

DESIGN AND ENERGY PERFORMANCE OF HOTEL BUILDING IN ASTANA, KAZAKHSTAN

(Capstone Project II)

**Bachelor of Engineering
(Civil Engineering)**



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2017

Declaration

We hereby declare that this report entitled “Design and Energy Performance of Hotel Building in Astana, Kazakhstan” is the result of my/our own project work except for quotations and citations which have been duly acknowledged. I/We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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11.04.2017

Acknowledgment

We wish to highly express our gratitude to Professor Shazim Memon, Professor Dichuan Zhang, Professor Jong Kim, Professor Hau Leung, Professor Minh Nguyen and Valeriya Tyo for their support, tolerance and timely feedback on the project work called “Design and Energy Performance of Hotel Building in Astana, Kazakhstan”. Also, we would like to express our sincere gratitude to our classmates from Civil Engineering department for their cooperation and help throughout the term. Moreover, Nazarbayev University library database really helped us to make research on our project.

Abstract

The following paper on the Capstone II report is based on the design project of a 15-storey reinforced concrete moment frame hotel building with the total area of 20725.2 m² and the area occupied by the building on the location space of 930.25 m². The hotel building has a name of “Utopia Hotel Astana”, and its main contractor is Efficient Energy Construction (EEC) company.

In this report, the detailed architectural design, including technical drawings of the proposed hotel, is provided. It is also based on the Kazakhstani codes, such as SNiPs and RDS RK. In order to provide the technical drawings, AutoCAD software was used; whereas, the 3D model was rendered in Google SketchUp software. In addition, using the same software, the outside parking area with available usual 38 parking places and 3 parking places for disabled people is shown as well. Moreover, hand and software calculations for the structural analysis of the building are presented. In order to fulfill the construction requirements of the high-rise building and to design structural members, such as slabs, columns and beams, EEC company has followed the regulations established by the UK code and ACI 318-11 code. Furthermore, for the geotechnical part, also hand calculations and in-situ tests were performed with the aim of finding the bearing capacity of the foundation, its dimensions, settlement of the soil, final decision on its type, and the final design of the foundation. Thus, to select the type of the foundation, the in-situ conditions were analyzed and the code provisions were followed; whereas, to calculate the necessary values, the procedures, formulae and equations were taken from the book, called “Principles of Geotechnical Engineering” of Braja M. Das.

In addition, in order to accomplish the project in a proper way, the preliminary cost estimation and the overall feasibility of the project are included into the report together with the project construction scheduling and work breakdown structure, involving Gantt chart and Flow chart for the project. And finally, on the strength of the aim of the project, literature review on the materials used to make the contribution into the energy efficiency performance is included. Through thermal properties analysis, three main insulating materials were selected for energy performance evaluation. Additionally, simulation on determining the effectiveness of these materials was generated in Design Builder software. According to two simulations results, application

of insulting materials is more energy saving. Hence, by the end of this paper, the discussions on the work done and accompanying conclusions are provided as well.

Table of Contents

1	INTRODUCTION	1.1
1.1	Project Description.....	1.1
1.2	Project Scope.....	1.1
1.3	Team Members.....	1.2
2	ARCHITECTURAL DESIGN	2.1
2.1	Design Statement	2.1
2.1.1	Main Function of the Building.....	2.1
2.1.2	Site Location	2.1
2.1.3	Preliminary Design	2.2
2.1.4	General Building Height and Areas.....	2.9
2.1.5	Area Distribution in the Hotel	2.9
2.2	Non-Structural Materials.....	2.10
2.2.1	Flooring.....	2.10
2.2.2	Ceiling.....	2.11
2.2.3	Glazing.....	2.12
2.2.4	Drainage Systems	2.13
2.3	Parking Area.....	2.13
2.4	Site Layout	2.14
2.5	Safety Requirements	2.15
2.5.1	Durability and Maintainability.....	2.15
2.5.2	Fire-Technical Requirements.....	2.16
2.6	Stairs and Staircases	2.17
2.7	Ensuring the Safety of People	2.17
2.7.1	Evacuation and Emergency Exits	2.17
2.7.2	Escape Routes	2.19
2.7.3	Evacuation by Stairs and Staircases.....	2.20
2.7.4	Elevators and elevator shafts	2.22
2.8	Partition.....	2.23
2.9	Building Envelope Thermophysical Properties	2.24
3	STRUCTURAL DESIGN	3.1
3.1	Design Load Calculation.....	3.1

3.1.1	Dead Load.....	3.1
3.1.2	Live Load.....	3.2
3.1.3	Wind Load	3.2
3.2	Structural Analysis Methods	3.6
3.2.1	Analysis under Dead Load and Wind Load.....	3.6
3.2.2	Portal Method	3.7
3.3	Structural Member Dimensioning.....	3.8
3.3.1	Column Sizing	3.8
3.3.2	Size of Column II.....	3.12
3.3.3	Size of Column III	3.13
3.3.4	Beam Sizing.....	3.13
3.4	Analysis under Wind Load.....	3.14
3.4.1	Hand Calculations.....	3.14
3.4.2	SAP 2000 Analysis.....	3.19
3.5	Analysis Under Dead Load	3.23
3.5.1	Hand calculations.....	3.23
3.5.2	SAP 2000 analysis	3.26
3.6	Structural Member Design	3.27
3.6.1	Beam Design.....	3.28
3.6.2	Column Design	3.30
3.6.3	Slab Design	3.32
4	GEOTECHNICAL DESIGN.....	4.1
4.1	Location of the Project	4.1
4.2	Soil Profile	4.2
4.3	Foundation Type	4.3
4.3.1	Analysis Methods	4.4
4.3.2	Shallow Foundation	4.4
4.3.3	Deep Foundation.....	4.15
4.3.4	Comparison of Shallow and Deep Foundation Types	4.16
4.3.5	Installation Method	4.21
5	INSULATING MATERIALS.....	5.1
5.1	Properties of Insulating Materials	5.1
5.2	Types of Insulating Materials.....	5.2

