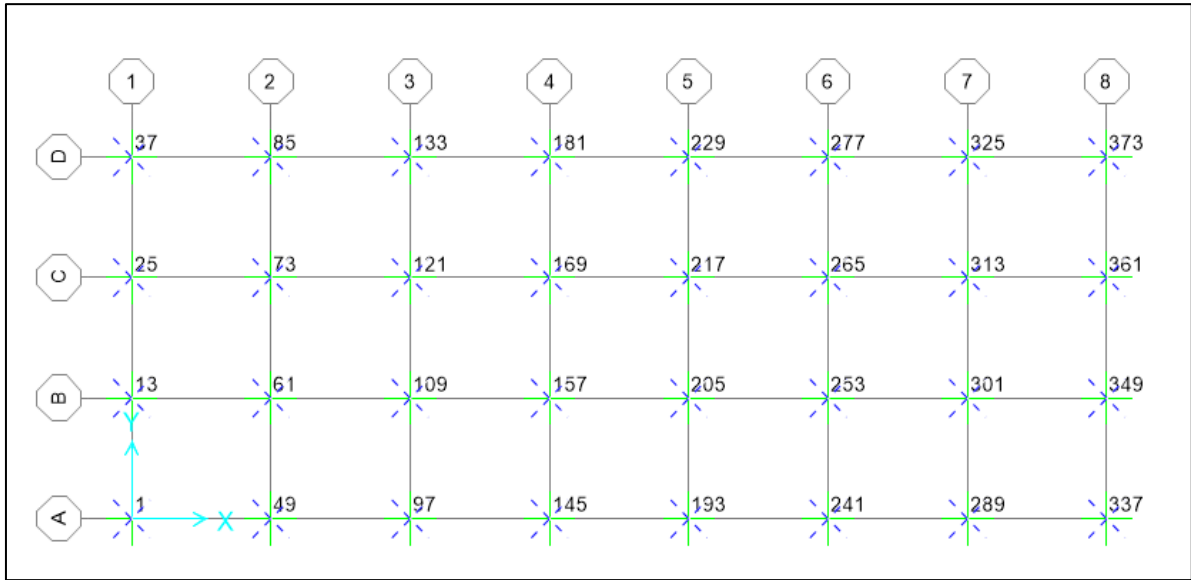


Figure 4.4.1 Base joints location (obtained from SAP2000 software)

4.5 Ultimate ($Q_{g(u)}$) and allowable ($Q_{g(all)}$) bearing capacities of the Pile Group

As mentioned before, piles should be grouped efficiently, so that the load-carrying capacity of the pile group is not less than the sum of the ultimate capacity of individual piles in the group. Generally, bearing capacity of the pile group is obtained by multiplying total ultimate bearing capacity of piles in the group by efficiency factor (η) as shown in equation (4.3.1):

$$Q_{g(u)} = \eta \sum Q_u \quad (4.3.1)$$

However, Viggiani et al states that for driven piles in cohesionless soil, group efficiency is greater than one for center to center spacing of piles between 2.5d and 3.5d (2012). When the pile spacing is increased and become more than 6d, efficiency tends to unity (ibid). Vesic (1967) also proved this through tests on 4 and 9 pile groups, which can be seen in Figure 4.3.3.

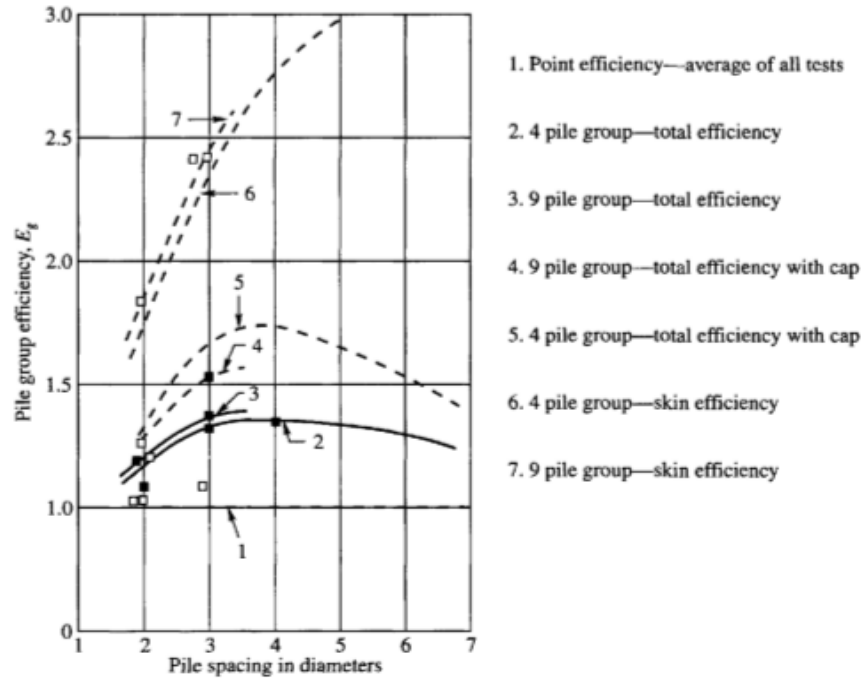


Figure 4.5.1 Efficiency of pile groups in sand (Vesic, 1967)

Therefore, for the case under consideration, where a pile is driven into a thick layer of sand, group efficiency can be conservatively assumed to be one, which means load carrying capacity of the pile group is equal to the sum of ultimate bearing capacities of piles in the group.

Ultimate bearing capacity of pile group should be subjected to a factor of safety to account for uncertainties. According to Allowable Stress Design (ASD), allowable bearing capacity can be obtained as follows:

$$Q_{all} = \frac{Q_{ult}}{FS} \quad (4.3.2)$$

The typical range of the safety factor is between 2 and 4 (Tomlinson, and Woodward, 2014). For the proposed project under design, global safety factor of 2.5 is used for residential building in inhomogeneous soil (Figure 4.5.1). This FS was also taken since, according to Eurocode 7, the most critical design approach is when pile resistance is factored by resistance factor (1.3 for driven pile for both base and shaft resistance) and model factor (1.5), which equals to approximately 2, which is less than 2.5 (1.3x1.5=1.95) (Bond and Harris, 2008).

Table 4.5.1 Suggested values of the overall factor of safety FS for pile foundations

<i>Characteristics of the structure</i>	<i>Homogeneous subsoil; exhaustive site and laboratory investigations</i>	<i>Unsatisfactory subsoil characterization because of either heterogeneous soil or poor investigations</i>
Maximum design loads occur frequently; the consequences of a collapse would be catastrophic (e.g. chemical or nuclear plant)	3	3.5
Maximum design loads occur rarely; the consequences of a collapse would be heavy (e.g. road bridges)	2.5	3
Maximum design loads are very improbable (e.g. residential buildings)	2	2.5

Calculation of ultimate bearing capacity and allowable load-carrying capacity of the single pile was made in Excel spreadsheet and shown in Table 4.5.2. Preliminary pile grouping based on maximum vertical loads from Table 4.4.1 was also done. Possible pile grouping for each major load cases was highlighted in this Table. It should be noted that they were selected taking into account maximum vertical loads, additional load of pile cap and lateral load actions. For further pile design, only piles with cross-section of 0.5 m x 0.5 m will be considered since they require less number of piles in group with smaller length, consequently, less cost with comparison to pile with square cross-section of 0.4 m x 0.4 m.

Table 4.5.2 Calculation of ultimate and allowable bearing capacity of the pile group

Pile width = 0.4 m (Ab=0.16 m ²)														
Reference	Qu of single pile				5 piles in group		6 piles in group		8 piles in group		11 piles in group		12 piles in group	
Pile Length, L (m)	Qp	Qs	Qult	Qall	pile #	Q g(all)	pile #	Q g(u)	pile #	Q g(u)	pile #	Q g(u)	pile #	Q g(u)
6	1064.88	664.96	1729.84	691.935	5	3459.67	6	4151.61	8	5535.477	11	7611.28	12	8303.215
7	1137.64	664.96	1802.60	721.04	5	3605.20	6	4326.24	8	5768.319	11	7931.439	12	8652.478
8	1205.20	664.96	1870.16	748.064	5	3740.32	6	4488.38	8	5984.512	11	8228.704	12	8976.768
9	1257.18	664.96	1922.13	768.852	5	3844.26	6	4613.11	8	6150.819	11	8457.376	12	9226.228
10	1322.09	664.96	1987.04	794.817	5	3974.09	6	4768.9	8	6358.538	11	8742.99	12	9537.808
11.5	1401.08	664.96	2066.03	826.414	5	4132.07	6	4958.48	8	6611.31	11	9090.552	12	9916.965
Pile width = 0.5m (Ab=0.25 m ²)														
Reference	Qu of single pile (kN)				4 piles in group		5 piles in group		6 piles in group		7 piles in group		8 piles in group	
Pile Length, L (m)	Qp	Qs	Qult	Qall	pile #	Q g(all)	pile #	Q g(all)	pile #	Q g(all)	pile #	Q g(all)	pile #	Q g(all)
6	1649.77	1167.26	2817.04	1126.82	4	4507.26	5	5634.08	6	6760.891	7	7887.706	8	9014.522
7	1748.61	1167.26	2915.87	1166.35	4	4665.39	5	5831.74	6	6998.089	7	8164.437	8	9330.786
8	1859.38	1167.26	3026.65	1210.66	4	4842.64	5	6053.3	6	7263.958	7	8474.618	8	9685.278
9	1969.70	1167.26	3136.96	1254.78	4	5019.14	5	6273.92	6	7528.708	7	8783.493	8	10038.28
10	2077.71	1167.26	3244.97	1297.99	4	5191.96	5	6489.94	6	7787.934	7	9085.923	8	10383.91
11.5	2202.97	1167.26	3370.23	1348.09	4	5392.37	5	6740.46	6	8088.556	7	9436.649	8	10784.74

4.6 Vertical resistance check of pile group

To check the vertical resistance, it is necessary to find the vertical load on single pile in pile group. Since pile group will be fixed with pile cap, loads on pile cap as in Figure 4.6.1 should be considered.

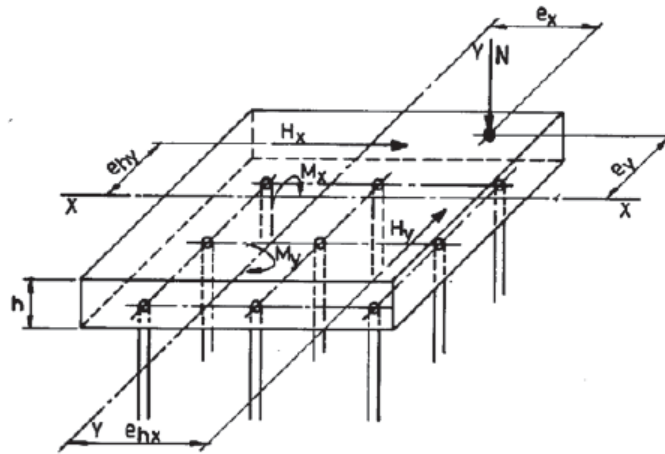


Figure 4.6.1 Loads and eccentricity on pile cap (Ray, 1995: pp. 303)

N- combined vertical load on pile cap

M_x – combined moment about x-x direction

M_y – combined moment about y-y axis

H_x – combined horizontal load on pile cap in x-x direction

H_y – combined horizontal load on pile cap in y-y direction

e_x – eccentricity of N from Center of Gravity (CG) of pile group in x-x direction

e_y – eccentricity of N from CG of pile group in y-y direction

h – pile cap depth.

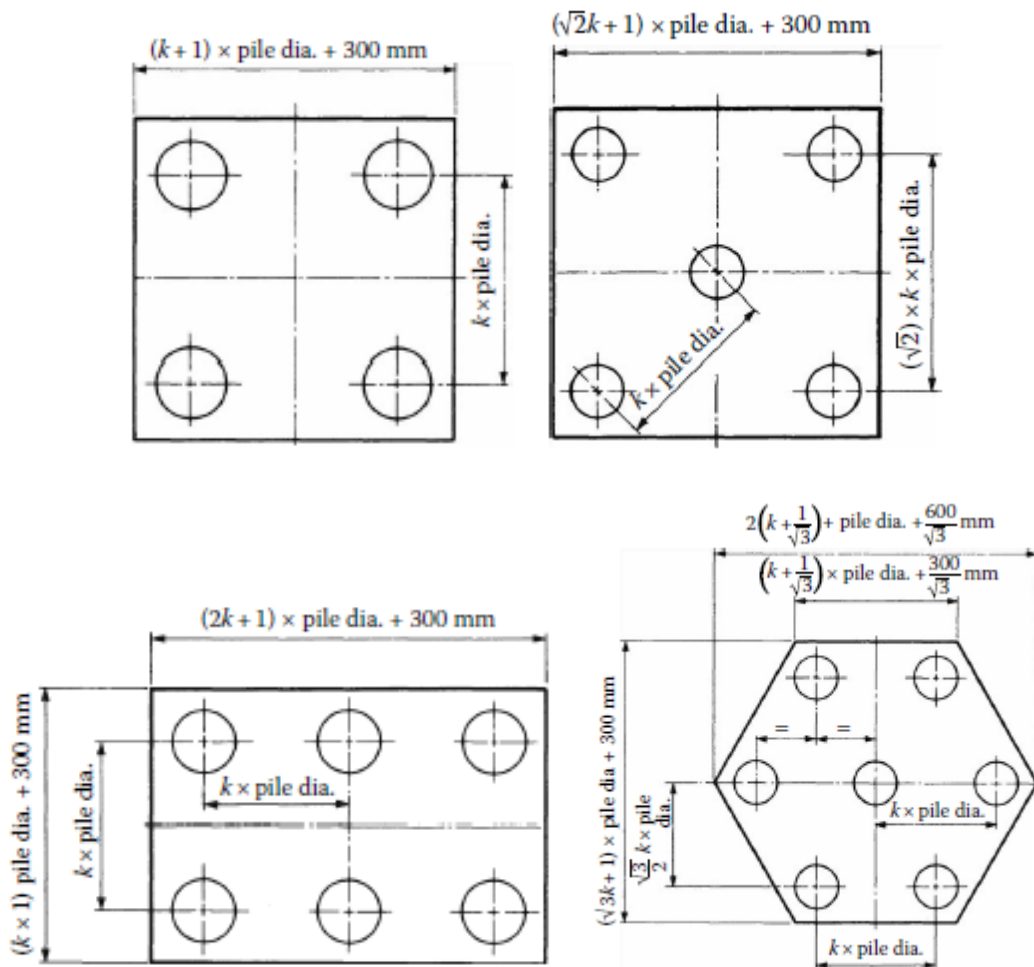
Since moment in both directions is negligible with comparison to maximum axial load, eccentricity is taken as zero.

Vertical load on a pile can be computed using equation (4.4.1):

$$\text{Vertical load on single pile} = \left(\frac{P}{R}\right) \pm \left(\frac{M_{xx}y}{I_{xx}}\right) \pm \left(\frac{M_{yy}x}{I_{yy}}\right) \quad (4.4.1)$$

Where P = vertical load on pile group = N + weight of pile cap; R is number of piles in group; M_{xx} = moment about x-x axis on pile group = $M_x + Ne_y + H_yh$; M_{yy} = moment about y-y axis on pile group = $M_y + Ne_x + H_xh$; $I_{xx} = \sum y^2$ and $I_{yy} = \sum x^2$ about x-x and y-y-axis, respectively, passing through CG of pile group; x , y = distances from the central axis to the pile in x-x and y-y directions, respectively.

To check vertical resistance of the single pile, preliminary pile cap assumption is necessary. As a base for assumptions for pile cap, standardized designs of pile cap as can be seen from Figure 4.6.2 for 4, 5, 6, 7, 8 and 9 piles in group was considered. Based on this Figure, preliminary sizes of pile cap are given in Table 4.6.1.



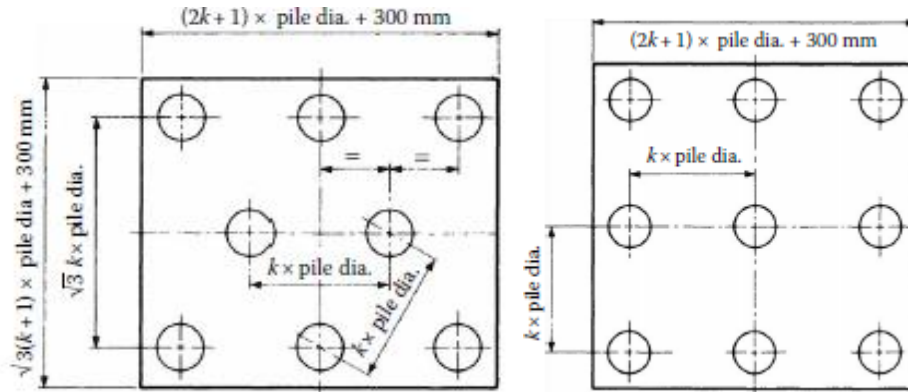


Figure 4.6.2 Standardized design of pile cap for 4, 5, 6, 7, 8 and 9 piles in group (Tomlinson and Woodward, 2010: pp. 392)

Table 4.6.1 Assumptions on pile cap size based on standardized pile cap design

Pile number in group	Pile cap size		
	Length (mm)	Width (mm)	Depth (mm)
4	2050	2050	1000
5	2568	2568	1000
6	3300	2050	1000
7	1712	1712	1000
8	3300	3330	1000
9	3300	3300	1000

Pile cap depth was assumed to be 1.0 m, unit weight of concrete, pile cap material, is 24 kN/m³. Thus, using these assumptions and load values from Table 4.4.1, vertical load on a pile for each critical load case and possible pile grouping was computed using equation (4.4.1) and shown in Table 4.6.2. It can be seen that vertical resistance check satisfies design criteria, meaning allowable carrying capacity of a pile is greater than the designed working load on pile. Final pile group for each major load case was highlighted on behalf of more safe, conservative and economic design. Allocation of pile groups in foundation of the residential building based on maximum vertical loading cases was also illustrated in Figure 4.6.3.

Table 4.6.2 Vertical resistance check of pile group

Case	N, kN	W, kN	P, kN	M _x , kNm	H _y , kN	M _{xx} , kNm	M _y , kNm	H _x , kN	M _{yy} , kNm	x, m	y, m	pile #	V, kN	<	Q _{all} , kN	OK	Pile length
1	3417	100.86	3517.46	-345.98	178.88	-167.1	-21.62	-34.3	-55.92	0.625	0.625	4	790.16	<	1126.82	OK	6 m
2	4393	100.86	4493.96	214.4	-118	96.38	69.11	95.2	164.31	0.625	0.625	4	1227.77	<	1254.78	OK	9 m
2	4393	158.27	4551.37	214.4	-118	96.38	69.11	95.2	164.31	0.884	0.884	5	984.00	<	1210.66	OK	8 m
2	4393	162.36	4555.46	214.4	-118	96.38	69.11	95.2	164.31	1.25	0.625	6	817.81	<	1210.66	OK	8 m
3	5692	158.27	5849.87	-381.78	232.26	-149.52	-1.32	0.21	-1.11	0.884	0.884	5	1127.38	<	1210.66	OK	8 m
3	5692	162.36	5853.96	-381.78	232.26	-149.52	-1.32	0.21	-1.11	1.25	0.625	6	935.57	<	1210.66	OK	8 m
3	5692	182.74	5874.34	-381.78	232.26	-149.52	-1.32	0.21	-1.11	0.938	1.083	7	804.44	<	1210.66	OK	8 m
4	7859	182.74	8041.24	229.24	-139.9	89.34	4.3	2.94	7.24	0.938	1.083	7	1170.83	<	1210.66	OK	6 m
4	7859	263.74	8122.24	229.24	-139.9	89.34	4.3	2.94	7.24	0.938	1.083	8	1029.94	<	1254.78	OK	9 m
4	7859	261.36	8119.86	229.24	-139.9	89.34	4.3	2.94	7.24	1.25	1.25	9	915.08	<	1254.78	OK	9 m

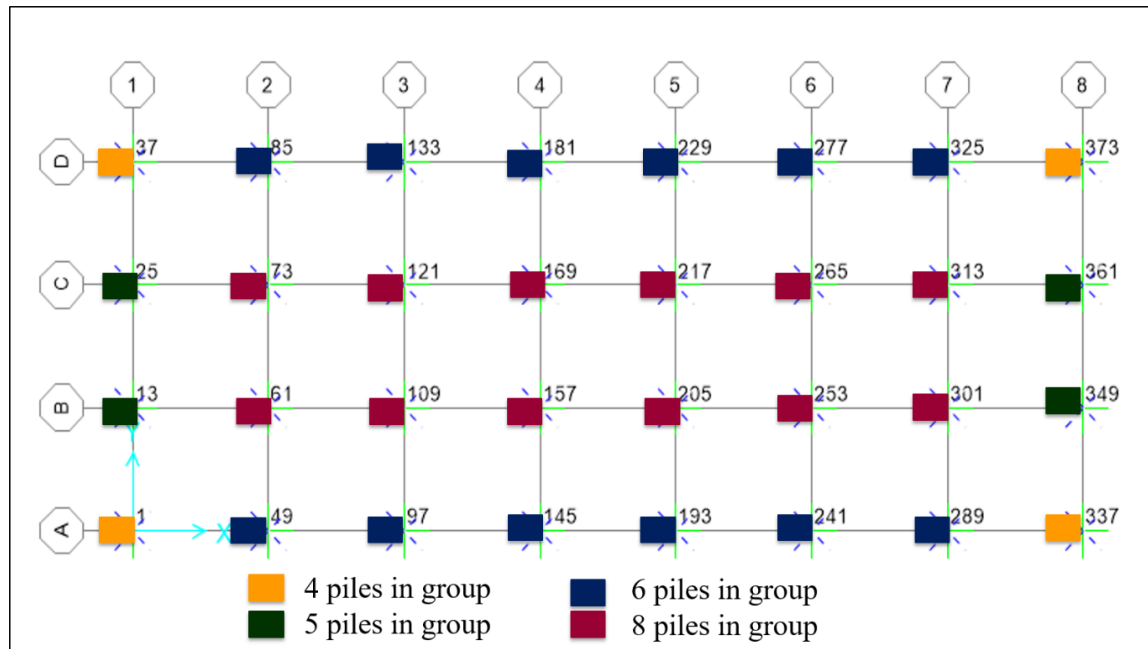


Figure 4.6.3 Pile group arrangement in foundation

4.7 Pile and pile cap design

To withstand the lateral loading, pile cap plays a key role in fixing pile heads. It should be reinforced and rigid to distribute the loads on piles evenly. Design of pile cap includes shape, depth of the pile cap, necessary steel amount, and reinforcement arrangement. Since critical load of 7858.5 kN occurs on internal piles, pile cap design is made for this critical case. Regarding pile, detailed design in terms of reinforcement will be provided. From previous section, since 8 piles in group are selected for internal load case, design of pile and pile cap will be done for pile group consisting of 8 piles.

Step 1. Pile cap dimension (method provided by S.S. Ray is followed, pp. 340-366)

Pile cap is designed according to standardized design given in Figure 4.6.2. Pile spacing is assumed to be twice as width of pile ($k=500 \times 2 = 1000$ mm) to keep spacing within economic limit. Base column size from structural part is 500 mm x 500 mm in the ground floor. Based on above data, pile cap with column is sketched and shown in Figure 4.7.1.

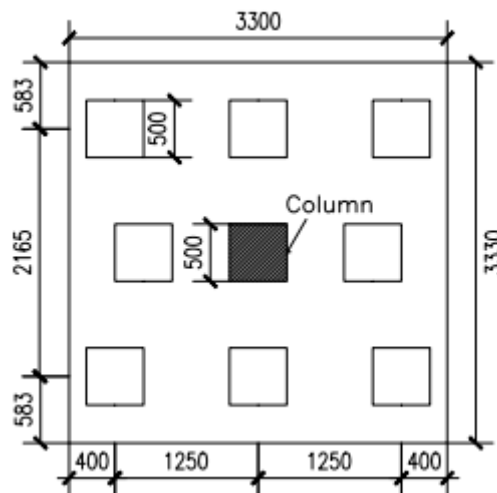


Figure 4.7.1 Plan of pile cap of 8 pile group

Step 2. Bending moment and shear force in pile cap

Critical sections used to calculate bending moment and shear force are shown in Figures 4.7.2 and 4.7.3, respectively. Sections 1-1 and 2-2 in pile cap are taken at the edge of column. Total moment is the sum of bending moment due to pile cap weight and pile reactions. Factored weight of pile cap = $1.0 \times 24 \times 1.5 = 36$ kN/m².

$$M_{11}' = \text{bending moment due to pile cap weight on section 1-1: } \frac{3.33 \times 36 \times 1.4^2}{2} = 117.5 \text{ kNm}$$

$M_{22}' = \text{bending moment due to pile cap weight on section 2-2: } \frac{3.3 \times 36 \times 1.415^2}{2} = 118.9 \text{ kNm}$

Pile cap reactions are Q_1, Q_2, Q_3 and Q_4 . $Q_1 = 1000.6 \text{ kN}$ – calculated using equation (4.4.1) with minus sign, $Q_3 = Q_4 = 1030 \text{ kN}$ from Table 4.6.2, Q_2 is average of Q_1 and Q_3 , and equals to 1015.3 kN . Based on these values and Figure 4.7.2:

$$M_{11}'' = 1.0 (1030 + 1030) = 2060.0 \text{ kNm}$$

$$M_{22}'' = 0.833 (1000.6 + 1015.3 + 1030) = 2535.7 \text{ kNm}$$

Thus, $M_{11} = M_{11}'' - M_{11}' = 2060 - 117.5 = 1942.4 \text{ kNm}$ and $M_{22} = 2416.8 \text{ kNm}$

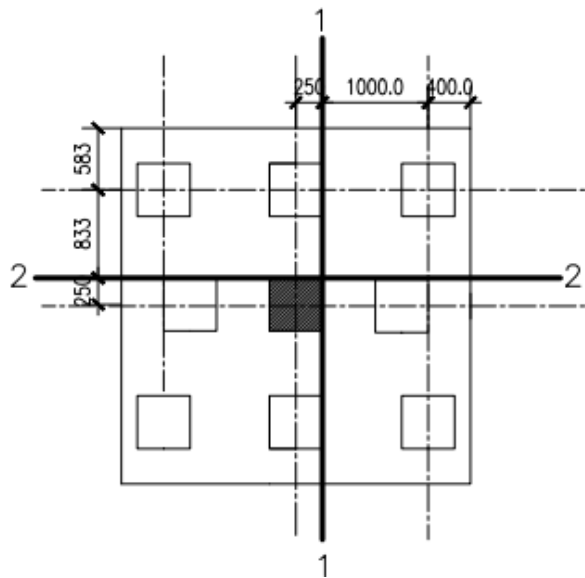


Figure 4.7.2 Critical sections for bending moment calculation

Total shear force is equal to shear in pile cap due to weight of pile cap and pile reactions.

Shear in pile cap due to weight of pile cap on section 3-3: $V_{33}' = 36 \times 3.33 \times 0.55 = 65.9 \text{ kN}$

Shear in pile cap due to weight of pile cap on section 4-4: $V_{44}' = 36 \times 3.3 \times 0.7325 = 87.0 \text{ kN}$

Shear force due to pile reaction on section 3-3: $V_{33}'' = (Q_3 + Q_4) = 2060 \text{ kN}$,

Shear force due to pile reaction on section 4-4: $V_{44}'' = (Q_1 + Q_2 + Q_3) = 3045.8 \text{ kN}$,

Thus, $V_{33} = V_{33}'' - V_{33}' = 1993.9 \text{ kN}$, $V_{44} = V_{44}'' - V_{44}' = 2958.8 \text{ kN}$

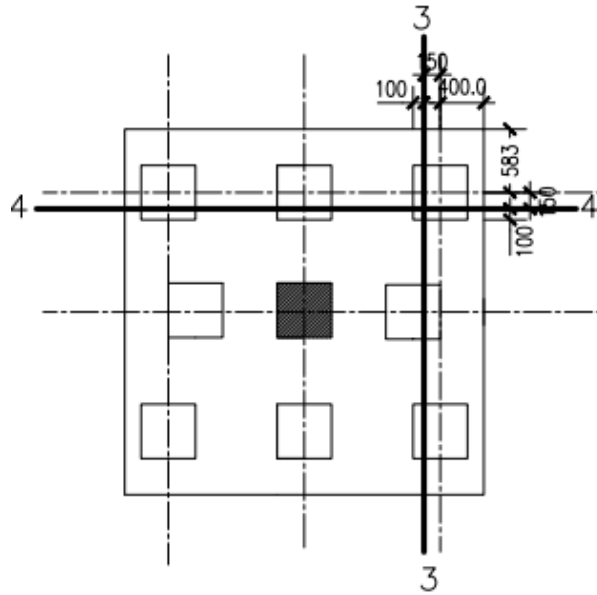


Figure 4.7.3 Critical sections for shear force calculation

Step 3. Determination of cover to reinforcement

From site investigation report, it is known that soil is susceptible to freeze and contaminated with sulfate. Taking into account that pile cap is in contact to ground and corrosive soil regarding steel, minimum cover to reinforcement should be 50 mm according to Table 1 in Appendix C-3. Cover to reinforcement is assumed to be 90 mm in pile cap.

Step 4. Calculation of reinforcement area in pile cap

1) $M_{11} = 1942.4$ kNm from Step 2.

Assume 20 mm diameter reinforcement.

Then, $d_x = 1000 - 90$ (cover) $- 10$ (half bar dia.) $= 900$ mm.

The concrete strength class of the cap and pile is chosen as C40/50 and C50/60, respectively. This is because cap is in contact with wet soil surface, and piles are fully embedded into the ground with seasonal water table change and freeze thaw actions. Thus, $f_{cu} = 40$ N / mm² for concrete in pile cap and $f_{cu} = 50$ N / mm² for piles.

$$K = \frac{M_{11}}{f_{cu}bd^2} = \frac{1942.4 * 10^6}{40 * 3330 * 500^2} = 0.058 \leq 0.156$$

$$z = d \left[0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} \right] = 0.93d \leq 0.95d = 837 \text{ mm}$$

$$A_{st} = \frac{M}{0.87f_y z} = \frac{1942.4 * 10^6}{0.87 * 500 * 837} = 5335 \text{ mm}^2$$

Ribbed steel reinforcing bar with $f_y = 500 \text{ N/mm}^2$ is used.

Area of 20 mm dia. bar = 314 mm^2 , hence, $17 * 314 = 5338 \text{ mm}^2 > 5335 \text{ mm}^2$

Use 17 no. 20 mm diameter bars equally spaced (spacing of bars = $(3330-180)/17=185$ mm, approximate to 180 mm) in longitudinal direction.

1) $M_{22} = 2416.8 \text{ kNm}$ (refer to Step 2)

Assume 16 mm diameter reinforcement. Then, $d_x = 1000 - 90$ (cover) $- 20$ (bar dia.) $- 8$ (half bar) = 882 mm . $f_{cu} = 40 \text{ N/mm}^2$ for concrete in pile cap.

$$K = \frac{M_{22}}{f_{cu} b d^2} = \frac{2416.8 * 10^6}{40 * 3300 * 500^2} = 0.07 \leq 0.156$$

$$z = d \left[0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} \right] = 0.91d \leq 0.95d = 803 \text{ mm}$$

$$A_{st} = \frac{M}{0.87f_y z} = \frac{2416.8 * 10^6}{0.87 * 500 * 803} = 6919 \text{ mm}^2$$

Area of 16 mm dia. bar = 201 mm^2 , hence, $35 * 201 = 7035 \text{ mm}^2 > 6919 \text{ mm}^2$

Use 33 no. 16 mm diameter bars equally spaced (spacing of bars = $(3300-180)/35=89$ mm, approximate to 85 mm) in the transverse direction.

Step 5: Check shear stress in pile cap

1) $V_{33} = 1993.9 \text{ kN}$ – shear on critical section 3-3 from Step 2.

$$a_v = \frac{3300}{2} - \frac{500}{2} \text{ (half column)} - 550 = 850 \text{ mm}$$

$$1.5d_x = 1.5 * 900 = 1350 \text{ mm}$$

$a_v < 1.5d_x$, therefore, enhancement of shear stress is allowed.

$$v = \frac{V}{bd} = \frac{1993.9 * 10^3}{3330 * 900} = 0.66 \text{ N/mm}^2$$

$$\rho = \frac{100A_s}{bd} = \frac{100 * 5338}{3330 * 900} = 0.18 \%$$

$$\frac{2d}{a_v} = 2 * \frac{900}{850} = 2.12$$

From Figure 1 in Appendix C-3, for $\rho = 0.18\%$ and $d > 400$ mm, $v_c = 0.4$ N/mm².

$$v_{c1} = v_c \left(\frac{2d}{a_v} \right) = 0.4 * 2.12 = 0.85 \frac{N}{mm^2} > 0.66 \frac{N}{mm^2} \quad \text{OK}$$

2) $V_{44} = 2958.8$ kN – shear on critical section 4-4 from Step 2.

$$a_v = \frac{3330}{2} - \frac{500}{2} \text{ (half column)} - 832.5 + 100 = 682.5 \text{ mm}$$

$$1.5d_y = 1.5 * 882 = 1323 \text{ mm} > a_v$$

$$\frac{2d_y}{a_v} = 2 * \frac{882}{682.5} = 2.6$$

$$v = \frac{V}{bd} = \frac{2958.8 * 10^3}{3300 * 882} = 1.0 \frac{N}{mm^2}$$

$$\rho = \frac{100A_s}{bd} = \frac{100 * 7035}{3300 * 882} = 0.24\%$$

From Figure 1 in Appendix C-3, for $\rho = 0.26\%$ and $d > 400$ mm, $v_c = 0.45$ N/mm²

$$v_{c1} = v_c \left(\frac{2d}{a_v} \right) = 0.45 * 2.6 = 1.2 \frac{N}{mm^2} > 1.0 \frac{N}{mm^2} \quad \text{OK}$$

Step 6. Check punching shear stress in pile cap

Punching shear stress check was made based on Figure 4.7.4.

$$U_1 = 2(500 + 500) = 2000 \text{ mm} = \text{column perimeter}$$

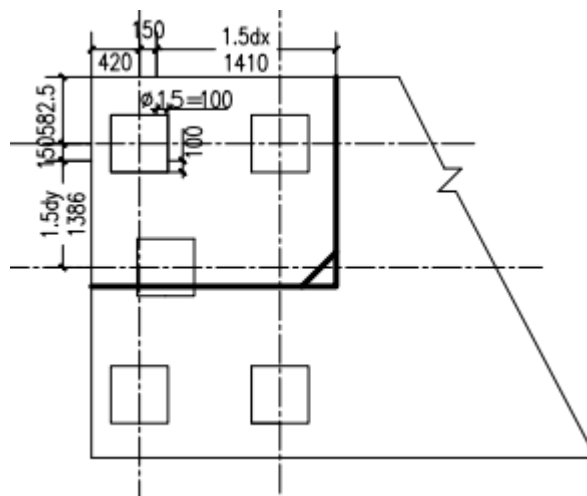


Figure 4.7.4 Critical planes for punching shear of piles in pile cap

It is not necessary to check punching shear stress at critical perimeter because pile spacing is less than 3 times of pile diameter.

$$U_2 = 1930 + 2218.5 = 4148.5 \text{ mm}$$

= perimeter on punching shear critical plane for pile load

Vertical maximum column load, $N = 7858.5 \text{ kN}$, vertical load on a pile = 1030 kN .

Column punching shear stress:

$$\frac{N}{U_1 d} = \frac{7858.5 \cdot 10^3}{2000 \cdot 0.5(900 + 882)} = 4.4 \frac{N}{\text{mm}^2} < 0.8\sqrt{f_u} \text{ or } 5 \text{ N/mm}^2 \quad \text{OK}$$

Punching shear stress at perimeter of pile:

$$\frac{1030 \cdot 10^3}{500 \cdot 4 \cdot 900} = 0.572 \frac{N}{\text{mm}^2} < 0.8\sqrt{f_u} \quad \text{OK}$$

Pile punching shear stress:

$$\frac{Q}{U_2 d} = \frac{1030 \cdot 10^3}{4148.5 \cdot 0.5(900 + 882)} = 0.28 \frac{N}{\text{mm}^2} < 0.4 \text{ N/mm}^2 \text{ for grade } 40 \frac{N}{\text{mm}^2} \text{ concrete} \quad \text{OK}$$

Step 7. Check area of reinforcement in pile

Assume that unsupported length of pile (l_o) is negligible. Assume $\frac{l_e}{h} < 10$, hence, pile is treated as a short column. From Table 4.2.8, $Q_{max} = 1030 \text{ kN}$ with $M = 346.65 \text{ kNm}$. Assume minimum cover is 75 mm . Allowing for links and bar diameter, assume $h_s = 350 \text{ mm}$:

$$\frac{h_s}{h} = \frac{350}{500} = 0.7 = k$$

$$f_{cu} = 50 \frac{N}{\text{mm}^2}, \quad e = \frac{M}{N} = \frac{346.65}{7858.5} = 0.044$$

$$\frac{e}{h} = \frac{0.044}{0.25} = 0.176$$

$$\frac{Q_{max}}{h^2} = \frac{1030 \cdot 10^3}{500 \cdot 500} = 4.12 \text{ N/mm}^2$$

According to Table 2 in Appendix C-3, minimum reinforcement can be used.

Step 8. Check shear capacity of RC pile

It is not required to check shear if $\frac{Mu}{Nu} \leq 0.6h$. Since moment on pile is zero, no shear check is needed.

$$\frac{H}{0.75A_c} = \frac{233.94 \times 10^3}{0.75 * 500 * 500} = 1.25 \frac{N}{mm^2} < 0.8\sqrt{f_{cu}} \quad OK$$

Step 9. Check minimum reinforcement in RC pile

$$\frac{100A_{sc}}{A_c} \geq 0.4, \quad A_{sc} = \frac{0.4A_c}{100} = 500 * 500 * \frac{0.4}{100} = 1000 \text{ mm}^2$$

Use 6 no. 16 mm dia. HT bars (6*201=1206 mm²).

Step 10. Containment of reinforcement in pile

Minimum diameter of links = 0.25 x bar dia.=4 mm > 6 mm

Maximum spacing of links 12 x smallest bar dia.=12x16=192 mm

6 mm dia. links at 175 mm centers are used.

Step 11. Minimum tension reinforcement in pile cap

$A_s \geq 0.0013bh$ in both directions. Minimum reinforcement in the x-x direction: 0.0013*3330*1000=4329 mm². Provided 5338 mm² (See Step 4).

Minimum reinforcement in the y-y direction: 0.0013*3300*1000=4290 mm²

Provided 7035 mm² (from Step 4).

Step 11: Curtailment of bars in pile cap

Minimum anchorage at ends of bars = 12xbar dia.

12x20=240 mm, 12x16=192 mm

Provide minimum 250 mm bent up length of pile bottom reinforcement.

Step 12: Spacing of bars in pile cap

$$\text{Maximum percentage of reinforcement} = \rho = \frac{100A_s}{bd} = \frac{100 * 5338}{3330 * 900} = 0.18 \%$$

Maximum allowed clear spacing for ρ less 0.3 % or 750 mm (Table below), whichever is less. Adopted spacing of bars is 180 mm.

Percentage of reinforcement, $100A_s/bd$ (%)	Maximum clear spacing of bars in pile cap (mm)
1 or over	160
0.75	210
0.5	320
0.3	530
Less than 0.3	$3d$ or 750

Step 13. Connection of pile to pile cap

From Table 3 in Appendix C-2:

Full anchorage bond length $=32\phi$: $32 \times 16 = 512$ mm

The bars from the pile will project 600 mm into pile cap. Detailed drawing of pile and pile cap is provided in technical drawings.

4.8 Lateral resistance of the pile group

To check the lateral resistance of the pile group, it is necessary to find horizontal load on each pile in a group. Lateral load on single pile can be computed using equation (4.5.1):

$$\text{Lateral load on single pile} = \frac{\sqrt{H_x^2 + H_y^2}}{R} \quad (4.5.1)$$

Lateral load on each pile group was accomplished using values from Table 4.6.2 and results are inserted in Table 4.8.1.

Table 4.8.1 Lateral load on single pile in pile group

Case	Hx, kN	Hy, kN	Pile no. in group	Lateral load, kN
1- corner piles	61.14	-193.66	4	50.8
2-external piles	92.12	-196.77	5	43.5
3-external piles	1.57	-251.64	6	41.9
4-internal piles	-11.06	234.40	8	29.3

Resistance of pile to lateral load depends on stiffness of the pile and soil and end fixity of the pile. Generally, piles can be classified as rigid or short piles and elastic or long piles (Das, 2007). To categorize piles used in this project, characteristic length of pile should be identified using equation (4.5.2).

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \quad (4.5.2)$$

Where $E_p = 4700 \sqrt{f_{cu}}$ is modulus of elasticity of pile material, $I_p = \frac{bh^3}{12}$ is moment of inertia of pile section and n_h is horizontal subgrade reaction constant. Considering concrete grade 50 ($f_{cu} = 50 \text{ N/mm}^2$) for pile, square cross-section pile with width of 500 mm, $E_p = 33.234 \text{ GPa}$ and $I_p = 5208 * 10^{-6} \text{ m}^4$. Further, n_h is assumed to be 4000 kN/m^3 for medium sand using Table 5 from Appendix C-1.

$$T = \sqrt[5]{\frac{33.234 * 10^6 * 5208 * 10^{-6}}{4000}} = 8.46 \text{ m}$$

If $L \geq 5T$, then pile is classified as long pile. For short piles, L should be equal or less than $2T$. Calculated length of piles in pile group is 6, 8 and 9 m (see Table 4.8.2).

Table 4.8.2 Pile classification with different pile lengths

Pile length, m	6	8	9
L/T	0.71	0.95	1.1

According to Table 4.8.2, piles under design are short piles.

Lateral resistance of the pile can be found using Brom's method, which is developed assuming shear failure in soil for short piles. According to soil profile, 6, 8 and 9 m long piles penetrate through loam to sand layer. Considering this, two cases of finding pile lateral resistance, assuming 1) soil is fully sand and 2) soil profile is homogeneous clay, are examined. Figure 4.8 shows Brom's solution for horizontal resistance of short piles in sand and clay.

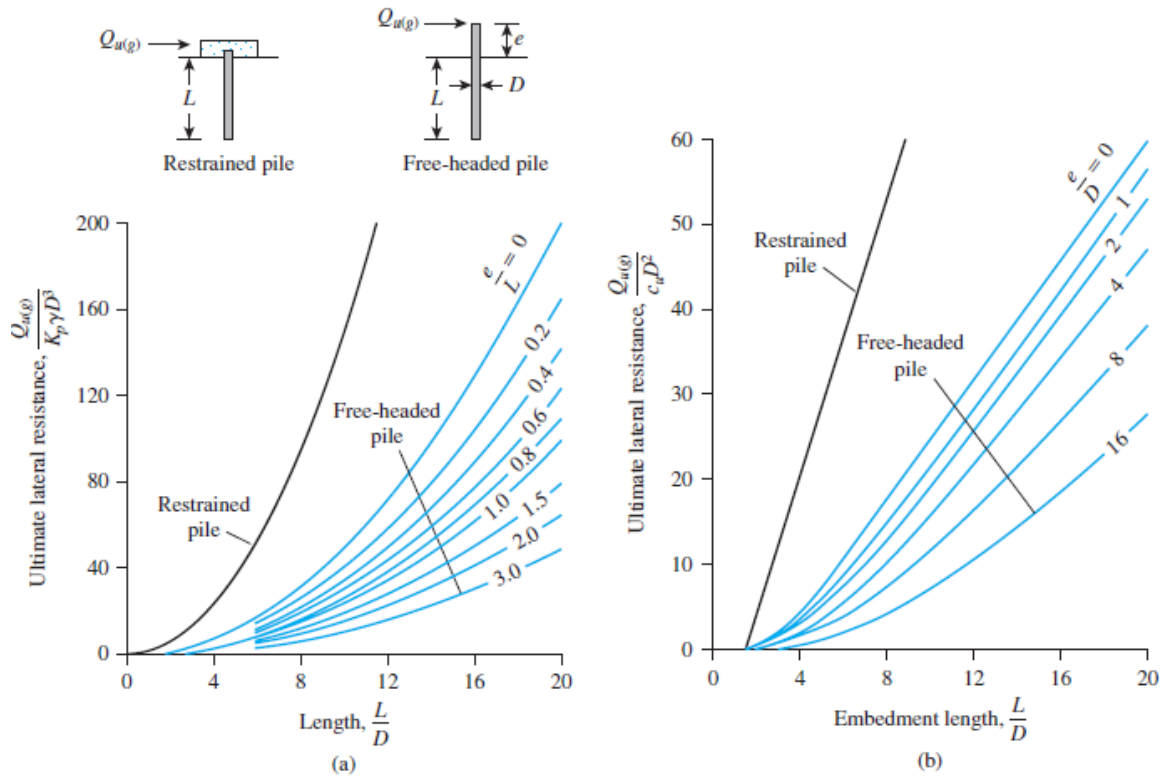


Figure 4.8.1 Brom's method for lateral resistance of short piles (a) in sand and (b) in clay (Das, 2007: pp.599)

Case 1: Short piles embedded in sand

From above Figure 4.8.1 (a):

$\frac{Q_{u(g)}}{K_p \gamma D^3} = 200$ for $\frac{L}{D} = \frac{6}{0.5}, \frac{8}{0.5} = 16$ and $\frac{9}{0.5} = 18$ for restrained pile, since pile head is fixed with pile cap.

Assuming $\gamma = 18 \text{ kN/m}^3$ and $\phi' = 30^\circ$ conservatively, Rankine passive earth pressure coefficient and lateral resistance of pile can be calculated:

$$K_p = \tan^2 \left(45 + \frac{\phi'}{2} \right) = \tan^2 \left(45 + \frac{30}{2} \right) = 3,$$

$$Q_{u(g)} = 200 K_p \gamma D^3 = 200 \times 3 \times 18 \times 0.5^3 = 1350 \text{ kN}$$

$$Q_{all} = \frac{Q_{u(g)}}{2.5} = \frac{1350}{2.5} = 540 \text{ kN}$$

Case 2: Short piles embedded in clay

From soil properties, $c_u = 18 \text{ kN/m}^2$. From Figure 4.8.1 (b), when $L/D=12, 16$ and 18 for fixed pile head, $\frac{Q_{u(g)}}{c_u D^2} = 60$.

$$Q_{u(g)} = 60c_u D^2 = 60 \times 18 \times 0.5^2 = 270 \text{ kN}$$

$$Q_{all} = \frac{Q_{u(g)}}{2.5} = \frac{270}{2.5} = 108 \text{ kN}$$

Allowable lateral resistance of single pile in both sand and clay is greater than lateral load on single pile given in Table 4.4.2. Thus, pile group design satisfies lateral load check.

4.9 Settlement of pile group

A significant concern in design of pile group is an elastic and consolidation settlement, which is greater than the settlement of the single pile under same applied load. The main reason of this is an extension of the stressed zone of the pile group to greater depth and width (see Figure 4.3.1). Short-term settlement calculations are considered in following sub-parts. However, long-term settlement (consolidation) is negligible since loam is upper layer of ground and pile is mainly derived their resisting capacity from the sand layer.

To ensure safe design and tolerable settlement, it is necessary to calculate the short-term settlement of the pile group. Elastic settlement of the pile group depends on the total settlement of the single pile, which is given by equation (4.5.1).

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)} \quad (4.5.1)$$

Where:

$S_{e(1)}$ = elastic settlement of pile

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$$

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

Q_{ws} = load carried by skin friction under working load condition

A_p = cross section area of pile

L = pile length

E_p = elastic modulus of pile material =

$$4700 \sqrt{f_{cu}} = 4700 * \sqrt{50} = 33.2 \text{ MPa}$$

$0.5 < \xi < 0.67$ (average of 0.585 will be taken)

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

D = pile width

$$S_{e(2)} = \frac{q_{wp}D}{E_s} (1 - \mu_s^2) I_{wp}$$

q_{wp} = point load per unit area at the pile point = $\frac{Q_{wp}}{A_p}$
 E_s = elastic modulus of soil below the pile point
 μ_s = Poisson's ratio of soil
 I_{wp} = influence factor ≈ 0.85

$S_{e(3)}$ = settlement of pile caused by the load transmitted along the pile shaft
 p = perimeter of the pile
 L = embedded pile length
 I_{ws} = influence factor
 $I_{ws} = 2 + 0.35\sqrt{(L/D)} = 3.57$

$$S_{e(3)} = \left(\frac{Q_{ws}}{pL}\right) \left(\frac{D}{E_s}\right) (1 - \mu_s^2) I_{ws}$$

Pile group settlement can be found using following equation:

$$S_{q(e)} = S_e \sqrt{\frac{B_g}{D}} \quad (4.5.2)$$

Elastic settlement of the single pile and pile group was calculated using above provided formulas and results are tabulated in Table 4.9.1.

Table 4.9.1 Elastic settlement of the single pile and pile group

pile # in group	Qp, kN	Qs, kN	L, m	Se(1), mm	Se(2), mm	Se(3), mm	Se(total), mm	Bg, mm	Sgroup, mm
4	1649.77	1167.26	6	1.686	11.719	0.007	13.413	2050	27.16
5	1859.38	1167.26	8	2.450	13.208	0.005	15.664	2568	35.50
6	1859.38	1167.26	8	2.450	13.208	0.005	15.664	2050	31.72
8	1969.70	1167.26	9	2.876	13.992	0.005	16.873	3330	43.54

Alternatively, settlement of single pile was analyzed in Plaxis 2D software. The focus of the simulation was to check the deformation under pile tip and stress effects during driving process on surrounding soil. For more realistic process, hard soil (HS) small model was applied to predict behavior of sand layer. Pile driving design in Plaxis software consists of two phases: construction stage and dynamic loading. While modeling driven concrete pile, interface was created to analyze the interaction between pile and soil. After a completed driven pile, model was simulated in a dynamic loading mode. Total displacement pattern due to driving can be seen in Figure 4.9.1. it is clear from the Figure that surrounding soil is displaced upward because of vibration effects, while soil under the pile toe becomes dense.

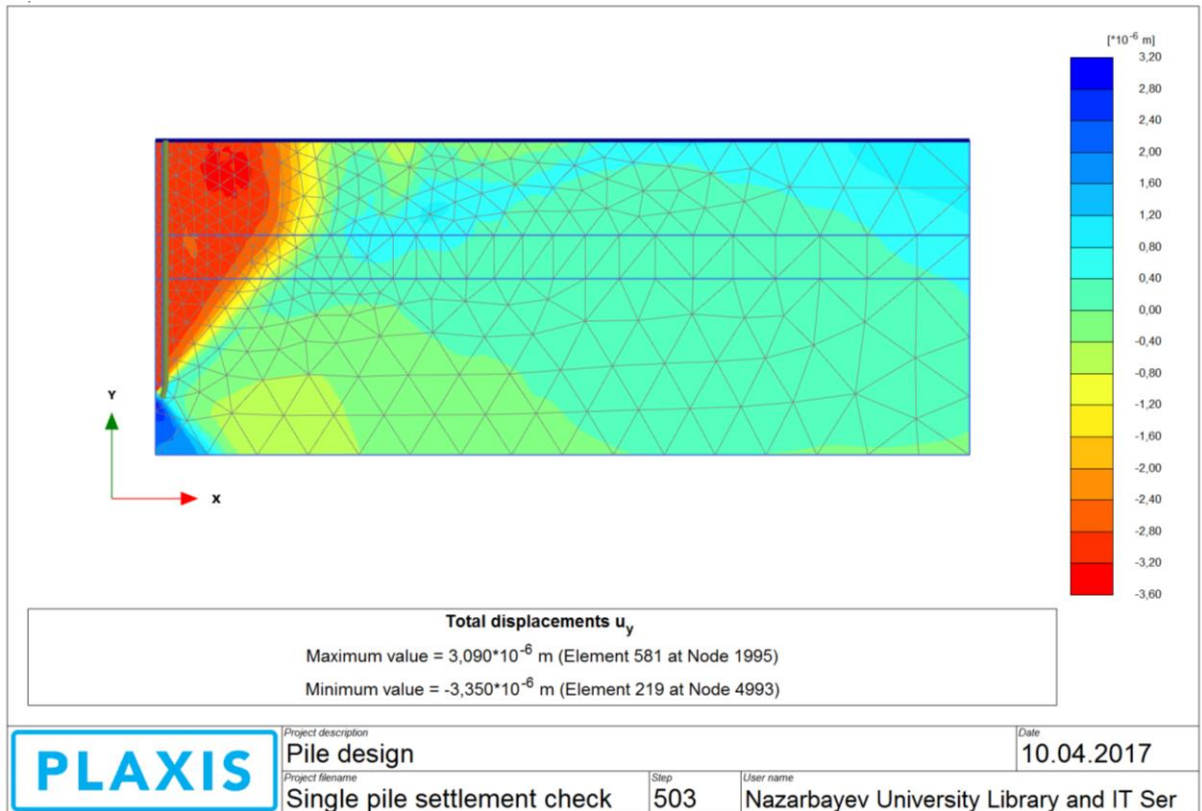


Figure 4.9.1 Total displacement of driving pile

As a result of simulation, graph of pile settlement versus time was also obtained. It can be inferred from the Figure 4.9.2 that maximum pile settlement occurs in initial stroke. Final settlement of the pile toe is about 0.003 mm. This value is negligible comparing to single pile settlement found manually. Therefore, calculated pile group settlement can be stated as conservative. Total principal stress pattern was also derived from Plaxis software and illustrated in Figure 4.9.3. Total stress propagation is decreasing when distance from driving point is getting far away. High stress around the pile can be explained by vibratory effect and dynamic action of pile installation.

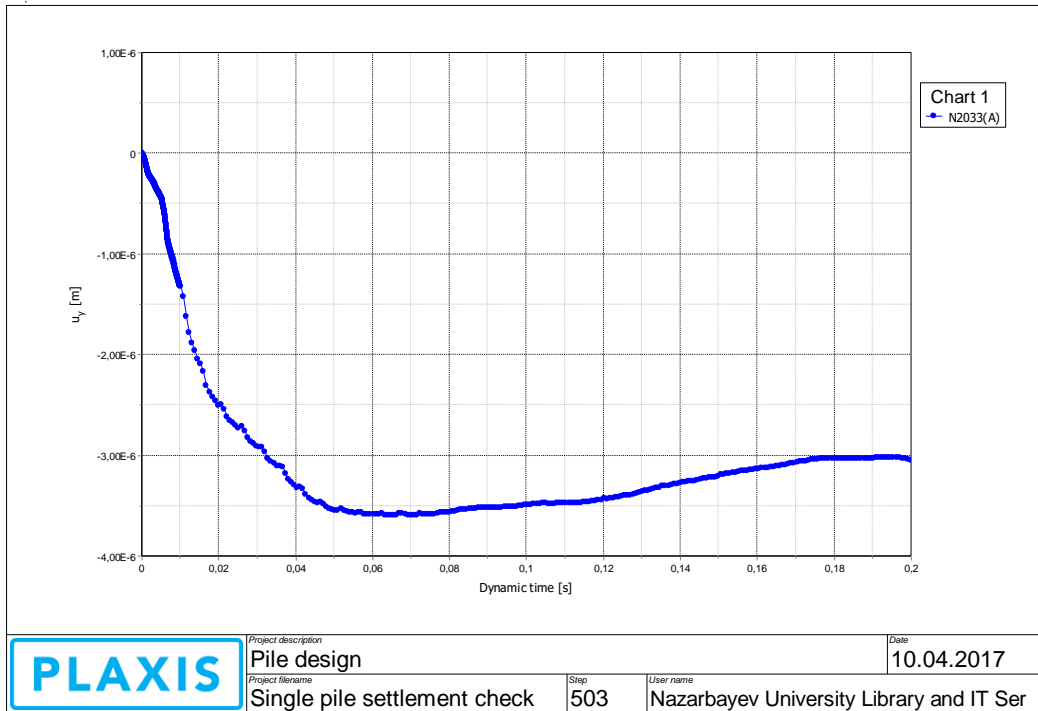


Figure 4.9.2 Plot of single pile settlement versus time

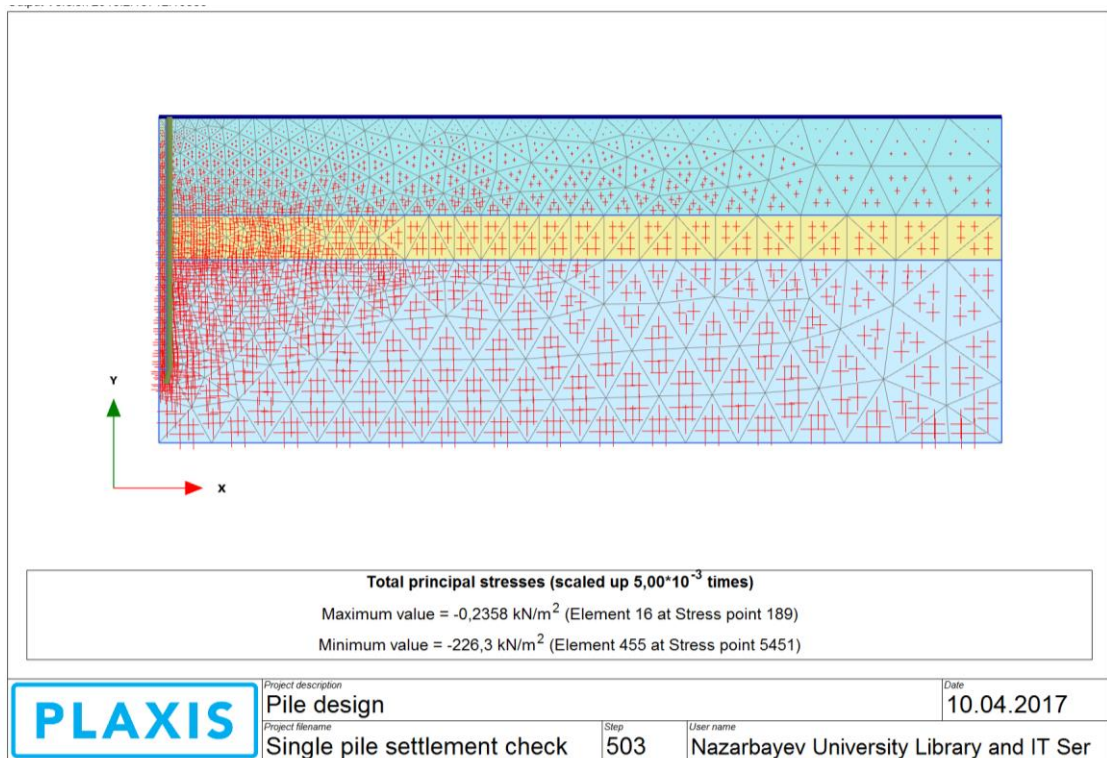


Figure 4.9.3 Total principal stress generated during pile driving

4.10 Design check against hydraulic failure

In cold climate regions, pile embedded into clay layer is subjected partly to seasonal freezing. Uplift of piles occurs when frost heave force is greater than the load applied downward. The value of upward force is related to the frost heaving intensity, surface area of the ground and tangential adfreeze strength, which refers to the bond between the frozen ground and pile surface area (Pewe, & Paige. 1963). Because of severe climate in Astana during winter and upper layer of ground, which is clayey loam susceptible to frost thaw action during seasonal change, uplift force action on piles should be considered during design stage. Problem of heave and close ground water table existing on site lead to pile damage due to uplift force during driving process. To prevent loss of equilibrium caused by uplift, pile tension capacity against hydraulic failure need to be checked. Referring to Eurocode 7 -Geotechnical Design, failure by uplift can be checked with equation (4.6):

$$V_{dst;d} \leq G_{stb;d} + R_d \quad (4.6)$$

Where $V_{dst;d}$ = design vertical disturbing load, which is combined force of frost heave and pore water pressure, $G_{stb;d}$ = weight of the structure. Example of failure by uplift mechanism is illustrated in Figure 4.10.1.

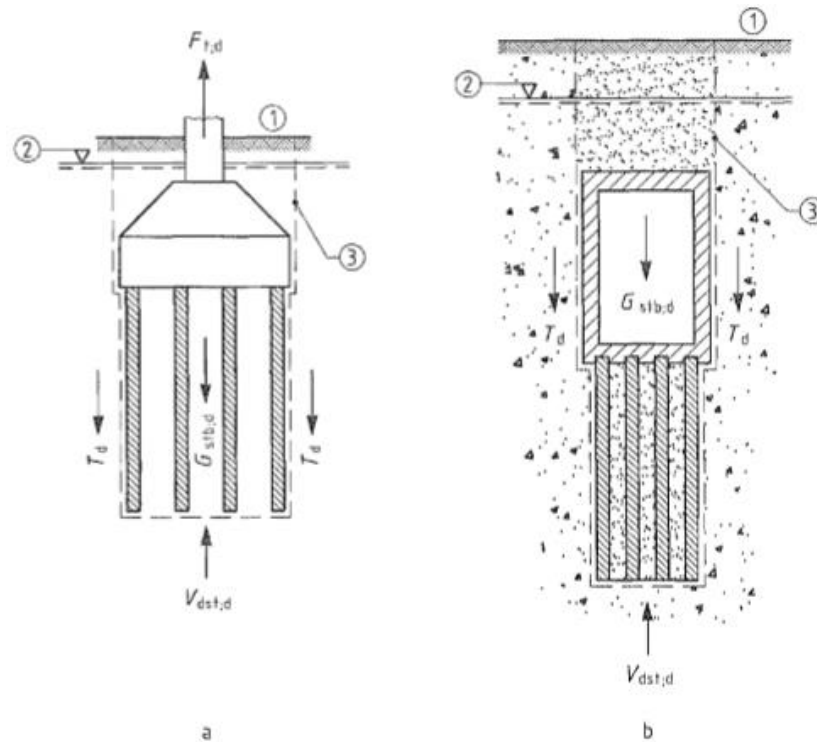


Figure 4.10.1 Uplift of a pile group: 1-ground surface, 2-ground-water level, 3-side of the 'block', where resistance T_d develops (EN 1997-1: 2004: pp. 84)

Because of scare study about frost heave in Astana, frost heave force was assumed to be 103 kN based on research study conducted in Canada (Ladanyi and Foriero, 1998). Pore water pressure acting on 6 m pile is $6 \times 9.81 = 58.86$ kN. Thus, total uplift force = $103 + 58.86 = 161.86$ kN.

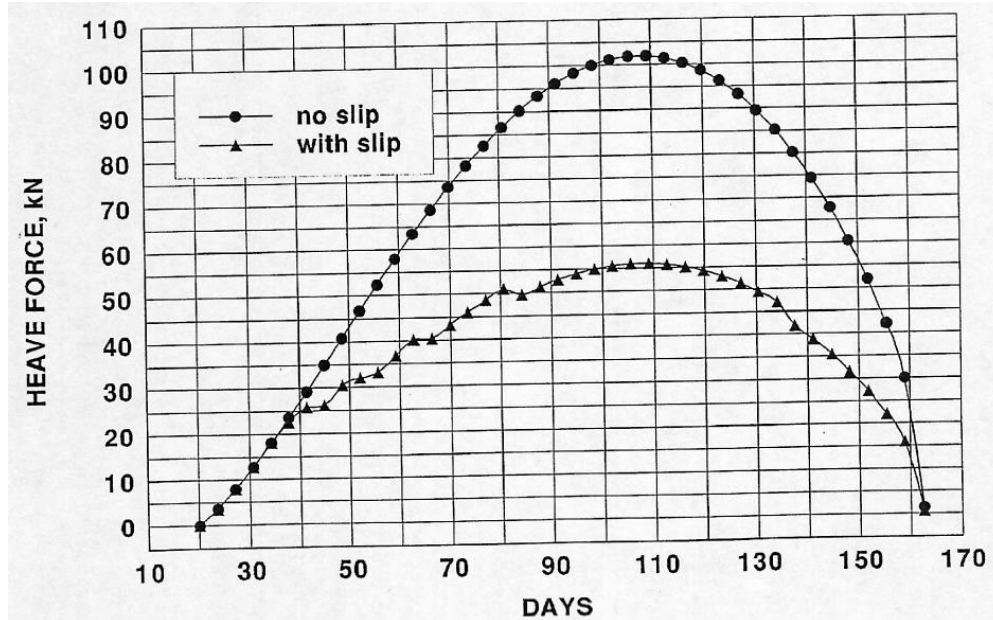


Figure 4.10.2 Uplift heave force acting on a pile (Ladanyi & Foriero, 1998: pp.628)

Since heavy superstructure load can easily handle uplift action, probable uplift of a pile during pile driving or after pile installation will be considered. In this way, stabilizing force against frost heave and pore water pressure becomes combined weight of pile cap and piles in group. Pile skin friction also contributes to the downward force. To check design against hydraulic failure, 4 pile group design in pile cap should be taken as critical case, because of smaller weight comparing to other pile group design. Thus, calculating for stabilizing force, following can be obtained:

$$\text{Total weight of 4 piles in group + pile cap} = 6 \times 24 \times 0.5^2 \times 4 + 1.0 \times 24 \times 2.05 \times 2.05 = 244.86 \text{ kN}$$

$$\text{Skin friction of 6 m length pile} = 354 \text{ kN (from Table 4.2.2-b)}$$

Total stabilizing force against uplift = 598.86 kN. Thus, design in terms of uplift check is satisfactory. However, design check is accomplished on the basis of approximate estimation. For example, actual frost heave force may be much higher in Astana than assumed one. Also, pile shaft may lose reasonable resistance due to sliding effect during warm season. In general, uplift design check is demonstrated to highlight an importance of considering uplift forces. This is a critical for Astana city because of close groundwater, soil freezing and cold climate. In the future, careful attention should be

given to uplift design check including more precise frost heave study and behavior of soil layer subjected to freeze thaw action.

4.11 Design of sheet pile wall

Sheet piling in construction industry plays a role of the waterfront structures that enable to resist an active earth pressure. Its use is strongly encouraged when high driving stresses present during piling because of hard soil (Das, 2007). The main situations indicating need in sheet pile wall for the project are close groundwater and frost heave in Astana. Since required trench depth is below the water table, occurrence of water in a trench after excavation creates unfavorable condition for pile driving. Moreover, frost heave forces can increase the lateral load on basement floor causing probable failure during construction of underground floor. Thus, integrating design of sheet piles to current project can solve these issues acting as a retaining wall.

There are two basic types of sheet piles as cantilever and anchored. Their use is distinguished based on dredge line. Cantilever sheet pile is recommended for height of 6 m or less above the dredge line. In case of deep excavation with depth of more than 6 m, anchored sheet piles are preferable. Because of excavation depth of 2.5 m, cantilever sheet pile walls are suitable one for building of student dormitory in Astana. These retaining structures can be made from concrete, steel or wood. Steel sheet piles with corrosion resistive coating can be chosen for further design taking into account their advantages as lightweight and reusability.

Inhomogeneous profile of geological structure of residential building project leads to complicated sheet pile design. Since loam layer is right after below excavated soil, sheet piling will be assumed to be penetrated in clay layer. Difficulty of sheet pile design in cohesive soil is in the change of clay strength with time, which is followed by corresponding change of lateral earth pressure with time. Because of these facts, design should be made for short-term stabilization and long-term stabilization. In short-term analysis, lateral earth pressure immediately after installation can be computed assuming prevailed undrained strength of clay. This means clay strength fully depends on cohesion and rejects internal friction. Assumed pressure distribution diagram on steel wall is illustrated in Figure 4.11.1 for short-term period.

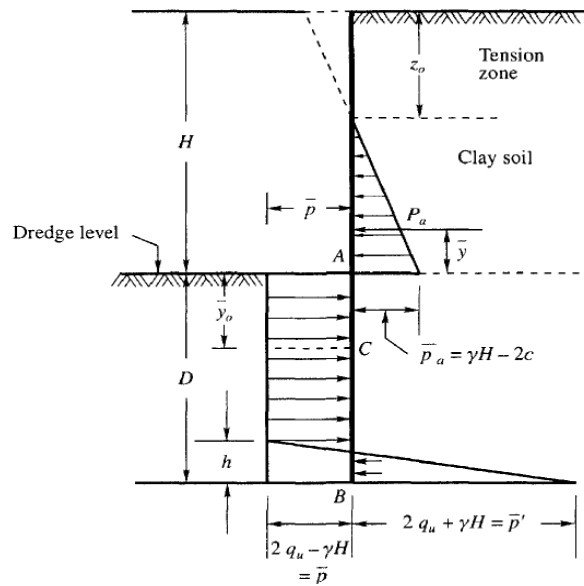


Figure 4.11.1 Pressure distribution diagram in clay (Murthy, 2002: pp.898)

In case of long-term evaluation, clay strength is changed and tends to zero. Hence clay starts to behave as cohesionless soil deriving its strength from internal friction. This enables application of pressure distribution diagram for sand shown in Figure 4.11.2 for long-term case.

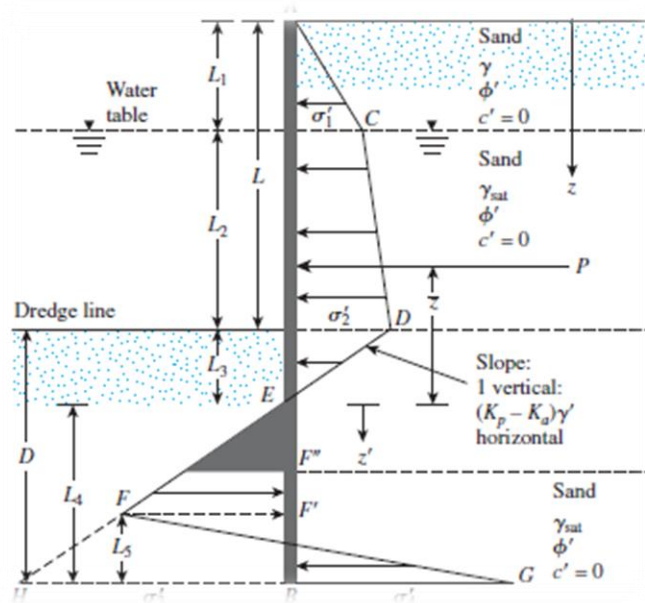


Figure 4.11.2 Pressure distribution diagram of sheet pile penetrating in sand (Das, 2007: pp.443)

General considerations of sheet pile design include estimation of forces and horizontal pressure, calculation of required penetration depth of the piling, determination of maximum bending moment and stresses in the sheet pile, and selection of suitable sheet pile section. Sheet pile design for short-term and long-term period was accomplished based on pressure distribution diagram in Figures 4.11.1 and 4.11.2. Hand

calculation is attached in Appendix C-4. The results of the calculation can be seen from Table 4.11.1 It is clear from this Table that long-term design requires longer penetration depth because of increased lateral active pressure. Since long-term period is of interest for current project, sheet pile with length of about 5.2 m and cross-section of not less than $2.21 \times 10^{-4} \text{ m}^3/\text{m}$ of wall should be used.

Table 4.11.1 Detailed sheet pile design

Case	Sheet pile penetration depth, m	Sheet pile length, m	Minimum required cross-section, m ³ /m of wall
Short-term	0.28 m	2.78 m	4.35×10^{-6}
Long-term	1.9 m	5.16 m	2.21×10^{-4}

Sheet pile wall was also modeled in Plaxis 2D software. According to calculations, length of sheet pile was taken as 5.2 m. Sheet pile is penetrated in clay and penetration depth is 2.66 m. The design of sheet pile in Plaxis is shown in Figure 4.11.3. From the Figure, it is obvious that sheet pile moves towards the left side with maximum displacement of 3.8 mm. This is due to lateral pressure on right side.

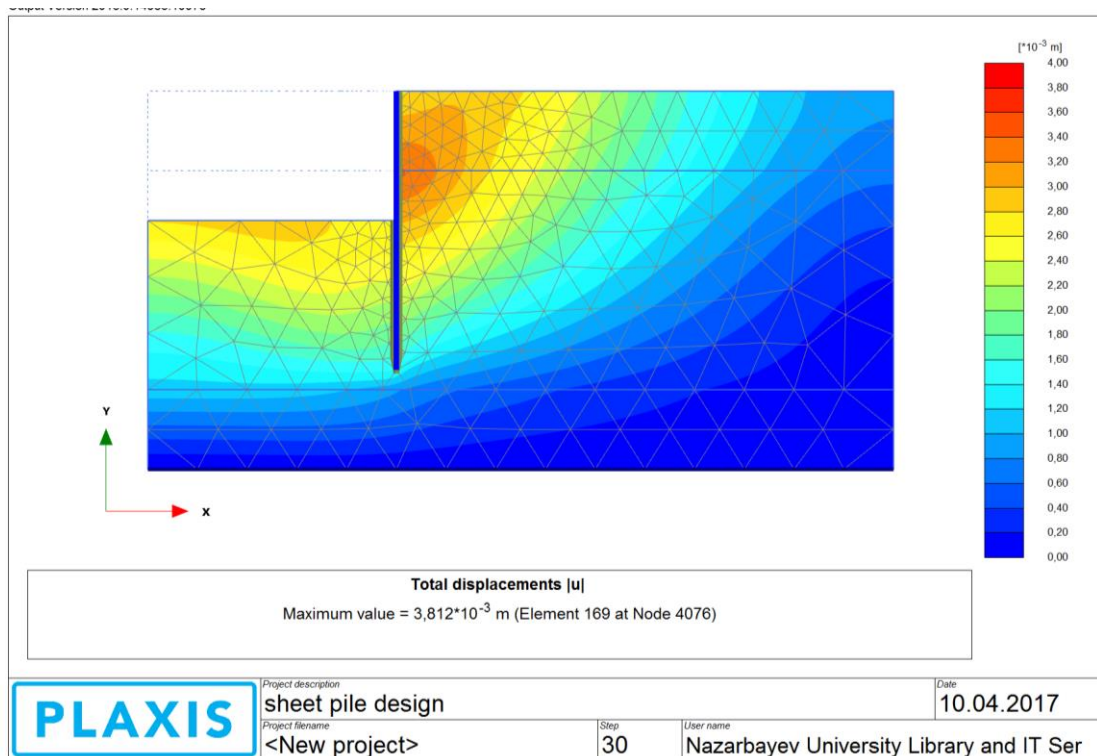


Figure 4.11.3 Total displacement of the sheet pile

Total stress spread from the sheet pile is also derived from the Plaxis software and shown in Figure 4.11.3. Total principal stress is quite uniformly propagated from

penetrated length of the sheet pile. It should be noted that on retaining side of sheet pile, considerable stress distribution is not observed.

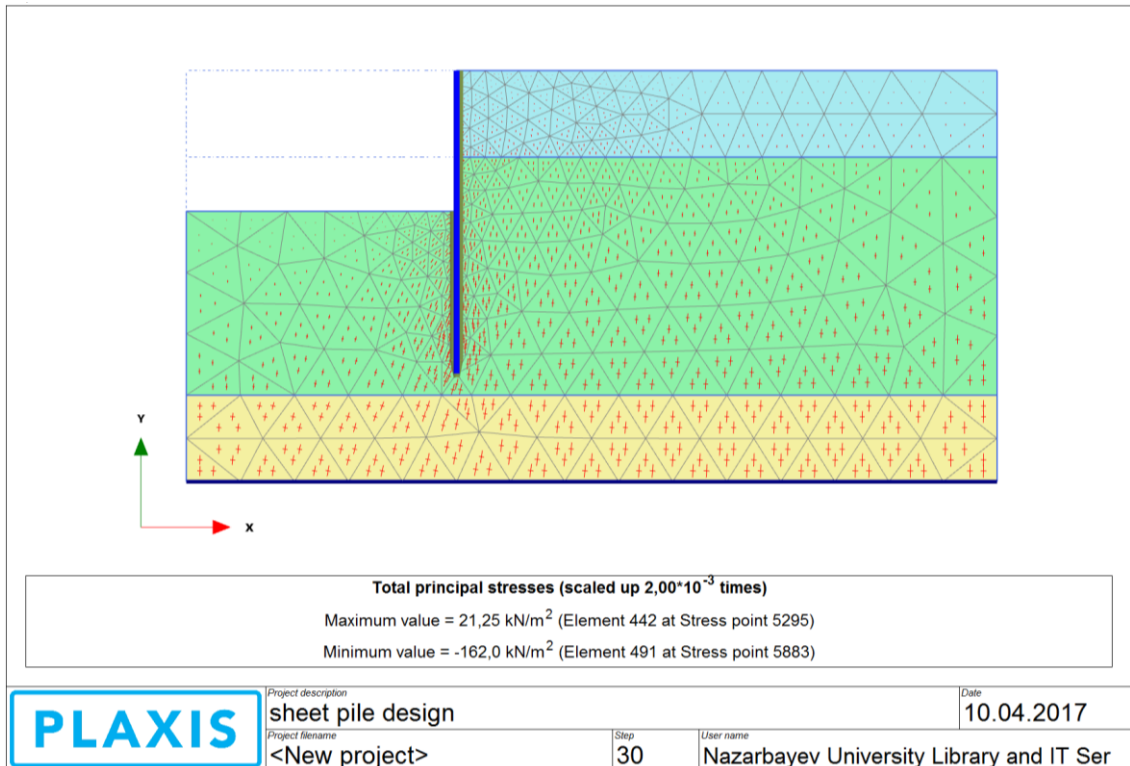


Figure 4.11.4 Total stress pattern on sheet pile wall

4.12 Foundation construction process

Construction of foundation is an essential part of a whole structure, which is the base for further building processing. Its stages should be planned carefully in advance to minimize the time delays and increase the efficiency. Mainly, construction of basement can be divided into following operations: excavation, dewatering, sheet pile construction, pile driving and pile quality check after installation. These foundation sub-parts are discussed below.

4.12.1 Excavation

After site works as clearing and removal of trees or existing structure, earthwork operations will be followed. According to project design, 2.5 m depth of earth should be excavated to locate underground floor. Before starting to excavate a ditch, it is required to establish corner benchmarks and identify the top and ground level by surveying. After these operations, excavation can be accomplished employing excavators or crane shovels. Considering wide use and availability of excavators in Astana, this equipment can be used for earthwork operations. In addition, especially, crawler mounting

excavator is in preference because of its excellent on-site mobility. Number of required excavators for current project can be calculated based on the volume of the earth. It is known that the area of the residential building under design is 42 m x 15 m. Taking additional 30 cm from each side of the area to locate sheet piles and considering required trench depth of 2.5 m, total volume of excavated soil is about 1618 m³. According to construction equipment market in Astana, excavator with bucket volume of 2 m³ can be hired with average rent cost of \$25/hour. Assuming production rate of 40 m³ per hour, time required for total excavation can be identified as following: 1618/40=40.45 hours. Taking 8 hour working day and unexpected whether conditions, excavation of trench can take more than 5 days. Therefore, to fasten the construction process, it is better to rent two excavators. Thus, with this amount, trench may be ready within 3 days..

4.12.2 Dewatering

As it is known from geological investigation that water table in Astana is approximately 2 m below the ground surface. During excavation of 2.5 m deep ditch, this situation creates a problem, which requires lowering of the ground water. Drainage of the site can be carried out using well point system as shown in Figure 4.12.1. This is the practical and economical drainage type where lowering level of up to 6 m is required. The main principle of the system is to locate several well points along a ditch and connect them to a common header. Then, this header is attached to well point pump, which drain the water out. Typical spacing of the well points is between 1 m and 3 m. Capacity of this drainage system varies depending on component sizes of the dewatering system and soil type (Nemati, 2005). To specify the sunk depth of the well point, quantity of pumps and well points and other drainage components, on site meticulous study of hydro-geological condition should be implemented.



Figure 4.12.1 Dewatering using well point system on site (from google website)

4.12.3 Sheet pile construction

Sheet pile installation as retaining wall and watertight structure was discussed in previous sections. Several methods are available to drive the piles into the ground. They include driving, trenching and jetting. When considering driving requirements, size of hammer, penetration depth and soil conditions should be taken into account. To avoid the case of excessive damage with impact hammer in piles, proper protective cap should be used. To keep the sheet piles vertically and aligned along the line, a guide frame or beam should be put on the ground. Piles are lifted through their holes by a crane and positioned to appropriate place. Powered hammer is typically used to install the sheet piles. Hydraulic hammer is of interest when noise reduction is required. Since piles may be tilted during driving, monitoring should be kept at all times (Murthy, 2002). Sheet piles are installed by interlocking each other. On site sheet pile driving can be seen from Figure 4.12.2.



Figure 4.12.2 Sheet pile driving using hydraulic hammer (from google website)

4.12.4 Pile driving

In pile foundation process, installation of piles is important process that impacts on bearing capacity of the piles, settlement and soil resistance. Therefore, pile driving machine should be chosen carefully to minimize the adverse effects during driving. To compare the results of dynamic load tests using diesel and hydraulic hammers, Ashley et al (n.d.) conducted the on-site test on the territory of Nazarbayev University. Test was performed on square precast pile with width of 30 cm. results of the static load and dynamic load tests reveals that piles are not fully driven to the required ground depth because of insufficient blowing energy of diesel hammer. However, excessive blowing can cause the breaking and cracking of the piles during installation. Therefore, suitable hammer type with adjusted blowing should be selected for safe and adequate pile driving. To choose the appropriate hammer type, preliminary estimations in terms of necessary blowing energy and penetration depth per blow should be accomplished. Moreover, availability of the equipment in the market of Astana also vital.

To check the carrying capacity of pile on site and identify suitability of a driving hammer, monitoring of driving process should be accomplished. Moreover, it is important to perform pile strength check after installation since during driving ground support may be weakened and piles can be damaged. Therefore, pile construction monitoring and pile quality inspection after installation will be discussed further.

4.12.5 Construction monitoring procedure

Since piles under design will be driven to the ground, they are subjected high driving stressed during installation process. Exceed in these stresses may cause the structural damage in piles. To prevent this undesirable case, pile installation can be monitored using dynamic testing approach. This testing method can be accomplished during pile testing phase or permanent pile installation. Results of dynamic monitoring is used to confirm the bearing capacity of the pile and its structural integrity. Basic steps of monitoring of pile construction are listed below (Hussein & Likins, 2005).

Step 1. Data preparation

Data regarding pile driving is predicted based on soil behavior during pile installation and using wave equation analysis. This is necessary for selection of suitable hammer type and forecast blow count, assessment of stresses induced in pile wall.

Acquired data will be used further to compare with the results of dynamic pile monitoring.

Step 2. Sensor attachment

For monitoring purposes, reusable strain transducers and accelerometers are used as an attachment to the pile. They are placed small distance apart from the top of the pile. These sensors are reusable. Monitoring equipment is light and easy to handle. Signals produced by the hammer are caught by sensors and send to the monitoring equipment for further analysis. Basic principle of measuring system is use of wave equation analysis, which gives more accurate outcome for hammer performance and pile capacity.

Step 3. Dynamic pile testing

Data from in-situ dynamic pile testing is acquired by special computer system called Pile Driving Analyzer (PDA). Results obtained instantly after each hammer blow are used for following purposes: to evaluate driving hammer performance to assess its productivity; stresses induced by driving piles to mitigate the possible risk of pile damage; to measure the structural integrity condition of the pile; to evaluate soil strength, and static bearing capacity of the pile. Furthermore, CAPWAP, “signal matching” computer program can be utilized to analyze recorded information and produce a graph of pile static load versus movement (Likins & Rausche, n.d.).

During in-situ dynamic pile monitoring, number of blow, stresses caused by hammer impact on pile wall, energy transfer to the pile, real soil resistance to driving will be obtained from the penetration depth of the pile. These data are used to confirm the pile design and adequacy of selected hammer.

Step 4. Data evaluation

Geotechnical engineers analyze the data recorded from monitoring and compare it with predicted data. End pile capacity after pile installation is evaluated by performing signal matching. Ultimately, this information enables to decide on pile acceptability.

4.12.6 Pile quality control after installation

The performance of pile after installation depends on several factors as material strength, soil characteristics and construction method. Convenient and economic approach to assess pile quality after placement is pile integrity test (PIT). This is non-

destructive testing method based on low strain integrity. To perform the PIT, small hand-held hammer is used to impact the pile top or shaft. This impact creates compression wave along the pile shaft. Alteration in pile “impedance”, pile tip, and soil resistance effects leads to reflection of low-strain wave. Response by low-strain wave can be measured with an accelerometer. Test results is typically given as velocity of the pile top, which comprises impact and wave reflection. Acquired test records can be analyzed by visual inspection or computer. Analysis using computer is based on simulations and numerical methods (Hussein & Likins, 2005). Figure 11 shows pile integrity test in the field.



Figure 4.12.3 Performance of PIT in the field (Hussein & Likins, 2005)

Since concrete piles are popular construction materials, novel methods of its long-term health monitoring have been developing. An innovative approach on the basis of piezoceramic transducers provide continuous monitoring of concrete piles and allow to detect the signs of structural damage. This automated inspection device works using smart aggregates embedded in concrete piles. These smart aggregates illustrated in Figure 4.12.4 (a) are associated with piezoceramic patch with lead wires. Firstly, for water proof purposes, insulation is used to coat the piezoceramic patch, and then these smart aggregates are inserted into a small concrete block. These small blocks are necessary to protect the aggregates from effect of vibration during concrete placing ((Song et al, 2010)). Piezoceramic patch with coating and electric wires can be seen from Figure 4.12.4 (b).

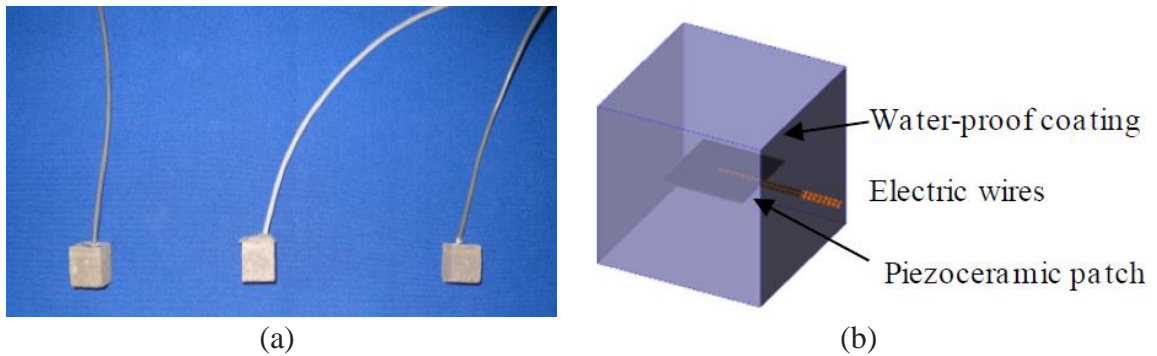


Figure 4.12.4 (a) Piezoceramic based smart aggregates; (b) smart aggregate with coating (Song et al, 2010)

Health monitoring approach used for long-term concrete pile check is based on the active sensing of smart aggregates. These aggregates distributed at different locations of the concrete piles play a role of an actuator and sensor to produce the required wave and to detect the responses of these waves, respectively. The pattern of this phenomenon is demonstrated in Figure 4.12.5.