5.2.1	Glass Wool.....	5.4
5.2.2	Polyurethane rigid foam (PUR)	5.4
5.2.3	Vacuum Insulation Panel (VIP)	5.5
5.3	How Insulation Works	5.7
5.3.1	Installation Method	5.7
5.4	Significance of Insulating Materials	5.8
6	ENERGY PERFORMANCE	6.1
6.1	Methodology	6.1
6.1.1	EnergyPlus Software.....	6.1
6.1.2	Design Builder	6.2
6.2	Description of simulated building.....	6.2
6.3	Climate in Astana.....	6.3
6.4	Analysis and Results Discussion.....	6.4
6.4.1	Simulation on EnergyPlus	6.5
6.4.2	Simulation on Design Builder.....	6.9
6.4.3	Model Verification.....	6.11
6.4.4	Parametric Optimization	6.13
6.4.5	Model simulation with PUR	6.14
7	CONSTRUCTION PROJECT MANAGEMENT.....	7.1
7.1	Cost Estimation	7.1
7.1.1	Rough Order of Magnitude (ROM) Estimates	7.2
7.1.2	Square Foot Estimate	7.2
7.1.3	Assemblies Estimates	7.2
7.1.4	Bottom-up Method.....	7.3
7.1.5	Top-down Method	7.3
7.2	Project cost estimation	7.3
7.3	Feasibility Analysis	7.6
7.3.1	Market Demand	7.7
7.3.2	Economic Feasibility	7.7
7.3.3	Operational Feasibility.....	7.8
7.4	Project Planning and Scheduling	7.8
8	CONCLUSIONS.....	8.1
9	REFERENCE LIST	9.1

10 APPENDICES.....	10.1
Appendix A	10.1
Appendix B	10.5
Appendix C	10.67
Appendix D	10.76

List of Tables

Table 2.1. The composition and the floor space of a floor maintenance	2.10
Table 2.2. Design loads of materials in a floor layer	2.11
Table 2.3. Percentage of parking places depending on a category of the hotel.....	2.13
Table 2.4. Limits of fire resistance of fire barriers.	2.16
Table 2.5. Limits of fire resistance of the filling openings in fire barriers	2.16
Table 2.6. Exit access travel distance	2.20
Table 2.7. Building Envelope Properties (Mastter 2011) and (Matbase n.d.)	2.25
Table 3.1 Calculation of Dead Loads	3.1
Table 3.2 Minimum Uniformly Distributed Live Loads, L_0 (Table 4-1, ASCE 7-05) .	3.2
Table 3.3. Characteristic values for each building floor.....	3.4
Table 3.4. Wind loading calculation results	3.5
Table 3.5. Design loads of materials in a floor layer	3.8
Table 3.6. Design load of material in a roofing	3.9
Table 3.7. Minimum uniformly distributed live loads.....	3.10
Table 3.8. Comparison of SAP2000 simulation and Portal Method results for structural analysis under wind load for points A to O.	3.22
Table 3.9. Minimum thickness of nonprestressed beams or one-way slabs unless deflections are calculated.....	3.32
Table 4.1. Soil Characteristics	4.2
Table 4.2. Bearing Capacity of Shallow Foundation.....	4.13
Table 4.3. Settlement of Shallow Foundation.....	4.14
Table 4.4. Foundation Characteristics	4.16
Table 5.1. Classification of insulating materials (Margit et al. 2007)	5.3
Table 5.2. Thermal properties and size formats of insulating materials (Margit et al. 2007).	5.6
Table 5.3. Benefits and Drawbacks of building insulation (Shpresa 2011)	5.9
Table 6.1. Thermal Properties of insulating materials.....	6.5
Table 6.2. Energy uses for different insulating materials for a whole year	6.7
Table 6.3. Energy Uses for different insulating materials during winter season.....	6.8
Table 6.4. Energy Uses for different insulating materials during for summer season ..	6.8
Table 6.5. Energy use of a room without insulating material.....	6.8
Table 6.6. Exterior wall details	6.11

Table 6.7. Amount of energy and related costs of insulating materials.....	6.13
Table 6.8. Total annual energy use with PUR, 0.2 m.....	6.18
Table 6.9. Comparison of total annual energy uses.....	6.18
Table 7.1. Calculation of costs of concrete for each floor (Satu 2016).....	7.4
Table 7.2. Procurement costs of insulating materials (TIU 2016) and (Satu 2016).....	7.4
Table 7.3. Amount of rooms and rates per night of five star hotels in Astana (Booking 2016).....	7.5
Table 7.4. Amount of rooms and annual estimated incomes of top hotels in Astana (Forbes 2016).....	7.6
Table 7.5. Calculation of Payback period.....	7.6

List of Figures

Figure 2.1. Chosen location for Utopia Hotel Astana project (Google, Earth)	2.2
Figure 2.2. Initial design of a hotel (AutoCAD model).....	2.2
Figure 2.3. Second design of a hotel building	2.3
Figure 2.4. Floor plans of lower (a), middle (b) and higher (c) sections respectively...	2.5
Figure 2.5. Final Preliminary design of a “Utopia Hotel Astana” in Capstone I (3D model)	2.6
Figure 2.6. Technical drawing of forth floor plan. Top view of a corner of a building	2.7
Figure 2.7. Final design of a “Utopia Hotel Astana” (AutoCAD 3D model).....	2.7
Figure 2.8. Typical floor plan of a “Utopia Hotel Astana”.....	2.8
Figure 2.9. Google Earth visualization of 3D model of a building	2.9
Figure 2.10. Floor section for lobby Figure 2.11. Floor section for rooms	2.11
Figure 2.12. Ceiling components (Google Sketch Up 3D model).....	2.12
Figure 2.13. Glazing of a hotel (AutoCAD 3D model, Rendering model).....	2.12
Figure 2.14. Isometric view of a main entrance (AutoCAD 3D model, Rendering model)	2.13
Figure 2.15. Main entrance of a hotel (Google Sketch Up 3D model).....	2.15
Figure 2.16. Parking lots of a hotel (Google Sketch Up 3D model).....	2.15
Figure 2.17. Exterior fire escape (Google Sketch Up 3D model).....	2.17
Figure 2.18. Evacuation routes for typical floor of a building	2.19
Figure 2.19. Views of staircase (Google Sketch Up 3D model).....	2.21
Figure 2.20. Views of staircase (AutoCAD 3D model).....	2.21
Figure 2.21. Custom view of the first floor with shaded elevator shafts including fire lift’s shaft (AutoCAD 3D model)	2.23
Figure 2.22. Fire resistant gypsum board (OSPANOFF, 2013)	2.23
Figure 2.23. Ceiling Profile (OSPANOFF, 2013)	2.24
Figure 2.24. Angular profile Figure 2.25. Double layered profile connector (OSPANOFF, 2013)	2.24
Figure 3.1. Deflected shape and bending moment diagram for fixed beam.....	3.7
Figure 3.2. Approximation for practical building frame	3.7
Figure 3.3. Assumed shape of frame under lateral loading and predicted zero moment points.....	3.8
Figure 3.4. The assumed reaction forces of the frame.....	3.8