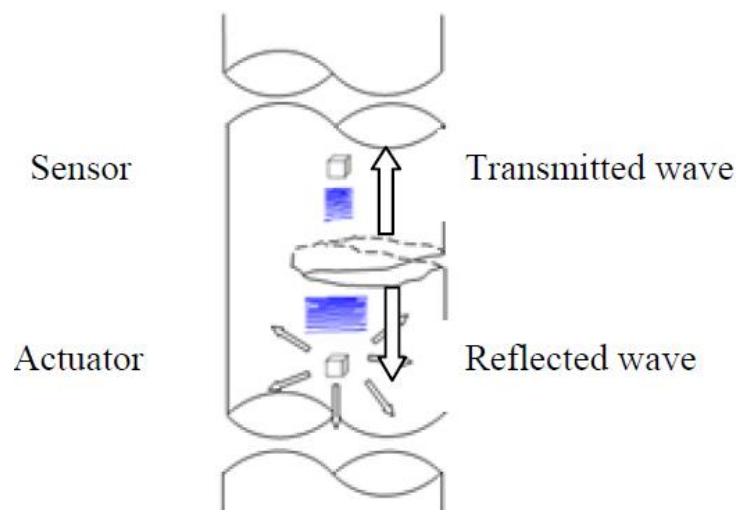


Figure 4.12.5 Active sensing system for concrete pile (Song et al, 2010)

Damage in the concrete structure can be identified through the energy of propagated wave, which is reduced due to the existence of a crack. When expanded wave energy passes along the crack, acoustic impedance differs in cracked area. Suitable signal processing can be employed for further analysis of the wave response to assess the severity of the damage. Eventually, distinguishing the energy distribution vector between healthy and damaged concrete piles, damage index can be found (Song et al, 2010).

5 ENERGY PERFORMANCE

5.1 Phase Change Material

5.1.1 Introduction to the phase change material

Phase change material (PCM) is the manufactured petroleum based and/or hydrated salt solution substance (Sutterlin, 2011). As the term implies, it is capable to change its state (solidifying/liquifying) at a particular point of temperature scale. During that process a certain amount of the energy is stored or released. Figure 5.1.1 below shows the solid/liquid phase transition curve of the PCM. The energy performance of the PCM can be described with the help of sensible and latent heat terms (Sutterlin, 2011). Sensible heat is the energy storage of the material where the temperature increases with every added BTU. Latent heat is the energy storage of the material where the temperature does not change with increasing BTU. As it can be seen from the Figure 5.1.1 when the temperature goes up, the PCM material will get warmer until it reaches a phase changing temperature and will start to store a sensible heat. Eventually, as soon as the temperature is at a phase changing scale (melting point), the PCM will store a large amount of energy performing like a latent heat storage substance keeping the temperature constant until it melts fully. When the PCM is fully melted, it will perform like a sensible heat storage substance again. In case the temperature goes below the cooling cycle will occur with the reverse procedure to heating cycle, i.e. the energy will be released during phase changing process of PCM (Sutterlin, 2011).

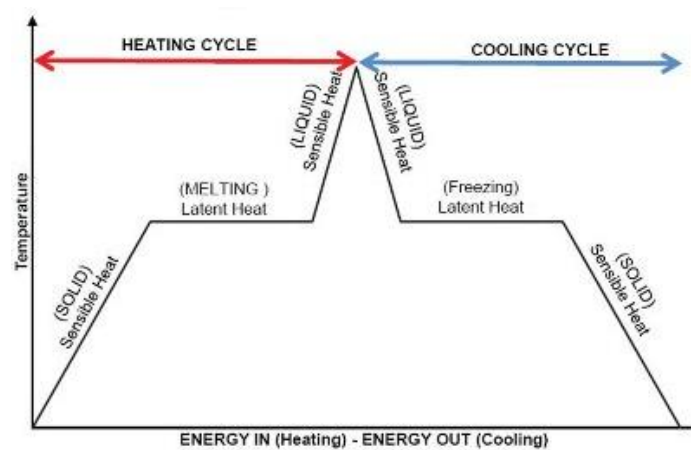


Figure 5.1.1 Latent Heat curve for solid/liquid phase transition (source: Pharmateutical Outsourcing)

5.1.2 Classification of the phase changing material

There are three main categories of the PCM which can be seen from Figure 5.1.2 The most common PCM for the incorporation to the building purposes are paraffin (organic) and hydrated salt (inorganic) (Fang, 2009). The following Table 5.1.1 describes these two PCM types in terms of their composition, latent heat of fusion and other properties. Eutectic type of the PCM is the mixture of organic and inorganic PCMs. But some properties of this type of PCM are

not investigated yet, and it will be neglected in this project. Thus, major focus is given to organic and inorganic PCMs.

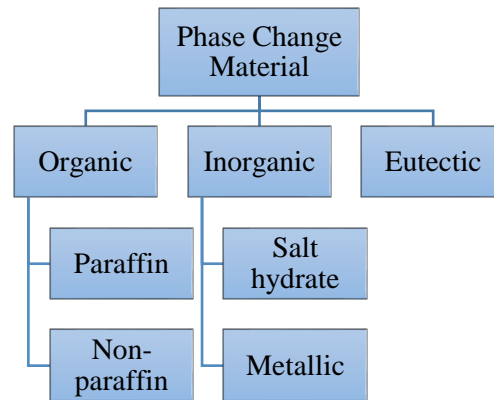


Figure 5.1.2 Classification of PCMs

Table 5.1.1 PCM properties (Fang, 2009) and (Memon, 2014)

Name	Composition	Advantage	Disadvantage
Paraffin (organic)	<ul style="list-style-type: none"> • Crude oil; • Coal; • Other organic materials. 	<ul style="list-style-type: none"> • Large temperature range availability; • Compatibility with conventional material of construction ; • No segregation; • Non-corrosive, non-toxic and chemically stable; • Less super cooling; • Melts congruently; • Recyclable; • Biodegradable. 	<ul style="list-style-type: none"> • Lower thermal storage capacity; • Low thermal conductivity; • Flammable.
Hydrated salt (inorganic)	<ul style="list-style-type: none"> • Anhydrous salts and water molecules. 	<ul style="list-style-type: none"> • High volumetric latent heat storage capacity; • Low cost and easy availability; • Non-flammable; • Sharp phase change; • High thermal conductivity. 	<ul style="list-style-type: none"> • High volume change; • Supercooling; • Segregation ; • Corrosive.

5.1.3 Phase change material's application to the buildings

Due to the rapid rising population and its increasing demands on good life conditions the consumption of the energy is becoming huge as time passes. Most of the energy is obtained by the fossil fuel, the usage of which results in an emission of the carbon dioxide. According to

International Energy Agency (IEA), world electricity generation by fossil fuel increased about 4 times from 1971 to 2010, which can be seen from Figure 5.1.3 Building sector accounts for the major part of the energy consumption. As an example, the statistics in US is taken. Figure 5.1.4 shows the distribution of the energy consumption to different sector, among energy consumption for the residential purposes is more than a one quarter. (Energy USDO, 2011). Additionally, Fang (2009) states that according to the most recent data of year 2001, air conditioning was estimated to be 16% of total annual consumption in US. And it is evident that this percentage increased due this date. Analyzing these data, seek of alternative energy producing or storing technologies or substances are of a huge concern. Consequently, PCM is proposed as one of the efficient solution and it has been on the top of the research works topics for the last 20 years (Zalba et al, 2003).

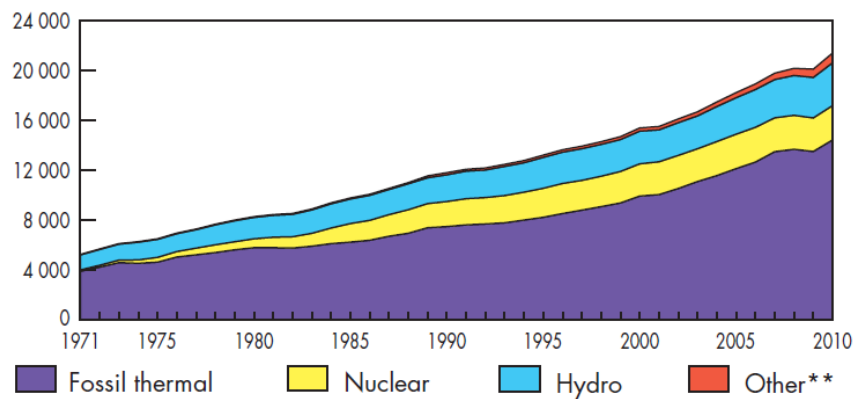


Figure 5.1.3 World electricity generation by fuel (TWh)

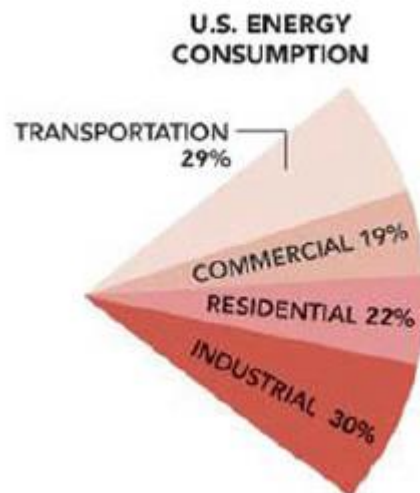


Figure 5.1.4 Energy consumption by different sectors

There are several methods of PCM application to the building (Chan, 2011):

- 1) Chilled ceiling;
- 2) Underfloor air- conditioning;

- 3) Building roof;
- 4) Window glazing;
- 5) External wall of the building

It was investigated that the incorporation of the PCM to the chilled ceiling, underfloor air-conditioning and building roof is not efficient, because it consumes a space of the floor, which limited due to the installation of the floor materials and false ceiling devices.

According to Chan (2011), the experiment that estimates the heat loss reduction with the application of the PCM was performed. The main objective was to analyze doubled glazing window with and without PCM. As a result, it was obtained that the heat loss was reduced to 30% when the PCM was used. Although, the transmittance was acceptable to use daylight in the room, architects and customers were not satisfied with this method because of inhomogeneous appearance of the PCM.

Wide spread method of the PCM application to the building is the PCM enhanced wallboard. With reference to the research papers it was obtained that the room temperature during summer season was decreased to 4.2 °C (Kuznik and Virgone, 2009).

5.1.4 Phase change material's integration to the wallboard

There are various methods of PCM integration to the building walls. The most common methods are described in Table 5.1.2 below (Fang, 2009).

Table 5.1.2 PCM integration methods and their description(Fang, 2009) and (Memon, 2014).

PCM integration method	Method description	Advantage	Disadvantage
Direct mixing	<ul style="list-style-type: none"> • PCM is mixed with conventional construction materials. 	<ul style="list-style-type: none"> • Most economical and simple method. 	<ul style="list-style-type: none"> • Risk of PCM leak, it has an adverse impact on durability and mechanical properties of construction materials
Encapsulation	<p>Micro-encapsulation: PCM particles ranging 1µm~1000 µm is encapsulated to the thin solid shell.</p> <p>Macro-encapsulation:</p> <ul style="list-style-type: none"> • PCM is 	<p>Micro-encapsulation:</p> <ul style="list-style-type: none"> • Prevent leakage of PCM; • resist volume change during phase transition; • improved thermal reliability; <p>Macro-encapsulation:</p> <ul style="list-style-type: none"> • Easy to transport; • Reduce volume change 	<p>Micro-encapsulation:</p> <ul style="list-style-type: none"> • Decreased heat transfer rate because of shell rigidity; • High cost <p>Macro-encapsulation:</p> <ol style="list-style-type: none"> a) Low thermal conductivity; b) Need more work

	encapsulated to containers of spherical and tubic shapes.		and labors to install; <ul style="list-style-type: none"> • Low heat transfer because of corner and edge solidification
Layer method	<ul style="list-style-type: none"> • PCM is placed between two thin polymer sheets like ordinary fiberglass insulation. 	<ul style="list-style-type: none"> • Easy to manufacture • Easy to transport • More practical to install than micro and macro-encapsulation • No oxidation and hygroscopic problems 	

5.1.5 Phase change material details used in this project

It was investigated that the efficiency of the PCM depends on the PCM type, application and integration methods, PCM amount, PCM location and the local climate (Alam et al, 2014).

5.2 PCM selection

According to Khudhair and Farid (2004), those PCMs are preferred to use for building application purposes which suit particular criteria, such as:

- High thermal conductivity;
- High specific heat capacity;
- Small volume change;
- Non-corrosive;
- Non-toxic;
- Low supercooling;
- Low segregation;
- Phase change temperature close to human comfort temperature.

Initially, it is obvious that inorganic PCMs are desirable to use for this purpose because of major advantages such as low cost and high volumetric latent heat storage capacity. However, as it was mentioned in Table 5.2 they tend to supercool, which means that the PCM will not change its phase at phase changing temperature, for example, water may not freeze at temperature below 0°C. Additionally, when using inorganic substances corrosion and segregation occurs, which means that it is risky to use them in a building wall where exists other construction materials; and they does not melt congruently. These three negative sides of the inorganic PCM make them unuseful for the building application. Thus, the current researches are

focusing mostly on the usage of organic PCM besides its high cost and low heat storage capacity. Nevertheless, they are non-corrosive, non-toxic, have no supercooling and available at a wide range of phase change temperature. Paraffin based organic PCM temperatures are available at human comfort zone, they are chemically inert and biodegradable (Memon, 2014). Biswas and Abhari (2014) and Alam et al (2014) used paraffinic bioPCM in building application, because it is biodegradable and has low cost. Also, BioPCMs are available at a wide range of melting temperatures which provides comfortable human thermal zone. Hence, there will be used BioPCM for the dormitory construction. As for the melting point of the PCM, it should be selected so as to provide a thermal condition in a human comfort zone. Souayfane (2016) states that in order to keep indoor temperature within human comfort range (23-27°C), the PCM must have melting temperature between 19-24°C. Thus, in dormitory construction the PCM with 24°C melting temperature will be used.

5.2.1 PCM application and integration method

Another concern about PCM usage as an energy storage device in the building is the method of application. There were introduced several types of the application, which are application to the ceiling chills, roof, floor, window glazing and wallboard. Relying on the past experiments, it was decided to use wallboard enhancement in terms of further reasons:

- It provides the room temperature in the range of human comfort and contributes to the reduction of temperature fluctuation (Kuznik and Virgone, 2009);
- It has an insignificant effect on the room area as it does has when applied to the roof and floor (Chan, 2011).

As for integration method, it was chosen to be layer method because it has several advantages over other ones. In layer method PCM is encapsulated in thin polymer sheets preventing direct contact with coatings and wall finishes, while direct mixing method does. Secondly, this method is superior to macro and micro-encapsulation methods, because it is more practical to install, hence require less work and labor on site. To be more specific here, in macro-encapsulation the installation of individual pipes is required, while in layer method the priority is given to reduce the pipe distances in order to make continuous section in which the pipes are placed next to another. The main objective of this method is to improve the reduction of peak heat flux which depends on PCM placement (Fang, 2009).

5.2.2 PCM location

The location of the PCM on the wall is a significant factor of PCM efficiency. Several investigations have been made by Jin et al (2012). In this study the thermal performance of the PCM placed at 7 different locations were observed as it can be seen from the Figure 5.2.1 The

result for the measurement of the heat flux across walls can be seen from Figure 5.2.2 It can be seen that the lowest heat flux was obtained in the wall where PCM is installed at location $1/5L$, in which L is the width of the insulating material. Figure 5.2.3 shows the cross-section view of the integrated wall that will be considered in this capstone project.

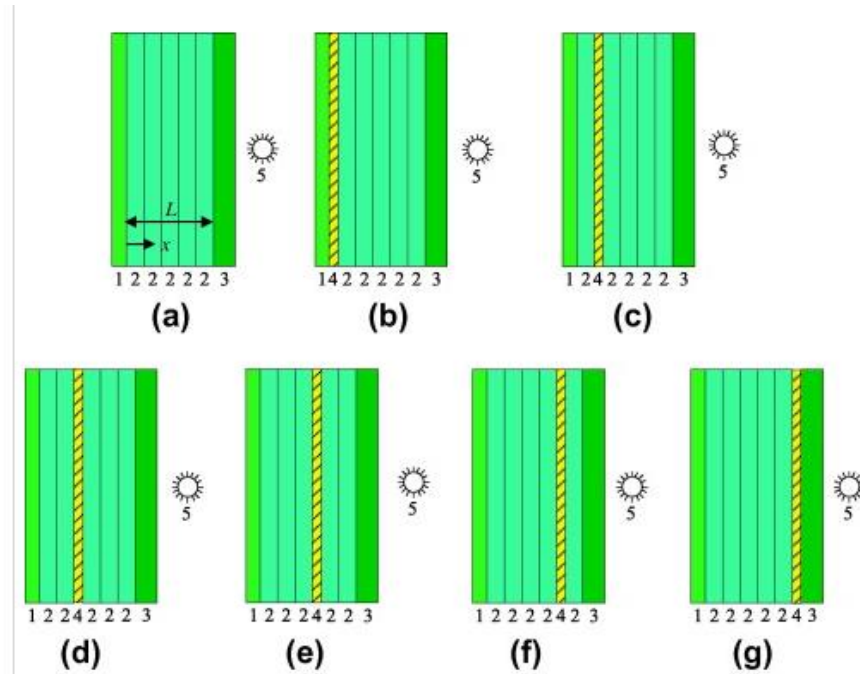


Figure 5.2.1 Schematic of wall construction

a) control wall, b) PCM enhanced wall at $0/5L$, c) PCM enhanced wall at $1/5L$, d) PCM enhanced wall at $2/5L$, e) PCM enhanced wall at $3/5L$, f) PCM enhanced wall at $4/5L$, g) PCM enhanced wall at $5/5L$. 1- gypsum plaster, 2-insulating layer, 3-OSB, 4- PCM, 5- heat source (outdoor)

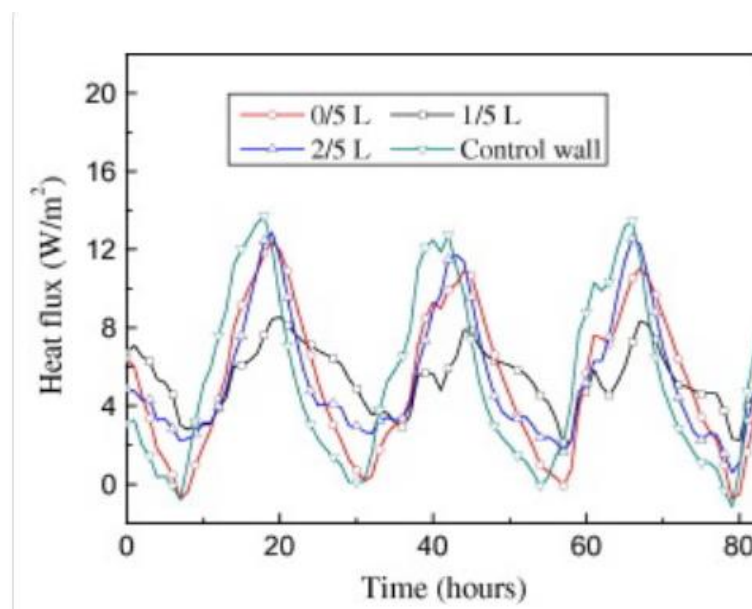


Figure 5.2.2 Heat flux measurement across wall

Also, the position of the PCM layer in the building wall is an important factor of the efficient PCM usage. A study made by Chan (2011) observed the energy consumption reduction

performance of the building when PCM was integrated to north, west, south and east walls. The results show that in Hong Kong west wall achieved the highest energy consumption reduction in summer time, because those walls received the highest amount of solar radiation. As for the climate of Astana, Kenzhetayev et al (2010) observed that the highest intensity of the solar radiation was in the south-west and south-east directions. Thus, in order to analyze the energy performance of the building, one section located on the east side of the building is selected.

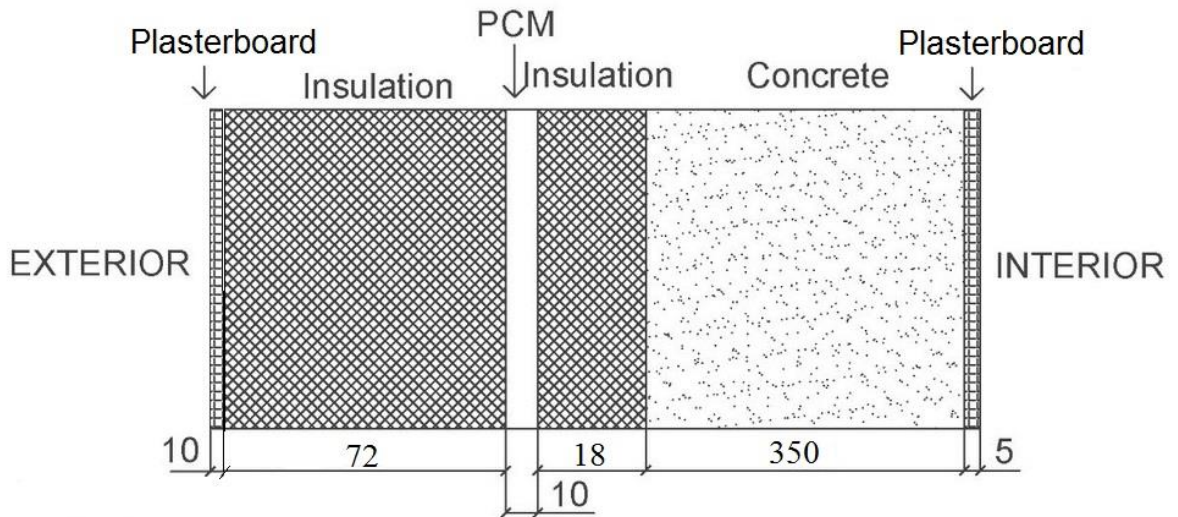


Figure 5.2.3 Cross-sectional view of the wall

5.3 Energy analysis

5.3.1 Selection of algorithm

Energy analysis is made by the program Energy Plus. Energy Plus software is the simulation program that analyzes energy performance, heating and cooling loads applied to maintain a certain thermal condition in the building. Energy Plus software uses heat balance method to estimate heating and cooling loads.

There are several types of building energy modeling programs. Their features are described in Table 5.1.3 with reference to (Aleem et al, 2016).

Table 5.3.1 Energy modeling programs

Program	Strength	Weakness
DOE-2	Easy to use with graphical user free	No modern engineering manual, difficult to understand calculation method.

TRNSYS	Easy to use; wide modeling capabilities; higher accuracy than DOE-2; possible to add new systems.	Expensive
Energy Plus	Improved accuracy; Possible to add new systems; good model library.	Requires programming skills.

Additionally, there are several building modeling programs that work in couple with Energy Plus software and obtain energy performance results. Common programs are Sketchup and DesignBuilder. In this project Design Builder is used, because it has a wide variety of PCM materials in the library, and it allows the user to change the properties.



Figure 5.3.1 Design Builder running with Energy plus

Because PCM has variable heat storing capacity, Finite Difference algorithm for calculation option is used. The screen of this option selection can be seen in Figure 5.1.2 The following Eq. 3.1 describes the node formulation of the PCM (Chan, 2016):

$$\frac{\rho C_p \Delta x (T_i^{n+1} - T_i^n)}{\Delta t} = \frac{k(T_{i-1}^{n+1} - T_i^{n+1})}{\Delta x} + \frac{k(T_{i+1}^{n+1} - T_i^{n+1})}{\Delta x}$$

Where,

- ρ - density (kg/m³);
- C_p - specific heat capacity (kJ/kg*K);
- Δx - layer thickness;
- T - node temperature;
- $n + 1$ - new time step;
- n - previous time step;
- i - node being modeled;
- $i - 1$ - adjacent node to exterior of construction;
- $i + 1$ - adjacent node to interior of construction;
- Δt - calculation time;
- k - thermal conductivity (kW/mK).

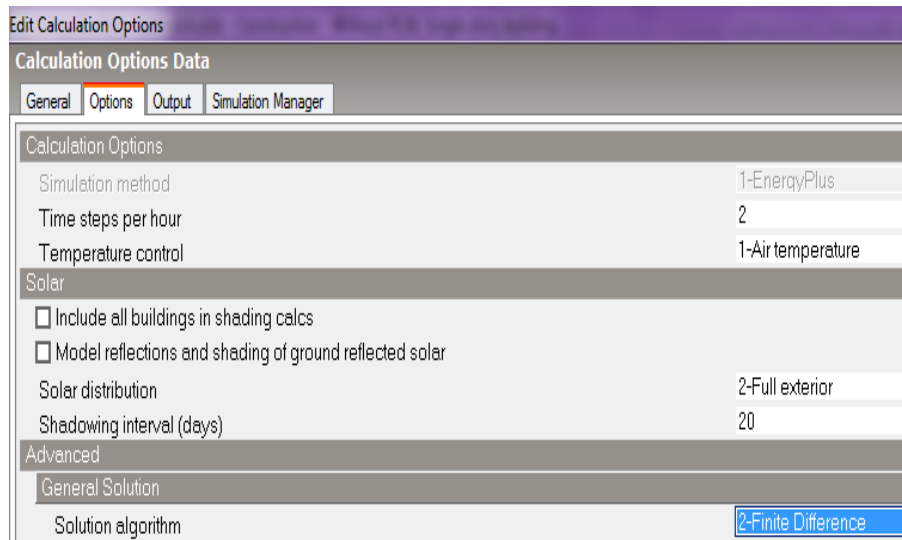


Figure 5.3.2 Finite difference algorithm selection screen

5.3.2 Selection of Phase change enthalpy-temperature function

The Finite Difference algorithm employs enthalpy-temperature method to arrange the equations for liquid and solid phases of the PCM to one simple equation:

$$h = h(T) \quad (\text{Eq. 3.2})$$

Souayfane (2016) states that in order to keep indoor temperature within human comfort range (23-27°C), the PCM must have melting temperature between 19-24°C. Thus, in dormitory building the PCM with 23 °C melting temperature will be used. For this kind of PCM the enthalpy-temperature function is indicated in Figure 5.3.3

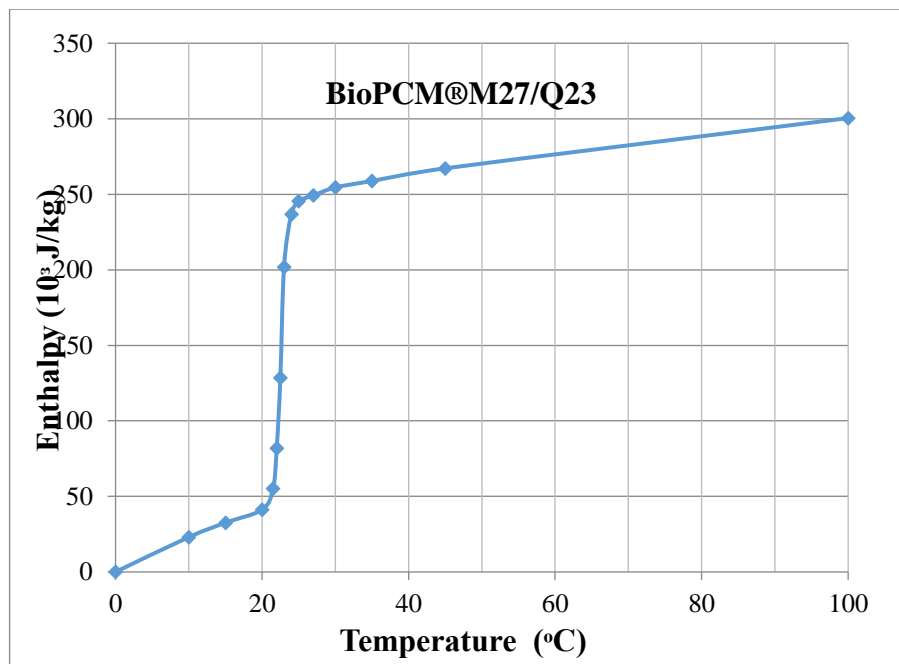


Figure 5.3.3 Enthalpy-temperature function of BioPCM®M27/Q23

5.3.3 Validation of model by comparing with data available in literature

In this study, the performance of PCM and the Energy Plus algorithm will be verified against past experiment done by Ozdenefe and Dewsbury (2012). This paper was selected as a datum, because it is more recent report where updated version of the program can be used. Also, this paper focused on the simulation of the single room house. This experiment was done on Design builder. Figure 5.3.4 shows the appearance of the simple house.

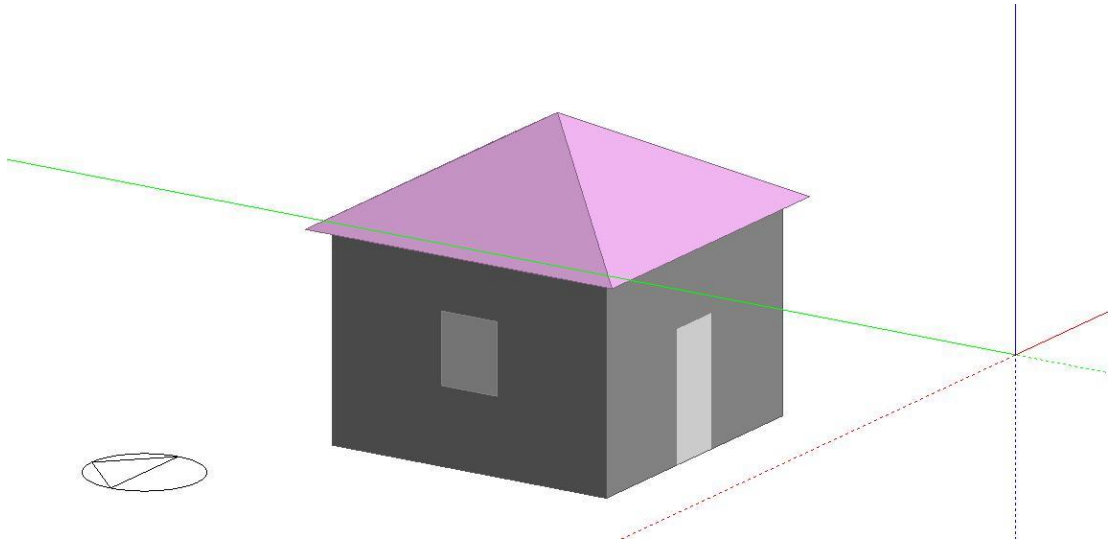


Figure 5.3.4 Single room house

The dimensions are shown in Figure 5.3.5

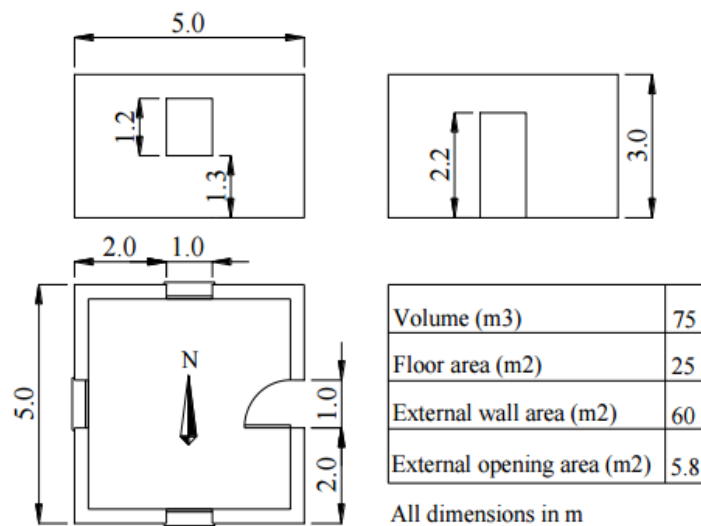


Figure 5.3.5 Plan view of the modeled building

The experiment was done in Cyprus, thus the weather data for Cyprus is loaded.

Then, materials are set as follows:

Category	Walls
Region	CYPRUS
Definition	
Definition method	1-Layers
Calculation Settings	
Layers	
Number of layers	3
Outermost layer	
Material	Copy of Plasterboard
Thickness (m)	0,0250
<input type="checkbox"/> Bridged?	
Layer 2	
Material	Copy of Brick
Thickness (m)	0,2000
<input type="checkbox"/> Bridged?	
Innermost layer	
Material	Copy of BioPCM® M27/Q25
Thickness (m)	0,0150
<input type="checkbox"/> Bridged?	

Figure 5.3.6 Wall specification

Category	Roofs
Region	CYPRUS
Definition	
Definition method	1-Layers
Calculation Settings	
Layers	
Number of layers	4
Outermost layer	
Material	Asphalt
Thickness (m)	0,0100
<input type="checkbox"/> Bridged?	
Layer 2	
Material	MW Glass Wool (rolls)
Thickness (m)	0,1445
<input type="checkbox"/> Bridged?	
Layer 3	
Material	Air gap >=25mm
Thickness (not used in thermal calcs) (m)	0,2000
Innermost layer	
Material	Plasterboard
Thickness (m)	0,0130
<input type="checkbox"/> Bridged?	

Figure 5.3.7 Roof specification

Category	Floors (ground)
Region	CYPRUS
Definition	
Definition method	1-Layers
Calculation Settings	
Layers	
Number of layers	5
Outermost layer	
Material	Copy of Tile Bedding
Thickness (m)	0,1500
<input type="checkbox"/> Bridged?	
Layer 2	
Material	Cast Concrete
Thickness (m)	0,1000
<input type="checkbox"/> Bridged?	
Layer 3	
Material	Copy of Sand and gravel
Thickness (m)	0,0500
<input type="checkbox"/> Bridged?	
Layer 4	
Material	Copy of Floor/Roof Screed
Thickness (m)	0,0200
<input type="checkbox"/> Bridged?	
Innermost layer	
Material	Copy of Marble
Thickness (m)	0,0300
<input type="checkbox"/> Bridged?	

Figure 5.3.8 Floor specification

Edit material - Copy of BioPCM® M27/Q25

Materials Data

General | Surface properties | Green roof | Embodied carbon | Phase change | Cost

General

Name Copy of BioPCM® M27/Q25

Description

Source Phase Change Energy Solutions

Category Phase change

Region General

Material Layer Thickness

Force thickness

Default thickness (m) 0,0150

Thermal Properties

Detailed properties

Thermal Bulk Properties

Conductivity (W/m-K) 0,1960

Specific Heat (J/kg-K) 1970,00

Density (kg/m3) 770,00

Resistance (R-value)

Vapour Resistance >>

Moisture Transfer >>

Figure 5.3.9 BioPCM®M27/Q25 specification

Then, simulation is conducted. Recording for a certain month (July) was selected to make the comparison easier. Simulation result for PCM applied building had 3.85% difference from the experiment result from the literature. The overlapped results can be seen from Figure 5.3.10

$$T_{ave\ experiment} = 26.1^{\circ}\text{C}$$

$$T_{ave\ simulation} = 27.1048^{\circ}\text{C}$$

$$Error = \frac{|T_{ave\ experiment} - T_{ave\ simulation}|}{T_{ave\ experiment}} * 100\% = 3.85\%$$

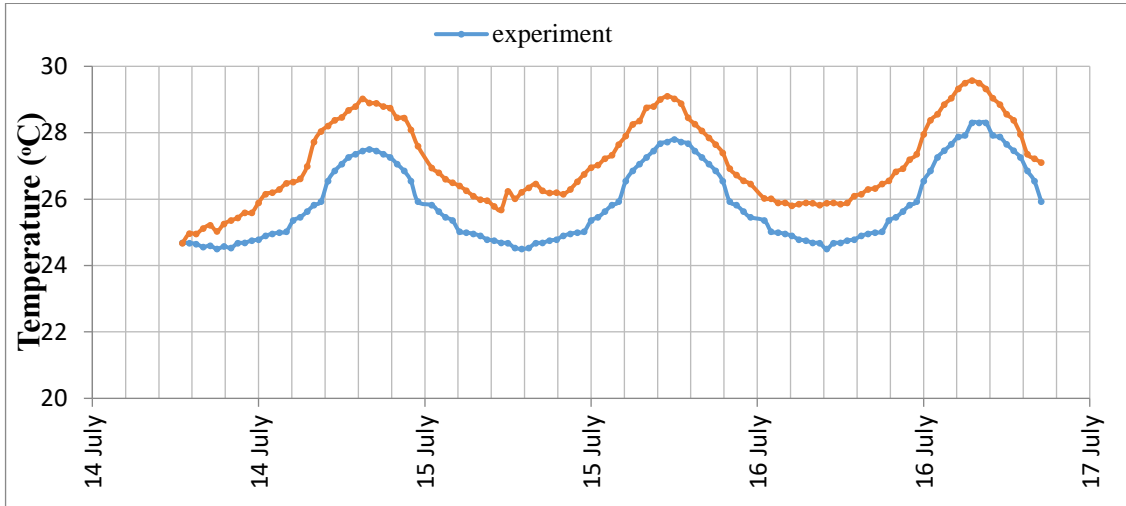


Figure 5.3.10 Comparison of experimental and simulation result for 14,15,16 July

Eventually, the reduction of the electricity consumption for cooling of the building was observed (Figure 5.3.11).

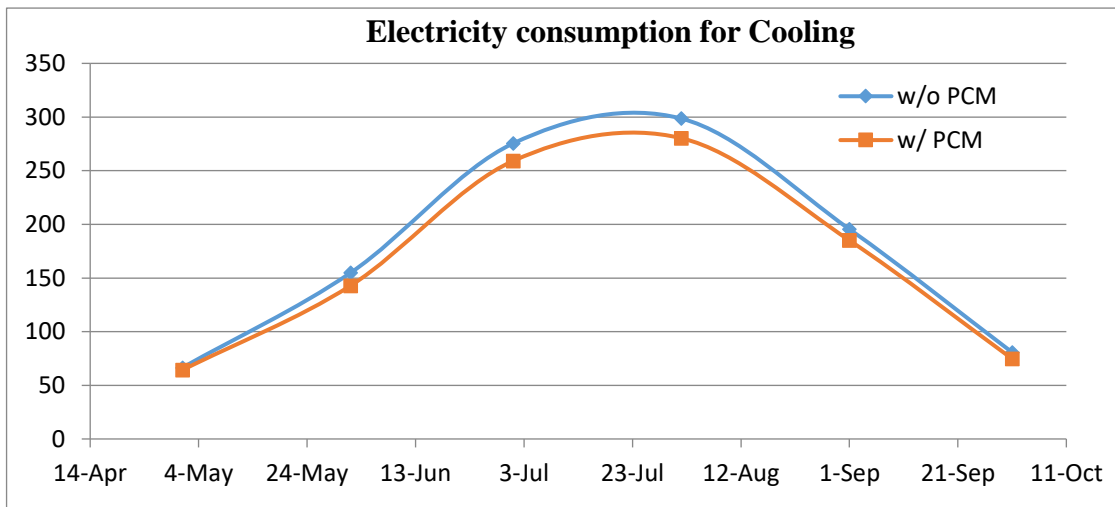


Figure 5.3.11 Electricity consumption to cool the building with and without PCM

5.3.4 Analysis of energy performance during summer

To analyze the energy performance of the building facility during summer, one section of the typical floor which contains two bedrooms, one toilet, one bathroom and an entry way was selected (Figure 5.3.12).

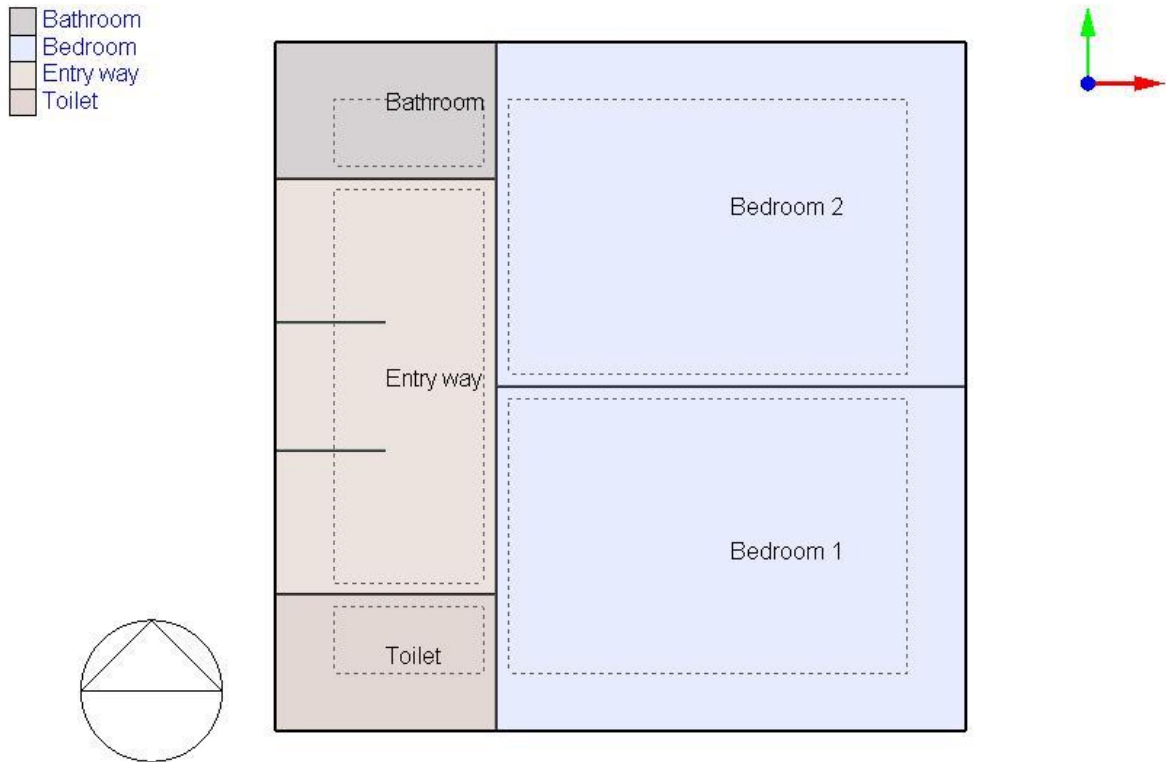


Figure 5.3.12 Plan view of the section to be simulated

The following Table 5.3.2, Table 5.3.3 and Figure 5.3.13 show the material profiles for the wall, roof and floor.