Figure 3.5. Tributary area of one column	3.11
Figure 3.6. Wind loading condition for portal method analysis	3.15
Figure 3.7. Internal forces determination for 15 th floor using portal method	3.16
Figure 3.8. Internal forces for points from A to O for the given frame	3.18
Figure 3.9. Shear, axial and moment diagrams for points from A to O for the given frame.	3.19
Figure 3.10. Applied wind load to the frame and deformation due to wind load.....	3.20
Figure 3.11. Axial force diagram from SAP 2000 and shear force diagram from SAP 2000	3.21
Figure 3.12. Moment diagram from SAP 2000	3.21
Figure 3.13. Representation of major and minor beams allocation	3.24
Figure 3.14. Manual calculation of internal forces for typical beam.....	3.25
Figure 3.15. Dead Load applied to 2d frame and deflected shape after simulation	3.26
Figure 3.16. Moment diagram and Shear Force diagram by SAP 2000.....	3.27
Figure 3.17. Exact values of Moment diagram and Shear Force diagram by SAP 2000	3.27
Figure 3.18. Resultant moment for a critical beam by SAP2000	3.28
Figure 3.19. Reinforced beam.....	3.30
Figure 3.20 Factored axial load and moment provided by SAP2000.....	3.30
Figure 3.21. Interaction diagram for rectangular column with $\gamma=0.7$	3.31
Figure 4.1. Location of the Project	4.2
Figure 4.2. Strip Footing (Mishra, 2016) Figure 4.3. Shallow Strip Foundation (Das, 2011)	4.5
Figure 4.4. Spread Footing (Mishra, 2016)	4.5
Figure 4.5. Combined Footing (Mishra, 2016).....	4.6
Figure 4.6. Cantilever Footing (Mishra, 2016).....	4.6
Figure 4.7. Raft Foundation (Mishra, 2016).....	4.7
Figure 4.8. Hand calculations of parameters for the settlement equation.....	4.11
Figure 4.9. Cross Sectional Shape for Piles (Ben C. Gerwick, n.d.)	4.19
Figure 4.10. Foundation Design	4.21
Figure 4.11. Pile Driver	4.21
Figure 5.1. Glass wool insulating material (Margit et al. 2007).....	5.4
Figure 5.2. Polyurethane rigid foam (Margit et al. 2007).....	5.5

Figure 5.3. Vacuum Insulation panel (Margit et al. 2007)	5.6
Figure 5.4. Exterior wall insulation with Glass wool (RITES 2007)	5.7
Figure 5.5. Exterior wall insulation with VIP (RITES 2007).....	5.8
Figure 6.1. Floor plan of typical corner room.....	6.3
Figure 6.2. Corner typical floor plan created in Design Builder	6.3
Figure 6.3. Daily low (blue) and high (red) temperatures for Astana, in 2015 (Weatherspark.com, 2015).....	6.4
Figure 6.4. Sketchup 3D model integrated with EnergyPlus software.....	6.6
Figure 6.5. Total Energy uses with different IMs by seasons.....	6.7
Figure 6.6. Total annual energy consumptions at different cases.....	6.9
Figure 6.7. 3D model of a typical corner room developed in Design Builder.....	6.10
Figure 6.8. Model of a typical room with three zones (Zone1- Bedroom, Zone 2 – Bathroom and Zone 3 – Assembly area)	6.10
Figure 6.9. Comparison of simulation outputs for summer season	6.12
Figure 6.10. Comparison of simulation outputs for winter season	6.12
Figure 6.11. Comparison of simulation outputs for whole year	6.13
Figure 6.12. Insulation thickness effect on annual energy use	6.14
Figure 6.13. External Wall layer specifications with PUR.....	6.15
Figure 6.14. Annual Heating (in red) and Cooling (in blue) Energy consumptions....	6.16
Figure 6.15. Monthly CO2 production with PUR	6.17
Figure 6.16. Monthly CO2 production without IM	6.17
Figure 7.1. Work Breakdown Structure for the project implementation	7.10
Figure 7.2. Gantt Chart for the Capstone I implementation	7.11
Figure 7.3. Gantt Chart for the Capstone I implementation	7.12
Figure 7.4. Work Breakdown Structure for the hotel construction	7.13

List of Abbreviations/Notations/Glossary of Terms

GRFFS = General Requirements for Fire Safety

A_c = area of critical section

A_{cp} = area enclosed by outside cross section perimeter of concrete

A_g = gross area of concrete section

A_l = total area of longitudinal reinforcement to resist torsion

A_{st} = cross-sectional area of the steel

A_{tr} = tributary area

a_{max} = peak horizontal acceleration at the ground surface

A_o = gross area enclosed by torsional shear flow path

A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement

A_p = pile tip area

A_{ps} = area of prestressed longitudinal tension reinforcement

A_s = area of non-prestressed longitudinal tension reinforcement

$A_{s,min}$ = minimum flexural reinforcement area

A_{st} = total area of non-prestressed longitudinal reinforcement including bars or steel shapes

A_t = area of one leg of a closed stirrup, hoop, or tie resisting torsion within spacing s / tributary area in m^2

A_v = shear reinforcement area

$A_{v,min}$ = minimum shear reinforcement area

b_t = width of that part of cross section containing the closed stirrups resisting torsion

b_w = web width or diameter of circular section

b_1 = critical section dimension measured in direction of span

b_2 = critical section dimension measured in direction perpendicular to b_1

B' = $B/2$ for foundation center, B for foundation corner

- c = cohesion
- c_c = clear cover of reinforcement
- $c_l c_r$ = distance from centroid of critical section to left and right faces
- C_B = correction factor for diameter of borehole
- C_c = compression index
- C_p = external pressure coefficient
- C_R = correction factor for the length of rod
- C_{swell} = swell index
- C_t = approximate period parameters for moment resisting frame type
- C_u = upper limit coefficient for calculated period
- C_{vx} = vertical distribution factor
- C_α = secondary consolidation index
- d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement
- d_{agg} = maximum size of coarse aggregate
- d_b = diameter of bar
- D = effect of dead loads
- $D_{flooring}$ = dead loads of the flooring
- D_{slab} = dead loads of the slab
- e_p = void ration at the end of primary consolidation
- e_0 = initial void ratio
- E_{cb} = modulus of elasticity for concrete beam
- E_{cs} = modulus of elasticity for concrete slab
- E_S = average modulus of elasticity
- f = unit friction at depth z
- f'_c = specified compressive strength of concrete (MPa)
- f_{pe} = concrete compressive strength due to effective prestress forces

- f_{pu} = tensile strength of prestressing reinforcement
 f_{se} = effective strength of prestressed reinforcement
 $f_{s'}$ = compressive stress in reinforcement under factored loads
 f_y' = specified yield strength for non-prestressed reinforcement
 f_{yt} = yield strength of transverse reinforcement
 F_a = site coefficient
 F_s = shape factor
 F_d = depth factor
 F_i = load inclination factor
 F_{PGA} = site coefficient
 FS = factor of safety
 F_x = lateral vertical force at any level
 F_v = site coefficient
 G = Gust Effect Factor
 h_h = structural height of the building
 h_i = the proportion of the structure height assigned to level i
 h_L = roof height including parapet
 h_x = the proportion of the structure height assigned to level x
 h = thickness, depth, or height of the member
 H = thickness of clay layer
 I = importance factor
 I_b = moment of inertia of beam
 I_{b1}, I_{b2} = negative moment sections at at beam ends
 I_{bm} = effective moment of inertia for midspan
 I_{cr} = moment of inertia of cracked section transformed to concrete
 I_e = effective moment of inertia for calculation of inertia

- I_f = depth factor
- I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
- I_s = moment of inertia of slab
- I_{sh} = shape factor
- J_c = property analogous to polar moment of inertia
- k = exponent related structure period / effective length factor
- K_d = wind directionality factor
- K_h = velocity pressure exposure coefficient
- K_z = velocity pressure exposure coefficient
- K_{zt} = Topographic factor
- K_{LL} = live load element factor, values are given in the Appendix B, Table 1
- l = beam span length
- l_a = additional embedment length beyond centerline of support or point of inflection
- l_c = length of compressive member
- l_d = development length in tension of deformed bar
- l_n = clear span in the long direction
- l_u = unsupported length, distance between floor slabs or beams
- L = effect of service live load / length of foundation/ reduced design live load per m^2 of area supported by the member
- L_0 = unreduced design live load per m^2 of area supported by the member
- L_{eff} = effective length
- L_i = the building length at level i parallel to the wind direction
- L_r = effect of service live load of roof
- L_{short} = short span length
- m = mean value live load

- M_a = maximum moment in member due to service loads at stage deflection
- M_{cr} = cracking moment
- M_n = nominal flexural strength
- M_{sc} = factored slab moment resisted by column
- M_u = factored moment
- M_w = moment magnitude
- $N_{c,q,\gamma}^*$ = bearing capacity factors
- N_{floor} = load applied to the floor
- N_m = standard resistance of penetration
- N_u = factored axial force
- p_{cp} = perimeter of cross section of concrete
- p_h = perimeter of centerline of outermost closed transverse torsional reinforcement
- P = design wind pressure
- PL = leeward face design pressure
- P_{LX} = leeward face design pressure acting in the x principal axis
- P_{LY} = leeward face design pressure acting in the y principal axis
- P_n = nominal axial compressive strength of member
- $P_{n,max}$ = maximum nominal axial compressive strength
- P_o = nominal axial strength
- P_u = factored axial force; to be taken as positive for compression and negative for tension
- q = effective stress
- Q_{all} = maximum allowable load bearing capacity
- Q_u = ultimate bearing capacity
- Q_p = point bearing capacity at the tip of the pile
- Q_s = skin frictional resistance of the pile
- r_d = stress reduction coefficient

R = response modification coefficient
 s = longitudinal spacing of reinforcement
 S = load effect of snow
 S_c = primary consolidation settlement
 S_n = nominal moment, shear, axial, torsional, or bearing strength
 S_e = elastic settlement
 S_{max} = maximum settlement
 S_{sc} = secondary consolidation settlement
 S_T = total settlement
 T_a = approximate fundamental period of the structure
 T_{cr} = cracking torsional moment
 T_L = long-period transition period of the structure
 T_n = nominal torsional moment strength
 T_{th} = threshold torsional moment
 T_u = factored torsional moment
 v = basic wind speed
 v_c = mean rate of occurrence of transient load
 v_n = concrete strength in correspondence with nominal two-way strength
 v_u = maximum factored two-way shear stress
 V_n = nominal shear strength
 V_c = volume of concrete/nominal shear strength by concrete for prestressed concrete
 V_n = nominal shear strength
 V_s = volume of steel/nominal shear strength by shear reinforcement
 V_{us} = storey shear
 γ = unit weight
 w_c = unit weight of the concrete (kg/m^3)

- x = approximate period parameters for moment resisting frame type
- y_t = distance from centroidal axis of gross section, neglecting reinforcement, to tension face
- α = angle defining the orientation of reinforcement
- α_f = flexural thickness ratio for beams
- α_{fm} = average flexural thickness ration for beams
- β = the ratio of clear span in the direction from long to short span
- β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section
- γ_c = partial safety factor of concrete
- γ_f = factor for determination of fraction of M_{sc}
- γ_s = partial safety factor of steel
- ΔL = incremental length of pile over which p and f are constant
- ΔS_{max} = maximum differential settlement
- $\Delta \sigma$ = net applied pressure on the foundation
- $\Delta \sigma'$ = effective net pressure
- Δ_0 = first order relative deflection between bottom and top
- ε_t = net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature
- λ = modification factor
- λ_{Δ} = multiplier used for additional deflection due to long-term effects
- μ_s = Poisson's ratio of soil
- η = compression factor
- ζ = time-dependent factor for sustained load
- ϕ = strength reduction factor
- τ_s = duration of average sustained load occupancy
- σ_c' = effective preconsolidation pressure

1 INTRODUCTION

1.1 Project Description

The project of the construction of Utopia Hotel Astana is located in Astana, Kazakhstan. The main objectives of this project for the EEC company are to design safe and modern hotel building, performing architectural, structural, and geotechnical analyses, and to evaluate an effect of insulating materials integration on energy consumption of hotel. The Utopia Hotel Astana would be a high-rise building with the total number of storeys of 15 and the total height of 51.35 meters. As Astana city is a young growing city, there is a need in such modern high-rise buildings, as our Utopia Hotel Astana appears to be. However, this building is also designed in accordance with the local design regulations and standards, written in the SNIps.