Table 5.3.2 Input parameters for the wall with PCM

Material	Thermal conductivity (W/mK)	Density (kg/ m ³)	Specific Heat	Thickness (mm)
Plaster	0.16	950	840	5
Concrete block	0.037	600	1000	350
Insulation	0.0415	145	1000	100
BioPCM ($T_{\text{melting}}=23^{\circ}\text{C}$) (Alam et al, 2014)	0.2	235	1970	10
Plaster	0.16	950	840	10

Table 5.3.3 Input parameters for the floor and roof

Material	Thermal conductivity (W/mK)	Density (kg/ m ³)	Specific Heat	Thickness (mm)
Roof insulation	0.0415	75	750	0.15
Slab	0.7	2548.42	750	
Floor insulation	0.0415	105	1000	0.02
Concrete screed for flooring	0.7	2446.48	750	0.05
Ceramic tiles (for bathroom and toilet)	0.8	1700	850	0.005
Linoleum (for bedroom)	0.17	1200	1400	0.005

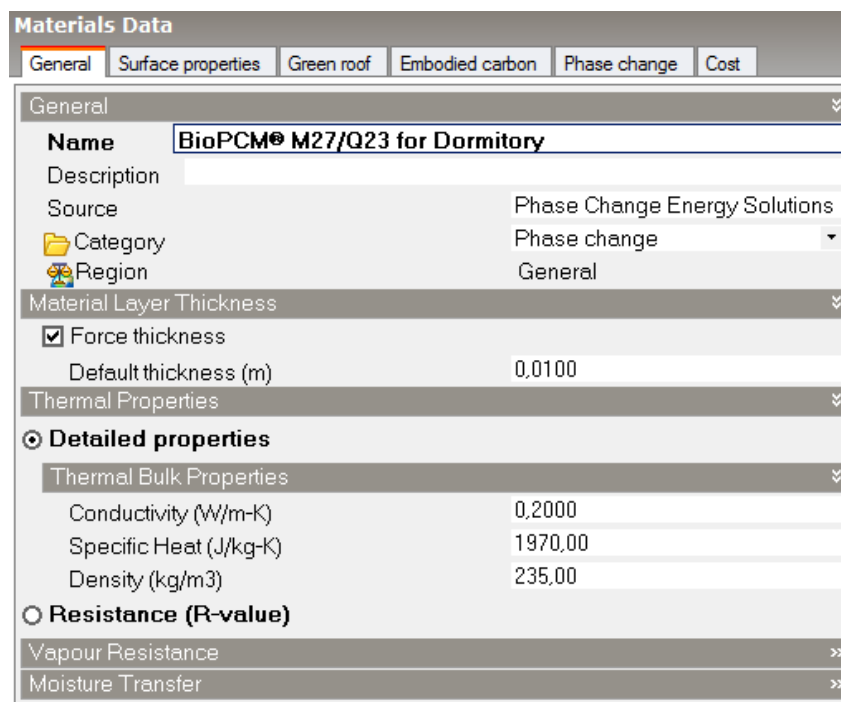


Figure 5.3.13 BioPCM®M27/Q23 specification

For the simulation details, the time step of 4 was selected in order to get more accurate values. Also, the simulation period was during the summer, which is 1st June – 30th August. The following Figure 5.3.14 shows the result of the simulation for three months. It can be seen that there is a slight difference between the mean air temperature for the conventional and PCM integrated walls. The average temperature changed to insignificant value (0.1 °C), but it does not mean that PCM does not work. It is important to know that the function of the PCM is not to increase or decrease the room temperature, but to keep it stable throughout the day and nighttime. Thus, in order to see a detailed behavior of the PCM, the result of the day and nighttime for the hottest day of the summer of Astana was exported (Figure 5.3.15). It shows that PCM mostly kept the temperature in the comfort range.

Moreover, the effectiveness of the PCM can also be shown as an electricity economy. Figure 5.3.16 depicts the function of the monthly electricity consumption for cooling during the summer. A significant difference for the July is noticed, which is explained by the fact that the July is the hottest month of the Astana. It was recorded that the average electricity consumption for cooling decreased to about 0.5 kWh, while the consumption for the whole summer can be calculated as follows:

$$\begin{aligned}
 & \% \text{Electricity consumption reduction} \\
 &= \frac{\text{Electricity consumption without PCM} - \text{Electricity consumption with PCM}}{\text{Electricity consumption without PCM}} \\
 &= \frac{131.69 - 86.76}{131.69} = 34\%
 \end{aligned}$$

As a result, PCM reduced electricity consumption for cooling to 34%, which means that the integration of the PCM to the wall is cost-effective and beneficial in acquiring a room temperature in the comfort range for the human.

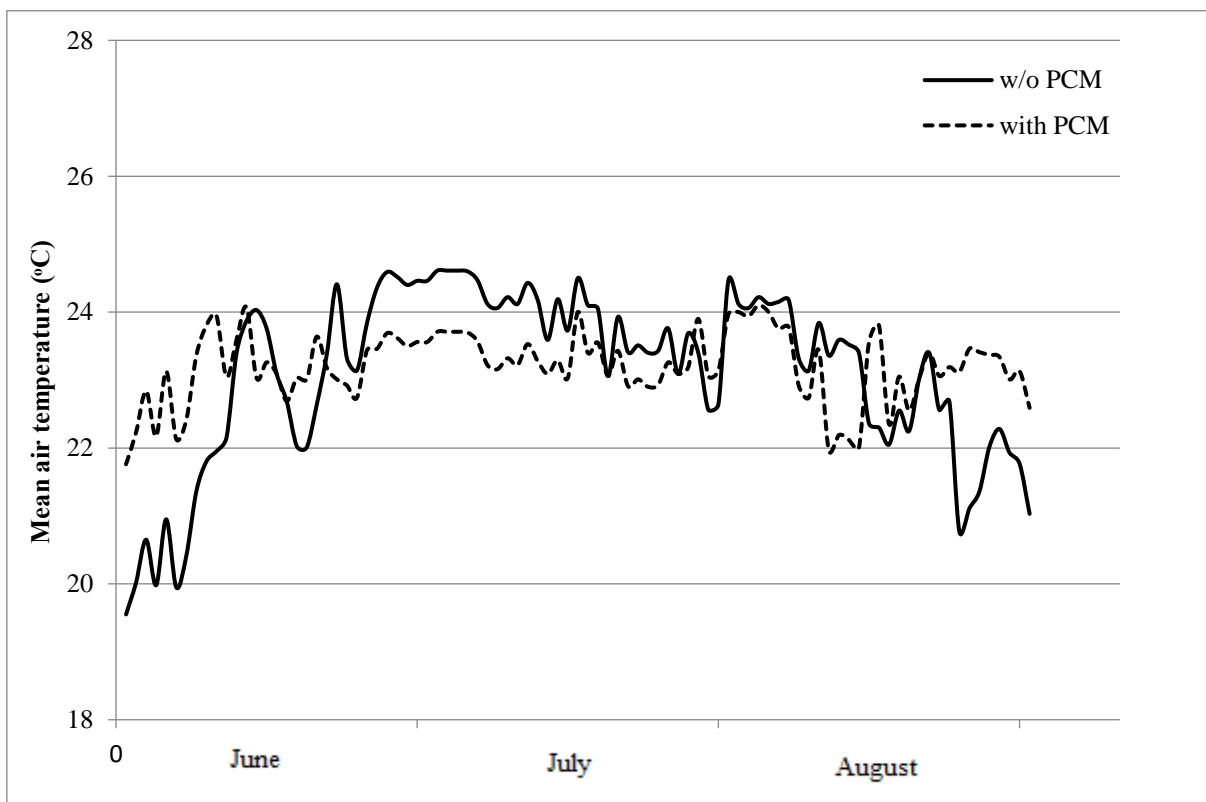


Figure 5.3.14 Interior mean air temperature of the section data for with and without PCM

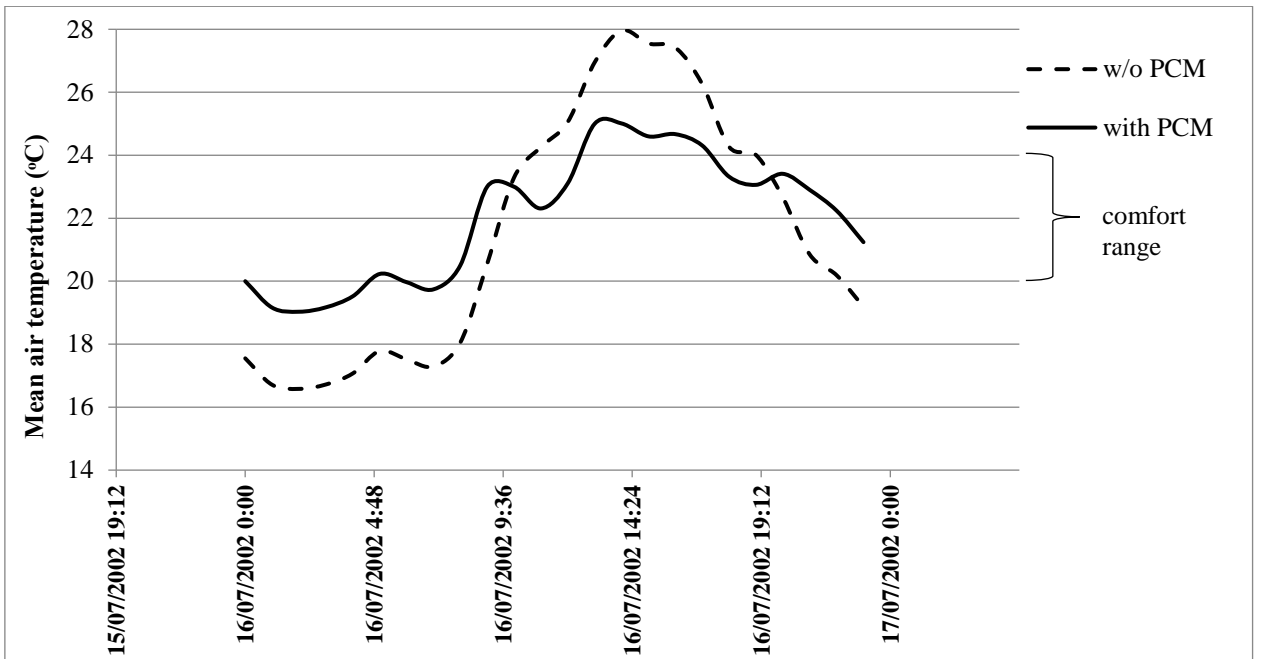


Figure 5.3.15 Hourly mean air temperature data for 16th of July with and without PCM

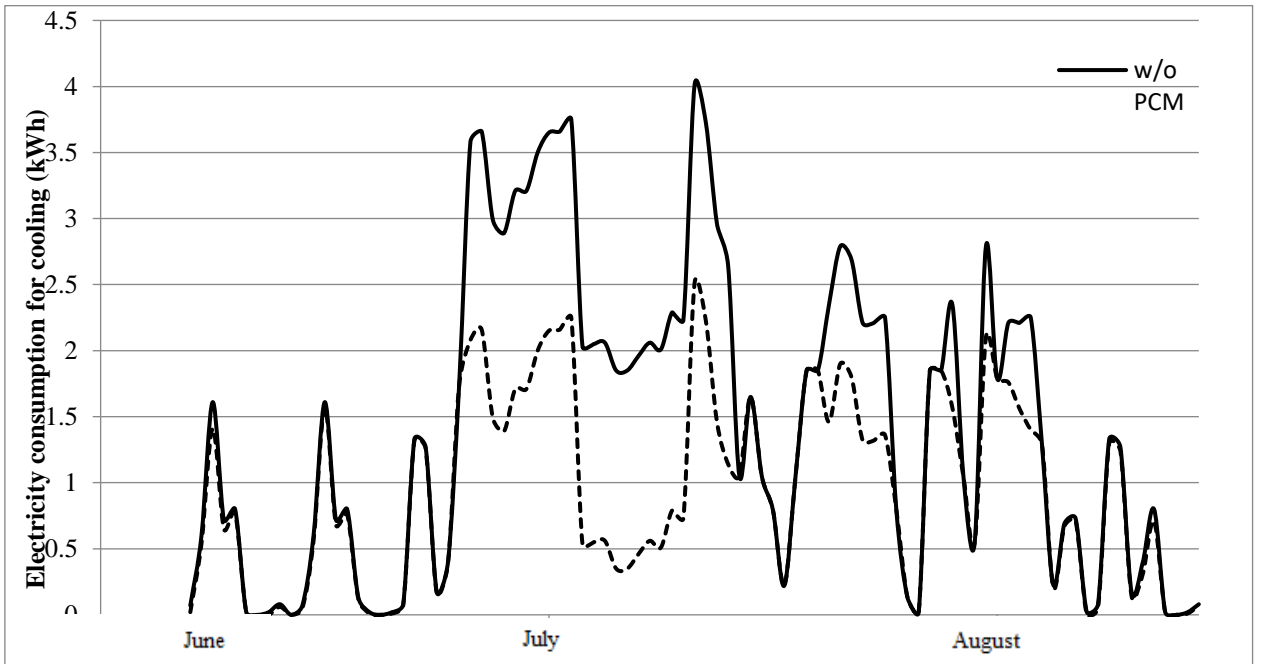


Figure 5.3.16 Electricity consumption for cooling data for with and without PCM

6 CONSTRUCTION MANAGEMENT

To deliver the project on time, within allocated budget and resources, role of construction management is important. This section includes construction delivery method, scheduling of main activities, cost estimate, work breakdown structure and risk management.

6.1 Project delivery

Project is planning to be delivered through design-build contract type. In this contract, owner negotiates with single entity, who will design the project and deal with construction. Advantage of design-build contract is less coordination issues. Also, construction process can be started before the end of final design. This contract type falls into a cost-reimbursement contract category. Complexity of design-build contract is in assessment of supplier's proposals. Hierarchy of management for design-build contract is shown in Figure 6.1.1.

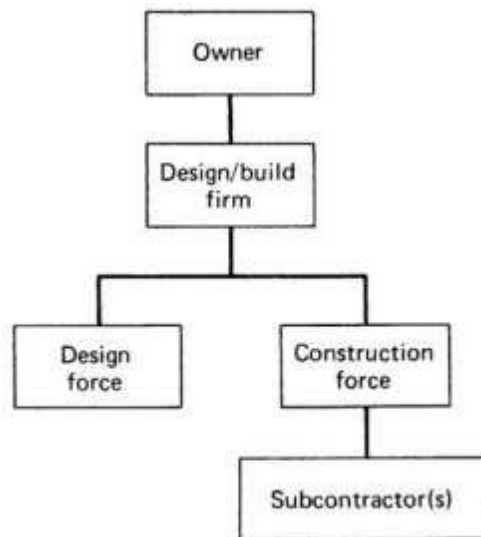


Figure 6.1.1 Design-build contract hierarchy (Shon, 2015)

6.2 Cost estimate

Estimation of project cost consists of three phases, which account during design development and project implementation. Based on the stages of the project development, cost estimate can be categorized as conceptual and preliminary estimate, detailed estimate, and definitive estimate. Conceptual estimating is approximated in the early stage of the project, while more accurate cost is given in detailed and definitive estimates in intermediate and final stages of the project, when more specific information as reinforcing details, material volume, and exact architectural drawings is known. Nevertheless, it should be noted that the accuracy of the detailed (or definitive) estimation is in the range of -5% and +10% (Kerzinger, 2013). Final cost estimate for student resident building is given in Table 6.2.1

Table 6.2.1 Cost Estimate for student resident building in Astana

Item	Quantity	Unit Cost	Total cost
Foundation:			
Site clearance	7 days	\$720 /day	\$ 5,040
Excavator rent	2x24 hrs=48 hrs	\$25/hr	\$ 1,200
Sheet pile	212 pieces	\$160 /pcs	\$ 33,920
Precast concrete pile (cross-section 0.5 m x0.5 m)			
with length 6 m	16 pieces	\$100/pcs	\$ 1,600
with length 8 m	128 pieces	\$130/pcs	\$ 16,640
with length 9 m	96 pieces	\$170/pcs	\$ 16,320
reinforcement for pile cap	520 tons	\$167/ton	\$ 87,054
Concrete for pile cap with depth=1m			
4 piles in group	1m x 2.05 mx 2.05 m= 4.2025 m ³	\$38/m ³	\$ 151
5 piles in group	1m x 2.568 mx 2.568 m=6.59 m ³	\$38/m ³	\$ 250
6 piles in group	1m x 3.3mx 2.05 m=6.77 m ³	\$38/m ³	\$ 257
8 piles in group	1m x 3.3m x 3.33m=11 m ³	\$38/m ³	\$ 418
Basement (d=1 m)	1m x 42 m x 15m=630 m ³	\$38/m ³	\$ 23,940
Labor and Equipment	20%		\$ 37,358
Total			\$ 224,148
Underground floor			
Columns (0.5m x 0.5 m)	2.2m x 0.5m x0.5 m x 32 columns=17.6 m ³	\$38/m ³	\$ 668.8
Short beams (0.5 m x 0.3 m)	5m x 0.5m x 0.3m x24 = 18 m ³	\$38/m ³	\$ 684.0
Long beams (0.4 m x 0.3 m)	6m x 0.4m x 0.3m x 28 = 20.16 m ³	\$38/m ³	\$ 766.1
Labor and Equipment	20%		\$ 423.8
<u>Subtotal</u>			<u>\$ 2,542.7</u>
1st floor			

Columns (0.5m x 0.5 m)	3.6m x 0.5m x 0.5 m x32 columns = 28.8 m3	\$38/m3	\$ 1,094.4
Short beams (0.5 m x 0.3 m)	5m x 0.5m x 0.3m x24 beams= 18 m3	\$38/m4	\$ 684.0
Long beams (0.4 m x 0.3 m)	6m x 0.4m x 0.3m x 28 beams = 20.16 m3	\$38/m3	\$ 766.1
Slab (0.2 m thick)	0.2m x 42m x 15m =125 m3	\$38/m4	\$ 4,750.0
<u>Subtotal</u>			<u>\$ 1,458.9</u>
2-4 floors			
Columns (0.45m x 0.45 m)	3.6m x 0.45m x 0.45 m x 32 columns x 3 floors = 69.98 m3	\$38/m3	\$2,659.24
Short beams (0.5 m x 0.3 m)	5m x 0.5m x 0.3m x24 beams x 3 floors= 54 m3	\$38/m4	\$2,052.00
Long beams (0.4 m x 0.3 m)	6m x 0.4m x 0.3m x 28 beams x 3 floors = 60.48 m3	\$38/m3	\$2,298.24
Slab (0.2 m thick)	0.2m x 42m x 15m =125 m3	\$38/m4	\$4,750.00
<u>Subtotal</u>			<u>\$11,759.48</u>
5-10 floors			
Columns (0.4m x 0.4 m)	3.6m x 0.4m x 0.4 m x 32 columns x 6 floors= 110.6 m3	\$38/m3	4202.8
Short beams (0.5 m x 0.3 m)	5m x 0.5m x 0.3m x24 beams x 6 floors= 108 m3	\$38/m4	4104
Long beams (0.4 m x 0.3 m)	6m x 0.4m x 0.3m x 28 beams x 6 floors = 120.96 m3	\$38/m3	4596.5
Slab (0.2 m thick)	0.2m x 42m x 15m =125 m3	\$38/m4	4750
<u>Subtotal</u>			<u>\$17,653.30</u>
Roof			
Slab (0.15 m thick)	0.15m x 42m x 15m =94.5 m3	\$38/m3	3591
Labor and Equipment	20%		718.2
<u>Subtotal</u>			<u>\$4,309.20</u>
Total			\$261,872.02
Interior			
Partitions: 1 cm PCM wall	42m x 2 x 39m=3276 m3	\$2/m3	\$6,552

Stairways: 2 stairs per floor	2x 11 floors	\$2,500	\$55,000
Flooring material: ceramic	42m x 15m =630 m2	\$180/m2	\$113,400
Flooring: Linoleum for 2-10 floors	9x 42m x 15 m=5670 m3	\$150/m2	\$850,500
Elevators	2 pieces	\$50,000	\$100,000
Doors	10floors x 10 rooms = 100 doors	\$400	\$40,000
Total			\$1,165,452
Exterior			
Curtain wall material	2 x 39 x (15+42) = 4446 m2	\$150/m2	\$666,900
installation	20%		\$133,380
Total			\$800,280
			\$2,454,295.15
	Construction of roads and civil works	5%	\$122,714.76
	Engineering permits	1%	\$24,542.95
	Project management	2%	\$49,085.90
	Final cleanup	0.5%	\$12,271.48
	Taxes	10%	\$245,429.52
	Rate of inflation	1%	\$122.71
		Total:	\$2,908,462.47

According to cost estimate, project worth 2.91 million dollars. Assuming 10-year payback period for student dormitory, rent rate per person can be calculated. Dormitory building with capacity of 200 residents will cost:

$$\frac{\$2.91 \times 10^6}{200 \times 10 \text{ years} \times 12 \text{ months}} = \$ 121.25 \text{ per month}$$

Converting to national currency, it is \$121.25 *313 = 37,951 tenge. In Astana, dormitory renting costs vary from 35,000 tenge to 45,000 tenge. Taking this into account, above renting price can be said as reasonable.

6.3 PCM Cost and Energy saving

Although, there is an immense potential for production of PCM material because of rich oil deposits in Kazakhstan, lack of manufacturers in the country can be explained by poor development and use of PCM. Nevertheless, PCM wallboard can be delivered from near countries. China can be an optimal supplier choice for PCM material because of the reasonable cost of the PCM, transportation time and cost. As indicated Miet all (2016) as well as suppliers' information, 10 mm thick PCM wallboard costs \$2 per square meter. According to architectural drawings and PCM material necessity for the project, PCM will be integrated to west and east walls. The data for PCM cost estimation is shown on Table 6.3.1

Table 6.3.1 PCM cost estimation data

East and west wall length (m)	Height of the walls (m)	Area of the PCM covered (m ²)	Cost of the PCM per square meter (\$/m ²)	Cost of the PCM for the required area (\$)	Delivery cost	Total PCM cost
42m*2	39	3276	2	6552	\$700	\$ 7252

Energy saving potential in terms of electricity consumption reduction for cooling was reduced to 34% when the PCM was applied. “Energy saving – 2020” program (2013) in Kazakhstan reveals that annual energy consumption by the building is 270 kWh/m². Total PCM cover area, total energy consumed and energy saving percentage led to the next calculation:

$$\text{Energy saving} = 0.34 \times 270 \frac{kWh}{m^2} \times 3276 m^2 = 300,736.8 kWh$$

Cost of saved energy is simply amount of saved energy and energy charge rate, which is 14.59 tenge/kWh for Astana residents. Since \$ 1 is equivalent to about 313 tenge, energy charge rate provided by “Astana Energosbyt” LLP is \$0.047/kWh. Hence, cost of preserved energy using PCM wallboard is

$$\text{Energy saving cost} = \frac{\$0.047}{\text{kWh}} \times 300,736.8 \text{ kWh} = \$14,134.6$$

6.4 Feasibility of PCM use

It is important to evaluate whether an integration of PCM is economically and technically feasible for decision-making purposes. The proper way of feasibility study is calculation of payback period, which is the indicator of how fast investment on project will be returned. Static payback period (SPP) can be used to evaluate the feasibility of PCM material. SPP can be found as following:

$$SPP = \frac{C_{PCM}}{S} \quad (6.1)$$

Where C_{PCM} is the cost invested to purchase and install PCM material, S is the annual income due to energy savings. Assuming PCM construction cost as 30 % of material cost, payback period can be calculated using equation (6.1):

$$SPP = \frac{1.3 * \$7252}{\$14,134.7} = 0.67 \text{ months}$$

Thus, the payback period for PCM material is 0.67, which is a reasonable being within 1 year.

6.5 Project scheduling

Project planning and executing is one of the most important factors to complete the construction process successfully. The significant parts of the project planning are identifying the work clearly to project participants and allocation of those activities to a certain schedule. The method of presenting systematically the work, services and data required to produce end product is the Work Breakdown Structure (WBS) (Kerzner, 2013). The WBS for the dormitory construction is shown in Figure 6.5.1, while Gantt Chart showing project plan is given in Figure 6.5.2.

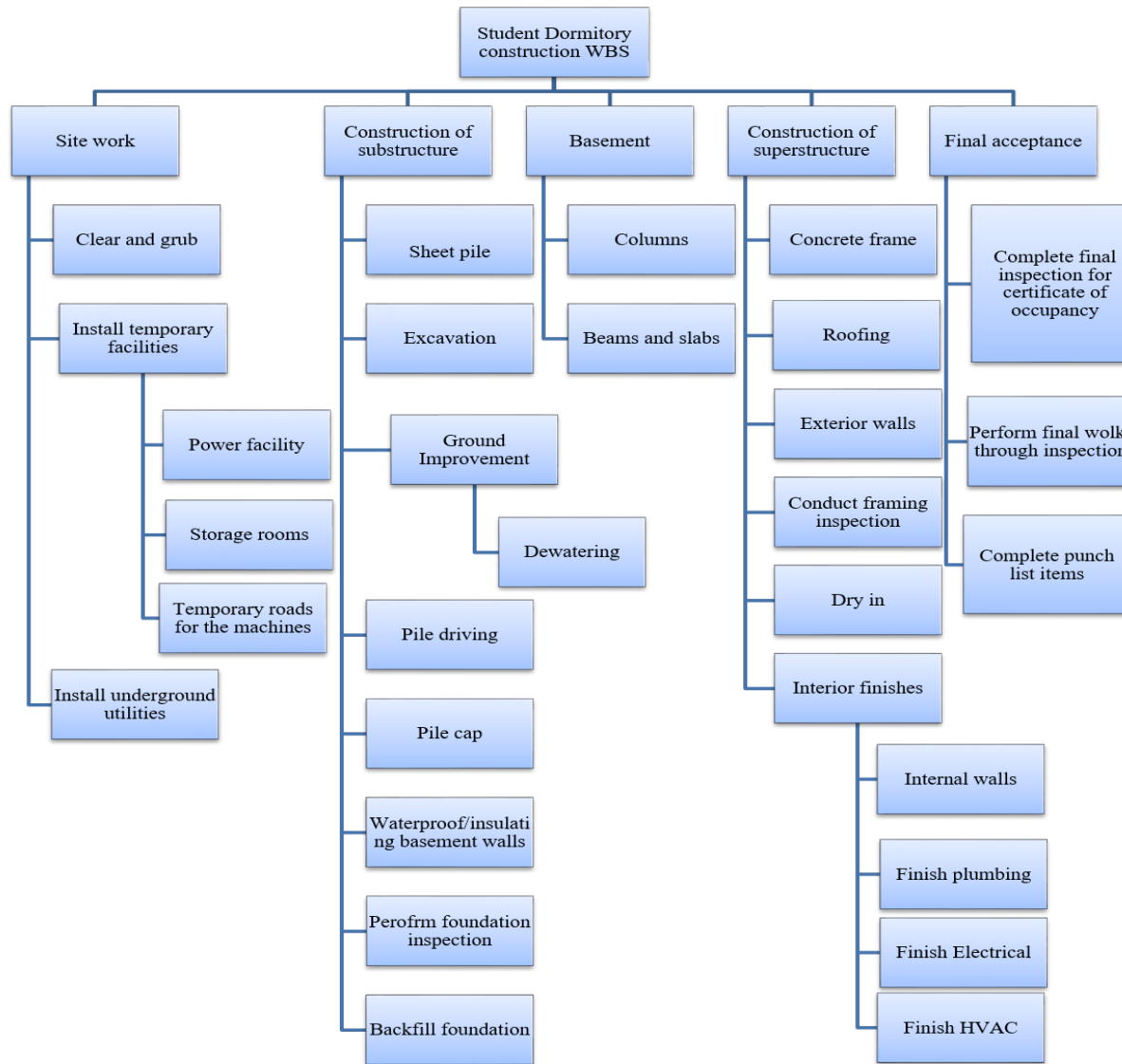
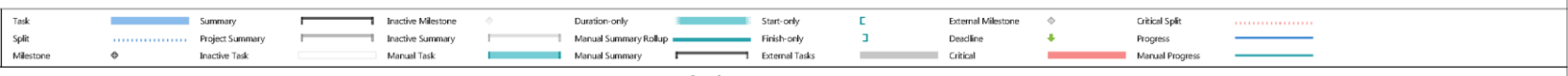
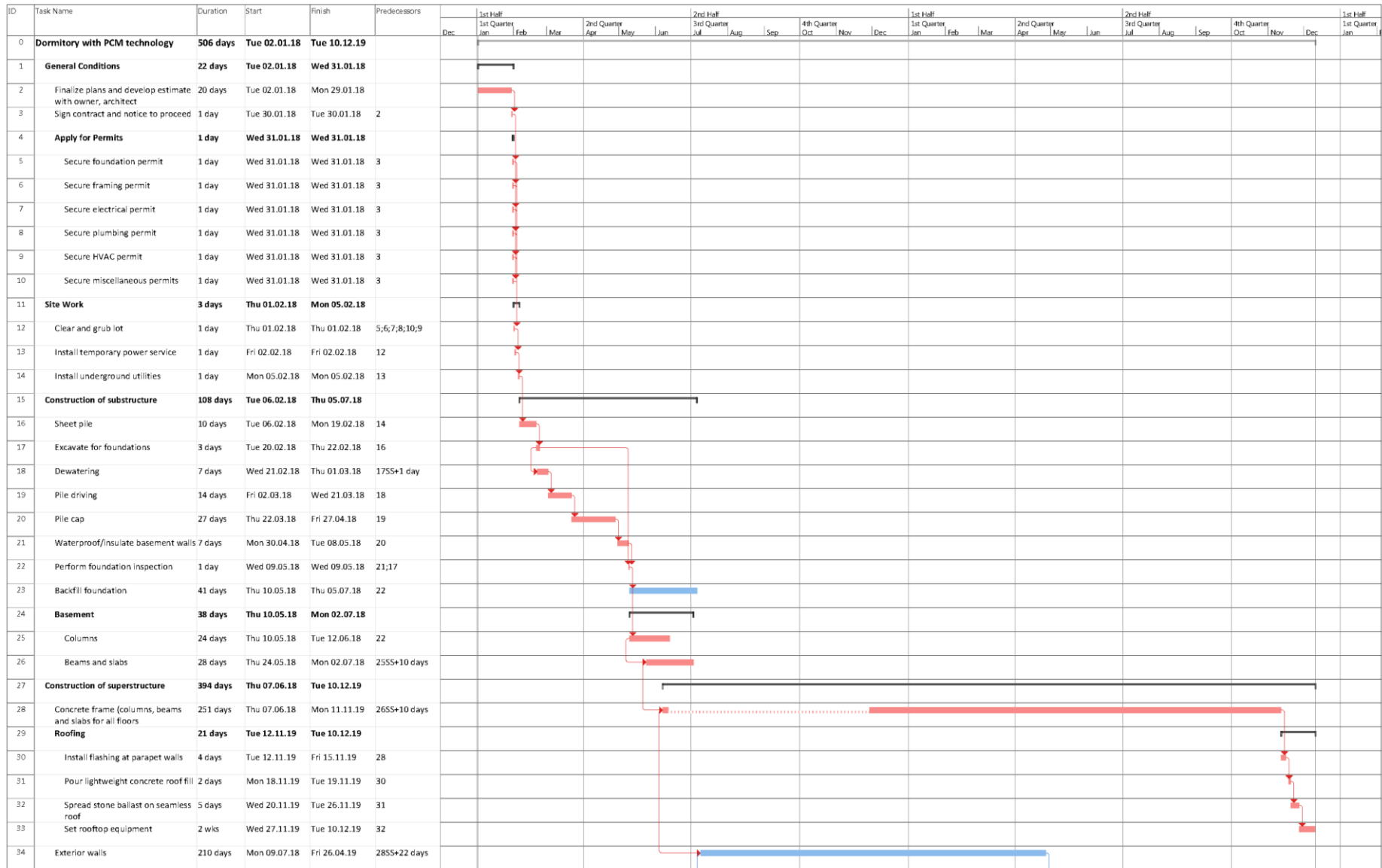


Figure 6.5.1 Work Breakdown Structure for student resident building



6.6 Risk management

Risk is the measure of failure in attaining an end up product of a certain project (Kerzner, 2013). In order to prevent the occurrence of any failure in the project, risk management must be conducted, where all possible risks are identified and the risk mitigation activities are undertaken.

6.6.1 Risk Identification

It is important for the risk management to identify the risk types. If all the possible risks are identified, then controlling and mitigating activities are undertaken, which means there is a high probability to neutralize all possible risks.

In any project there are different types of risks that might occur. An effective representation method of the all available risks is by creating Risk Breakdown Structure (RBS). The following Figure 6.6.1. shows the RBS for the Student Dormitory Construction project.

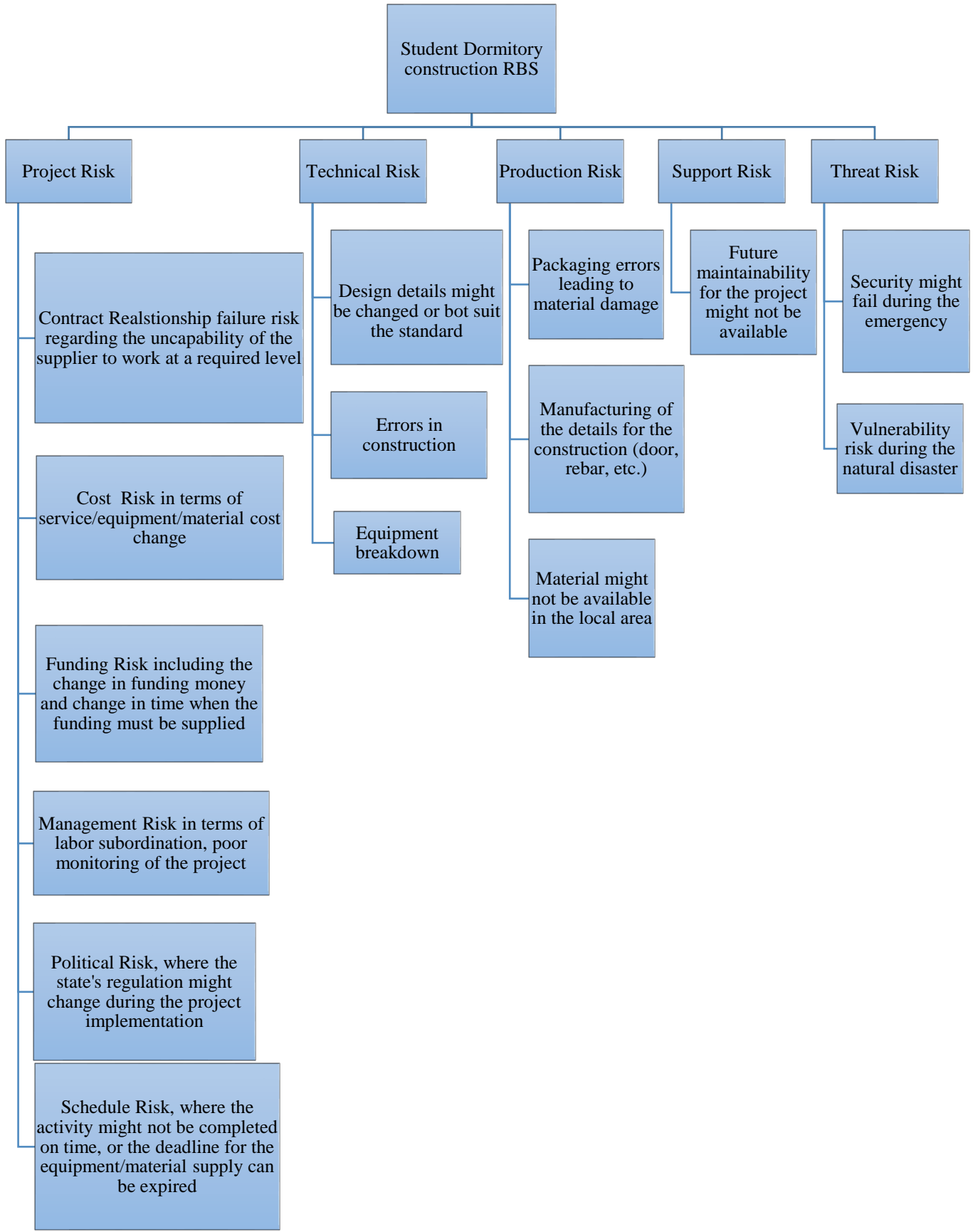


Figure 6.6.1 Risk Breakdown Structure of the Student Dormitory construction

6.6.2 Risk assessment

After the risk identification, there must be proposed a possible mitigation plan. Also, the risk always must be controlled. In order to control it, the risks should be classified in terms of impact and likelihood. Risk severity matrix is a method of classifying the risks regarding their impact to the environment and occurrence likelihood. Figure 6.6.1 shows that Risk severity matrix for the Student Dormitory construction.

Table 6.6.1 Risk severity matrix (Kerzner, 2013)

Likelihood	5	-	-	Cost Risk	Managem ent Risk	Errors in constructi on
	4	Equipment breakdown	-	Security Risk	Design change	Shedule Risk
	3		Packaging Risk	Manufacturin g Risk	Political Risk	Contract relationshi p Risk
	2	-	-	Maintainabilit y Risk	Packaging Risk	Vulnerabil ity Risk
	1	-	-	-	Material Risk	Funding Risk
		1	2	3	4	5
Impact						

Impact: 1 – negligible; 2 – minor, 4 – major, 5 – severe.

Likelihood: 1- rare; 2 – unlikely; 3 – possible; 5 – almost certain.

Table 6.6.1, shows that the errors in construction has the highest likelihood and has a greatest impact on the project. It means that this risk must be seriously taken to control and mitigating measurements must be planned to decrease the consequence. Also, the risk value can be calculated by the following formula:

$$\text{Risk Value} = \text{Impact} * \text{Probability}$$

The results for the Risk values are available in Table 6.6.2. According to this table, it can seen that the most critical risk is construction error with the highest risk value.

Table 6.6.2 Risk Values

Risk category	Risk value
Construction error	25
Schedule Risk	20
Management Risk	20
Design change Risk	16
Cost Risk	15
Security Risk	12
Political Risk	12
Vulnerability Risk	10
Manufacturing Risk	9
Packaging Risk	6
Maintainability Risk	6
Funding Risk	5
Equipment breakdown Risk	4
Material Risk	4

6.6.3 Risk control

In order to prevent the failure in the system, identified and assessed risks must be controlled and mitigated depending on the risk values. Table 6.6.3 shows how the identified risks must be mitigated.

Table 6.6.3 Risk mitigation plan

Risk category	Risk value	Mitigating method
Construction error	25	<ul style="list-style-type: none"> - Well qualified labors must be hired. - All the workers must be warn up about safety instructions.

Schedule Risk	20	<ul style="list-style-type: none"> - Good project manager must be hired. - Schedule of the completed/uncompleted work volume must be monitored and controlled. - The materials should be delivered on time. - Equipments must be used strongly by the schedule so that the overlapping will not happen.
Management Risk	20	<ul style="list-style-type: none"> - Highly qualified managers must be hired. - Good time management must be a requirement. - Good job atmosphere should present. - Subordination should present between the labors of different position.
Design change Risk	16	<ul style="list-style-type: none"> - Detailed, clear and reasonable design specification should be created. - Any error must be eliminated.
Cost Risk	15	<ul style="list-style-type: none"> - The labors should be noted about the working rate. - Equipments must be used strongly by the schedule so that the overlapping will not happen. - Any delay should be detected and accelerated. - Good project management is required.
Security Risk	12	<ul style="list-style-type: none"> - Good security stuff must be hired. - The stuff should be well-trained. - Security apparatus of good quality should be used.
Political Risk	12	<ul style="list-style-type: none"> - Any possible changes in the political risk should be anticipated. - Alternative method of solving the problem should present. - Clients and suppliers should be aware of political regulations of the construction place.
Vulnerability Risk	10	<ul style="list-style-type: none"> - Materials with high durability and strength should be used. - The structure should be designed according to natural phenomenon of

		the local place.
Manufacturing Risk	9	- Reliable manufacturer should be selected.
Packaging Risk	6	- Reliable supplier should be selected. - Be aware of the material/equipment condition not leaving the supplier's storage.
Maintainability Risk	6	- Further maintaining and operational plan and schedule should be observed.
Funding Risk	5	- Must have good contractor for the due date of the funding process.
Equipment breakdown Risk	4	- Labors should be informed about the equipment instructions. - Well-trained and highly qualified labors should be hired. - Control the maintenance of the equipment during a certain period of time.
Material Risk	4	- Materials that suit the standard and the design specifications should be selected. - It is recommended to select the materials available in the local place and in reasonable cost.

7 CONCLUSION AND RECOMMENDATIONS

In this report, “EASY Engineering” LLP proposed a 10-storey residential building project designated for students in Astana. Project design was accomplished following local construction standards and Eurocode. Notable feature of the design is integration of PCM into the building wall to enhance the energy performance of the building due to use of PCM.

In architectural design part, non-structural materials including ceiling, floor, and partitions were selected and provided with specifications. Design of the site layout with parking area, traffic flow and landscaping was drawn in AutoCAD software. Detailed sketch of building façade and floor plans for 1st floor and typical floors were provided in the technical drawings. Dimensional characteristics as width of corridors, stairs and thickness of the walls were designed taking into account requirements of the local construction standards and Eurocodes.

Various load actions as dead, live, wind and snow loads were calculated manually in structural design part. They were further used as an input for analysis of building frame in two-dimensional SAP 2000 software. Slab, column and beam details were accomplished both by hand calculation and software simulation. Results were found close to each other. Thus, comparison between manual and software design reveals a reasonable design and calculation of structural members. For structural analysis, critical cases under various load combinations were considered. Reinforced concrete frame was also analyzed with the aid of ETABS and SAFE software.

Geotechnical technical design for pile foundation included estimation of pile bearing capacity, settlement, and sheet pile design. After calculation of bearing capacity of the single pile based on static equations and due to heavy imposed load by superstructure, necessity in pile grouping was identified. Also, pile cap design for four, five, six and eight piles were accomplished to fix pile heads to withstand lateral loads. Considering applied loads by structure and bearing capacity of an individual pile, pile grouping was designed for four major load cases on external, internal and corner columns. Maximum settlement was in case of eight pile group with value of 43.5 mm. Sheet pile design for problematic ground condition was also provided for short-term and long-term period. Long-term case is of interest for the design of resident building. Both settlement and sheet pile were analyzed in Plaxis 2D software.

The proposed pile group design satisfies lateral load and uplift force check. Construction process for foundation was also discussed and new long-term monitoring system was considered as an efficient method to determine structural damage in the pile after pile installation. During pile driving, great control should be placed on dynamic pile monitoring to check actual bearing capacity of the pile and verify the suitability of the driving machine. Detailed pile foundation design with dimensions can be found in technical drawings.

Evaluating the different methods of the PCM integration to the building, it was decided to apply it to the wall by the layer method. The main reasons are simplicity and cost-effectiveness. Also, the PCM was located to the 1/5L of the insulation wall, because of energy effectiveness with reference to research investigations. Additionally, the PCM with phase changing temperature of 23 degrees was selected in order to provide comfort room temperature for the human. One section of the typical floor on the west side of the dormitory was selected in order to estimate an energy saving potential. Building modeling program Design Builder which estimates the energy efficiency by the Energy Plus software was used. The section temperature change and electricity consumption for the cooling during the summer was estimated. It was found that 34% of energy was saved when the wall was integrated with PCM.

In general, project goals are successfully reached by reasonable design of architectural, structural and geotechnical parts. Energy performance simulation and construction management are also satisfied the project scope. Nevertheless, some recommendations can improve the design of student building. Particularly, geotechnical design should be simulated by advanced software to obtain more accurate results. In addition, on site pile testing and more comprehensive study on frost heave in Astana are required. Regarding energy performance of the building, it is obvious that the function of the PCM is to keep the room temperature in a comfort range. However, when estimating the effectiveness of this material, mean zone temperature is not the only thing that must be monitored, but accurate hourly temperature change must also be evaluated. The reason is that by the daily mean zone temperature the effect of PCM might not be seen properly, because although the PCM keeps the temperature in stability during the night and day times, daily mean temperature might not change. Thus, to obtain an accurate and precise results, hourly and sub-hourly data should be extracted, and the simulation time step must be as high as possible (4 or more).