This building perfectly suits to the surrounding territory and buildings with its height, materials used for the façade sizes, and overall size. The design performed which is actually, presented later in this report, is enabled with moment frame; whereas, the reinforcement concrete makes the building resistant to the wind loads, which are common in Astana city, and any other external loads applied to the building.

In addition, it should be mentioned that by the end of this project report, it is necessary to perform the literature review and detailed calculations, analysis and design on architectural, structural and geotechnical design codes, insulating materials and its application in buildings and Energy analysis. Moreover, it is essential to show learning about the software, such as EnergyPlus, Design Builder, and SAP2000. Furthermore, in this paper, the architectural part is also aimed to include the site layout (landscaping, parking and traffic flow), 1st and typical floor design, choice of structural and non-structural materials, climate data of Astana city and the choice of insulating materials to be integrated.

1.2 Project Scope

The scope of this project is to develop architectural, structural and geotechnical design of the building according to literature review and calculations analysis, make feasibility analysis (cost estimation, scheduling, work breakdown structure) as a part of project management, perform literature review on the energy performance with integration of insulating materials and on software used for that, and do the energy

consumption analysis. Literature review contains studying design codes, standards and regulations together with the data and information extracted from the books and research papers.

1.3 Team Members

Our project team consists of four members:

1. *Amire Anuarbek* – Team Reflector. Responsible for structural design of the project.
2. *Nurassyl Battalgazy* – Team Explorer. Responsible for architectural design and materials selection of the project.
3. *Zauzat Zeken* – Team Recorder & Reporter. Responsible for energy efficiency methodology and feasibility of the project.
4. *Zarina Zharaspayeva* – Project Manager. Responsible for geotechnical design of the project.

2 ARCHITECTURAL DESIGN

2.1 Design Statement

2.1.1 Main Function of the Building

Utopia Hotel Astana” is designed for conducting hotel businesses. Classification of “Utopia Hotel Astana” is assigned as a public building according to Building Standards of the Republic of Kazakhstan. According to resolution of the government of Kazakhstan, buildings are divided into classes and sub-classes depending on fire hazards and extend to which the people’s safety is at risk in case of fire, with taking into account their age, physical condition, possibility of staying in a state of sleep, the kind of basic functional contingent and its amount. Hotels, hostels, dormitories and health resorts of the general type of holiday homes, campgrounds, and motels are categorized as residential buildings. The hotel consists of 154 rooms of different categories such as Standard, Triple or Family and Suite rooms. Besides, Utopia Hotel Astana will accommodate restaurant, food-courts and other services for residents’ need on a first floor.

2.1.2 Site Location

Location of Utopia Hotel Astana initially was proposed to be in the center of city. Taking into account following criteria, location as it is illustrated in Figure 2.1 was chosen.

- Since hotels are classified as public buildings, all necessary municipal services or city services should be near the hotel building. Basic municipal services include malls, schools, fire departments, police and health departments.

- Views from different sides should be aesthetically pleasant so that residents’ preferences could be fulfilled.

- The total territory should be within sufficiently large area in order to accommodate parking zone.

Furthermore, upcoming Light Rail Transport system (LRT), which is scheduled to be launched in late 2018, will pass through intersection of “Kabanbay Batyr” and “Syganak” avenues, which means that residents of hotel can reach main facilities without renting or using cars, in a short time (Alrt.kz, 2016). The selected site has the area of 67.30 m x 60.80 m.



Figure 2.1. Chosen location for Utopia Hotel Astana project (Google, Earth)

2.1.3 Preliminary Design

At the initial design stage, hotel was in the form of oval and dome roofs as illustrated in Figure 2.2. Despite the fact that it looks aesthetically pleasing, construction of this type of structure will be very difficult due to its complex shape. After brainstorming and reassessment of our opportunities and objectives, it was agreed to focus on structural safety as the primary aim of the project and design modern hotel building by performing accurate architectural, structural and geotechnical analysis.

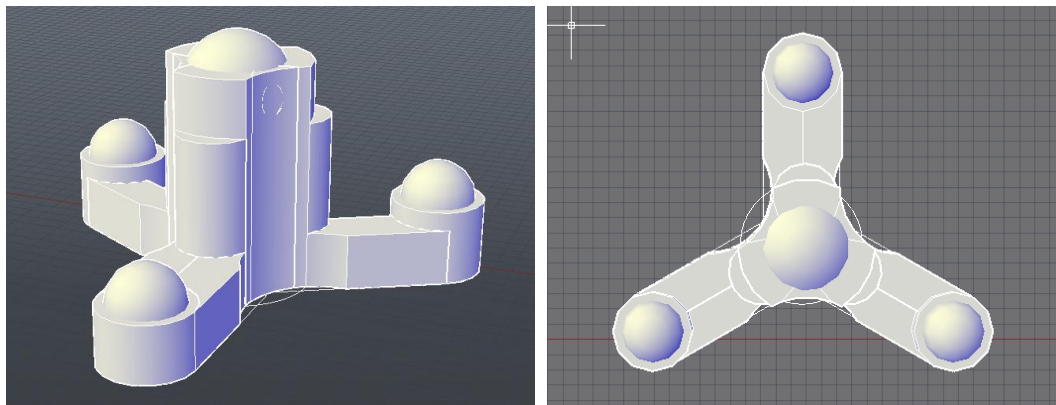


Figure 2.2. Initial design of a hotel (AutoCAD model)

Designing rectangular shaped structures is the best solution to develop precise structural and geotechnical investigations. Various versions of hotel design in rectangular shape were considered and leveled cake formed design was selected for further considerations. The 3D models generated using the Google Sketch Up and hand drawings are illustrated in Figure 2.3. As shown in drawings, hotel building comprises 3 blocks with varying dimensions and connected at different vertexes with each other. These three blocks are adjoined with the first stage and each block has 3 sections with

changing floor plans (see Figure 2.4). The total area of such design is 20725.2 m², but the project scope was very big to handle with. Furthermore, underground parking that was initially planned became another difficulty for geotechnical analysis in a group with least members in comparison with others. In addition, criteria such as price of ground, surrounding architecture and packing capacity of a building were not considered by group. However, during poster presentation, professors mentioned similar possible issues and some recommendations were provided to eliminate these problems. Therefore, taking into account professors' guidance to reduce the project size, it was decided to eliminate 2 blocks by keeping only one block and decrease the area of first stage from 130x70m to 30.5x30.5m. Moreover, with the sufficient area of selected location it is possible to construct parking lots outside of hotel building with the decreased area of 30.5x30.5m.

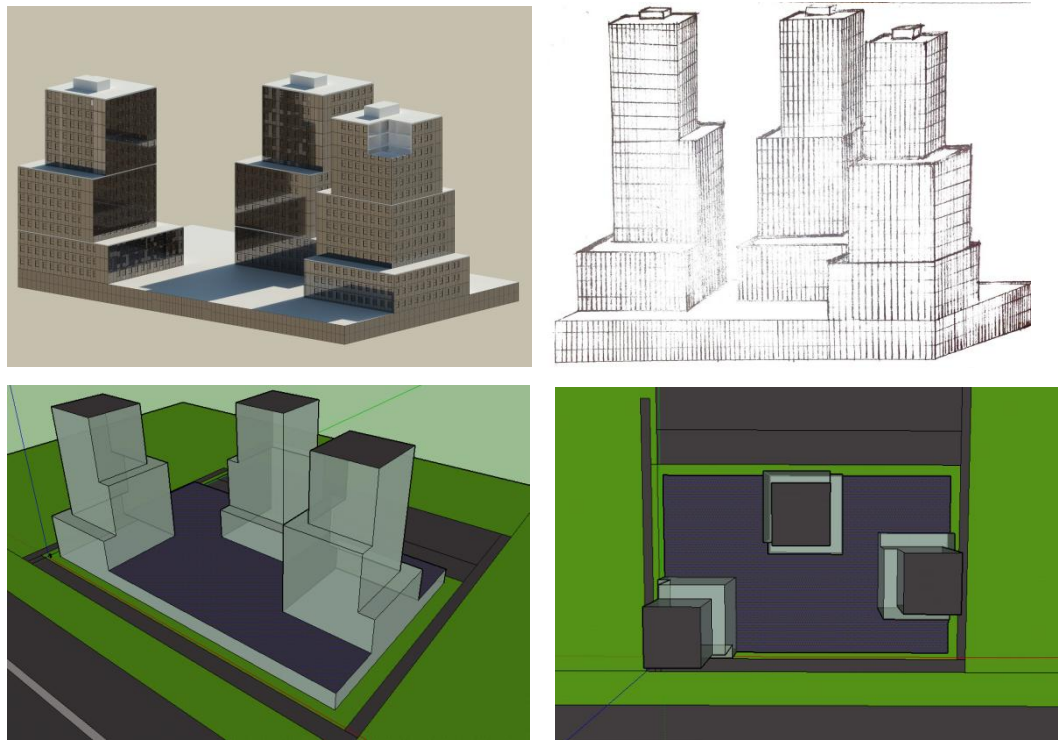


Figure 2.3. Second design of a hotel building

The final design of hotel building in Capstone I was as shown in Figure 2.5. As mentioned previously, decision to choose this type of building was made referring to following criteria:

- Design of the building with respect to surrounding architecture is essential for Astana. All preliminary designs of building are assessed by SI “Department of

Architecture and Town planning”. Therefore, rectangular shaped design of a hotel is suitable for this chosen area.

- Astana city is increasingly growing and site cost is also considerably rising, especially in center regions of the city. So, considering all these, hotel should be designed in a more efficient way in terms of place and square shape which has largest packing capacity.

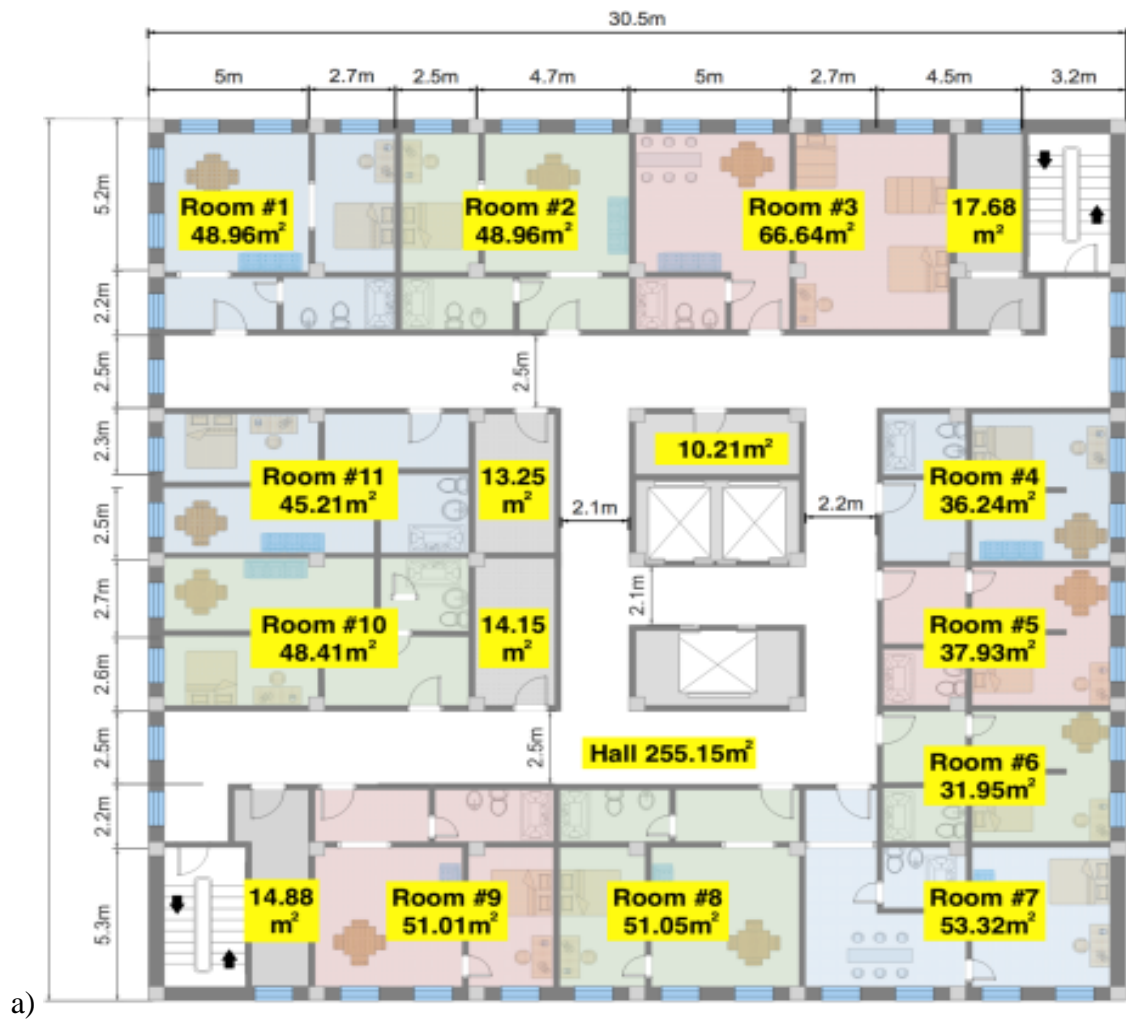




Figure 2.4. Floor plans of lower (a), middle (b) and higher (c) sections respectively