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Appendix A

Table 1. Design situations by Eurocode 0

Design Situations
<u>Persistent</u> , which refer to the conditions of normal use
<u>Transient</u> , which refer to temporary conditions applicable to the structure
<u>Accidental</u> , which refer to exceptional conditions applicable to the structure or to its exposure

Table 2. Actions classified by Eurocode 0;1

Classification of Actions:		
Permanent actions (G)	By their origin	
	<i>direct</i>	<i>indirect</i>
Variable actions (Q)	By their spatial	
	<i>fixed</i>	<i>free</i>
Accidental actions (A)	By their structural response	
	<i>static</i>	<i>dynamic</i>

Table 3. The category of use (Eurocode 1-1-1: Table 6.1)

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D ¹⁾)	<p>C1: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions.</p> <p>C2: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms.</p> <p>C3: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts.</p> <p>C4: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages.</p> <p>C5: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.</p>
D	Shopping areas	<p>D1: Areas in general retail shops</p> <p>D2: Areas in department stores</p>

Table 4. Imposed loads on roofs of category H (Eurocode 1-1-1: Table 6.10)

Roof	q_k [kN/m ²]	Q_k [kN]
Category H	q_k	Q_k
<p>NOTE 1 For category H q_k may be selected within the range 0,00 kN/m² to 1,0 kN/m² and Q_k may be selected within the range 0,9 kN to 1,5 kN.</p> <p>Where a range is given the values may be set by the National Annex. The recommended values are:</p> <p style="text-align: center;">$q_k = 0,4 \text{ kN/m}^2, Q_k = 1,0 \text{ kN}$</p> <p>NOTE 2 q_k may be varied by the National Annex dependent upon the roof slope.</p> <p>NOTE 3 q_k may be assumed to act on an area A which may be set by the National Annex. The recommended value for A is 10 m², within the range of zero to the whole area of the roof.</p> <p>NOTE 4 See also 3.3.2 (1)</p>		

Table 5. Design Situations for snow load and load arrangements to be used for different locations (Eurocode 1-1-4: Table A.1)

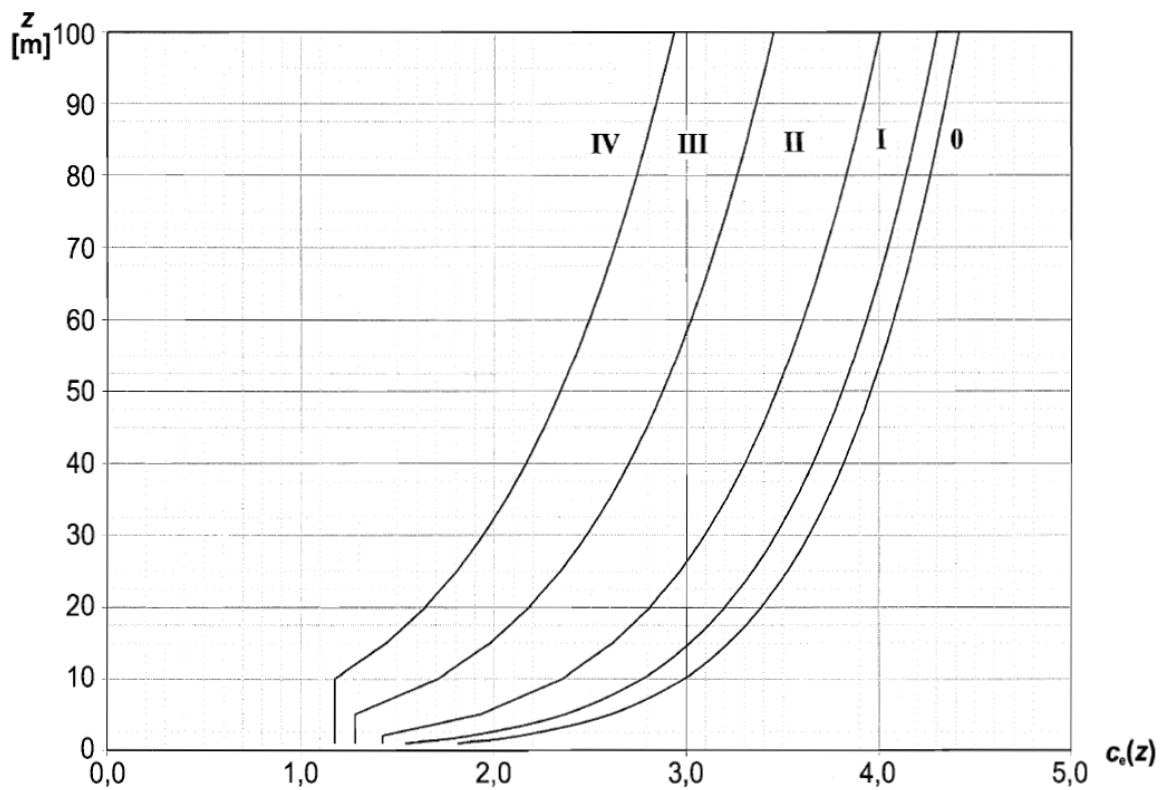
Normal	Exceptional conditions		
Case A	Case B1	Case B2	Case B3
No exceptional falls No exceptional drift	Exceptional falls No exceptional drift	No exceptional falls Exceptional drift	Exceptional falls Exceptional drift
3.2(1)	3.3(1)	3.3(2)	3.3(3)
<i>Persistent/transient design situation</i>	<i>Persistent/transient design situation</i>	<i>Persistent/transient design situation</i>	<i>Persistent/transient design situation</i>
[1] undrifted $\mu_i C_e C_t s_k$	[1] undrifted $\mu_i C_e C_t s_k$	[1] undrifted $\mu_i C_e C_t s_k$	[1] undrifted $\mu_i C_e C_t s_k$
[2] drifted $\mu_i C_e C_t s_k$	[2] drifted $\mu_i C_e C_t s_k$	[2] drifted $\mu_i C_e C_t s_k$ (except for roof shapes in AnnexB)	[2] drifted $\mu_i C_e C_t s_k$ (except for roof shapes in AnnexB)
	<i>Accidental design situation (where snow is the accidental action)</i>	<i>Accidental design situation (where snow is the accidental action)</i>	<i>Accidental design situation (where snow is the accidental action)</i>
	[3] undrifted $\mu_i C_e C_t C_{esl} s_k$	[3] drifted $\mu_i s_k$ (for roof shapes in AnnexB)	[3] undrifted $\mu_i C_e C_t C_{esl} s_k$
	[4] drifted $\mu_i C_e C_t C_{esl} s_k$		[4] drifted $\mu_i s_k$ (for roof shapes in AnnexB)
<p>NOTE 1: Exceptional conditions are defined according to the National Annex.</p> <p>NOTE 2: For cases B1 and B3 the National Annex may define design situations which apply for the particular local effects described in section 6.</p>			

Table 6. Terrain categories and terrain parameters (Eurocode 1-1-4: Table 4.1)

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

NOTE: The terrain categories are illustrated in A.1.

Figure 1. Graph of the exposure factor $c_e(z)$ for $c_0=1.0$ $k_1=1.0$



Appendix B1

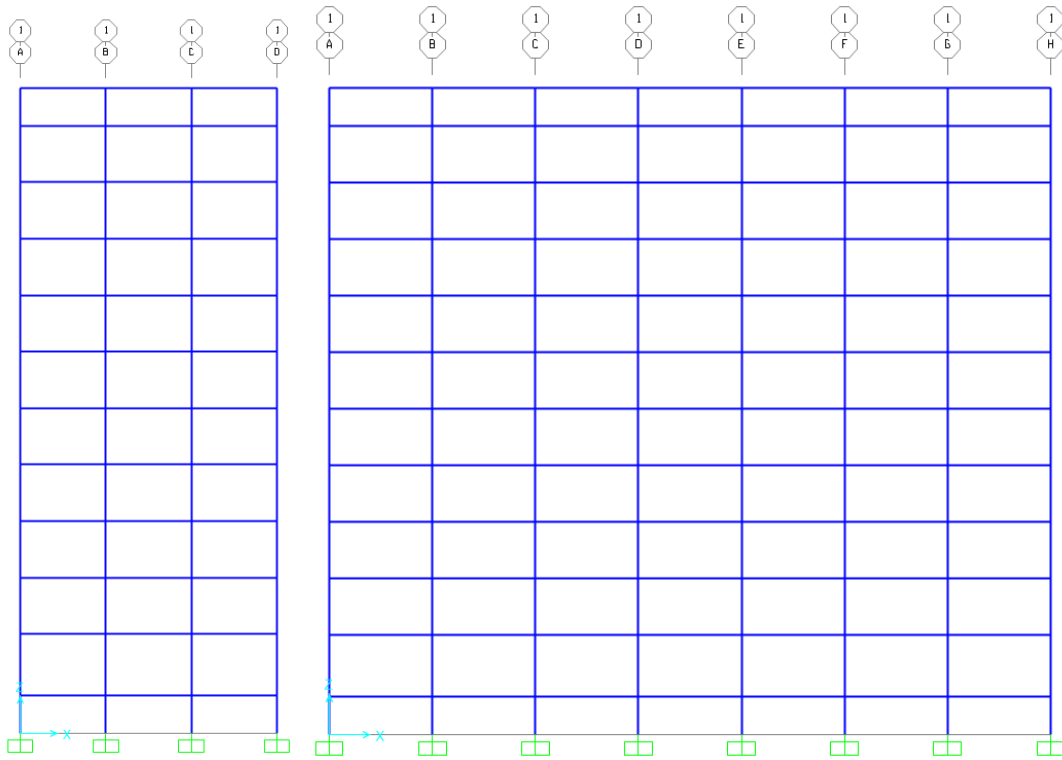


Figure B1.1. Small and large frame

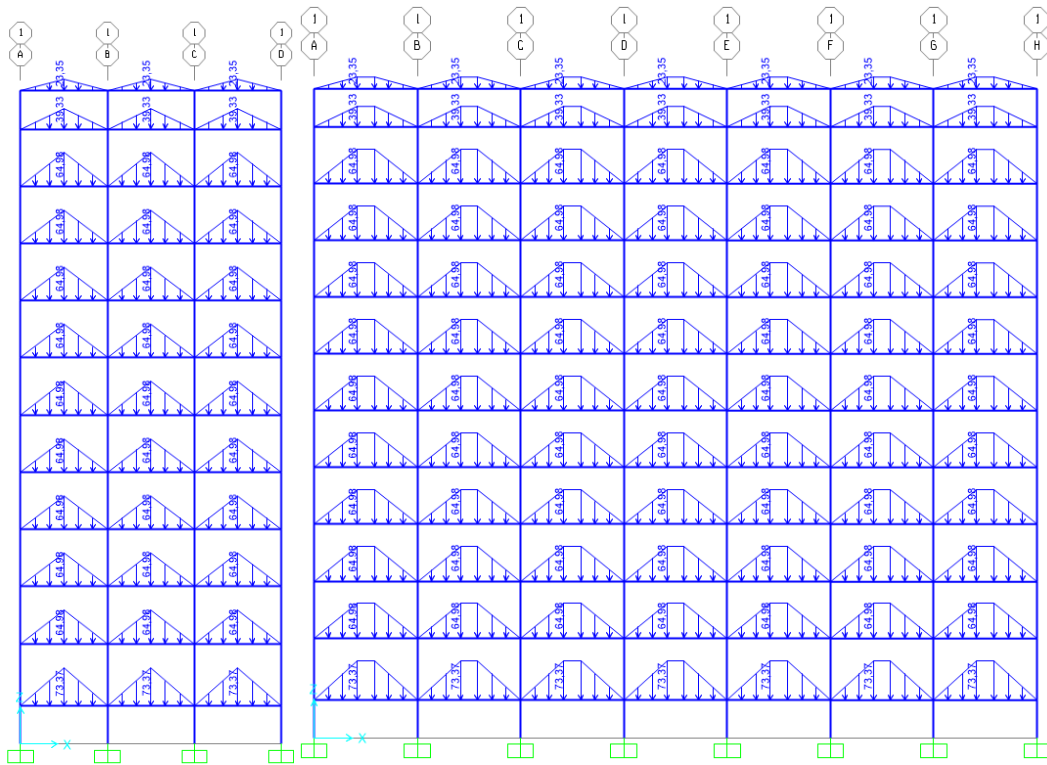


Figure B1.2 Dead load application on frames

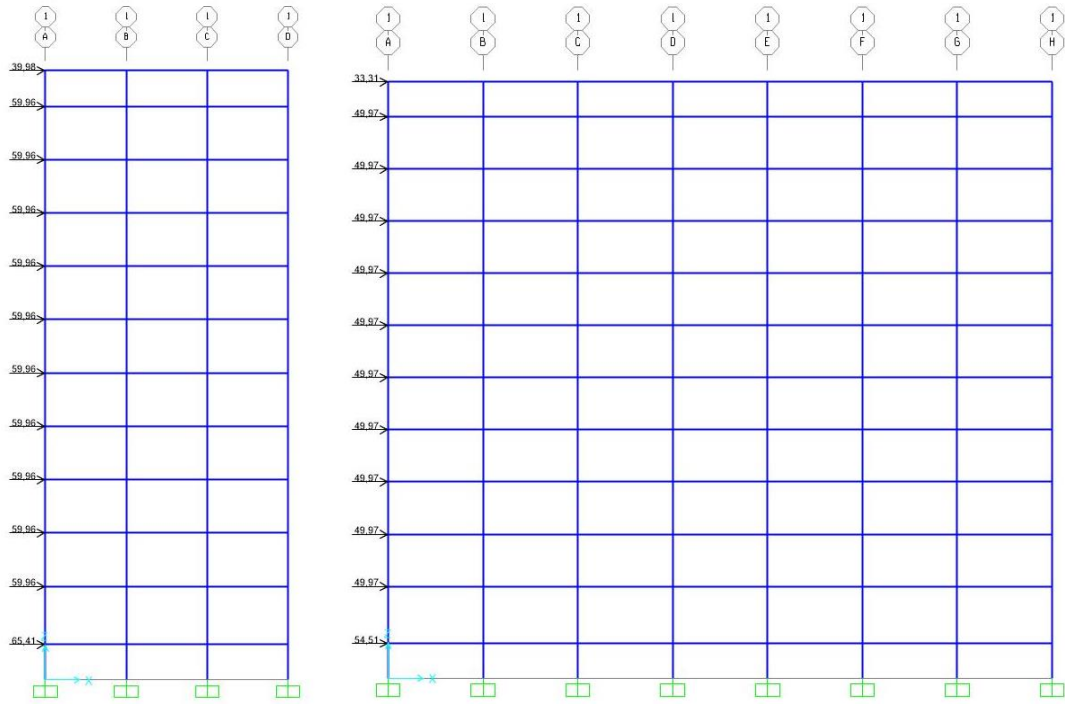
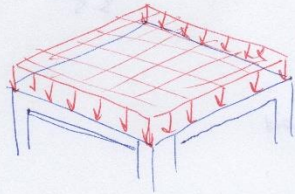


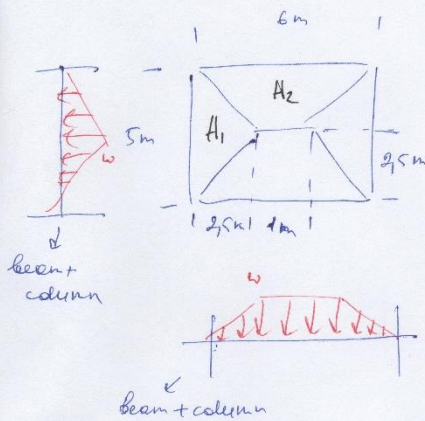
Figure B1.3 Wind load application on frames

Appendix B2.1

Load take-down



uniformly distributed slab load
 $w, \text{ kN/m}^2$
 slab panel



slab panel

Transformation from UDL
 to line load on beams:

$$\textcircled{1} \quad A_1 = 2.5 \cdot 2.5 \cdot \frac{1}{2} \cdot 2 = 6.25 \text{ m}^2$$

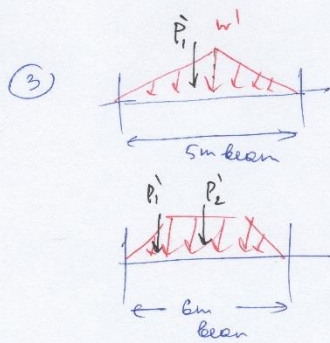
$$A_2 = 2.5 \cdot 2.5 \cdot \frac{1}{2} \cdot 2 + 1 \cdot 2.5 = 6.25 + 2.5 = 8.75 \text{ m}^2$$

$\textcircled{2}$ load at area A_1

$$P_1 = w \cdot 6.25$$

load at area A_2

$$P_2 = w \cdot 8.75 \text{ m}^2$$



$$P_1' = w' \cdot 2.5 \cdot \frac{1}{2}$$

$$\text{total} = 2P_1' = w' \cdot 2.5$$

$$P_1' = w' \cdot 2.5 \cdot \frac{1}{2}$$

$$P_2' = w' \cdot 1$$

$$\text{total} = 2P_1' + P_2' = 2.5w' + w' = 3.5w'$$

Appendix B2.2

shuttle frame \parallel

th on beam \uparrow
 \downarrow

$x = 0, 212$

load s $sw + w$
 3.75
 KN/m

5m

$P_4 = P_3$

$P_1 = 3.75(5-2x)$

$P_2 = w'(5-2x) s$
 $= \frac{24x}{5}(5-2x)$

$P_3 = P_4 = (\text{area } w') \left(\frac{5-2x}{2} \right) \cdot \frac{1}{2} s$
 $= \left(w - \frac{24x}{5} \right) \frac{(5-2x)}{4} s$
 $= w \left(1 - \frac{2x}{5} \right) \frac{(5-2x)}{4}$

$w' \Rightarrow w$

$\frac{w'}{w} = \frac{x}{2.5}$ $\frac{w}{w'} = \frac{2.5}{x}$

$w' = \frac{wx}{2.5}$ $w = \frac{2.5w'}{x}$

$w' s = \frac{24wx}{5}$

$P_1 @ (5-2x)/2$ $P_3 @ \frac{5-2x}{2} + \frac{1}{3} \frac{(5-2x)}{2} = \frac{5-2x}{2} \left(1 + \frac{1}{3} \right) = \frac{4}{3} \frac{(5-2x)}{2} s$

$P_2 @ (5-2x)/2$ $P_4 @ \frac{2}{3} \cdot \frac{(5-2x)}{2} = \frac{(5-2x)}{3}$
 $= \frac{(5-2x)}{3}$

$$\sum M_{\odot A'} = 0$$

$$V_{A'} \cdot (5-2x) = P_1 \frac{(5-2x)}{2} + P_2 \frac{(5-2x)}{2} + P_4 \frac{(5-2x)}{3} + P_3 \left(\frac{2}{3}(5-2x) \right)$$

$$V_{A'} = \frac{P_1 + P_2}{2} + \frac{1}{3}P_4 + \frac{2}{3}P_3 \quad | \quad P_4 = P_3$$

$$V_{A'} = \frac{P_1 + P_2}{2} + \frac{1}{3}P_3 + \frac{2}{3}P_3$$

$$V_{A'} = \frac{P_1 + P_2}{2} + P_3$$

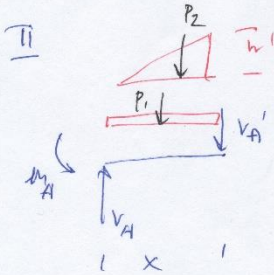
$$V_{B'} = \frac{3.75(5-2x) + \frac{2wx}{5}(5-2x)}{2} + w \left(\frac{1-2x}{5} \right) \frac{(5-2x)}{4}$$

$$= \frac{(5-2x)}{2} \left(3.75 + \frac{2wx}{5} + \frac{w}{2} \left(\frac{1-2x}{5} \right) \right)$$

$$\sum F = 0$$

$$V_{H'} = \frac{5-2x}{2} \left(3.75 + \frac{2wx}{5} + \frac{w}{2} - \frac{2wx}{10} \right) = \frac{5-2x}{2} \left(3.75 + \frac{w}{2} + \frac{wx}{5} \right)$$

$$\begin{aligned} 2 \cdot \frac{2wx - 2wx}{10} &= \frac{2wx}{10} = \frac{wx}{5} \end{aligned}$$



$$P_1 = 3.75x \quad \odot \quad x/2$$

$$P_2 = \frac{2wx}{5} \cdot x \cdot \frac{1}{2} = \frac{wx^2}{5} \quad \odot \quad \frac{2}{3}x$$

$$\sum M_{\odot A} = 0$$

$$M_A = V_{A'}x + P_1 \frac{x}{2} + P_2 \cdot \frac{2}{3}x$$

$$M_A = \frac{x(5-2x)}{2} \left(3.75 + \frac{w}{2} + \frac{wx}{5} \right) + \frac{3.75x^2}{2} + \frac{wx^2}{5} \cdot \frac{2}{3}x$$

$$\sum F = 0$$

$$V_A = V_A' + P_1 + P_2$$

$$V_A = \frac{5-x}{2} \left(3.75 + \frac{w}{2} + \frac{wx}{5} \right) + 3.75x + \frac{wx^2}{5}$$

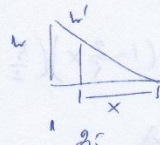
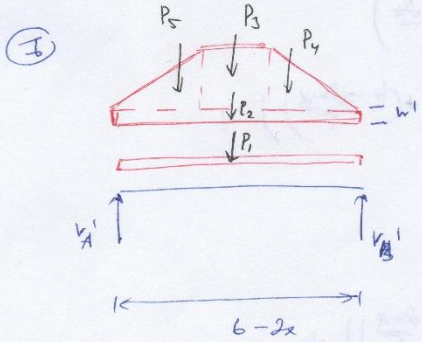
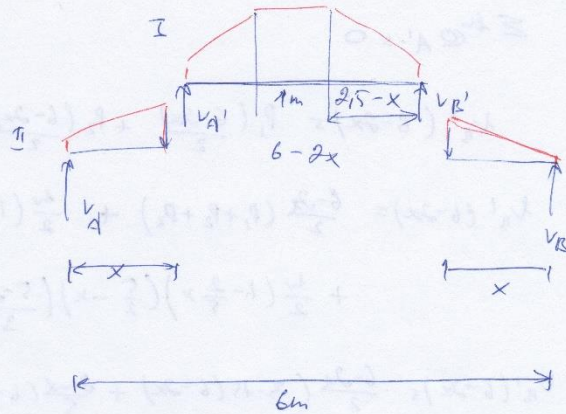
large frame

4

4
+ v beams

$x = \text{optimal (or other)}$

load = $8w + w$
 3.25



$$\frac{w}{w'} = \frac{2.5}{x}$$

$$w' = \frac{wx}{2.5} = \frac{wx}{5/2} = \frac{2wx}{5}$$

$$w - w' = w - \frac{2}{5}wx$$

$$P_1 = 3.75(6-2x) \text{ @ } \frac{6-2x}{2}$$

$$P_2 = w'(6-2x) = \frac{2wx}{5}(6-2x) \text{ @ } \frac{6-2x}{2}$$

$$P_3 = (w-w')(1) = \left(w - \frac{2wx}{5}\right) \cdot 1 \text{ @ } \frac{6-2x}{2}$$

$$P_4 = P_5 = (w-w')(2.5-x) \cdot \frac{1}{2}$$

$$= \left(w - \frac{2}{5}wx\right)(2.5-x) \cdot \frac{1}{2} =$$

$$= \frac{w}{2} \left(1 - \frac{2x}{5}\right)(2.5-x)$$

(from A) $P_5 = \frac{2}{5}(2.5-x)$
 $P_4 = (2.5-x) + 1 + \frac{1}{2}(2.5-x) \text{ or } (6-2x) - \frac{2}{5}(2.5-x)$

$$\equiv 4 \quad \textcircled{A} \leq 0$$

$$h_A = v_A' x + P_1 \frac{x}{2} + P_2 \frac{2}{3} x$$

$$= 11.25x - \underbrace{3.75x^2} + vx^2 + \frac{wx}{2} - \frac{2wx^3}{5} +$$

$$+ \frac{wx(1-\frac{2}{5}x)(\frac{5}{2}-x)(3-x)}{(6-2x)} + \frac{3.75x^2}{2} + \frac{2x3w}{15} =$$

$$= 11.25x - \frac{3.75x^2}{2} + vx^2 + \frac{wx}{2} - \frac{2wx^3}{15} + \frac{4x}{6-2x} (1-\frac{2}{5}x)(\frac{5}{2}-x)(3-x)$$

$$= 11.25x - \frac{3.75x^2}{2} + vx^2 + \frac{wx}{2} - \frac{2wx^3}{15} + \frac{4x}{6-2x} (1-\frac{2}{5}x)(\frac{5}{2}-x)(3-x)$$

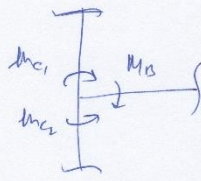
$$h_d = v_d' + P_1 + P_2$$

$$h_d = 11.25 - \cancel{3.75x} + vx + \frac{w}{2} - \frac{2wx^2}{5} + \frac{w(1-\frac{2}{5}x)(\frac{5}{2}-x)(3-x)}{(6-2x)}$$

$$+ \cancel{3.75x} + \frac{wx}{5}$$

$$h_d = 11.25 + vx + \frac{w}{2} - \frac{wx^2}{5} + \frac{w(1-\frac{2}{5}x)(\frac{5}{2}-x)(3-x)}{(6-2x)}$$

Min columns



$$M_B = M_{c1} + M_{c2}$$

$$l_1 = l_2 \quad M_{c1} = M_{c2} = M_B / 2$$

$l_1 \neq l_2$ (stick to typical, typical to ground, ground to basement)

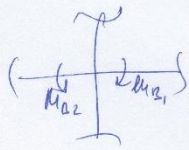
$$k = 4EI/L$$

$$DF = \frac{EI/L}{EI/L + EI/L}$$

$$M_{c1} = \frac{(EI)/l_1}{(EI)/l_1 + (EI)/l_2} \quad M_B =$$

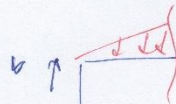
$$= \frac{1/l_1}{1/l_1 + 1/l_2} \quad M_B = \frac{l_2}{l_1 + l_2} M_B$$

$$M_{c2} = \frac{l_1}{l_1 + l_2} M_B$$



$$M_{B1} = M_{B2} \Rightarrow M_{c1} = M_{c2} = 0$$

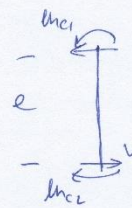
N in columns



st level end $V_{beam} + \text{column self weight}$

next level $(l_B + C_{sw}) + (M_{c1} + C_{sw})$

N columns



$$d = \frac{(M_{c1} + M_{c2})}{e}$$

Appendix B2.3

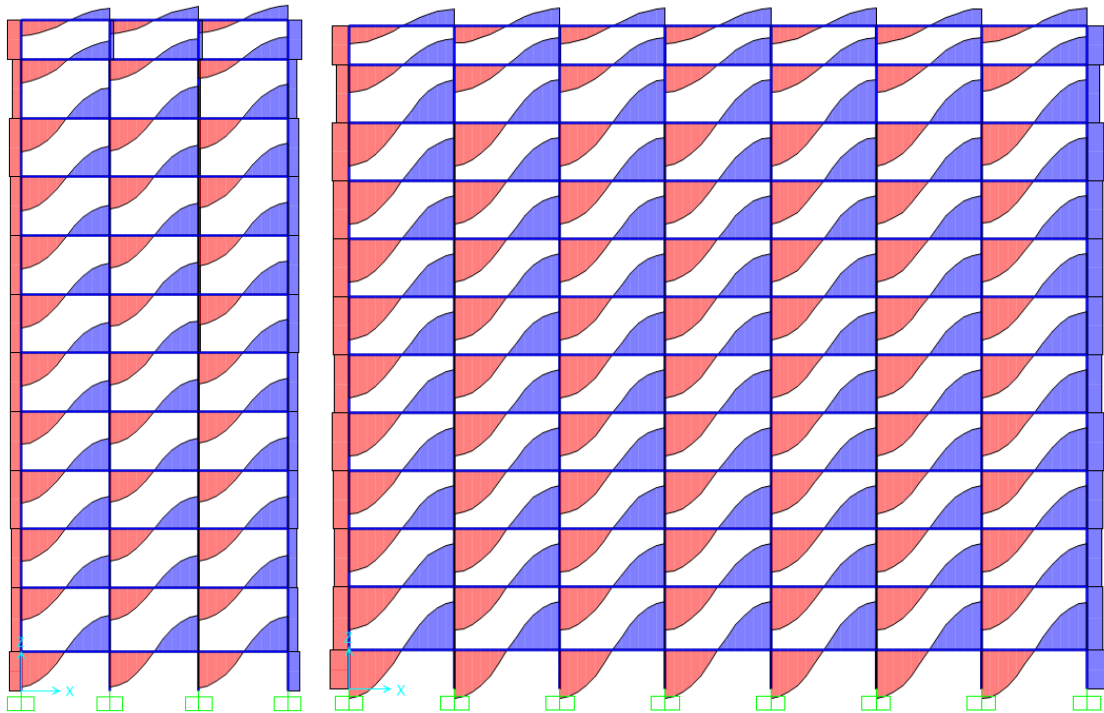


Figure B2.3.1. Shear diagram of dead load for frames

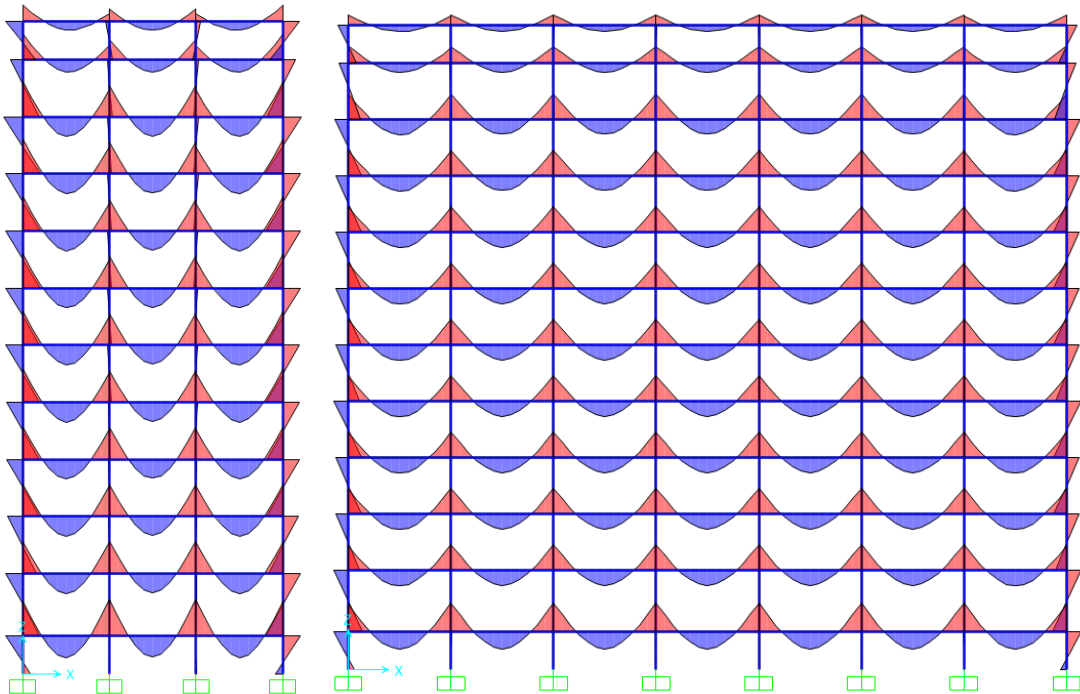


Figure B2.3.2. Moment diagram of dead load for frames

Table B2.3.1. Beam moments under dead load compared

level (slab)	w, kN/m	end moment, kNm	center, kNm	sap center moment	sap end moment	end moment error	center moment error
Small frame							
roof	23.352	37.483	21.0890	23.0927	37.2726	-0.56%	8.68%
attic	39.331	57.060	33.8975	35.0718	58.583	2.60%	3.35%
typical	64.979	87.869	57.4694	54.9090	92.3471	4.85%	-4.66%
ground	73.365	98.357	55.7590	61.1597	103.3993	4.88%	8.83%
Large frames							
roof	23.352	61.946	35.6885	35.7436	61.885	-0.10%	0.15%
attic	39.331	96.407	56.4878	56.1734	96.7163	0.32%	-0.56%
typical	64.979	151.721	89.8729	88.9799	152.6194	0.59%	-1.00%
ground	73.365	169.807	100.7886	99.7576	170.833	0.60%	-1.03%

Table B2.3.2. Beam shear under dead load compared

level (slab)	w, kN/m	end shear, kN	sap end shear	error
Small frame				
roof	23.352	38.565	38.562	-0.01%
attic	39.331	58.539	58.536	0.00%
typical	64.979	90.599	90.596	0.00%
ground	73.365	101.081	101.078	0.00%
Large frame				
roof	23.352	52.116	52.113	-0.01%
attic	39.331	80.079	80.076	0.00%
typical	64.979	124.963	124.96	0.00%
ground	73.365	139.639	139.635	0.00%

Appendix B2.4

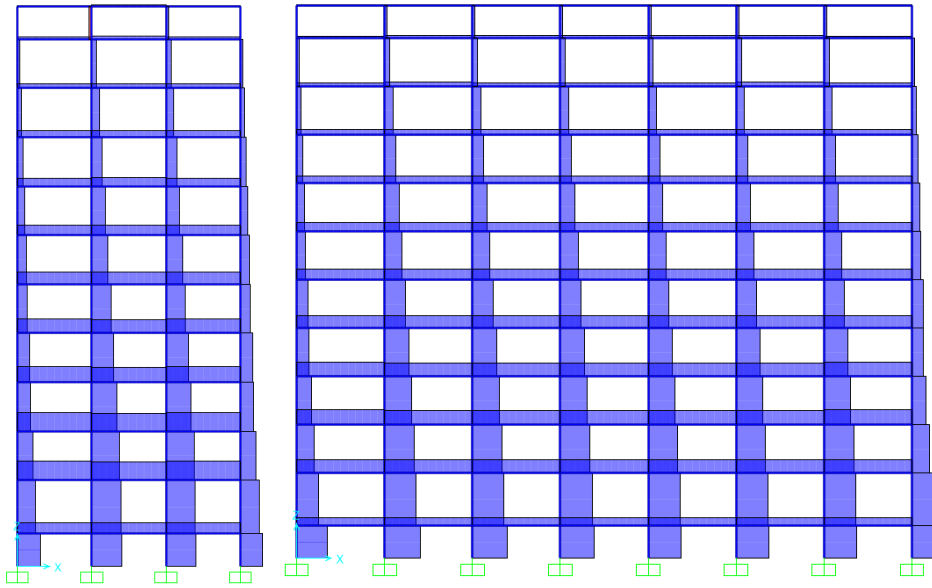


Figure B.2.4.1. Shear diagram of wind load for frames

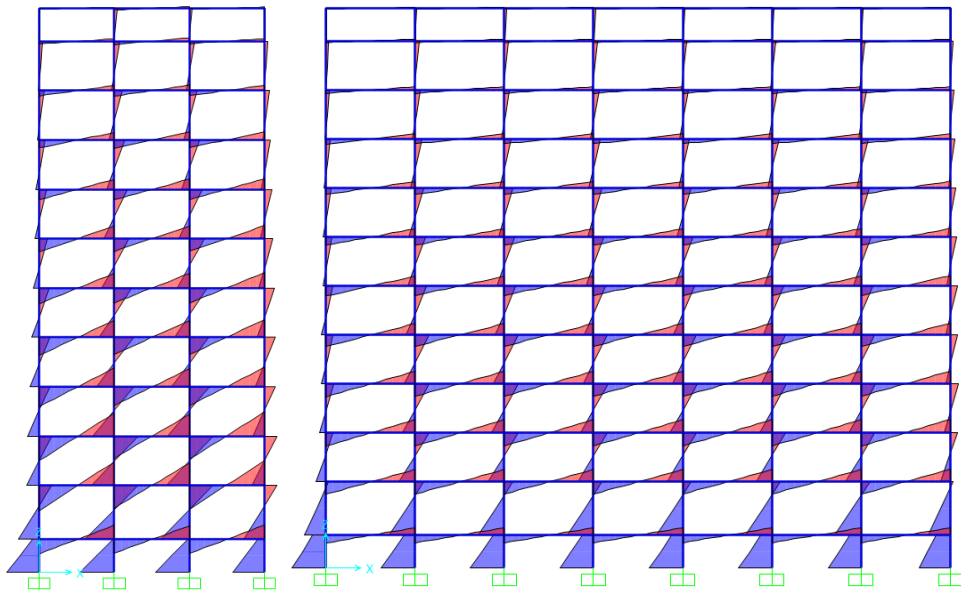


Figure B.2.4.2. Moment diagram of wind load for frames

Appendix B3.1

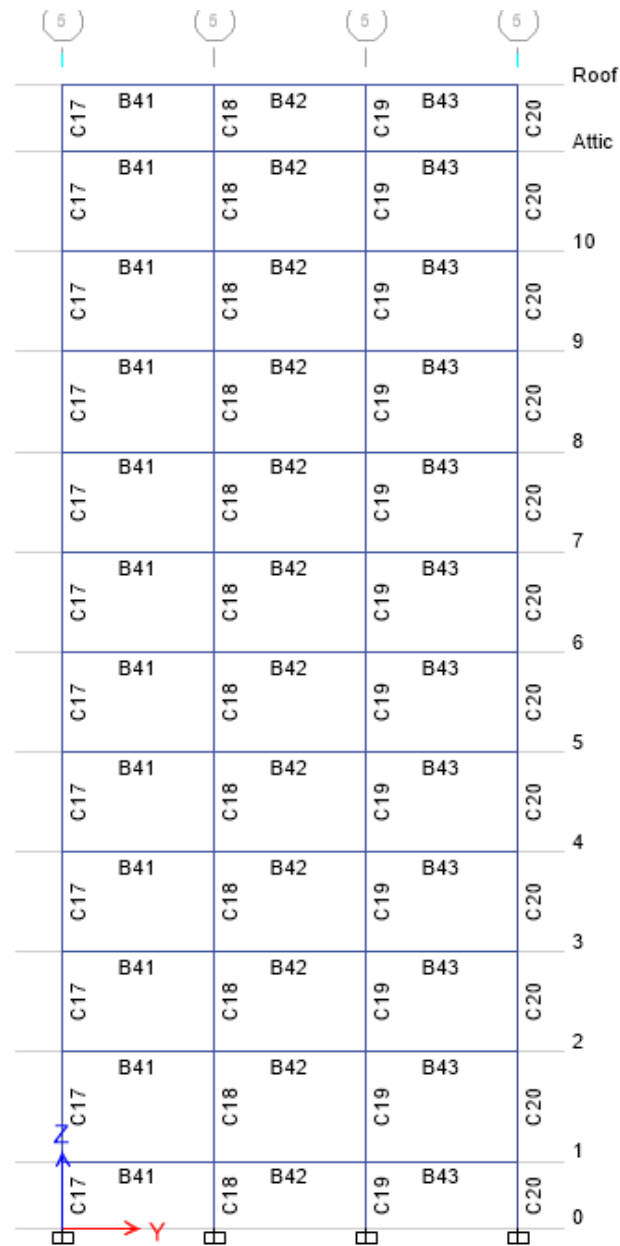


Figure B3.1.1. Frame ID for small frame

Table B3.1.1- Concrete Beam Summary - Eurocode 2-2004 (Part 1 of 2)											
Story	Label	Unique Name	Station mm	Design Section	Status	As Top Combo	As,min Top mm ²	As Top mm ²	As Bottom Combo	As,min Bottom mm ²	As Bottom mm ²
Roof	B41	425	0	Beam	No	COMB11	562	572	COMB13	562	562
Roof	B41	425	500	Beam	No	COMB13	562	562	COMB13	562	562
Roof	B41	425	1000	Beam	No	COMB13	562	562	COMB13	562	562
Roof	B41	425	1500	Beam	No	COMB13	562	562	COMB13	562	562
Roof	B41	425	2000	Beam	No	COMB13	562	562	COMB13	562	562
Roof	B41	425	2500	Beam	No	COMB13	562	562	COMB13	562	562

				500x300	Message						
Roof	B41	425	3000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B41	425	3500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B41	425	4000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B41	425	4500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B41	425	5000	Beam	No	COMB2	562	562	COMB13	562	562
				500x300	Message						
Roof	B42	426	0	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B42	426	500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B42	426	1000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B42	426	1500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B42	426	2000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B42	426	2500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B42	426	3000	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B42	426	3500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B42	426	4000	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B42	426	4500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	0	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	1000	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	1500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	2000	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	2500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	3000	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	3500	Beam	No	COMB13	562	562	COMB13	562	562
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Roof	B43	427	4000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B43	427	4500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Roof	B43	427	5000	Beam	No	COMB6	562	572	COMB13	562	562
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Attic	B41	477	1000	Beam	No	COMB13	562	562	COMB13	562	562
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Attic	B41	477	2000	Beam	No	COMB13	562	562	COMB13	562	562
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				500x300	Message						
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				500x300	Message						
Attic	B41	477	4000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
Attic	B41	477	4500	Beam	No	COMB13	562	562	COMB13	562	562

B1.1

				500x300	Message						
Attic	B41	477	5000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
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				500x300	Message						
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				500x300	Message						
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				500x300	Message						
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B1.1

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9	B43	583	1000	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
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8	B41	633	2500	Beam	No	COMB13	562	562	COMB13	562	562
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8	B41	633	3000	Beam	No	COMB13	562	562	COMB11	562	590
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8	B41	633	3500	Beam	No	COMB13	562	562	COMB11	562	662
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8	B41	633	4000	Beam	No	COMB13	562	562	COMB11	562	650
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8	B41	633	4500	Beam	No	COMB7	562	951	COMB11	562	578
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8	B41	633	5000	Beam	No	COMB7	562	1498	COMB7	562	721
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8	B42	634	0	Beam	No	COMB12	562	1784	COMB12	562	851
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8	B42	634	500	Beam	No	COMB12	562	1176	COMB13	562	562
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8	B42	634	1000	Beam	No	COMB12	562	637	COMB13	562	562
				500x300	Message						
8	B42	634	1500	Beam	No	COMB13	562	562	COMB6	562	603
				500x300	Message						
8	B42	634	2000	Beam	No	COMB13	562	562	COMB6	562	574
				500x300	Message						
8	B42	634	2500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
8	B42	634	3000	Beam	No	COMB13	562	562	COMB11	562	574
				500x300	Message						
8	B42	634	3500	Beam	No	COMB13	562	562	COMB11	562	603
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8	B42	634	4000	Beam	No	COMB7	562	637	COMB13	562	562
				500x300	Message						
8	B42	634	4500	Beam	No	COMB7	562	1176	COMB13	562	562
				500x300	Message						
8	B42	634	5000	Beam	No	COMB7	562	1784	COMB7	562	851

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8	B43	635	0	Beam	No	COMB12	562	1498	COMB12	562	721
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8	B43	635	500	Beam	No	COMB12	562	951	COMB6	562	578
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8	B43	635	1000	Beam	No	COMB13	562	562	COMB6	562	650
				500x300	Message						
8	B43	635	1500	Beam	No	COMB13	562	562	COMB6	562	662
				500x300	Message						
8	B43	635	2000	Beam	No	COMB13	562	562	COMB6	562	590
				500x300	Message						
8	B43	635	2500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
8	B43	635	3000	Beam	No	COMB13	562	562	COMB12	562	583
				500x300	Message						
8	B43	635	3500	Beam	No	COMB13	562	562	COMB12	562	570
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8	B43	635	4000	Beam	No	COMB6	562	800	COMB13	562	562
				500x300	Message						
8	B43	635	4500	Beam	No	COMB6	562	1395	COMB13	562	562
				500x300	Message						
8	B43	635	5000	Beam	No	COMB6	562	2071	COMB6	562	979
				500x300	Message						
7	B41	685	0	Beam	No	COMB11	562	2368	COMB11	562	1110
				500x300	Message						
7	B41	685	500	Beam	No	COMB11	562	1616	COMB13	562	562
				500x300	Message						
7	B41	685	1000	Beam	No	COMB11	562	957	COMB7	562	652
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7	B41	685	1500	Beam	No	COMB13	562	562	COMB7	562	693
				500x300	Message						
7	B41	685	2000	Beam	No	COMB13	562	562	COMB7	562	648
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7	B41	685	2500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
7	B41	685	3000	Beam	No	COMB13	562	562	COMB11	562	629
				500x300	Message						
7	B41	685	3500	Beam	No	COMB13	562	562	COMB11	562	751
				500x300	Message						
7	B41	685	4000	Beam	No	COMB7	562	628	COMB11	562	787
				500x300	Message						
7	B41	685	4500	Beam	No	COMB7	562	1183	COMB11	562	763
				500x300	Message						
7	B41	685	5000	Beam	No	COMB7	562	1810	COMB7	562	863
				500x300	Message						
7	B42	686	0	Beam	No	COMB12	562	2056	COMB12	562	973
				500x300	Message						
7	B42	686	500	Beam	No	COMB12	562	1379	COMB6	562	617
				500x300	Message						
7	B42	686	1000	Beam	No	COMB12	562	781	COMB6	562	686
				500x300	Message						
7	B42	686	1500	Beam	No	COMB13	562	562	COMB6	562	695
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7	B42	686	2000	Beam	No	COMB13	562	562	COMB6	562	620
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7	B42	686	2500	Beam	No	COMB13	562	562	COMB13	562	562
				500x300	Message						
7	B42	686	3000	Beam	No	COMB13	562	562	COMB11	562	620
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7	B42	686	3500	Beam	No	COMB13	562	562	COMB11	562	695
				500x300	Message						
7	B42	686	4000	Beam	No	COMB7	562	781	COMB11	562	686
				500x300	Message						
7	B42	686	4500	Beam	No	COMB7	562	1379	COMB11	562	617
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7	B42	686	5000	Beam	No	COMB7	562	2056	COMB7	562	973
				500x300	Message						
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				500x300	Message						
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				500x300	Message						
7	B43	687	1000	Beam	No	COMB12	562	628	COMB6	562	787
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7	B43	687	2000	Beam	No	COMB13	562	562	COMB6	562	629
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7	B43	687	2500	Beam	No	COMB13	562	562	COMB13	562	562
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7	B43	687	3000	Beam	No	COMB13	562	562	COMB12	562	648
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7	B43	687	3500	Beam	No	COMB13	562	562	COMB12	562	693
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7	B43	687	4000	Beam	No	COMB6	562	957	COMB12	562	652
				500x300	Message						
7	B43	687	4500	Beam	No	COMB6	562	1616	COMB13	562	562
				500x300	Message						
7	B43	687	5000	Beam	No	COMB6	562	2368	COMB6	562	1110
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6	B41	737	0	Beam	No	COMB11	562	2671	COMB11	562	1240
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6	B41	737	500	Beam	No	COMB11	562	1840	COMB7	562	812
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6	B41	737	1000	Beam	No	COMB11	562	1115	COMB7	562	851
				500x300	Message						
6	B41	737	1500	Beam	No	COMB11	562	601	COMB7	562	830
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6	B41	737	2000	Beam	No	COMB11	562	601	COMB7	562	723
				500x300	Message						
6	B41	737	2500	Beam	No	COMB11	562	601	COMB11	562	601
				500x300	Message						
6	B41	737	3000	Beam	No	COMB11	562	601	COMB11	562	666
				500x300	Message						
6	B41	737	3500	Beam	No	COMB11	562	601	COMB11	562	836
				500x300	Message						
6	B41	737	4000	Beam	No	COMB7	562	800	COMB11	562	921
				500x300	Message						
6	B41	737	4500	Beam	No	COMB7	562	1431	COMB11	562	945
				500x300	Message						
6	B41	737	5000	Beam	No	COMB7	562	2148	COMB7	562	1014
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6	B42	738	0	Beam	No	COMB12	562	2330	COMB12	562	1093
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6	B42	738	500	Beam	No	COMB12	562	1581	COMB6	562	798
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6	B42	738	1000	Beam	No	COMB12	562	924	COMB6	562	822
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6	B42	738	1500	Beam	No	COMB13	562	562	COMB6	562	785
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6	B42	738	2000	Beam	No	COMB13	562	562	COMB6	562	663
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6	B42	738	2500	Beam	No	COMB13	562	562	COMB13	562	562
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6	B42	738	3000	Beam	No	COMB13	562	562	COMB11	562	663
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6	B42	738	3500	Beam	No	COMB13	562	562	COMB11	562	785
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6	B42	738	4000	Beam	No	COMB7	562	924	COMB11	562	822
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6	B42	738	4500	Beam	No	COMB7	562	1581	COMB11	562	798
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6	B42	738	5000	Beam	No	COMB7	562	2330	COMB7	562	1093
				500x300	Message						
6	B43	739	0	Beam	No	COMB12	562	2148	COMB12	562	1014
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6	B43	739	500	Beam	No	COMB12	562	1431	COMB6	562	945
				500x300	Message						
6	B43	739	1000	Beam	No	COMB12	562	800	COMB6	562	921
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6	B43	739	1500	Beam	No	COMB6	562	601	COMB6	562	836
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6	B43	739	2000	Beam	No	COMB6	562	601	COMB6	562	666
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6	B43	739	2500	Beam	No	COMB6	562	601	COMB6	562	601
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6	B43	739	3000	Beam	No	COMB6	562	601	COMB12	562	723
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6	B43	739	3500	Beam	No	COMB6	562	601	COMB12	562	830

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6	B43	739	4000	Beam	No	COMB6	562	1115	COMB12	562	851
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6	B43	739	4500	Beam	No	COMB6	562	1840	COMB12	562	812
				500x300	Message						
6	B43	739	5000	Beam	No	COMB6	562	2671	COMB6	562	1240
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5	B41	789	0	Beam	No	COMB11	562	3002	COMB11	562	1378
				500x300	Message						
5	B41	789	500	Beam	No	COMB11	562	2078	COMB7	562	1027
				500x300	Message						
5	B41	789	1000	Beam	No	COMB11	562	1280	COMB7	562	1009
				500x300	Message						
5	B41	789	1500	Beam	No	COMB11	562	665	COMB7	562	931
				500x300	Message						
5	B41	789	2000	Beam	No	COMB11	562	665	COMB7	562	766
				500x300	Message						
5	B41	789	2500	Beam	No	COMB11	562	665	COMB11	562	665
				500x300	Message						
5	B41	789	3000	Beam	No	COMB11	562	665	COMB11	562	715
				500x300	Message						
5	B41	789	3500	Beam	No	COMB11	562	665	COMB11	562	938
				500x300	Message						
5	B41	789	4000	Beam	No	COMB7	562	984	COMB11	562	1077
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5	B41	789	4500	Beam	No	COMB7	562	1690	COMB11	562	1156
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5	B41	789	5000	Beam	No	COMB7	562	2498	COMB11	562	1200
				500x300	Message						
5	B42	790	0	Beam	No	COMB12	562	2669	COMB12	562	1239
				500x300	Message						
5	B42	790	500	Beam	No	COMB12	562	1829	COMB6	562	1017
				500x300	Message						
5	B42	790	1000	Beam	No	COMB12	562	1097	COMB6	562	985
				500x300	Message						
5	B42	790	1500	Beam	No	COMB7	562	600	COMB6	562	892
				500x300	Message						
5	B42	790	2000	Beam	No	COMB7	562	600	COMB6	562	713
				500x300	Message						
5	B42	790	2500	Beam	No	COMB7	562	600	COMB7	562	600
				500x300	Message						
5	B42	790	3000	Beam	No	COMB7	562	600	COMB11	562	713
				500x300	Message						
5	B42	790	3500	Beam	No	COMB7	562	600	COMB11	562	892
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5	B42	790	4000	Beam	No	COMB7	562	1097	COMB11	562	985
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5	B42	790	4500	Beam	No	COMB7	562	1829	COMB11	562	1017
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5	B43	791	0	Beam	No	COMB12	562	2498	COMB6	562	1200
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5	B43	791	500	Beam	No	COMB12	562	1690	COMB6	562	1156
				500x300	Message						
5	B43	791	1000	Beam	No	COMB12	562	984	COMB6	562	1077
				500x300	Message						
5	B43	791	1500	Beam	No	COMB6	562	665	COMB6	562	938
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5	B43	791	2000	Beam	No	COMB6	562	665	COMB6	562	715
				500x300	Message						
5	B43	791	2500	Beam	No	COMB6	562	665	COMB6	562	665
				500x300	Message						
5	B43	791	3000	Beam	No	COMB6	562	665	COMB12	562	766
				500x300	Message						
5	B43	791	3500	Beam	No	COMB6	562	665	COMB12	562	931
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				500x300	Message						
5	B43	791	4500	Beam	No	COMB6	562	2078	COMB12	562	1027
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5	B43	791	5000	Beam	No	COMB6	562	3002	COMB6	562	1378
				500x300	Message						
4	B41	841	0	Beam	No	COMB11	562	3197	COMB11	562	1457

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4	B41	841	500	Beam	No	COMB11	562	2215	COMB7	562	1167
				500x300	Message						
4	B41	841	1000	Beam	No	COMB11	562	1372	COMB7	562	1110
				500x300	Message						
4	B41	841	1500	Beam	No	COMB11	562	702	COMB7	562	992
				500x300	Message						
4	B41	841	2000	Beam	No	COMB11	562	702	COMB7	562	788
				500x300	Message						
4	B41	841	2500	Beam	No	COMB11	562	702	COMB11	562	702
				500x300	Message						
4	B41	841	3000	Beam	No	COMB11	562	702	COMB11	562	750
				500x300	Message						
4	B41	841	3500	Beam	No	COMB11	562	702	COMB11	562	1006
				500x300	Message						
4	B41	841	4000	Beam	No	COMB7	562	1116	COMB11	562	1178
				500x300	Message						
4	B41	841	4500	Beam	No	COMB7	562	1875	COMB11	562	1289
				500x300	Message						
4	B41	841	5000	Beam	No	COMB7	562	2749	COMB11	562	1367
				500x300	Message						
4	B42	842	0	Beam	No	COMB12	562	2910	COMB12	562	1340
				500x300	Message						
4	B42	842	500	Beam	No	COMB12	562	2003	COMB6	562	1169
				500x300	Message						
4	B42	842	1000	Beam	No	COMB12	562	1219	COMB6	562	1097
				500x300	Message						
4	B42	842	1500	Beam	No	COMB7	562	647	COMB6	562	965
				500x300	Message						
4	B42	842	2000	Beam	No	COMB7	562	647	COMB6	562	747
				500x300	Message						
4	B42	842	2500	Beam	No	COMB7	562	647	COMB7	562	647
				500x300	Message						
4	B42	842	3000	Beam	No	COMB7	562	647	COMB11	562	747
				500x300	Message						
4	B42	842	3500	Beam	No	COMB7	562	647	COMB11	562	965
				500x300	Message						
4	B42	842	4000	Beam	No	COMB7	562	1219	COMB11	562	1097
				500x300	Message						
4	B42	842	4500	Beam	No	COMB7	562	2003	COMB11	562	1169
				500x300	Message						
4	B42	842	5000	Beam	No	COMB7	562	2910	COMB7	562	1340
				500x300	Message						
4	B43	843	0	Beam	No	COMB12	562	2749	COMB6	562	1367
				500x300	Message						
4	B43	843	500	Beam	No	COMB12	562	1875	COMB6	562	1289
				500x300	Message						
4	B43	843	1000	Beam	No	COMB12	562	1116	COMB6	562	1178
				500x300	Message						
4	B43	843	1500	Beam	No	COMB6	562	702	COMB6	562	1006
				500x300	Message						
4	B43	843	2000	Beam	No	COMB6	562	702	COMB6	562	750
				500x300	Message						
4	B43	843	2500	Beam	No	COMB6	562	702	COMB6	562	702
				500x300	Message						
4	B43	843	3000	Beam	No	COMB6	562	702	COMB12	562	788
				500x300	Message						
4	B43	843	3500	Beam	No	COMB6	562	702	COMB12	562	992
				500x300	Message						
4	B43	843	4000	Beam	No	COMB6	562	1372	COMB12	562	1110
				500x300	Message						
4	B43	843	4500	Beam	No	COMB6	562	2215	COMB12	562	1167
				500x300	Message						
4	B43	843	5000	Beam	No	COMB6	562	3197	COMB6	562	1457
				500x300	Message						
3	B41	893	0	Beam	No	COMB11	623	3358	COMB7	623	1609
				550x300	Message						
3	B41	893	500	Beam	No	COMB11	623	2376	COMB7	623	1487
				550x300	Message						
3	B41	893	1000	Beam	No	COMB11	623	1521	COMB7	623	1334
				550x300	Message						
3	B41	893	1500	Beam	No	COMB11	623	786	COMB7	623	1127
				550x300	Message						
3	B41	893	2000	Beam	No	COMB11	623	743	COMB7	623	845

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3	B41	893	2500	Beam	No	COMB11	623	743	COMB11	623	743
				550x300	Message						
3	B41	893	3000	Beam	No	COMB11	623	743	COMB11	623	743
				550x300	Message						
3	B41	893	3500	Beam	No	COMB11	623	743	COMB11	623	1050
				550x300	Message						
3	B41	893	4000	Beam	No	COMB7	623	1254	COMB11	623	1292
				550x300	Message						
3	B41	893	4500	Beam	No	COMB7	623	2046	COMB11	623	1482
				550x300	Message						
3	B41	893	5000	Beam	No	COMB7	623	2947	COMB11	623	1642
				550x300	Message						
3	B42	894	0	Beam	No	COMB12	623	2988	COMB6	623	1453
				550x300	Message						
3	B42	894	500	Beam	No	COMB12	623	2096	COMB6	623	1345
				550x300	Message						
3	B42	894	1000	Beam	No	COMB12	623	1312	COMB6	623	1205
				550x300	Message						
3	B42	894	1500	Beam	No	COMB7	623	671	COMB6	623	1013
				550x300	Message						
3	B42	894	2000	Beam	No	COMB7	623	671	COMB6	623	745
				550x300	Message						
3	B42	894	2500	Beam	No	COMB7	623	671	COMB7	623	671
				550x300	Message						
3	B42	894	3000	Beam	No	COMB7	623	671	COMB11	623	745
				550x300	Message						
3	B42	894	3500	Beam	No	COMB7	623	671	COMB11	623	1013
				550x300	Message						
3	B42	894	4000	Beam	No	COMB7	623	1312	COMB11	623	1205
				550x300	Message						
3	B42	894	4500	Beam	No	COMB7	623	2096	COMB11	623	1345
				550x300	Message						
3	B42	894	5000	Beam	No	COMB7	623	2988	COMB11	623	1453
				550x300	Message						
3	B43	895	0	Beam	No	COMB12	623	2947	COMB6	623	1642
				550x300	Message						
3	B43	895	500	Beam	No	COMB12	623	2046	COMB6	623	1482
				550x300	Message						
3	B43	895	1000	Beam	No	COMB12	623	1254	COMB6	623	1292
				550x300	Message						
3	B43	895	1500	Beam	No	COMB6	623	743	COMB6	623	1050
				550x300	Message						
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				550x300	Message						
3	B43	895	2500	Beam	No	COMB6	623	743	COMB6	623	743
				550x300	Message						
3	B43	895	3000	Beam	No	COMB6	623	743	COMB12	623	845
				550x300	Message						
3	B43	895	3500	Beam	No	COMB6	623	786	COMB12	623	1127
				550x300	Message						
3	B43	895	4000	Beam	No	COMB6	623	1521	COMB12	623	1334
				550x300	Message						
3	B43	895	4500	Beam	No	COMB6	623	2376	COMB12	623	1487
				550x300	Message						
3	B43	895	5000	Beam	No	COMB6	623	3358	COMB12	623	1609
				550x300	Message						
2	B41	945	0	Beam	No	COMB11	623	3406	COMB7	623	1712
				550x300	Message						
2	B41	945	500	Beam	No	COMB11	623	2411	COMB7	623	1566
				550x300	Message						
2	B41	945	1000	Beam	No	COMB11	623	1545	COMB7	623	1389
				550x300	Message						
2	B41	945	1500	Beam	No	COMB11	623	801	COMB7	623	1159
				550x300	Message						
2	B41	945	2000	Beam	No	COMB11	623	752	COMB7	623	854
				550x300	Message						
2	B41	945	2500	Beam	No	COMB11	623	752	COMB11	623	752
				550x300	Message						
2	B41	945	3000	Beam	No	COMB11	623	752	COMB11	623	752
				550x300	Message						
2	B41	945	3500	Beam	No	COMB11	623	752	COMB11	623	1064
				550x300	Message						
2	B41	945	4000	Beam	No	COMB7	623	1336	COMB11	623	1314

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				550x300	Message						
2	B41	945	4500	Beam	No	COMB7	623	2159	COMB11	623	1512
				550x300	Message						
2	B41	945	5000	Beam	No	COMB7	623	3099	COMB11	623	1680
				550x300	Message						
2	B42	946	0	Beam	No	COMB12	623	3074	COMB6	623	1518
				550x300	Message						
2	B42	946	500	Beam	No	COMB12	623	2159	COMB6	623	1395
				550x300	Message						
2	B42	946	1000	Beam	No	COMB12	623	1357	COMB6	623	1242
				550x300	Message						
2	B42	946	1500	Beam	No	COMB7	623	688	COMB6	623	1035
				550x300	Message						
2	B42	946	2000	Beam	No	COMB7	623	688	COMB6	623	754
				550x300	Message						
2	B42	946	2500	Beam	No	COMB7	623	688	COMB7	623	688
				550x300	Message						
2	B42	946	3000	Beam	No	COMB7	623	688	COMB11	623	754
				550x300	Message						
2	B42	946	3500	Beam	No	COMB7	623	688	COMB11	623	1035
				550x300	Message						
2	B42	946	4000	Beam	No	COMB7	623	1357	COMB11	623	1242
				550x300	Message						
2	B42	946	4500	Beam	No	COMB7	623	2159	COMB11	623	1395
				550x300	Message						
2	B42	946	5000	Beam	No	COMB7	623	3074	COMB11	623	1518
				550x300	Message						
2	B43	947	0	Beam	No	COMB12	623	3099	COMB6	623	1680
				550x300	Message						
2	B43	947	500	Beam	No	COMB12	623	2159	COMB6	623	1512
				550x300	Message						
2	B43	947	1000	Beam	No	COMB12	623	1336	COMB6	623	1314
				550x300	Message						
2	B43	947	1500	Beam	No	COMB6	623	752	COMB6	623	1064
				550x300	Message						
2	B43	947	2000	Beam	No	COMB6	623	752	COMB6	623	752
				550x300	Message						
2	B43	947	2500	Beam	No	COMB6	623	752	COMB6	623	752
				550x300	Message						
2	B43	947	3000	Beam	No	COMB6	623	752	COMB12	623	854
				550x300	Message						
2	B43	947	3500	Beam	No	COMB6	623	801	COMB12	623	1159
				550x300	Message						
2	B43	947	4000	Beam	No	COMB6	623	1545	COMB12	623	1389
				550x300	Message						
2	B43	947	4500	Beam	No	COMB6	623	2411	COMB12	623	1566
				550x300	Message						
2	B43	947	5000	Beam	No	COMB6	623	3406	COMB12	623	1712
				550x300	Message						
1	B41	997	0	Beam	No	COMB11	562	2581	COMB11	562	1201
				500x300	Message						
1	B41	997	500	Beam	No	COMB11	562	1711	COMB7	562	746
				500x300	Message						
1	B41	997	1000	Beam	No	COMB11	562	960	COMB7	562	846
				500x300	Message						
1	B41	997	1500	Beam	No	COMB11	562	583	COMB7	562	872
				500x300	Message						
1	B41	997	2000	Beam	No	COMB11	562	583	COMB7	562	789
				500x300	Message						
1	B41	997	2500	Beam	No	COMB11	562	583	COMB3	562	744
				500x300	Message						
1	B41	997	3000	Beam	No	COMB11	562	583	COMB11	562	742
				500x300	Message						
1	B41	997	3500	Beam	No	COMB11	562	583	COMB11	562	819
				500x300	Message						
1	B41	997	4000	Beam	No	COMB7	562	935	COMB11	562	787
				500x300	Message						
1	B41	997	4500	Beam	No	COMB7	562	1690	COMB11	562	682
				500x300	Message						
1	B41	997	5000	Beam	No	COMB7	562	2565	COMB7	562	1195
				500x300	Message						
1	B42	998	0	Beam	No	COMB12	562	2539	COMB12	562	1183
				500x300	Message						
1	B42	998	500	Beam	No	COMB12	562	1680	COMB6	562	661

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				500x300	Message						
1	B42	998	1000	Beam	No	COMB12	562	938	COMB6	562	772
				500x300	Message						
1	B42	998	1500	Beam	No	COMB12	562	574	COMB6	562	809
				500x300	Message						
1	B42	998	2000	Beam	No	COMB12	562	574	COMB6	562	739
				500x300	Message						
1	B42	998	2500	Beam	No	COMB12	562	574	COMB3	562	731
				500x300	Message						
1	B42	998	3000	Beam	No	COMB12	562	574	COMB11	562	739
				500x300	Message						
1	B42	998	3500	Beam	No	COMB12	562	574	COMB11	562	809
				500x300	Message						
1	B42	998	4000	Beam	No	COMB7	562	938	COMB11	562	772
				500x300	Message						
1	B42	998	4500	Beam	No	COMB7	562	1680	COMB11	562	661
				500x300	Message						
1	B42	998	5000	Beam	No	COMB7	562	2539	COMB7	562	1183
				500x300	Message						
1	B43	999	0	Beam	No	COMB12	562	2565	COMB12	562	1195
				500x300	Message						
1	B43	999	500	Beam	No	COMB12	562	1690	COMB6	562	682
				500x300	Message						
1	B43	999	1000	Beam	No	COMB12	562	935	COMB6	562	787
				500x300	Message						
1	B43	999	1500	Beam	No	COMB6	562	583	COMB6	562	819
				500x300	Message						
1	B43	999	2000	Beam	No	COMB6	562	583	COMB6	562	742
				500x300	Message						
1	B43	999	2500	Beam	No	COMB6	562	583	COMB3	562	744
				500x300	Message						
1	B43	999	3000	Beam	No	COMB6	562	583	COMB12	562	789
				500x300	Message						
1	B43	999	3500	Beam	No	COMB6	562	583	COMB12	562	872
				500x300	Message						
1	B43	999	4000	Beam	No	COMB6	562	960	COMB12	562	846
				500x300	Message						
1	B43	999	4500	Beam	No	COMB6	562	1711	COMB12	562	746
				500x300	Message						
1	B43	999	5000	Beam	No	COMB6	562	2581	COMB6	562	1201
				500x300	Message						

Table B3.1.2 - Concrete Beam Summary - Eurocode 2-2004 (Part 2 of 2)

Story	Label	Unique Name	Station mm	V Combo	At Shear mm ² /m	Torsion Long Combo	Al Torsion mm ²	Torsion Tran Combo	At Torsion mm ² /m	Warnings	Errors
Roof	B41	425	0	COMB13	339,41	COMB7	4	COMB10	0,62	No Message	No Message
Roof	B41	425	500	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	1000	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	1500	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	2000	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	2500	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	3000	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	3500	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	4000	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	4500	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B41	425	5000	COMB13	339,41	COMB7	4	COMB13	0	No Message	No Message
Roof	B42	426	0	COMB13	339,41	COMB11	0,1528	COMB13	0	No Message	No Message
Roof	B42	426	500	COMB13	339,41	COMB11	0,1528	COMB13	0	No Message	No Message

Roof	B42	426	1000	COMB13	339,41	COMB11	0,1528	COMB13	0	No	No
Roof	B42	426	1500	COMB13	339,41	COMB11	0,1528	COMB13	0	Message	Message
Roof	B42	426	2000	COMB13	339,41	COMB11	0,1528	COMB13	0	No	No
Roof	B42	426	2500	COMB13	339,41	COMB11	0,1528	COMB13	0	Message	Message
Roof	B42	426	3000	COMB13	339,41	COMB11	0,1528	COMB13	0	No	No
Roof	B42	426	3500	COMB13	339,41	COMB11	0,1528	COMB13	0	Message	Message
Roof	B42	426	4000	COMB13	339,41	COMB11	0,1528	COMB13	0	No	No
Roof	B42	426	4500	COMB13	339,41	COMB11	0,1528	COMB13	0	Message	Message
Roof	B42	426	5000	COMB13	339,41	COMB11	0,1528	COMB13	0	No	No
Roof	B43	427	0	COMB13	339,41	COMB12	4	COMB13	0	Message	Message
Roof	B43	427	500	COMB13	339,41	COMB12	4	COMB13	0	No	No
Roof	B43	427	1000	COMB13	339,41	COMB12	4	COMB13	0	Message	Message
Roof	B43	427	1500	COMB13	339,41	COMB12	4	COMB13	0	No	No
Roof	B43	427	2000	COMB13	339,41	COMB12	4	COMB13	0	Message	Message
Roof	B43	427	2500	COMB13	339,41	COMB12	4	COMB13	0	No	No
Roof	B43	427	3000	COMB13	339,41	COMB12	4	COMB13	0	Message	Message
Roof	B43	427	3500	COMB13	339,41	COMB12	4	COMB13	0	No	No
Roof	B43	427	4000	COMB13	339,41	COMB12	4	COMB13	0	Message	Message
Roof	B43	427	4500	COMB13	339,41	COMB12	4	COMB13	0	No	No
Roof	B43	427	5000	COMB13	339,41	COMB12	4	COMB5	0,62	Message	Message
Attic	B41	477	0	COMB13	339,41	COMB7	2	COMB7	0,56	No	No
Attic	B41	477	500	COMB13	339,41	COMB7	2	COMB7	0,56	Message	Message
Attic	B41	477	1000	COMB13	339,41	COMB7	2	COMB7	0,56	No	No
Attic	B41	477	1500	COMB13	339,41	COMB7	2	COMB3	0,45	Message	Message
Attic	B41	477	2000	COMB13	339,41	COMB7	2	COMB13	0	No	No
Attic	B41	477	2500	COMB13	339,41	COMB7	2	COMB13	0	Message	Message
Attic	B41	477	3000	COMB13	339,41	COMB7	2	COMB13	0	No	No
Attic	B41	477	3500	COMB13	339,41	COMB7	2	COMB13	0	Message	Message
Attic	B41	477	4000	COMB13	339,41	COMB7	2	COMB13	0	No	No
Attic	B41	477	4500	COMB13	339,41	COMB7	2	COMB7	0,56	Message	Message
Attic	B41	477	5000	COMB13	339,41	COMB7	2	COMB7	0,56	No	No
Attic	B42	478	0	COMB13	339,41	COMB11	0,0219	COMB11	0,01	Message	Message
Attic	B42	478	500	COMB13	339,41	COMB11	0,0219	COMB11	0,01	No	No
Attic	B42	478	1000	COMB13	339,41	COMB11	0,0219	COMB11	0,01	Message	Message
Attic	B42	478	1500	COMB13	339,41	COMB11	0,0219	COMB11	0,01	No	No
Attic	B42	478	2000	COMB13	339,41	COMB11	0,0219	COMB13	0	Message	Message
Attic	B42	478	2500	COMB13	339,41	COMB11	0,0219	COMB13	0	No	No
Attic	B42	478	2500	COMB13	339,41	COMB11	0,0219	COMB13	0	Message	Message

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Attic	B42	478	3000	COMB13	339,41	COMB11	0,0219	COMB13	0	No	No
Attic	B42	478	3500	COMB13	339,41	COMB11	0,0219	COMB7	0,01	Message	Message
Attic	B42	478	4000	COMB13	339,41	COMB11	0,0219	COMB7	0,01	No	No
Attic	B42	478	4500	COMB13	339,41	COMB11	0,0219	COMB7	0,01	Message	Message
Attic	B42	478	5000	COMB13	339,41	COMB11	0,0219	COMB7	0,01	No	No
Attic	B43	479	0	COMB13	339,41	COMB12	2	COMB12	0,56	Message	Message
Attic	B43	479	500	COMB13	339,41	COMB12	2	COMB12	0,56	No	No
Attic	B43	479	1000	COMB13	339,41	COMB12	2	COMB13	0	Message	Message
Attic	B43	479	1500	COMB13	339,41	COMB12	2	COMB13	0	No	No
Attic	B43	479	2000	COMB13	339,41	COMB12	2	COMB13	0	Message	Message
Attic	B43	479	2500	COMB13	339,41	COMB12	2	COMB13	0	No	No
Attic	B43	479	3000	COMB13	339,41	COMB12	2	COMB13	0	Message	Message
Attic	B43	479	3500	COMB13	339,41	COMB12	2	COMB3	0,45	No	No
Attic	B43	479	4000	COMB13	339,41	COMB12	2	COMB12	0,56	Message	Message
Attic	B43	479	4500	COMB13	339,41	COMB12	2	COMB12	0,56	No	No
Attic	B43	479	5000	COMB13	339,41	COMB12	2	COMB12	0,56	Message	Message
10	B41	529	0	COMB11	425,05	COMB11	1	COMB11	0,27	No	No
10	B41	529	500	COMB11	408,51	COMB11	1	COMB11	0,27	Message	Message
10	B41	529	1000	COMB11	370,14	COMB11	1	COMB11	0,27	No	No
10	B41	529	1500	COMB13	339,41	COMB11	1	COMB11	0,27	Message	Message
10	B41	529	2000	COMB13	339,41	COMB11	1	COMB11	0,27	No	No
10	B41	529	2500	COMB13	339,41	COMB11	1	COMB13	0	Message	Message
10	B41	529	3000	COMB13	339,41	COMB11	1	COMB13	0	No	No
10	B41	529	3500	COMB13	339,41	COMB11	1	COMB7	0,06	Message	Message
10	B41	529	4000	COMB13	339,41	COMB11	1	COMB3	0,11	No	No
10	B41	529	4500	COMB13	339,41	COMB11	1	COMB9	0,2	Message	Message
10	B41	529	5000	COMB7	346,43	COMB11	1	COMB11	0,27	No	No
10	B42	530	0	COMB12	393,47	COMB11	0,05859	COMB11	0,02	Message	Message
10	B42	530	500	COMB12	376,93	COMB11	0,05859	COMB11	0,02	No	No
10	B42	530	1000	COMB13	339,41	COMB11	0,05859	COMB11	0,02	Message	Message
10	B42	530	1500	COMB13	339,41	COMB11	0,05859	COMB11	0,02	No	No
10	B42	530	2000	COMB13	339,41	COMB11	0,05859	COMB11	0,02	Message	Message
10	B42	530	2500	COMB13	339,41	COMB11	0,05859	COMB13	0	No	No
10	B42	530	3000	COMB13	339,41	COMB11	0,05859	COMB7	0,02	Message	Message
10	B42	530	3500	COMB13	339,41	COMB11	0,05859	COMB7	0,02	No	No
10	B42	530	4000	COMB13	339,41	COMB11	0,05859	COMB7	0,02	Message	Message
10	B42	530	4500	COMB7	376,93	COMB11	0,05859	COMB11	0,02	No	No
										Message	Message

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10	B42	530	5000	COMB7	393,47	COMB11	0,05859	COMB7	0,02	No	No
10	B43	531	0	COMB12	346,43	COMB6	1	COMB6	0,27	Message	Message
10	B43	531	500	COMB13	339,41	COMB6	1	COMB4	0,2	No	No
10	B43	531	1000	COMB13	339,41	COMB6	1	COMB3	0,11	Message	Message
10	B43	531	1500	COMB13	339,41	COMB6	1	COMB12	0,06	No	No
10	B43	531	2000	COMB13	339,41	COMB6	1	COMB13	0	Message	Message
10	B43	531	2500	COMB13	339,41	COMB6	1	COMB13	0	No	No
10	B43	531	3000	COMB13	339,41	COMB6	1	COMB6	0,27	Message	Message
10	B43	531	3500	COMB13	339,41	COMB6	1	COMB6	0,27	No	No
10	B43	531	4000	COMB6	370,14	COMB6	1	COMB6	0,27	Message	Message
10	B43	531	4500	COMB6	408,51	COMB6	1	COMB6	0,27	No	No
10	B43	531	5000	COMB6	425,05	COMB6	1	COMB6	0,27	Message	Message
9	B41	581	0	COMB11	468,06	COMB7	1	COMB8	0,14	No	No
9	B41	581	500	COMB11	451,52	COMB7	1	COMB7	0,2	Message	Message
9	B41	581	1000	COMB11	413,14	COMB7	1	COMB8	0,14	No	No
9	B41	581	1500	COMB11	352,94	COMB7	1	COMB11	0,1	Message	Message
9	B41	581	2000	COMB13	339,41	COMB7	1	COMB11	0,1	No	No
9	B41	581	2500	COMB13	339,41	COMB7	1	COMB11	0,1	Message	Message
9	B41	581	3000	COMB13	339,41	COMB7	1	COMB7	0,2	No	No
9	B41	581	3500	COMB13	339,41	COMB7	1	COMB7	0,2	Message	Message
9	B41	581	4000	COMB13	339,41	COMB7	1	COMB7	0,2	No	No
9	B41	581	4500	COMB7	373,55	COMB7	1	COMB7	0,2	Message	Message
9	B41	581	5000	COMB7	390,09	COMB7	1	COMB7	0,2	No	No
9	B42	582	0	COMB12	432,75	COMB11	0,05664	COMB11	0,01	Message	Message
9	B42	582	500	COMB12	416,21	COMB11	0,05664	COMB11	0,01	No	No
9	B42	582	1000	COMB12	377,83	COMB11	0,05664	COMB11	0,01	Message	Message
9	B42	582	1500	COMB13	339,41	COMB11	0,05664	COMB11	0,01	No	No
9	B42	582	2000	COMB13	339,41	COMB11	0,05664	COMB11	0,01	Message	Message
9	B42	582	2500	COMB13	339,41	COMB11	0,05664	COMB13	0	No	No
9	B42	582	3000	COMB13	339,41	COMB11	0,05664	COMB7	0,01	Message	Message
9	B42	582	3500	COMB13	339,41	COMB11	0,05664	COMB7	0,01	No	No
9	B42	582	4000	COMB7	377,83	COMB11	0,05664	COMB7	0,01	Message	Message
9	B42	582	4500	COMB7	416,21	COMB11	0,05664	COMB7	0,01	No	No
9	B42	582	5000	COMB7	432,75	COMB11	0,05664	COMB7	0,01	Message	Message
9	B43	583	0	COMB12	390,09	COMB12	1	COMB12	0,2	No	No
9	B43	583	500	COMB12	373,55	COMB12	1	COMB12	0,2	Message	Message
9	B43	583	1000	COMB13	339,41	COMB12	1	COMB12	0,2	No	No
9	B43	583	1000	COMB13	339,41	COMB12	1	COMB12	0,2	Message	Message

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9	B43	583	1500	COMB13	339,41	COMB12	1	COMB12	0,2	No	No
9	B43	583	2000	COMB13	339,41	COMB12	1	COMB12	0,2	Message	Message
9	B43	583	2500	COMB13	339,41	COMB12	1	COMB6	0,1	No	No
9	B43	583	3000	COMB13	339,41	COMB12	1	COMB6	0,1	Message	Message
9	B43	583	3500	COMB6	352,94	COMB12	1	COMB6	0,1	No	No
9	B43	583	4000	COMB6	413,14	COMB12	1	COMB13	0,14	Message	Message
9	B43	583	4500	COMB6	451,52	COMB12	1	COMB12	0,2	No	No
9	B43	583	5000	COMB6	468,06	COMB12	1	COMB13	0,14	Message	Message
8	B41	633	0	COMB11	508,37	COMB11	1	COMB11	0,16	No	No
8	B41	633	500	COMB11	491,83	COMB11	1	COMB11	0,16	Message	Message
8	B41	633	1000	COMB11	453,46	COMB11	1	COMB11	0,16	No	No
8	B41	633	1500	COMB11	393,25	COMB11	1	COMB11	0,16	Message	Message
8	B41	633	2000	COMB13	339,41	COMB11	1	COMB11	0,16	No	No
8	B41	633	2500	COMB13	339,41	COMB11	1	COMB11	0,16	Message	Message
8	B41	633	3000	COMB13	339,41	COMB11	1	COMB7	0,13	No	No
8	B41	633	3500	COMB13	339,41	COMB11	1	COMB7	0,13	Message	Message
8	B41	633	4000	COMB7	381,94	COMB11	1	COMB7	0,13	No	No
8	B41	633	4500	COMB7	420,32	COMB11	1	COMB7	0,13	Message	Message
8	B41	633	5000	COMB7	436,86	COMB11	1	COMB7	0,13	No	No
8	B42	634	0	COMB12	471,46	COMB11	0,06113	COMB11	0,02	Message	Message
8	B42	634	500	COMB12	454,92	COMB11	0,06113	COMB11	0,02	No	No
8	B42	634	1000	COMB12	416,54	COMB11	0,06113	COMB11	0,02	Message	Message
8	B42	634	1500	COMB12	356,34	COMB11	0,06113	COMB11	0,02	No	No
8	B42	634	2000	COMB13	339,41	COMB11	0,06113	COMB11	0,02	Message	Message
8	B42	634	2500	COMB13	339,41	COMB11	0,06113	COMB11	0,02	No	No
8	B42	634	3000	COMB13	339,41	COMB11	0,06113	COMB7	0,02	Message	Message
8	B42	634	3500	COMB7	356,34	COMB11	0,06113	COMB7	0,02	No	No
8	B42	634	4000	COMB7	416,54	COMB11	0,06113	COMB7	0,02	Message	Message
8	B42	634	4500	COMB7	454,92	COMB11	0,06113	COMB7	0,02	No	No
8	B42	634	5000	COMB7	471,46	COMB11	0,06113	COMB7	0,02	Message	Message
8	B43	635	0	COMB12	436,86	COMB6	1	COMB12	0,13	No	No
8	B43	635	500	COMB12	420,32	COMB6	1	COMB12	0,13	Message	Message
8	B43	635	1000	COMB12	381,94	COMB6	1	COMB12	0,13	No	No
8	B43	635	1500	COMB13	339,41	COMB6	1	COMB12	0,13	Message	Message
8	B43	635	2000	COMB13	339,41	COMB6	1	COMB12	0,13	No	No
8	B43	635	2500	COMB13	339,41	COMB6	1	COMB6	0,16	Message	Message
8	B43	635	3000	COMB13	339,41	COMB6	1	COMB6	0,16	No	No
8	B43	635	3000	COMB13	339,41	COMB6	1	COMB6	0,16	Message	Message

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8	B43	635	3500	COMB6	393,25	COMB6	1	COMB6	0,16	No	No
8	B43	635	4000	COMB6	453,46	COMB6	1	COMB6	0,16	Message	Message
8	B43	635	4500	COMB6	491,83	COMB6	1	COMB6	0,16	No	No
8	B43	635	5000	COMB6	508,37	COMB6	1	COMB6	0,16	Message	Message
7	B41	685	0	COMB11	548,62	COMB7	1	COMB11	0,11	No	No
7	B41	685	500	COMB11	532,08	COMB7	1	COMB11	0,11	Message	Message
7	B41	685	1000	COMB11	493,71	COMB7	1	COMB11	0,11	No	No
7	B41	685	1500	COMB11	433,5	COMB7	1	COMB11	0,11	Message	Message
7	B41	685	2000	COMB11	351,46	COMB7	1	COMB11	0,11	No	No
7	B41	685	2500	COMB13	339,41	COMB7	1	COMB7	0,15	Message	Message
7	B41	685	3000	COMB13	339,41	COMB7	1	COMB7	0,15	No	No
7	B41	685	3500	COMB7	369,82	COMB7	1	COMB7	0,15	Message	Message
7	B41	685	4000	COMB7	430,02	COMB7	1	COMB7	0,15	No	No
7	B41	685	4500	COMB7	468,4	COMB7	1	COMB7	0,15	Message	Message
7	B41	685	5000	COMB7	484,94	COMB7	1	COMB7	0,15	No	No
7	B42	686	0	COMB12	510,77	COMB11	0,06344	COMB11	0,02	Message	Message
7	B42	686	500	COMB12	494,23	COMB11	0,06344	COMB11	0,02	No	No
7	B42	686	1000	COMB12	455,86	COMB11	0,06344	COMB11	0,02	Message	Message
7	B42	686	1500	COMB12	395,65	COMB11	0,06344	COMB11	0,02	No	No
7	B42	686	2000	COMB13	339,41	COMB11	0,06344	COMB11	0,02	Message	Message
7	B42	686	2500	COMB13	339,41	COMB11	0,06344	COMB11	0,02	No	No
7	B42	686	3000	COMB13	339,41	COMB11	0,06344	COMB7	0,02	Message	Message
7	B42	686	3500	COMB7	395,65	COMB11	0,06344	COMB7	0,02	No	No
7	B42	686	4000	COMB7	455,86	COMB11	0,06344	COMB7	0,02	Message	Message
7	B42	686	4500	COMB7	494,23	COMB11	0,06344	COMB7	0,02	No	No
7	B42	686	5000	COMB7	510,77	COMB11	0,06344	COMB7	0,02	Message	Message
7	B43	687	0	COMB12	484,94	COMB12	1	COMB12	0,15	No	No
7	B43	687	500	COMB12	468,4	COMB12	1	COMB12	0,15	Message	Message
7	B43	687	1000	COMB12	430,02	COMB12	1	COMB12	0,15	No	No
7	B43	687	1500	COMB12	369,82	COMB12	1	COMB12	0,15	Message	Message
7	B43	687	2000	COMB13	339,41	COMB12	1	COMB12	0,15	No	No
7	B43	687	2500	COMB13	339,41	COMB12	1	COMB12	0,15	Message	Message
7	B43	687	3000	COMB6	351,46	COMB12	1	COMB6	0,11	No	No
7	B43	687	3500	COMB6	433,5	COMB12	1	COMB6	0,11	Message	Message
7	B43	687	4000	COMB6	493,71	COMB12	1	COMB6	0,11	No	No
7	B43	687	4500	COMB6	532,08	COMB12	1	COMB6	0,11	Message	Message
7	B43	687	5000	COMB6	548,62	COMB12	1	COMB6	0,11	No	No
										Message	Message

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6	B41	737	0	COMB11	588,03	COMB11	1	COMB11	0,17	No	No
6	B41	737	500	COMB11	571,49	COMB11	1	COMB11	0,17	Message	Message
6	B41	737	1000	COMB11	533,12	COMB11	1	COMB11	0,17	No	No
6	B41	737	1500	COMB11	472,91	COMB11	1	COMB11	0,17	Message	Message
6	B41	737	2000	COMB11	390,87	COMB11	1	COMB11	0,17	No	No
6	B41	737	2500	COMB13	339,41	COMB11	1	COMB11	0,17	Message	Message
6	B41	737	3000	COMB13	339,41	COMB11	1	COMB11	0,17	No	No
6	B41	737	3500	COMB7	420,88	COMB11	1	COMB7	0,06	Message	Message
6	B41	737	4000	COMB7	481,09	COMB11	1	COMB7	0,06	No	No
6	B41	737	4500	COMB7	519,46	COMB11	1	COMB7	0,06	Message	Message
6	B41	737	5000	COMB7	536	COMB11	1	COMB7	0,06	No	No
6	B42	738	0	COMB12	548,98	COMB11	0,07133	COMB11	0,02	Message	Message
6	B42	738	500	COMB12	532,44	COMB11	0,07133	COMB11	0,02	No	No
6	B42	738	1000	COMB12	494,06	COMB11	0,07133	COMB11	0,02	Message	Message
6	B42	738	1500	COMB12	433,86	COMB11	0,07133	COMB11	0,02	No	No
6	B42	738	2000	COMB12	351,82	COMB11	0,07133	COMB11	0,02	Message	Message
6	B42	738	2500	COMB13	339,41	COMB11	0,07133	COMB11	0,02	No	No
6	B42	738	3000	COMB7	351,82	COMB11	0,07133	COMB7	0,02	Message	Message
6	B42	738	3500	COMB7	433,86	COMB11	0,07133	COMB7	0,02	No	No
6	B42	738	4000	COMB7	494,06	COMB11	0,07133	COMB7	0,02	Message	Message
6	B42	738	4500	COMB7	532,44	COMB11	0,07133	COMB7	0,02	No	No
6	B42	738	5000	COMB7	548,98	COMB11	0,07133	COMB7	0,02	Message	Message
6	B43	739	0	COMB12	536	COMB6	1	COMB12	0,06	No	No
6	B43	739	500	COMB12	519,46	COMB6	1	COMB12	0,06	Message	Message
6	B43	739	1000	COMB12	481,09	COMB6	1	COMB12	0,06	No	No
6	B43	739	1500	COMB12	420,88	COMB6	1	COMB12	0,06	Message	Message
6	B43	739	2000	COMB13	339,41	COMB6	1	COMB6	0,17	No	No
6	B43	739	2500	COMB13	339,41	COMB6	1	COMB6	0,17	Message	Message
6	B43	739	3000	COMB6	390,87	COMB6	1	COMB6	0,17	No	No
6	B43	739	3500	COMB6	472,91	COMB6	1	COMB6	0,17	Message	Message
6	B43	739	4000	COMB6	533,12	COMB6	1	COMB6	0,17	No	No
6	B43	739	4500	COMB6	571,49	COMB6	1	COMB6	0,17	Message	Message
6	B43	739	5000	COMB6	588,03	COMB6	1	COMB6	0,17	No	No
5	B41	789	0	COMB11	630,63	COMB7	0,3304	COMB11	0,05	Message	Message
5	B41	789	500	COMB11	614,09	COMB7	0,3304	COMB11	0,05	No	No
5	B41	789	1000	COMB11	575,72	COMB7	0,3304	COMB11	0,05	Message	Message
5	B41	789	1500	COMB11	515,51	COMB7	0,3304	COMB11	0,05	No	No
										Message	Message