The preliminary design of the hotel building (for Capstone I) has a 30.5 x 30.5 m. The hotel has three sections, each having different floor plans; higher section, middle

section and lower section. Figure 2.4 illustrates these floor plans with assigned interior design. The first top section is 6-storey high with 5 rooms at each floor, while the middle section is 5- storey high with 8 rooms at each one. The lowest bottom section is 3-storey high with 11 rooms at each floor. Accordingly, the total number of rooms was 103.

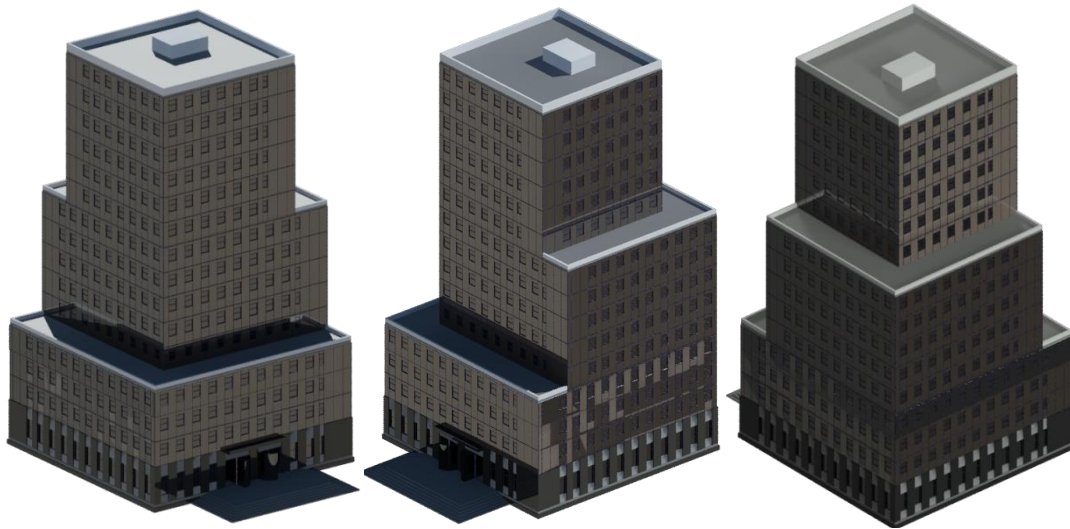


Figure 2.5. Final Preliminary design of a “Utopia Hotel Astana” in Capstone I (3D model)

However, the building design with three sections at different vertices resulted in another challenge. Despite the fact that it is technically possible to construct building of such type, the major problem has arisen with staircase shafts. The staircase shafts at each section are placed in different locations. Namely, lower and middle sections’ shafts have to be connected by changing the shaft’s location at 4th floor. Consequently, the same method should be done between middle and upper section of the building. The scheme of a fourth floor is shown in Figure 2.6 to illustrate the main issue. Moreover, shafts with different locations do not only make the structural analysis complex, but also can create inconvenience to hotel residents that leads to weak safety conditions.

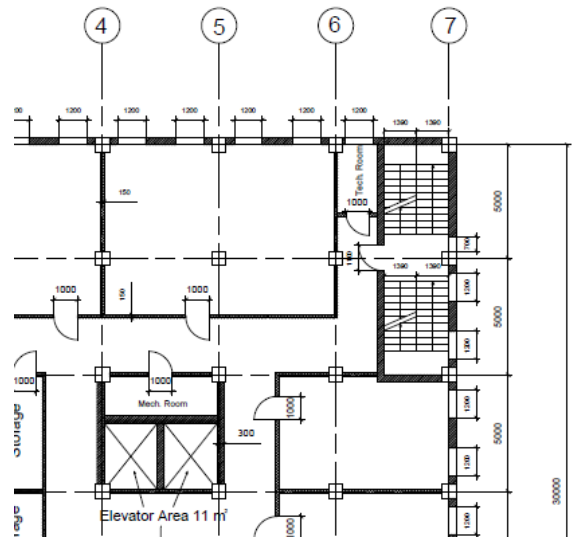


Figure 2.6. Technical drawing of fourth floor plan. Top view of a corner of a building

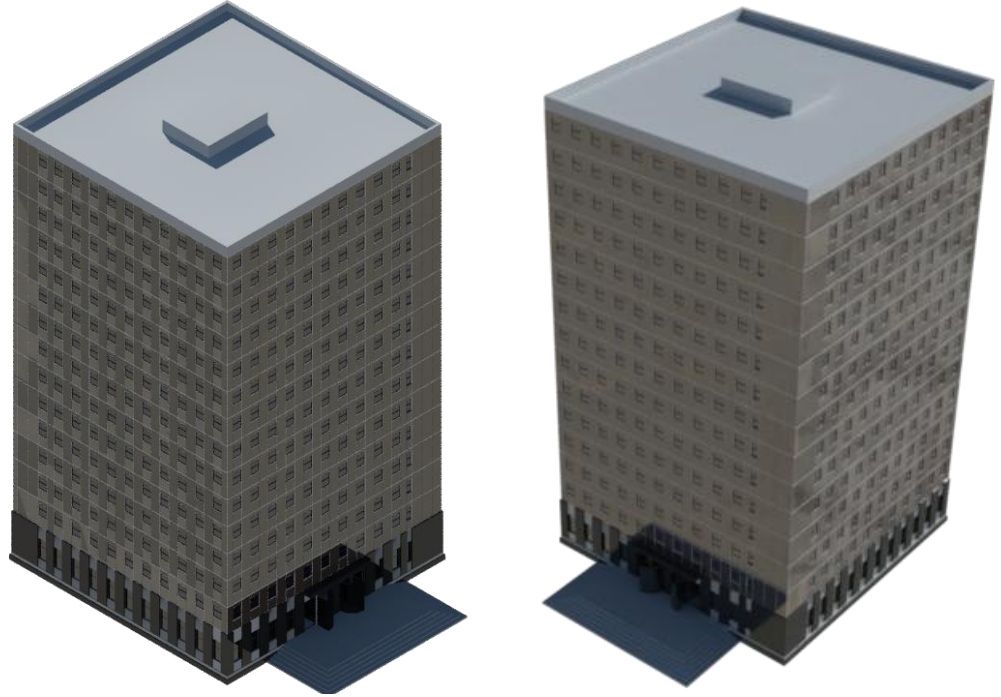


Figure 2.7. Final design of a “Utopia Hotel Astana” (AutoCAD 3D model).