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5	B41	789	2000	COMB11	433,47	COMB7	0,3304	COMB11	0,05	No	No
5	B41	789	2500	COMB13	339,41	COMB7	0,3304	COMB7	0,08	Message	Message
5	B41	789	3000	COMB7	385,22	COMB7	0,3304	COMB7	0,08	No	No
5	B41	789	3500	COMB7	467,26	COMB7	0,3304	COMB7	0,08	Message	Message
5	B41	789	4000	COMB7	527,47	COMB7	0,3304	COMB7	0,08	No	No
5	B41	789	4500	COMB7	565,84	COMB7	0,3304	COMB7	0,08	Message	Message
5	B41	789	5000	COMB7	582,38	COMB7	0,3304	COMB7	0,08	No	No
5	B42	790	0	COMB12	594,27	COMB11	0,05305	COMB11	0,01	Message	Message
5	B42	790	500	COMB12	577,73	COMB11	0,05305	COMB11	0,01	No	No
5	B42	790	1000	COMB12	539,35	COMB11	0,05305	COMB11	0,01	Message	Message
5	B42	790	1500	COMB12	479,15	COMB11	0,05305	COMB11	0,01	No	No
5	B42	790	2000	COMB12	397,11	COMB11	0,05305	COMB11	0,01	Message	Message
5	B42	790	2500	COMB13	339,41	COMB11	0,05305	COMB11	0,01	No	No
5	B42	790	3000	COMB7	397,11	COMB11	0,05305	COMB11	0,01	Message	Message
5	B42	790	3500	COMB7	479,15	COMB11	0,05305	COMB7	0,01	No	No
5	B42	790	4000	COMB7	539,35	COMB11	0,05305	COMB7	0,01	Message	Message
5	B42	790	4500	COMB7	577,73	COMB11	0,05305	COMB7	0,01	No	No
5	B42	790	5000	COMB7	594,27	COMB11	0,05305	COMB7	0,01	Message	Message
5	B43	791	0	COMB12	582,38	COMB12	0,3304	COMB12	0,08	No	No
5	B43	791	500	COMB12	565,84	COMB12	0,3304	COMB12	0,08	Message	Message
5	B43	791	1000	COMB12	527,47	COMB12	0,3304	COMB12	0,08	No	No
5	B43	791	1500	COMB12	467,26	COMB12	0,3304	COMB12	0,08	Message	Message
5	B43	791	2000	COMB12	385,22	COMB12	0,3304	COMB12	0,08	No	No
5	B43	791	2500	COMB13	339,41	COMB12	0,3304	COMB12	0,08	Message	Message
5	B43	791	3000	COMB6	433,47	COMB12	0,3304	COMB6	0,05	No	No
5	B43	791	3500	COMB6	515,51	COMB12	0,3304	COMB6	0,05	Message	Message
5	B43	791	4000	COMB6	575,72	COMB12	0,3304	COMB6	0,05	No	No
5	B43	791	4500	COMB6	614,09	COMB12	0,3304	COMB6	0,05	Message	Message
5	B43	791	5000	COMB6	630,63	COMB12	0,3304	COMB6	0,05	No	No
4	B41	841	0	COMB11	655,91	COMB7	0,4565	COMB3	0,08	Message	Message
4	B41	841	500	COMB11	639,37	COMB7	0,4565	COMB3	0,08	No	No
4	B41	841	1000	COMB11	601	COMB7	0,4565	COMB3	0,08	Message	Message
4	B41	841	1500	COMB11	540,79	COMB7	0,4565	COMB3	0,08	No	No
4	B41	841	2000	COMB11	458,75	COMB7	0,4565	COMB7	0,12	Message	Message
4	B41	841	2500	COMB11	354,88	COMB7	0,4565	COMB7	0,12	No	No
4	B41	841	3000	COMB7	416,24	COMB7	0,4565	COMB7	0,12	Message	Message
4	B41	841	3500	COMB7	498,28	COMB7	0,4565	COMB7	0,12	No	No
										Message	Message

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4	B41	841	4000	COMB7	558,49	COMB7	0,4565	COMB7	0,12	No	No
4	B41	841	4500	COMB7	596,86	COMB7	0,4565	COMB7	0,12	Message	Message
4	B41	841	5000	COMB7	613,4	COMB7	0,4565	COMB7	0,12	No	No
4	B42	842	0	COMB12	625,3	COMB7	0,04396	COMB11	0,01	Message	Message
4	B42	842	500	COMB12	608,76	COMB7	0,04396	COMB11	0,01	No	No
4	B42	842	1000	COMB12	570,39	COMB7	0,04396	COMB11	0,01	Message	Message
4	B42	842	1500	COMB12	510,18	COMB7	0,04396	COMB11	0,01	No	No
4	B42	842	2000	COMB12	428,14	COMB7	0,04396	COMB7	0,01	Message	Message
4	B42	842	2500	COMB13	339,41	COMB7	0,04396	COMB7	0,01	No	No
4	B42	842	3000	COMB7	428,14	COMB7	0,04396	COMB7	0,01	Message	Message
4	B42	842	3500	COMB7	510,18	COMB7	0,04396	COMB7	0,01	No	No
4	B42	842	4000	COMB7	570,39	COMB7	0,04396	COMB7	0,01	Message	Message
4	B42	842	4500	COMB7	608,76	COMB7	0,04396	COMB7	0,01	No	No
4	B42	842	5000	COMB7	625,3	COMB7	0,04396	COMB7	0,01	Message	Message
4	B43	843	0	COMB12	613,4	COMB12	0,4565	COMB12	0,12	No	No
4	B43	843	500	COMB12	596,86	COMB12	0,4565	COMB12	0,12	Message	Message
4	B43	843	1000	COMB12	558,49	COMB12	0,4565	COMB12	0,12	No	No
4	B43	843	1500	COMB12	498,28	COMB12	0,4565	COMB12	0,12	Message	Message
4	B43	843	2000	COMB12	416,24	COMB12	0,4565	COMB12	0,12	No	No
4	B43	843	2500	COMB6	354,88	COMB12	0,4565	COMB12	0,12	Message	Message
4	B43	843	3000	COMB6	458,75	COMB12	0,4565	COMB12	0,12	No	No
4	B43	843	3500	COMB6	540,79	COMB12	0,4565	COMB3	0,08	Message	Message
4	B43	843	4000	COMB6	601	COMB12	0,4565	COMB3	0,08	No	No
4	B43	843	4500	COMB6	639,37	COMB12	0,4565	COMB3	0,08	Message	Message
4	B43	843	5000	COMB6	655,91	COMB12	0,4565	COMB3	0,08	No	No
3	B41	893	0	COMB11	665,58	COMB11	1	COMB11	0,12	Message	Message
3	B41	893	500	COMB11	650,16	COMB11	1	COMB11	0,12	No	No
3	B41	893	1000	COMB11	615,04	COMB11	1	COMB11	0,12	Message	Message
3	B41	893	1500	COMB11	560,23	COMB11	1	COMB11	0,12	No	No
3	B41	893	2000	COMB11	485,72	COMB11	1	COMB11	0,12	Message	Message
3	B41	893	2500	COMB11	391,52	COMB11	1	COMB11	0,12	No	No
3	B41	893	3000	COMB7	456,44	COMB11	1	COMB11	0,12	Message	Message
3	B41	893	3500	COMB7	530,94	COMB11	1	COMB11	0,12	No	No
3	B41	893	4000	COMB7	585,75	COMB11	1	COMB3	0,09	Message	Message
3	B41	893	4500	COMB7	620,87	COMB11	1	COMB3	0,09	No	No
3	B41	893	5000	COMB7	636,3	COMB11	1	COMB4	0,06	Message	Message
3	B42	894	0	COMB12	626,95	COMB11	0,04479	COMB11	0,01	No	No
										Message	Message

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3	B42	894	500	COMB12	611,53	COMB11	0,04479	COMB11	0,01	No	No
3	B42	894	1000	COMB12	576,41	COMB11	0,04479	COMB11	0,01	Message	Message
3	B42	894	1500	COMB12	521,6	COMB11	0,04479	COMB11	0,01	No	No
3	B42	894	2000	COMB12	447,09	COMB11	0,04479	COMB11	0,01	Message	Message
3	B42	894	2500	COMB7	352,9	COMB11	0,04479	COMB11	0,01	No	No
3	B42	894	3000	COMB7	447,09	COMB11	0,04479	COMB11	0,01	Message	Message
3	B42	894	3500	COMB7	521,6	COMB11	0,04479	COMB7	0,01	No	No
3	B42	894	4000	COMB7	576,41	COMB11	0,04479	COMB7	0,01	Message	Message
3	B42	894	4500	COMB7	611,53	COMB11	0,04479	COMB7	0,01	No	No
3	B42	894	5000	COMB7	626,95	COMB11	0,04479	COMB7	0,01	Message	Message
3	B43	895	0	COMB12	636,3	COMB6	1	COMB9	0,06	No	No
3	B43	895	500	COMB12	620,87	COMB6	1	COMB3	0,09	Message	Message
3	B43	895	1000	COMB12	585,75	COMB6	1	COMB3	0,09	No	No
3	B43	895	1500	COMB12	530,94	COMB6	1	COMB6	0,12	Message	Message
3	B43	895	2000	COMB12	456,44	COMB6	1	COMB6	0,12	No	No
3	B43	895	2500	COMB6	391,52	COMB6	1	COMB6	0,12	Message	Message
3	B43	895	3000	COMB6	485,72	COMB6	1	COMB6	0,12	No	No
3	B43	895	3500	COMB6	560,23	COMB6	1	COMB6	0,12	Message	Message
3	B43	895	4000	COMB6	615,04	COMB6	1	COMB6	0,12	No	No
3	B43	895	4500	COMB6	650,16	COMB6	1	COMB6	0,12	Message	Message
3	B43	895	5000	COMB6	665,58	COMB6	1	COMB6	0,12	No	No
2	B41	945	0	COMB11	671,54	COMB9	0,4725	COMB9	0,11	Message	Message
2	B41	945	500	COMB11	656,12	COMB9	0,4725	COMB9	0,11	No	No
2	B41	945	1000	COMB11	621	COMB9	0,4725	COMB9	0,11	Message	Message
2	B41	945	1500	COMB11	566,19	COMB9	0,4725	COMB9	0,11	No	No
2	B41	945	2000	COMB11	491,68	COMB9	0,4725	COMB9	0,11	Message	Message
2	B41	945	2500	COMB11	397,49	COMB9	0,4725	COMB9	0,11	No	No
2	B41	945	3000	COMB7	474,37	COMB9	0,4725	COMB4	0,11	Message	Message
2	B41	945	3500	COMB7	548,88	COMB9	0,4725	COMB4	0,11	No	No
2	B41	945	4000	COMB7	603,69	COMB9	0,4725	COMB3	0,11	Message	Message
2	B41	945	4500	COMB7	638,8	COMB9	0,4725	COMB3	0,11	No	No
2	B41	945	5000	COMB7	654,23	COMB9	0,4725	COMB4	0,11	Message	Message
2	B42	946	0	COMB12	637,7	COMB11	0,04676	COMB11	0,01	No	No
2	B42	946	500	COMB12	622,28	COMB11	0,04676	COMB11	0,01	Message	Message
2	B42	946	1000	COMB12	587,16	COMB11	0,04676	COMB11	0,01	No	No
2	B42	946	1500	COMB12	532,35	COMB11	0,04676	COMB11	0,01	Message	Message
2	B42	946	2000	COMB12	457,84	COMB11	0,04676	COMB11	0,01	No	No
										Message	Message

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2	B42	946	2500	COMB7	363,65	COMB11	0,04676	COMB11	0,01	No	No
2	B42	946	3000	COMB7	457,84	COMB11	0,04676	COMB11	0,01	Message	Message
2	B42	946	3500	COMB7	532,35	COMB11	0,04676	COMB11	0,01	No	No
2	B42	946	4000	COMB7	587,16	COMB11	0,04676	COMB7	0,01	Message	Message
2	B42	946	4500	COMB7	622,28	COMB11	0,04676	COMB7	0,01	No	No
2	B42	946	5000	COMB7	637,7	COMB11	0,04676	COMB7	0,01	Message	Message
2	B43	947	0	COMB12	654,23	COMB4	0,4725	COMB9	0,11	No	No
2	B43	947	500	COMB12	638,8	COMB4	0,4725	COMB3	0,11	Message	Message
2	B43	947	1000	COMB12	603,69	COMB4	0,4725	COMB3	0,11	No	No
2	B43	947	1500	COMB12	548,88	COMB4	0,4725	COMB9	0,11	Message	Message
2	B43	947	2000	COMB12	474,37	COMB4	0,4725	COMB9	0,11	No	No
2	B43	947	2500	COMB6	397,49	COMB4	0,4725	COMB4	0,11	Message	Message
2	B43	947	3000	COMB6	491,68	COMB4	0,4725	COMB4	0,11	No	No
2	B43	947	3500	COMB6	566,19	COMB4	0,4725	COMB4	0,11	Message	Message
2	B43	947	4000	COMB6	621	COMB4	0,4725	COMB4	0,11	No	No
2	B43	947	4500	COMB6	656,12	COMB4	0,4725	COMB4	0,11	Message	Message
2	B43	947	5000	COMB6	671,54	COMB4	0,4725	COMB4	0,11	No	No
1	B41	997	0	COMB11	624,11	COMB10	1	COMB10	0,17	Message	Message
1	B41	997	500	COMB11	604,57	COMB10	1	COMB10	0,17	No	No
1	B41	997	1000	COMB11	557,19	COMB10	1	COMB10	0,17	Message	Message
1	B41	997	1500	COMB11	481,99	COMB10	1	COMB10	0,17	No	No
1	B41	997	2000	COMB11	378,95	COMB10	1	COMB10	0,17	Message	Message
1	B41	997	2500	COMB13	339,41	COMB10	1	COMB12	0,16	No	No
1	B41	997	3000	COMB7	383,63	COMB10	1	COMB5	0,12	Message	Message
1	B41	997	3500	COMB7	486,66	COMB10	1	COMB3	0,15	No	No
1	B41	997	4000	COMB7	561,87	COMB10	1	COMB10	0,17	Message	Message
1	B41	997	4500	COMB7	609,24	COMB10	1	COMB10	0,17	No	No
1	B41	997	5000	COMB7	628,78	COMB10	1	COMB3	0,15	Message	Message
1	B42	998	0	COMB12	618,97	COMB7	0,05429	COMB11	0,01	No	No
1	B42	998	500	COMB12	599,43	COMB7	0,05429	COMB11	0,01	Message	Message
1	B42	998	1000	COMB12	552,06	COMB7	0,05429	COMB11	0,01	No	No
1	B42	998	1500	COMB12	476,85	COMB7	0,05429	COMB11	0,01	Message	Message
1	B42	998	2000	COMB12	373,82	COMB7	0,05429	COMB11	0,01	No	No
1	B42	998	2500	COMB13	339,41	COMB7	0,05429	COMB7	0,01	Message	Message
1	B42	998	3000	COMB7	373,82	COMB7	0,05429	COMB7	0,01	No	No
1	B42	998	3500	COMB7	476,85	COMB7	0,05429	COMB7	0,01	Message	Message
1	B42	998	4000	COMB7	552,06	COMB7	0,05429	COMB7	0,01	No	No
										Message	Message

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1	B42	998	4500	COMB7	599,43	COMB7	0,05429	COMB7	0,01	No Message	No Message
1	B42	998	5000	COMB7	618,97	COMB7	0,05429	COMB7	0,01	No Message	No Message
1	B43	999	0	COMB12	628,78	COMB5	1	COMB3	0,15	No Message	No Message
1	B43	999	500	COMB12	609,24	COMB5	1	COMB5	0,17	No Message	No Message
1	B43	999	1000	COMB12	561,87	COMB5	1	COMB5	0,17	No Message	No Message
1	B43	999	1500	COMB12	486,66	COMB5	1	COMB3	0,15	No Message	No Message
1	B43	999	2000	COMB12	383,63	COMB5	1	COMB10	0,12	No Message	No Message
1	B43	999	2500	COMB13	339,41	COMB5	1	COMB7	0,16	No Message	No Message
1	B43	999	3000	COMB6	378,95	COMB5	1	COMB5	0,17	No Message	No Message
1	B43	999	3500	COMB6	481,99	COMB5	1	COMB5	0,17	No Message	No Message
1	B43	999	4000	COMB6	557,19	COMB5	1	COMB5	0,17	No Message	No Message
1	B43	999	4500	COMB6	604,57	COMB5	1	COMB5	0,17	No Message	No Message
1	B43	999	5000	COMB6	624,11	COMB5	1	COMB5	0,17	No Message	No Message

Appendix B3.2

Table B3.2.1 - Concrete Column Summary - Eurocode 2-2004 (Part 1 of 2)

Story	Label	Unique Name	Station mm	Design Section	Design/Check	Status	PMM Ratio	PMM Combo	As,min mm ²	As mm ²
Roof	C17	17	0	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C17	17	1100	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C17	17	2200	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C18	18	0	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C18	18	1100	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C18	18	2200	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C19	19	0	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C19	19	1100	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C19	19	2200	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C20	20	0	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C20	20	1100	Col 400x400	Design	No Message		COMB13	1600	1600
Roof	C20	20	2200	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C17	49	0	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C17	49	1650	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C17	49	3300	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C18	50	0	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C18	50	1650	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C18	50	3300	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C19	51	0	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C19	51	1650	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C19	51	3300	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C20	52	0	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C20	52	1650	Col 400x400	Design	No Message		COMB13	1600	1600
Attic	C20	52	3300	Col 400x400	Design	No Message		COMB13	1600	1600
10	C17	81	0	Col 400x400	Design	No Message		COMB13	1600	1600
10	C17	81	1650	Col 400x400	Design	No Message		COMB13	1600	1600
10	C17	81	3300	Col 400x400	Design	No Message		COMB13	1600	1600
10	C18	82	0	Col 400x400	Design	No Message		COMB13	1600	1600
10	C18	82	1650	Col 400x400	Design	No Message		COMB13	1600	1600
10	C18	82	3300	Col 400x400	Design	No Message		COMB13	1600	1600
10	C19	83	0	Col 400x400	Design	No Message		COMB13	1600	1600
10	C19	83	1650	Col 400x400	Design	No Message		COMB13	1600	1600
10	C19	83	3300	Col 400x400	Design	No Message		COMB13	1600	1600
10	C20	84	0	Col 400x400	Design	No Message		COMB13	1600	1600

10	C20	84	1650	Col 400x400	Design	No Message	COMB13	1600	1600
10	C20	84	3300	Col 400x400	Design	No Message	COMB13	1600	1600
9	C17	113	0	Col 400x400	Design	No Message	COMB13	1600	1600
9	C17	113	1650	Col 400x400	Design	No Message	COMB13	1600	1600
9	C17	113	3300	Col 400x400	Design	No Message	COMB13	1600	1600
9	C18	114	0	Col 400x400	Design	No Message	COMB13	1600	1600
9	C18	114	1650	Col 400x400	Design	No Message	COMB13	1600	1600
9	C18	114	3300	Col 400x400	Design	No Message	COMB13	1600	1600
9	C19	115	0	Col 400x400	Design	No Message	COMB13	1600	1600
9	C19	115	1650	Col 400x400	Design	No Message	COMB13	1600	1600
9	C19	115	3300	Col 400x400	Design	No Message	COMB13	1600	1600
9	C20	116	0	Col 400x400	Design	No Message	COMB13	1600	1600
9	C20	116	1650	Col 400x400	Design	No Message	COMB13	1600	1600
9	C20	116	3300	Col 400x400	Design	No Message	COMB13	1600	1600
8	C17	145	0	Col 400x400	Design	No Message	COMB13	1600	1600
8	C17	145	1650	Col 400x400	Design	No Message	COMB13	1600	1600
8	C17	145	3300	Col 400x400	Design	No Message	COMB13	1600	1600
8	C18	146	0	Col 400x400	Design	No Message	COMB13	1600	1600
8	C18	146	1650	Col 400x400	Design	No Message	COMB13	1600	1600
8	C18	146	3300	Col 400x400	Design	No Message	COMB13	1600	1600
8	C19	147	0	Col 400x400	Design	No Message	COMB13	1600	1600
8	C19	147	1650	Col 400x400	Design	No Message	COMB13	1600	1600
8	C19	147	3300	Col 400x400	Design	No Message	COMB13	1600	1600
8	C20	148	0	Col 400x400	Design	No Message	COMB13	1600	1600
8	C20	148	1650	Col 400x400	Design	No Message	COMB13	1600	1600
8	C20	148	3300	Col 400x400	Design	No Message	COMB13	1600	1600
7	C17	177	0	Col 400x400	Design	No Message	COMB13	1600	1600
7	C17	177	1650	Col 400x400	Design	No Message	COMB13	1600	1600
7	C17	177	3300	Col 400x400	Design	No Message	COMB13	1600	1600
7	C18	178	0	Col 400x400	Design	No Message	COMB11	1600	2033
7	C18	178	1650	Col 400x400	Design	No Message	COMB11	1600	2020
7	C18	178	3300	Col 400x400	Design	No Message	COMB11	1600	2006
7	C19	179	0	Col 400x400	Design	No Message	COMB6	1600	2033
7	C19	179	1650	Col 400x400	Design	No Message	COMB6	1600	2020
7	C19	179	3300	Col 400x400	Design	No Message	COMB6	1600	2006
7	C20	180	0	Col 400x400	Design	No Message	COMB13	1600	1600
7	C20	180	1650	Col 400x400	Design	No Message	COMB13	1600	1600

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7	C20	180	3300	Col 400x400	Design	No Message	COMB13	1600	1600
6	C17	209	0	Col 400x400	Design	No Message	COMB13	1600	1600
6	C17	209	1650	Col 400x400	Design	No Message	COMB13	1600	1600
6	C17	209	3300	Col 400x400	Design	No Message	COMB13	1600	1600
6	C18	210	0	Col 400x400	Design	No Message	COMB11	1600	4138
6	C18	210	1650	Col 400x400	Design	No Message	COMB11	1600	4121
6	C18	210	3300	Col 400x400	Design	No Message	COMB11	1600	4104
6	C19	211	0	Col 400x400	Design	No Message	COMB6	1600	4138
6	C19	211	1650	Col 400x400	Design	No Message	COMB6	1600	4121
6	C19	211	3300	Col 400x400	Design	No Message	COMB6	1600	4104
6	C20	212	0	Col 400x400	Design	No Message	COMB13	1600	1600
6	C20	212	1650	Col 400x400	Design	No Message	COMB13	1600	1600
6	C20	212	3300	Col 400x400	Design	No Message	COMB13	1600	1600
5	C17	241	0	Col 450x450	Design	No Message	COMB13	2025	2025
5	C17	241	1650	Col 450x450	Design	No Message	COMB13	2025	2025
5	C17	241	3300	Col 450x450	Design	No Message	COMB13	2025	2025
5	C18	242	0	Col 450x450	Design	No Message	COMB11	2025	2525
5	C18	242	1650	Col 450x450	Design	No Message	COMB11	2025	2507
5	C18	242	3300	Col 450x450	Design	No Message	COMB11	2025	2490
5	C19	243	0	Col 450x450	Design	No Message	COMB6	2025	2525
5	C19	243	1650	Col 450x450	Design	No Message	COMB6	2025	2507
5	C19	243	3300	Col 450x450	Design	No Message	COMB6	2025	2490
5	C20	244	0	Col 450x450	Design	No Message	COMB13	2025	2025
5	C20	244	1650	Col 450x450	Design	No Message	COMB13	2025	2025
5	C20	244	3300	Col 450x450	Design	No Message	COMB13	2025	2025
4	C17	273	0	Col 450x450	Design	No Message	COMB13	2025	2025
4	C17	273	1650	Col 450x450	Design	No Message	COMB13	2025	2025
4	C17	273	3300	Col 450x450	Design	No Message	COMB13	2025	2025
4	C18	274	0	Col 450x450	Design	No Message	COMB11	2025	4413
4	C18	274	1650	Col 450x450	Design	No Message	COMB11	2025	4390
4	C18	274	3300	Col 450x450	Design	No Message	COMB11	2025	4368
4	C19	275	0	Col 450x450	Design	No Message	COMB6	2025	4413
4	C19	275	1650	Col 450x450	Design	No Message	COMB6	2025	4390
4	C19	275	3300	Col 450x450	Design	No Message	COMB6	2025	4368
4	C20	276	0	Col 450x450	Design	No Message	COMB13	2025	2025
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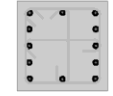
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3	C17	305	0	Col	Design	No	COMB13	2025	2025
				450x450		Message			
3	C17	305	1650	Col	Design	No	COMB13	2025	2025
				450x450		Message			
3	C17	305	3300	Col	Design	No	COMB13	2025	2025
				450x450		Message			
3	C18	306	0	Col	Design	No	COMB11	2025	6314
				450x450		Message			
3	C18	306	1650	Col	Design	No	COMB11	2025	6289
				450x450		Message			
3	C18	306	3300	Col	Design	No	COMB11	2025	6263
				450x450		Message			
3	C19	307	0	Col	Design	No	COMB6	2025	6314
				450x450		Message			
3	C19	307	1650	Col	Design	No	COMB6	2025	6289
				450x450		Message			
3	C19	307	3300	Col	Design	No	COMB6	2025	6263
				450x450		Message			
3	C20	308	0	Col	Design	No	COMB13	2025	2025
				450x450		Message			
3	C20	308	1650	Col	Design	No	COMB13	2025	2025
				450x450		Message			
3	C20	308	3300	Col	Design	No	COMB13	2025	2025
				450x450		Message			
2	C17	337	0	Col	Design	No	COMB13	2500	2500
				500x500		Message			
2	C17	337	1800	Col	Design	No	COMB13	2500	2500
				500x500		Message			
2	C17	337	3600	Col	Design	No	COMB13	2500	2500
				500x500		Message			
2	C18	338	0	Col	Design	No	COMB11	2500	4855
				500x500		Message			
2	C18	338	1800	Col	Design	No	COMB11	2500	4821
				500x500		Message			
2	C18	338	3600	Col	Design	No	COMB11	2500	4786
				500x500		Message			
2	C19	339	0	Col	Design	No	COMB6	2500	4855
				500x500		Message			
2	C19	339	1800	Col	Design	No	COMB6	2500	4821
				500x500		Message			
2	C19	339	3600	Col	Design	No	COMB6	2500	4786
				500x500		Message			
2	C20	340	0	Col	Design	No	COMB13	2500	2500
				500x500		Message			
2	C20	340	1800	Col	Design	No	COMB13	2500	2500
				500x500		Message			
2	C20	340	3600	Col	Design	No	COMB13	2500	2500
				500x500		Message			
1	C17	369	0	Col	Design	No	COMB11	2500	4168
				500x500		Message			
1	C17	369	1100	Col	Design	No	COMB11	2500	4156
				500x500		Message			
1	C17	369	2200	Col	Design	No	COMB11	2500	4144
				500x500		Message			
1	C18	370	0	Col	Design	No	COMB6	2500	7851
				500x500		Message			
1	C18	370	1100	Col	Design	No	COMB6	2500	7830
				500x500		Message			
1	C18	370	2200	Col	Design	No	COMB6	2500	7809
				500x500		Message			
1	C19	371	0	Col	Design	No	COMB11	2500	7851
				500x500		Message			
1	C19	371	1100	Col	Design	No	COMB11	2500	7830
				500x500		Message			
1	C19	371	2200	Col	Design	No	COMB11	2500	7809
				500x500		Message			
1	C20	372	0	Col	Design	No	COMB6	2500	4168
				500x500		Message			
1	C20	372	1100	Col	Design	No	COMB6	2500	4156
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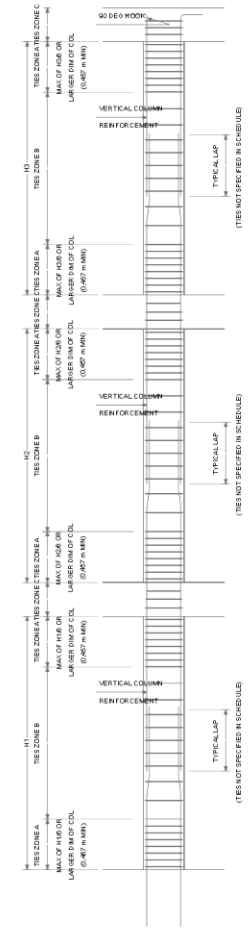
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Concrete Column Schedule

	CC1			CC2			CC3			CC4			CC5			CC6			CC7					
	COLUMN SIZE	SECTION	REINFORCING	TIES ZONE-A	TIES ZONE-B	TIES ZONE-C	COLUMN SIZE	SECTION	REINFORCING	TIES ZONE-A	TIES ZONE-B	TIES ZONE-C	COLUMN SIZE	SECTION	REINFORCING	TIES ZONE-A	TIES ZONE-B	TIES ZONE-C	COLUMN SIZE	SECTION	REINFORCING	TIES ZONE-A	TIES ZONE-B	TIES ZONE-C
Roof																								
Attic			12-14 (1 600)J0																					
10																								
9																								
8	400 MMx400 MM		12-14 (1 600)J0				400 MMx400 MM		12-14 (1 600)J0				400 MMx400 MM		12-14 (1 600)J0				400 MMx400 MM		12-14 (1 600)J0			
7			12-14 (1 600)J0						12-14 (1 600)J0						12-14 (1 600)J0						12-14 (1 600)J0			
6		A	12-16 (2 025)J0	100@100 MM	100@100 MM	100@100 MM		A	12-14 (1 600)J0	100@100 MM	100@100 MM	100@100 MM		A	12-14 (1 600)J0	100@100 MM	100@100 MM	100@100 MM		A	12-16 (2 025)J2	100@100 MM	100@100 MM	100@100 MM
5																								
4	400 MMx450 MM		12-16 (2 025)J0				400 MMx450 MM		12-16 (2 025)J0				400 MMx450 MM		12-20 (3 204)J1				400 MMx450 MM		12-20 (3 204)J1			
3			12-16 (2 025)J0						12-16 (2 025)J0						12-25 (4 973)J5						12-25 (4 973)J5			
2			12-30 (3 195)J0						12-16 (2 025)J0						12-25 (4 973)J5						12-25 (4 973)J5			
1	400 MMx500 MM		12-30 (3 195)J0				400 MMx500 MM		12-18 (2 500)J0				400 MMx500 MM		12-28 (7 258)J4				400 MMx500 MM		12-32 (7 950)J5			
Base																								



Column Section-A



Concrete Column Typical Elevation - Special

CONSULTANTS

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EASY ENGINEERING COMPANY

DESIGN TEAM MEMBERS

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YERNIYAZ ABILDIN

Design and Energy performance of student resident building integrated with Phase Change Material in Astana, Kazakhstan

Capstone project

DRAWING

Column Schedule

STRUCTURE

Dormitory

CLIENT

Nazarbayev University

PROJECT NUMBER: 1

DATE: _____

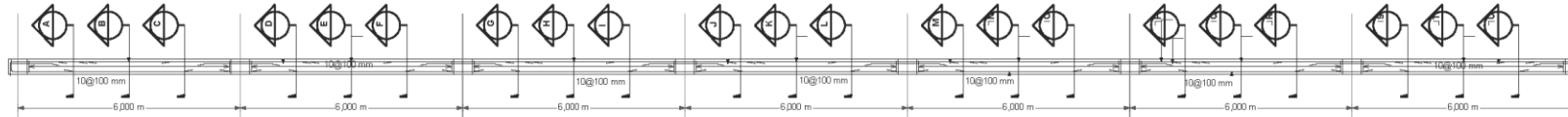
DESIGNED BY: Yerniyaz Abildin

CHECKED BY: Gulzhan Negmetova

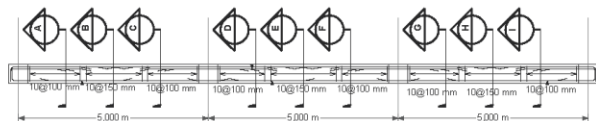
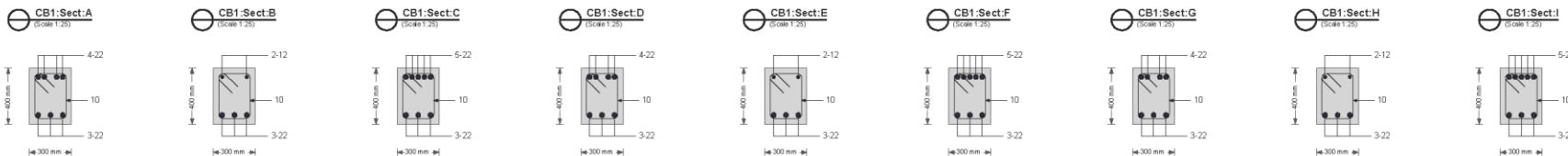
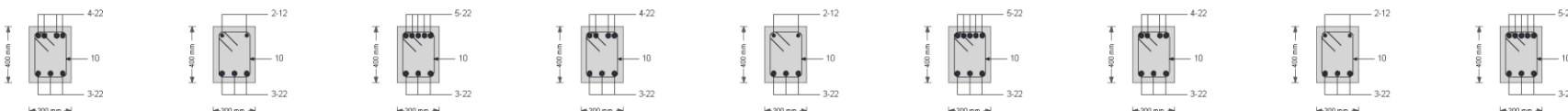
S-05

SCALE: _____

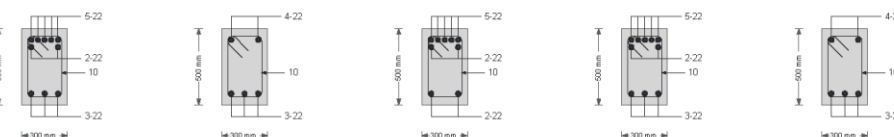
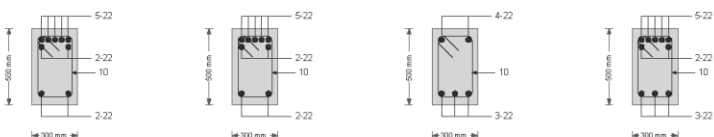
PROJECT NUMBER	1
DATE	
DESIGNED BY	Yemiyaz Abildin
CHECKED BY	Gulzhan Negmetova



CB1:Elevation
(Scale 1:100)



CB2:Elevation
(Scale 1:100)



**DR. SHAZIM MEMON
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**EASY
ENGINEERING
COMPANY**

**BAKHYTBEK MAKHIN
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YERKEZHAN MADENOVA
YERNIYAZ ABILDIN**

*Design and Energy performance of
student resident building integrated
with Phase Change Material in
Astana, Kazakhstan*

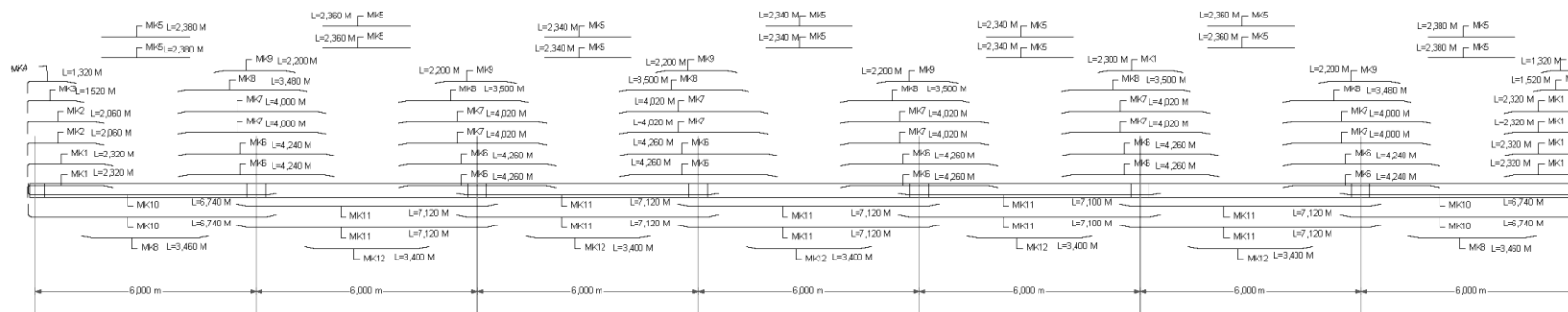
Capstone project

**Beam
Reinforcement:
Level 1**

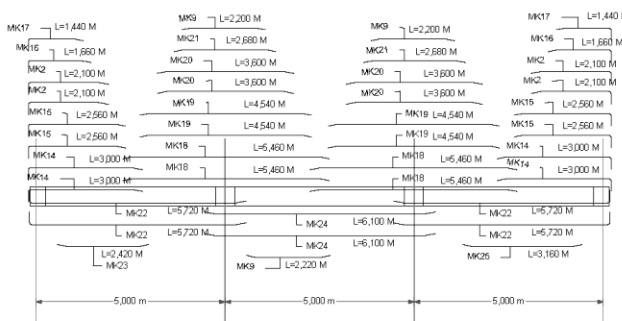
Dormitory

**Nazarbayev
University**

PROJECT NUMBER	1
DATE	
DESIGNED BY	Yerniyaz Abildin
CHECKED BY	Gulzhan Negmetova
S-07	
SCALE	



CB1: Elevation - All Bars
(Scale 1:100)



CB2: Elevation - All Bars
(Scale 1:100)

REBAR SHAPE CODES

CODE	SHAPE
00	
11	
12	
13	
14	
51	
52	
62	
63	

BEAMS REBAR TABLE

SR. NO.	BAR MARK	DIA.	SHAP E CODE	CUT LENGTH (M)	NUMBERS	TOTAL LENGTH (M)
1	MK1	22	11	2,620	28	73,380
2	MK2	22	11	2,380	40	94,820
3	MK3	22	11	1,820	8	14,560
4	MK4	22	11	1,620	8	12,960
5	MK5	12	00	2,380	56	133,280
6	MK6	22	00	4,260	48	204,060
7	MK7	22	00	4,020	48	192,880
8	MK8	22	00	3,500	32	111,900
9	MK9	22	00	2,220	44	97,680
10	MK10	22	11	7,040	16	112,720
11	MK11	22	00	7,120	40	284,780
12	MK12	22	00	3,400	20	68,020
13	MK13	10	52	1,360	1491	2040,760
14	MK14	22	11	3,320	32	106,100
15	MK15	22	11	2,860	32	91,700
16	MK16	22	11	1,960	16	31,440
17	MK17	22	11	1,760	16	28,000
18	MK18	22	00	5,460	32	175,000
19	MK19	22	00	4,540	32	145,340
20	MK20	22	00	3,620	32	115,700
21	MK21	22	00	2,680	16	43,040
22	MK22	22	11	6,040	32	193,140
23	MK23	22	00	2,420	8	19,380
24	MK24	22	00	6,120	16	97,920
25	MK25	22	00	3,160	8	25,300
26	MK26	10	52	1,520	842	1287,140

Appendix B3.3

Table B3.3.1 Slab design loads

Loads, kN/m ²	Unfactored		Factored		Combination	
	Dead	Live	Dead	Live	ULS	SLS
roof	4.712	0.5	6.361	0.750	7.111	5.212
attic	7.936	2	10.713	3.000	13.713	9.936
typical floor	13.110	2	17.699	3.000	20.699	15.110
ground floor	14.802	5	19.983	7.500	27.483	19.802

Table B3.3.2 Slab design coefficients

Panel type	Short span coefficient		Long span coefficient		Short span coefficient		Long span coefficient	
	Cont. edge negative M	Center positive M	Cont. edge negative M	Center positive M	Continuous edge V	Discontinuous edge V	Continuous edge V	Discontinuous edge V
interior	0.042	0.032	0.032	0.024	0.39		0.33	
1 short edge disc	0.048	0.036	0.037	0.028	0.42		0.36	0.24
1 long edge disc	0.056	0.042	0.037	0.028	0.44	0.29	0.36	
corner	0.063	0.047	0.045	0.034	0.47	0.31	0.4	0.26

Table B3.3.3 Slab design dimensions

slab dimensions	roof	other
h	150	200
d1	120	169
d2	110	157

Table B3.3.4 Slab design details

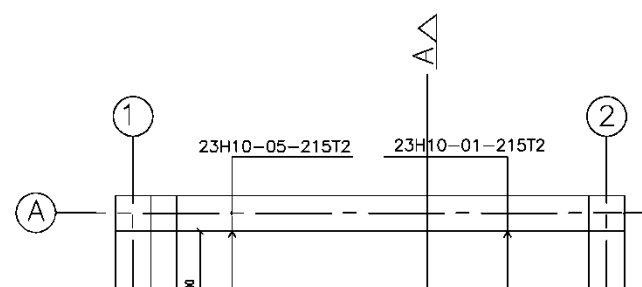
Level	Panel type	Location	Span	Moment M, kNm/m	k	z/d	As required, mm ² /m	As provided, mm ² /m	Spacing, s	Adjusted spacing	As min	Edge shear force, kN	Shear capacity, kN
roof	interior	cont support	short	7.46630812	0.01234	0.741	211	262	307	300	256	13.8660008	149.9396494
As max			long	5.68861571	0.00940	0.743	160	262	307	300	156	11.73276991	
4800		center	short	5.68861571	0.00940	0.743	160	265	196	190	8 at 190		
			long	4.26646178	0.00705	0.745	120	265	196	190			
	corner	cont support	short	11.1994622	0.01851	0.736	318	327	247	240		16.71030865	187.4245617
			long	7.99961584	0.01322	0.740	226	262	307	300		14.22153928	149.9396494
		center	short	8.35515433	0.01381	0.740	236	265	196	190			
			long	6.04415419	0.00999	0.743	170	265	196	190			
		free edge	short				118	265	196	190		11.02169294	151.5179615
			long				85	265	196	190		9.244000531	151.5179615
	1 short edge disc	cont support	short	8.53292357	0.01410	0.739	241	262	307	300		14.93261624	149.9396494
			long	6.57746192	0.01087	0.742	185	262	307	300		12.79938535	149.9396494
		center	short	6.39969268	0.01058	0.742	180	265	196	190			
			long	4.97753875	0.00823	0.744	140	265	196	190			
		free edge	short				90	265	196	190		8.532923567	151.5179615
	1 long edge disc	cont support	short	9.9550775	0.01645	0.738	282	291	278	270		15.64369321	166.5996104
			long	6.57746192	0.01087	0.742	185	262	307	300		12.79938535	149.9396494
		center	short	7.46630812	0.01234	0.741	211	265	196	190			
			long	4.97753875	0.00823	0.744	140	265	196	190			
		free edge	long				70	265	196	190		10.31061598	151.5179615

Level	Panel type	Location	Span	Moment M, kNm/m	k	z/d	As required, mm ² /m	As provided, mm ² /m	Spacing, s	Adjusted spacing	As min	Edge shear force, kN	Shear capacity, kN
attic	interior	cont support	short	14.3988182	0.01168	0.741	285	365	218	215	360	26.74066236	179.3932703
As max			long	10.9705281	0.00890	0.743	216	365	218	215	219.7	22.62671431	179.3932703
6760		center	short	10.9705281	0.00890	0.743	216	365	218	215	10 at 215		
			long	8.22789611	0.00668	0.745	162	365	218	215			
	corner	cont support	short	21.5982273	0.01752	0.737	429	436	183	180		32.22592643	214.2752951
			long	15.4273052	0.01252	0.741	305	365	218	215		27.42632037	179.3932703
		center	short	16.1129632	0.01307	0.740	319	365	218	215			
			long	11.6561862	0.00946	0.743	230	365	218	215			
		free edge	short				159	365	218	215		21.25539829	179.3932703
			long				115	365	218	215		17.82710824	179.3932703
	1 short edge disc	cont support	short	16.4557922	0.01335	0.740	326	365	218	215		28.79763639	179.3932703
			long	12.6846732	0.01029	0.742	250	365	218	215		24.68368833	179.3932703
		center	short	12.3418442	0.01001	0.742	244	365	218	215			
			long	9.59921213	0.00779	0.744	189	365	218	215			
		free edge	short				122	365	218	215		16.45579222	
	1 long edge disc	cont support	short	19.1984243	0.01558	0.738	381	393	206	200		30.16895241	192.8477656
			long	12.6846732	0.01029	0.742	250	365	218	215		24.68368833	179.3932703
		center	short	14.3988182	0.01168	0.741	285	365	218	215			
			long	9.59921213	0.00779	0.744	189	365	218	215			179.3932703
		free edge	long				94	365	218	215		19.88408227	
Level	Panel type	Location	Span	Moment M, kNm/m	k	z/d	As required, mm ² /m	As provided, mm ² /m	Spacing, s	Adjusted spacing	As min	Edge shear force, kN	Shear capacity, kN

							mm ² /m	mm ² /m					
typical	interior	cont support	short	21.7340558	0.01763	0.737	432	436	182	180	360	40.36324643	214.2752951
As max			long	16.5592806	0.01344	0.740	328	365	218	215	219.7	34.15351621	179.3932703
6760		center	short	16.5592806	0.01344	0.740	328	365	218	215	10 at 215		
			long	12.4194604	0.01008	0.742	245	365	218	215			
	corner	cont support	short	32.6010837	0.02645	0.730	654	654	120	120		48.64288673	321.4129427
			long	23.2864883	0.01889	0.736	464	476	169	165		41.39820147	233.7548674
		center	short	24.3214434	0.01973	0.735	485	491	162	160			
			long	17.5942356	0.01428	0.739	349	365	218	215			
		free edge	short				242	365	218	215		32.08360614	179.3932703
			long				174	365	218	215		26.90883096	179.3932703
	1 short edge disc	cont support	short	24.8389209	0.02015	0.735	495	507	159	155		43.46811154	248.8358266
			long	19.1466682	0.01554	0.738	380	393	207	200		37.25838132	192.8477656
		center	short	18.6291907	0.01512	0.739	369	374	213	210			
			long	14.4893705	0.01176	0.741	286	365	218	215			
		free edge	short				185	365	218	215		24.83892088	
	1 long edge disc	cont support	short	28.978741	0.02351	0.732	580	582	135	135		45.53802162	285.7003935
			long	19.1466682	0.01554	0.738	380	393	207	200		37.25838132	192.8477656
		center	short	21.7340558	0.01763	0.737	432	436	182	180			
			long	14.4893705	0.01176	0.741	286	365	218	215			
		free edge	long				143	365	218	215		30.01369607	179.3932703
Level	Panel type	Location	Span	Moment M, kNm/m	k	z/d	As required, mm²/m	As provided, mm²/m	Spacing, s	Adjusted spacing	As min	Edge shear force, kN	Shear capacity, kN
ground	interior	cont	short	28.8575316	0.02341	0.732	577	582	136	135	360	53.59255875	285.7003935

B1.1

		support											
As max			long	21.9866908	0.01784	0.737	437	449	180	175	219.7	45.34754971	220.3974464
6760		center	short	21.9866908	0.01784	0.737	437	449	180	175	10 at 215		
			long	16.4900181	0.01338	0.740	326	365	218	215			
	corner	cont support	short	43.2862975	0.03512	0.724	876	905	129	125	250	64.58590414	444.3212519
			long	30.9187839	0.02509	0.731	619	628	127	125		54.96672693	308.556425
		center	short	32.2929521	0.02620	0.730	648	654	121	120			
			long	23.3608589	0.01895	0.736	465	462	169	170			
		free edge	short				324	365	218	215		42.59921337	179.3932703
			long				233	365	218	215		35.7283725	179.3932703
	1 short edge disc	cont support	short	32.9800362	0.02676	0.730	662	683	119	115		57.71506327	335.3874184
			long	25.4221112	0.02063	0.735	507	507	155	155		49.47005423	248.8358266
		center	short	24.7350271	0.02007	0.735	493	507	159	155			
			long	19.2383544	0.01561	0.738	382	393	206	200			
		free edge	short				247	365	218	215		32.98003616	179.3932703
			cont support	short	38.4767088	0.03122	0.727	776	785	101	100		60.46339962



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Appendix B4

Table B4.1.1 Deflection calculation for slab

level	panel	slab effective depth	short span center reinforcement, mm ² /m	K	reinforcement ratio	limiting l/d	actual l/d		
roof	interior	110	265	1.5	0.00241	154.57	45.45		
	corner		265	1.3	0.00241	133.96			
	edge (long edge dsc)		265	1.3	0.00241	133.96			
	edge (short disc)		265	1.3	0.00241	133.96			
attic	interior	157	365	1.5	0.00232	163.90	31.85		
	corner		365	1.3	0.00232	142.04			
	edge (long edge dsc)		365	1.3	0.00232	142.04			
	edge (short disc)		365	1.3	0.00232	142.04			
typical	interior	157	365	1.5	0.00232	163.90		31.85	
	corner		491	1.3	0.00313	87.13			
	edge (long edge dsc)		436	1.3	0.00278	105.97			
	edge (short disc)		374	1.3	0.00238	136.46			
ground	interior	157	449	1.5	0.00286	116.49			31.85
	corner		654	1.3	0.00417	54.84			
	edge (long edge dsc)		582	1.3	0.00371	66.03			
	edge (short disc)		507	1.3	0.00323	82.66			

Appendix C-1

Table 1. Interpolated values of N_{σ}^* based on Meyerhof's Theory (Das, 2007: pp.558)

Soil friction angle, ϕ (deg)	N_{σ}^*
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
--	---

Table 2. Bearing capacity factor N_{σ}^* r based on the theory of expansion of cavities (Das, 2007: pp562)

Table 11.7 Bearing Capacity Factors N_q^* Based on the Theory of Expansion of Cavities

ϕ'	I_{cr}							
	10	20	40	60	80	100	200	300
25	12.12	15.95	20.98	24.64	27.61	30.16	39.70	46.61
26	13.18	17.47	23.15	27.30	30.69	33.60	44.53	52.51
27	14.33	19.12	25.52	30.21	34.06	37.37	49.88	59.05
28	15.57	20.91	28.10	33.40	37.75	41.51	55.77	66.29
29	16.90	22.85	30.90	36.87	41.79	46.05	62.27	74.30
30	18.24	24.95	33.95	40.66	46.21	51.02	69.43	83.14
31	19.88	27.22	37.27	44.79	51.03	56.46	77.31	92.90
32	21.55	29.68	40.88	49.30	56.30	62.41	85.96	103.66
33	23.34	32.34	44.80	54.20	62.05	68.92	95.46	115.51
34	25.28	35.21	49.05	59.54	68.33	76.02	105.90	128.55
35	27.36	38.32	53.67	65.36	75.17	83.78	117.33	142.89
36	29.60	41.68	58.68	71.69	82.62	92.24	129.87	158.65
37	32.02	45.31	64.13	78.57	90.75	101.48	143.61	175.95
38	34.63	49.24	70.03	86.05	99.60	111.56	158.65	194.94
39	37.44	53.50	76.45	94.20	109.24	122.54	175.11	215.78
40	40.47	58.10	83.40	103.05	119.74	134.52	193.13	238.62
41	43.74	63.07	90.96	112.68	131.18	147.59	212.84	263.67
42	47.27	68.46	99.16	123.16	143.64	161.83	234.40	291.13
43	51.08	74.30	108.08	134.56	157.21	177.36	257.99	321.22
44	55.20	80.62	117.76	146.97	172.00	194.31	283.80	354.20
45	59.66	87.48	128.28	160.48	188.12	212.79	312.03	390.35

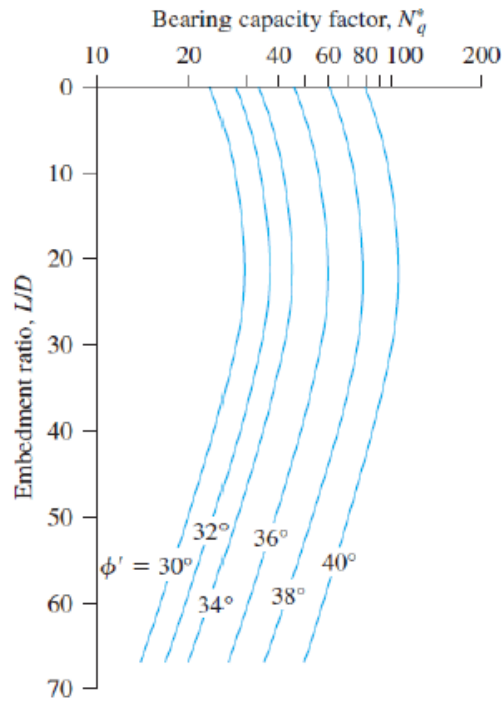


Figure 1. Variation of N_q^* with L/D (Das, 2007: pp564)

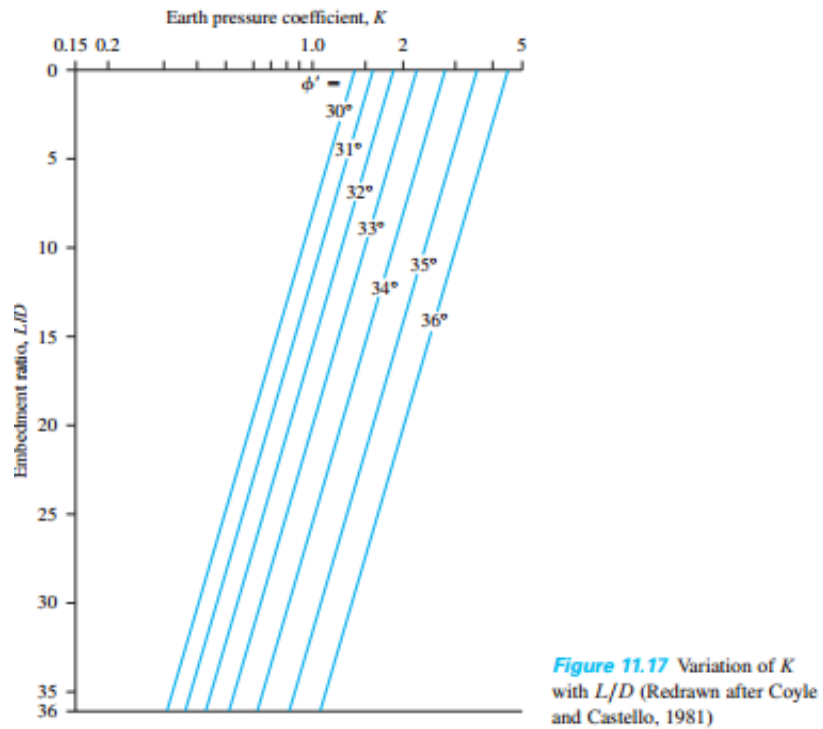


Figure 2. Variation of K with L/D (Das, 2007: pp. 571)

Table 3. Variation of λ with pile embedment length, L (Das, 2007: pp. 576)

	Embedment length, L (m)	λ
Depth	0	0.5
	5	0.336
	10	0.245
	15	0.200
	20	0.173
	25	0.150
	30	0.136
	35	0.132
	40	0.127
	50	0.118
	60	0.113
	70	0.110
	80	0.110
90	0.110	

Table 4. Variation of α (Interpolation values based on Terzaghi, Peck and Mersi, 1996) (Das, 2007: pp. 577)

	$\frac{c_u}{p_a}$	α
eS	≤ 0.1	1.00
	0.2	0.92
	0.3	0.82
	0.4	0.74
	0.6	0.62
	0.8	0.54
	1.0	0.48
	1.2	0.42
	1.4	0.40
	1.6	0.38
	1.8	0.36
	2.0	0.35
	2.4	0.34
2.8	0.34	

Note: p_a = atmospheric pressure
 $\approx 100 \text{ kN/m}^2$

Table 5. Representative values of n_h (Das, 2007: pp. 595)

Table 11.15 Representative Values of n_h

Soil	n_h kN/m³
Dry or moist sand	
Loose	1800–2200
Medium	5500–7000
Dense	15,000–18,000
Submerged sand	
Loose	1000–1400
Medium	3500–4500
Dense	9000–12,000

Appendix C-2

Results of cone penetration test (Technical report provided by “KaragandaGIIZ and K”
firm, 2015)

№№	Borehole number	Penetration points	Testing interval, m	Unit cone resistance, MPa	Average cone resistance, MPa	Unit sleeve friction, kPa	Average sleeve friction, kPa
1	2	3	4	5	6	7	8
Fill							
1	222-15	1	0,0-0,4	1,5-4,3	2,6	79-255	153
2	224-15	2	0,0-0,5	1,1-3,8	2,1	111-300	218
3		3	0,0-0,4	1,1-1,5	1,3	67-93	77
4		4	0,0-0,6	1,0-12,5	6,7	127-187	150
5	228-15	5	0,0-3,0	0,8-25,0	3,6	14-384	93
6	219-15	6	0,0-0,4	1,8-2,6	2,1	103-168	129
7	225-15	7	0,0-1,8	0,7-1,5	1,0	15-89	45
8	226-15	8	0,0-1,50	1,1-3,3	1,9	46-146	91
9	220-15	9	0,0-0,4	1,8-7,7	3,9	122-228	181
10	227-15	10	0,0-0,6	2,1-9,5	4,9	50-216	128
11	223-15	11	0,0-0,50	1,2-1,3	1,3	55-62	58
12	221-15	12	0,0-0,8	1,3-4,5	2,2	96-221	144
			n		12		12
			min		1,0		45
			max		6,7		218
			A		2,8		122
1	2	3	4	5	6	7	8
Loam							
1	222-15	1	0,4-5,6	0,2-7,9	1,8	22-98	48
2	224-15	2	0,5-5,3	0,2-4,3	1,2	19-101	44
3		3	0,4-5,4	0,7-4,1	1,6	10-94	44
4		4	0,0-0,6	0,6-4,0	1,2	17-96	38
5	228-15	5	3,0-5,0	0,8-6,1	2,5	12-65	41
6	219-15	6	0,4-5,0	0,6-4,5	1,2	14-81	38
7	225-15	7	1,8-5,4	0,8-8,9	1,6	14-58	35
8	226-15	8	1,5-4,5	0,5-2,0	1,2	24-81	47
9	220-15	9	0,4-5,5	0,6-2,7	1,1	14-93	34
10	227-15	10	0,6-4,6	0,9-4,0	1,6	2-29	10
11	223-15	11	0,5-4,8	0,8-5,5	1,8	21-62	35
12	221-15	12	0,8-5,2	0,6-5,7	1,6	19-77	46
			n		12		12
			min		1,1		10
			max		2,5		48
			A		1,5		38

Medium dense sand							
1	222-15	1	5,6-6,2	16,0-25,0	20,1	101-195	147
2		3	5,4-6,4	1,1-10,6	5,9	41-168	97
3		4	4,0-4,8	17,0-25,0	19,3	87-209	168
4	226-15	8	4,5-6,5	4,0-18,0	11,3	34-86	59
5	220-15	9	5,5-7,0	1,3-8,7	5,0	21-50	33
6	227-15	10	4,6-7,0	1,2-20,8	9,1	5-72	21
			n		6		6
			min		5,0		21
			max		20,1		168
			A		11,8		87
Coarse sand							
1	224-15	2	5,3-6,4	1,1-25,0	13,2	48-281	129
2		3	6,4-6,8	17,7-25,0	21,3	108-247	177
3	225-15	7	5,4-6,2	19,4-24,0	21,0	7-146	115
			n		3		3
			min		13,2		115
			max		21,3		177
			A		18,5		140

Appendix C-3

Table 1. Minimum cover to reinforcement (BS 8110-1-1997, Clause 3.3.7: pp.21)

Table 3.3 — Nominal cover to all reinforcement (including links) to meet durability requirements (see NOTE 1)

Conditions of exposure (see 3.3.4)	Nominal cover Dimensions in millimetres				
	Mild	25	20	20 ^a	20 ^a
Moderate	—	35	30	25	20
Severe	—	—	40	30	25
Very severe	—	—	50 ^b	40 ^b	30
Most severe	—	—	—	—	50
Abrasive	—	—	—	See NOTE 3	See NOTE 3
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content (kg/m ³)	275	300	325	350	400
Lowest grade of concrete	C30	C35	C40	C45	C50

NOTE 1 This table relates to normal-weight aggregate of 20 mm nominal size. Adjustments to minimum cement contents for aggregates other than 20 mm nominal maximum size are detailed in Table 8 of BS 5328-1:1997.

NOTE 2 Use of sulfate resisting cement conforming to BS 4027. These cements have lower resistance to chloride ion migration. If they are used in reinforced concrete in very severe or most severe exposure conditions, the covers in Table 3.3 should be increased by 10 mm.

NOTE 3 Cover should be not less than the nominal value corresponding to the relevant environmental category plus any allowance for loss of cover due to abrasion.

^a These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm.

^b Where concrete is subject to freezing whilst wet, air-entrainment should be used (see 5.3.3 of BS 5328-1:1997) and the strength grade may be reduced by 5.

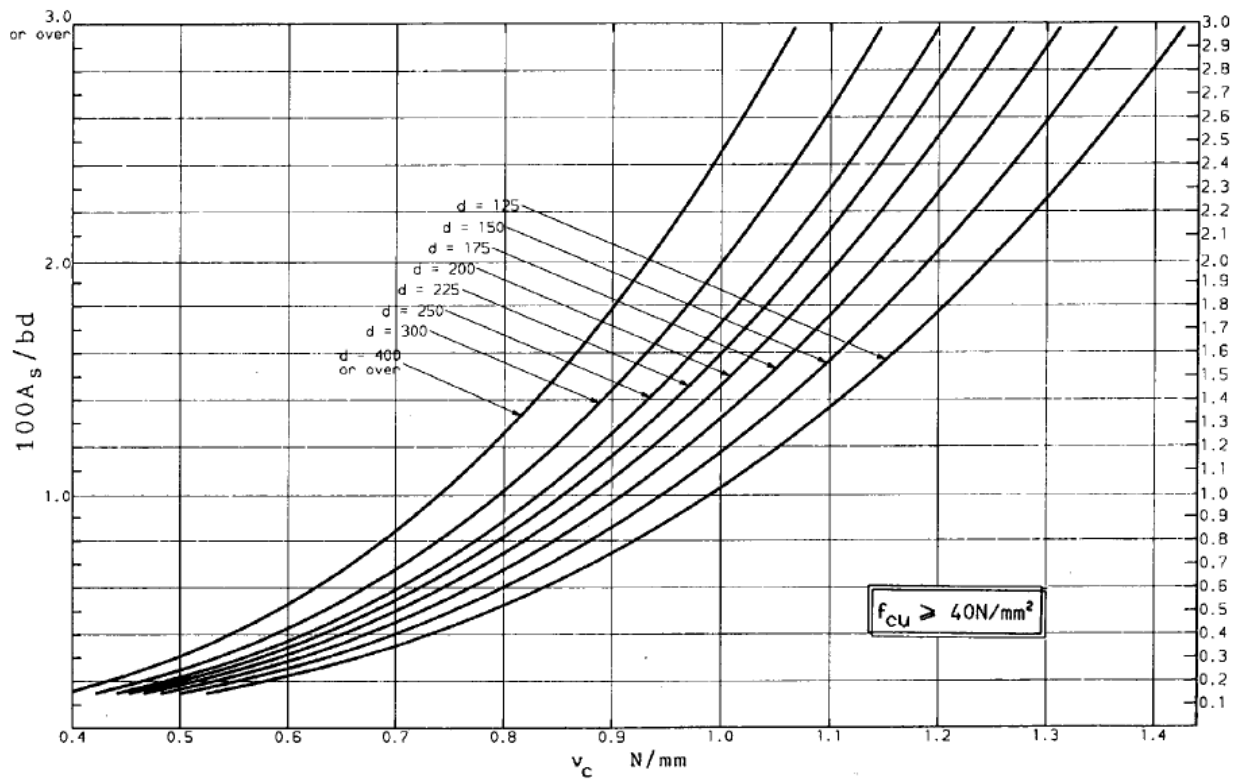


Figure 1. Values of v_c – design concrete shear stress (Ray, 1995: pp.492)

Table 2. $f_{cu} = 50 N/mm^2$, $k = 0.8$ (Ray, 1995: pp.517)

eh	p=0.4		p=1.0		p=2.0		p=3.0		p=4.0		p=5.0		p=6.0	
	Nbh	x/h	Nbh	x/h	Nbh	x/h	Nbh	x/h	Nbh	x/h	Nbh	x/h	Nbh	x/h
0.05	21.41	1.011	23.37	1.025	26.65	1.045	29.93	1.062	33.23	1.076	36.53	1.089	39.84	1.100
0.10	19.06	0.901	20.85	0.916	23.82	0.936	26.78	0.951	29.74	0.963	32.70	0.974	35.67	0.983
0.15	16.77	0.795	18.44	0.815	21.16	0.838	23.86	0.855	26.54	0.868	29.21	0.878	31.88	0.887
0.20	14.58	0.696	16.21	0.725	18.77	0.755	21.26	0.776	23.72	0.790	26.16	0.802	28.58	0.811
0.25	12.66	0.607	14.23	0.649	16.70	0.688	19.03	0.712	21.30	0.729	23.55	0.742	25.78	0.752
0.30	10.79	0.532	12.54	0.587	14.94	0.635	17.14	0.663	19.26	0.681	21.35	0.695	23.41	0.706
0.35	9.06	0.451	11.14	0.539	13.47	0.593	15.55	0.623	17.54	0.644	19.49	0.659	21.41	0.670
0.40	7.29	0.370	9.89	0.492	12.24	0.559	14.21	0.582	16.09	0.614	17.91	0.629	19.70	0.641
0.45	5.78	0.304	8.45	0.427	11.20	0.533	13.08	0.567	14.85	0.589	16.56	0.606	18.24	0.618
0.50	4.59	0.253	7.18	0.375	10.32	0.511	12.11	0.546	13.78	0.569	15.40	0.586	16.98	0.599
0.55	3.71	0.218	6.13	0.334	9.29	0.483	11.27	0.529	12.85	0.553	14.38	0.570	15.88	0.583
0.60	3.07	0.194	5.29	0.303	8.25	0.425	10.53	0.515	12.04	0.539	13.48	0.556	14.91	0.569
0.65	2.60	0.178	4.62	0.280	7.37	0.394	9.77	0.486	11.33	0.527	12.71	0.544	14.05	0.557
0.70	2.24	0.166	4.08	0.262	6.63	0.369	8.93	0.452	10.69	0.516	12.01	0.534	13.29	0.547
0.75	1.97	0.157	3.65	0.247	6.00	0.349	8.15	0.427	10.09	0.502	11.38	0.525	12.60	0.538
0.80	1.76	0.150	3.29	0.236	5.47	0.333	7.48	0.406	9.40	0.488	10.81	0.517	11.99	0.530
0.85	1.59	0.145	2.99	0.227	5.02	0.319	6.90	0.388	8.71	0.447	10.30	0.510	11.42	0.523
0.90	1.44	0.141	2.74	0.220	4.63	0.308	6.40	0.374	8.10	0.429	9.72	0.483	10.91	0.517
0.95	1.33	0.137	2.53	0.214	4.30	0.298	5.96	0.361	7.58	0.414	9.13	0.460	10.45	0.512
1.00	1.22	0.135	2.35	0.209	4.01	0.290	5.57	0.351	7.08	0.401	8.57	0.445	9.96	0.495
1.10	1.06	0.130	2.05	0.201	3.52	0.277	4.92	0.334	6.28	0.380	7.82	0.420	8.94	0.456
1.20	0.94	0.126	1.82	0.195	3.14	0.267	4.40	0.321	5.53	0.364	6.84	0.401	8.05	0.434
1.30	0.83	0.121	1.64	0.190	2.83	0.260	3.98	0.310	5.10	0.351	6.20	0.386	7.30	0.417
1.40	0.75	0.117	1.49	0.186	2.58	0.254	3.63	0.302	4.65	0.341	5.67	0.374	6.68	0.403
1.50	0.68	0.114	1.36	0.183	2.37	0.249	3.33	0.295	4.28	0.333	5.22	0.364	6.16	0.392
1.60	0.63	0.111	1.25	0.181	2.19	0.244	3.08	0.290	3.96	0.326	4.83	0.356	5.70	0.382
1.70	0.58	0.108	1.16	0.178	2.03	0.241	2.86	0.285	3.69	0.320	4.50	0.349	5.31	0.374
1.80	0.54	0.105	1.09	0.177	1.90	0.238	2.68	0.281	3.45	0.315	4.21	0.343	4.97	0.368
1.90	0.50	0.105	1.02	0.175	1.78	0.235	2.51	0.278	3.23	0.311	3.95	0.338	4.67	0.362
2.00	0.47	0.103	0.96	0.174	1.67	0.233	2.37	0.274	3.05	0.307	3.73	0.334	4.40	0.357
2.20	0.42	0.100	0.85	0.171	1.50	0.229	2.12	0.269	2.73	0.301	3.34	0.326	3.95	0.348
2.40	0.42	0.100	0.85	0.171	1.50	0.229	2.12	0.269	2.73	0.301	3.34	0.326	3.95	0.348
2.60			0.77	0.169	1.36	0.226	1.92	0.265	2.48	0.296	3.03	0.321	3.58	0.342
2.80			0.70	0.168	1.24	0.224	1.75	0.262	2.26	0.292	2.77	0.316	3.27	0.336
3.00			0.65	0.167	1.14	0.222	1.61	0.259	2.08	0.288	2.55	0.312	3.02	0.332
3.20			0.60	0.166	1.05	0.220	1.49	0.257	1.93	0.285	2.36	0.309	2.79	0.328
3.40			0.56	0.165	0.98	0.218	1.39	0.255	1.80	0.283	2.20	0.306	2.60	0.325
3.60			0.52	0.164	0.92	0.217	1.30	0.253	1.68	0.281	2.06	0.303	2.44	0.322
3.80			0.49	0.163	0.86	0.216	1.22	0.252	1.58	0.279	1.94	0.301	2.29	0.320
4.00			0.46	0.163	0.81	0.215	1.15	0.251	1.49	0.278	1.83	0.299	2.16	0.318
5.00			0.44	0.162	0.77	0.214	1.09	0.249	1.41	0.276	1.73	0.298	2.05	0.316
6.00			0.34	0.160	0.61	0.211	0.86	0.245	1.11	0.271	1.36	0.292	1.61	0.309
7.00			0.28	0.159	0.50	0.209	0.71	0.242	0.92	0.268	1.12	0.288	1.33	0.305
8.00			0.24	0.158	0.42	0.208	0.60	0.241	0.78	0.265	0.96	0.285	1.13	0.302
10.00			0.21	0.157	0.37	0.206	0.53	0.239	0.68	0.264	0.83	0.283	0.99	0.299
			0.17	0.156	0.29	0.205	0.42	0.237	0.54	0.261	0.66	0.281	0.78	0.296

Table 3. ultimate anchorage bond length and lap lengths as multiples of bar size (BS 8110-1-1997, Clause 3.12.8.15: pp.82)

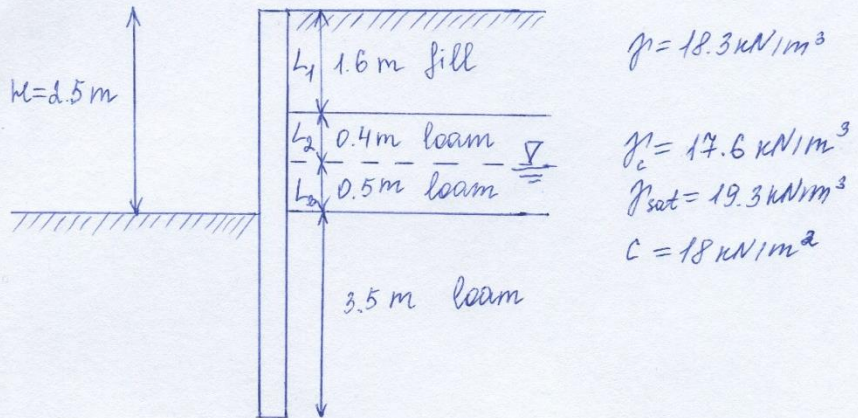
Table 3.27 — Ultimate anchorage bond lengths and lap lengths as multiples of bar size

Reinforcement types	Grade 250 plain	Grade 400			
		Plain	Deformed type 1	Deformed type 2	Fabric
Concrete cube strength 25					
Tension anchorage and lap length	43	79	55	44	34
1.4 × tension lap	60	110	77	62	48
2.0 × tension lap	85	157	110	88	68
Compression anchorage length	34	63	44	35	28
Compression lap length	43	79	55	44	34
Concrete cube strength 30					
Tension anchorage and lap length	39	72	50	40	31
1.4 × tension lap	55	100	70	56	44
2.0 × tension lap	78	143	100	80	62
Compression anchorage length	32	58	40	32	25
Compression lap length	39	72	50	40	31
Concrete cube strength 35					
Tension anchorage and lap length	36	67	47	38	29
1.4 × tension lap	51	93	65	52	40
2.0 × tension lap	72	133	93	75	57
Compression anchorage length	29	53	38	30	23
Compression lap length	36	67	47	38	29
Concrete cube strength 40					
Tension anchorage and lap length	34	62	44	35	27
1.4 × tension lap	48	87	61	49	38
2.0 × tension lap	68	124	81	70	54
Compression anchorage length	27	50	35	28	22
Compression lap length	34	62	44	35	27
NOTE The values are rounded up to the nearest whole number and the length derived from these values may differ slightly from those calculated directly for each bar or wire size.					

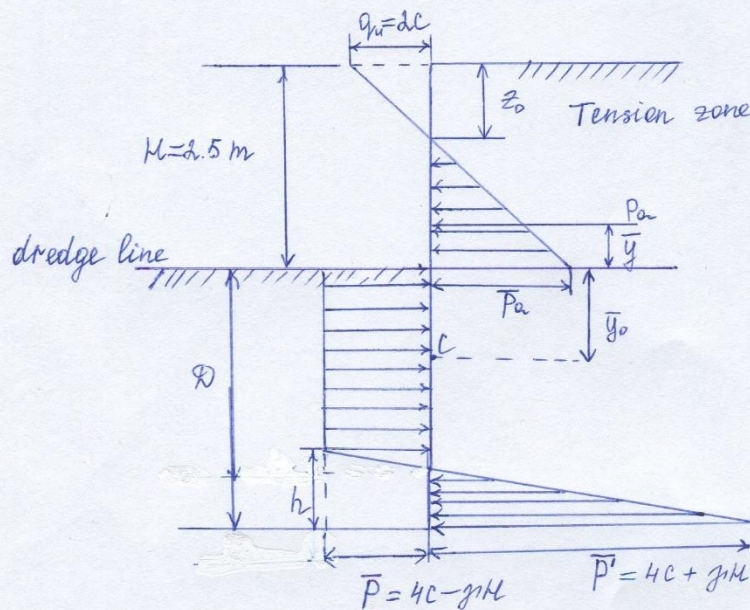
Appendix C-4

Sheet Pile Design

Short term analysis



For the case under consideration, loam will be treated as clay and pressure distribution in cohesive wall is taken in Figure below.



Pressure on the wall in tension zone up to a depth of z_0 is zero. The shaded triangle indicates net pressure distribution.

$$\begin{aligned}
 \text{For } \varphi_u = 0, \quad \bar{P}_a &= L_1 \gamma + \gamma_c L_2 + (\gamma_{\text{sat}} - \gamma_w) L_3 - 2C \\
 &= 1.6 \cdot 18.3 + 0.4 \cdot 17.6 + (19.3 - 9.81) \cdot 0.5 - 2 \cdot 18 = 5.9 \text{ kN/m}^2
 \end{aligned}$$

$$z_0 = \frac{2c}{\gamma} = \frac{2 \times 18}{18.3} = 1.97 \text{ m}$$

$$H - z_0 = 2.5 - 1.97 = 0.53 \text{ m}$$

$$P_a = \frac{1}{2} \bar{p}_a (H - z_0) = \frac{1}{2} \cdot 5.1 \times 0.53 = 1.35 \text{ kN/m}$$

$$\begin{aligned} \bar{p} &= 4c - (\gamma' L_1 + \gamma_2 L_2 + (\gamma_{sat} - \gamma_w) L_3) \\ &= 4 \times 18 - (18.3 \times 1.6 + 17.6 \times 0.4 + (19.3 - 9.81) \times 0.5) = 72 - 41.065 = 30.935 \text{ kN/m}^2 \\ &\approx 31 \text{ kN/m}^2 \end{aligned}$$

$$\begin{aligned} \bar{p}' &= 4c + (\gamma' L_1 + \gamma_2 L_2 + (\gamma_{sat} - \gamma_w) L_3) \\ &= 4 \times 18 + (18.3 \times 1.6 + 17.6 \times 0.4 + (19.3 - 9.81) \times 0.5) = 72 + 41.065 = 113.065 \text{ kN/m}^2 \\ &\approx 113.1 \text{ kN/m}^2 \end{aligned}$$

$$\bar{y} = \frac{1}{3} (H - z_0) = \frac{1}{3} \cdot 0.53 = 0.177 \text{ m}$$

For static equilibrium, $\Sigma F_H = 0$ and $\Sigma M_o = 0$

$$\Sigma F_H = 0: P_a \cdot (D + \bar{y}) - \bar{p} \frac{D^2}{2} + (\bar{p} + \bar{p}') \times \frac{1}{2} \times \frac{h}{3} = 0$$

$$\text{or } 1.35 (D + 0.177) - 31 \cdot \frac{D^2}{2} + (31 + 113.1) \times \frac{h^2}{6} = 0 \quad (1)$$

$$\Sigma F_H = 0: P_a - \bar{p} \cdot D + \frac{1}{2} (\bar{p} + \bar{p}') \cdot h = 0$$

$$1.35 - 31 \cdot D + \frac{1}{2} (31 + 113.1) \cdot h = 0$$

$$\Rightarrow h = \frac{31D - 1.35}{42.05} \quad (2)$$

Substituting h for (2):

$$\begin{aligned} 1.35D + 0.24 - 15.5D^2 + 24 \cdot \left[\frac{31D - 1.35}{42.05} \right]^2 &= 0 \\ -11.06D^2 + 0.963D + 0.248 &= 0 \end{aligned}$$

$$D = 0.2 \text{ m}$$

Increasing D by 40%, we have $D = 1.4 \times 0.2 = 0.28 \text{ m}$

Maximum bending moment

$$M_{max} = P_a (\bar{y}_0 + \bar{y}) - \frac{\bar{P} \bar{y}_0^2}{2}$$

$$\bar{y}_0 = \frac{P_a}{\bar{P}} = \frac{13.5}{31} = 0.04 \text{ m}$$

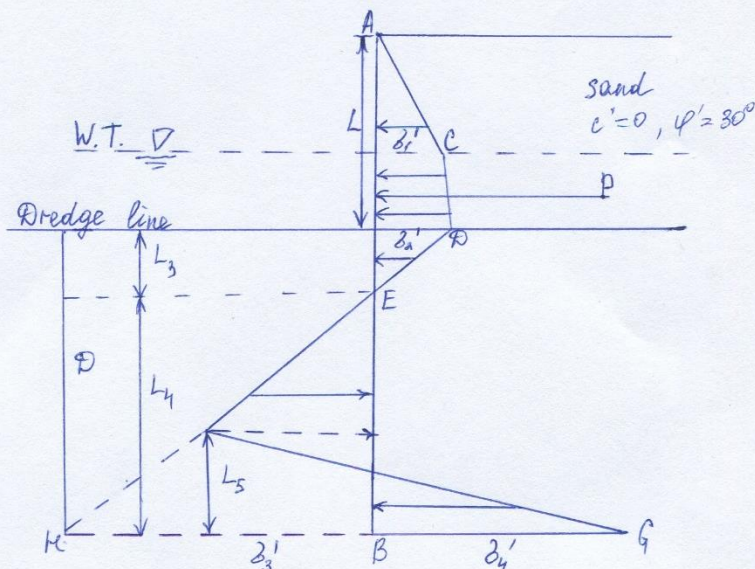
$$\bar{y} = 0.53 \text{ m}, \quad M_{max} = 1.35 (0.04 + 0.53) - \frac{31 \times 0.04^2}{2} = 0.74 \text{ kN}\cdot\text{m/m}$$

Assuming allowable flexural stress of the sheet pile, $f_b = 170 \text{ MN/m}^2$, minimum required section modulus can be calculated as following:

$$S = \frac{M_{max}}{f_b} = \frac{0.74 \text{ kN}\cdot\text{m/m}}{170 \text{ MN/m}^2} = 4.35 \times 10^{-6} \text{ m}^3/\text{m}$$

long term analysis

In this case, $c=0$ and φ can be assumed to be 30° . Clay will behave as sand, thus, pressure distribution of sand on sheet pile can be considered.



Step 1: $K_a = \tan^2(45 - \frac{\varphi'}{2}) = \tan^2(45 - 30/2) = 0.33$
 $K_p = \tan^2(45 + \frac{\varphi'}{2}) = \tan^2(45 + 30/2) = 3$

Step 2:

$$\delta_1' = \gamma L_1 k_a = 2 \times 18.3 \cdot 0.33 = 12.1 \text{ kN/m}^2$$

$$\delta_2' = (\gamma L_1 + \gamma' L_2) k_a = (2 \times 18.3 + (19.3 - 9.81) \times 0.5) \cdot 0.33 = 13.6 \text{ kN/m}^2$$

$$\text{Step 3: } L_3 = \frac{\delta_2'}{\gamma' (k_p - k_a)} = \frac{13.6}{(19.3 - 9.81)(3 - 0.33)} = 0.534 \text{ m}$$

$$\begin{aligned} \text{Step 4: } P &= \frac{1}{2} \delta_1' L_1 + \delta_1' L_2 + \frac{1}{2} (\delta_2' - \delta_1') \cdot L_2 + \frac{1}{2} \delta_2' L_3 \\ &= \frac{1}{2} \times 12.1 \times 2 + 0.5 \times 12.1 + \frac{1}{2} (13.6 - 12.1) \cdot 0.5 + \frac{1}{2} \times 13.6 \times 0.534 = \\ &= 22.2 \text{ kN/m} \end{aligned}$$

$$\begin{aligned} \text{Step 5: } \bar{z} &= \frac{\sum M_E}{P} = \frac{1}{22.2} \left[12.1 \cdot (0.534 + 0.5 + \frac{2}{3}) + 6.05 (0.66 + \frac{0.5}{2}) + 0.345 (0.66 + \frac{0.5}{2}) + \right. \\ &\quad \left. + 3.652 (0.66 + \frac{2}{3}) \right] = 0.81 \text{ m} \end{aligned}$$

Step 6:

$$\begin{aligned} \delta_5' &= (\gamma L_1 + \gamma' L_2) k_p + \gamma' L_3 (k_p - k_a) \\ &= (18.3 \times 2 + 9.49 \times 0.5) \times 3 + 9.49 \times 0.534 (3 - 0.33) \\ &= 137.6 \text{ kN/m}^2 \end{aligned}$$

$$\text{Step 7: } A_1 = \frac{\delta_5'}{\gamma' (k_p - k_a)} = \frac{137.6}{(19.3 - 9.81)(3 - 0.33)} = 5.43$$

$$A_2 = \frac{8P}{\gamma' (k_p - k_a)} = \frac{8 \times 22.2}{(19.3 - 9.81)(3 - 0.33)} = 7$$

$$\begin{aligned} A_3 &= \frac{6P [2\bar{z} \gamma' (k_p - k_a) + \delta_5']}{\gamma'^2 (k_p - k_a)^2} = \frac{6 \cdot 22.2 (2 \cdot 0.81 \cdot 9.49 (3 - 0.33) + 137.6)}{9.49^2 (3 - 0.33)^2} = \\ &= 34.1 \end{aligned}$$

$$A_4 = \frac{P(6\bar{z} \delta_5' + 4P)}{\gamma'^2 (k_p - k_a)^2} = \frac{22.2 (6 \cdot 0.81 \times 137.6 + 4 \times 22.2)}{9.49^2 (3 - 0.33)^2} = 26.2$$

$$\text{Step 8: } L_4^4 + A_1 L_4^3 + A_2 L_4^2 + A_3 L_4 - A_4 = 0$$

$$L_4^4 + 5.43 L_4^3 - 7 L_4^2 - 34.1 L_4 - 26.2 = 0$$

$$\therefore L_4 \approx 1.36 \text{ m}$$

$$\text{step 9: } D_{\text{req}} = L_3 + L_4 = 0.534 + 1.36 = 1.894 \text{ m} \approx 1.9 \text{ m}$$

$$D = 1.4 \times 1.9 = 2.66 \text{ m}$$

$$\text{step 10: Total length of sheet pile: } L_1 + L_2 + D = 2 + 0.5 + 2.66 = 5.16 \text{ m}$$

$$\text{step 11: } z' = \sqrt{\frac{2P}{(k_p - k_a) \gamma'}} = \sqrt{\frac{2 \cdot 22.2}{(3 - 0.33) \cdot 9.49}} = 1.32 \text{ m}$$

$$\text{step 12: } M_{\text{max}} = P(\bar{z} + z') - \frac{1}{2} \gamma' z'^2 (k_p - k_a) - \frac{z'}{3}$$

$$= 22.2 (0.81 + 1.32) - \frac{1}{2} \cdot (19.3 - 9.81) \cdot 1.32^2 (3 - 0.33) - \frac{1.32}{3} =$$

$$= 34.6 \text{ kN}\cdot\text{m/m}$$

$$\text{Assuming } z_{\text{all}} = 170 \text{ MN/m}^2,$$

$$\text{minimum required section modulus} \Rightarrow S = \frac{M_{\text{max}}}{z_{\text{all}}} = \frac{34.6 \text{ kN}\cdot\text{m/m}}{170 \times 10^3 \text{ kN/m}^2} = 2.01 \cdot 10^{-4} \text{ m}^3/\text{m}$$