Figure 2.8. Typical floor plan of a “Utopia Hotel Astana”

After several discussions, decision was made to change the hotel design again design taking into account all the problems mentioned above. In order to make staircase shafts straight without changing their locations from the corner of a building, the floor plan of the lowest bottom section was applied for rest 14 floors. The final design of a “Utopia Hotel Astana” is shown in Figure 2.7 and typical floor plan of a hotel is illustrated in Figure 2.8. This modification was admitted due to its simplicity and the way it solves the issues with staircase shafts. Moreover, this new design eliminates empty spaces between sections of old designs which were also one of the concerns. Furthermore, the number of rooms was also increased from 103 to 154 rooms. Consequently, number of parking places also should be predetermined which will be discussed in the following paragraphs.

In order to visualize the surrounding environment, the Google Earth was used with exact location of “Utopia Hotel Astana” building. Site layout including landscaping, parking and traffic flow was developed and shown in Figure 2.9. Additional illustrations can be observed from Figures A6-A9 in Appendix A.

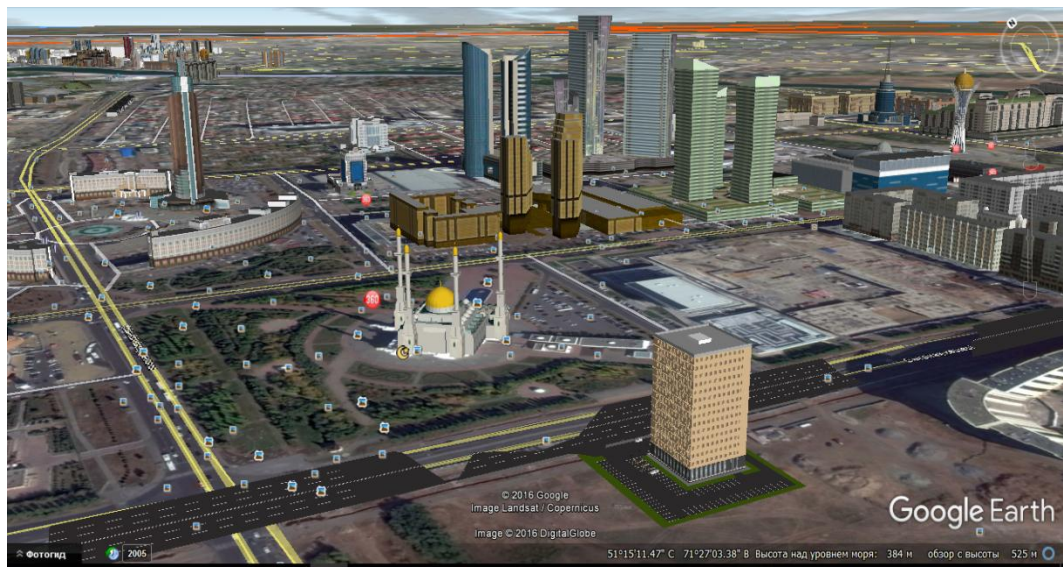


Figure 2.9. Google Earth visualization of 3D model of a building

2.1.4 General Building Height and Areas

According to Building Regulations (BR RK, 2011), in Fire protection requirements paragraph, it is recommended that hotel building accommodations should be designed with a height not exceeding 16 storeys. Utopia Hotel Astana is 51.35 meters height with 15 storeys (or up to 50 m of the average travel planning markers intended for fire engines entrance, to the level of the top floor of the floor).

2.1.5 Area Distribution in the Hotel

According to technical regulations of hotel buildings written in requirements of building code, every floor of hotel building should have special areas for various

purposes needed for staff and cleaning team. Area distribution of floors was designed according to Table 2.1.

Table 2.1. The composition and the floor space of a floor maintenance

Premises	Area (m ²)
The room staff on duty with built-in cabinets for clean linen	10 (16) ¹⁾
The room is older maid (layerwise economy, housekeeper) ²⁾	12
Storage dirty linen ³⁾	6
Store cleaning equipment	4
Disassembly dirty linen at laundry chutes	4
Room of consumer service ⁴⁾	6 - 8
Storage room for maid carts ⁵⁾	8 (12) ¹⁾
The room cleaning footwear ⁶⁾	8
WC Staff - toilet, sink, shower	4
<p>¹⁾ Figures in parentheses - for categories **** and *****;</p> <p>²⁾ In hotels with a capacity of 300 seats or more;</p> <p>³⁾ The hotel can accommodate up to 100 people are allowed to replace the cabinets;</p> <p>⁴⁾ In the hotel category *, **, *** for stays on the floor at least 30 people - may be placed through the floor.</p> <p>⁵⁾ For the category *** and above;</p> <p>⁶⁾ For hotel category **** and *****. ** For the categories of hotels and above can be used special equipment shoe cleanings on floors or in the lobby.</p>	

2.2 Non-Structural Materials

2.2.1 Flooring

According to State Standards in the field of Architecture, town-planning and construction, assembly and construction parts must be made of materials capable to resist possible moisture impacts, aggressive environment, and low temperatures, biological and other adverse factors. Generally, floor sections consist of four or five layers of materials with different characteristics depending on the type of building. In hotel buildings, it is highly recommended to use acoustic insulations in order to

eliminate noise from other stages. Concrete screeds are used to obtain defined level and to carry complete flooring and allow a wearing surface. For various building zones floor sections are different and they are shown in Figures 2.10 and 2.11 for room and lobby areas respectively. Ceramic material will be used for lobby and parquet will be used for rooms.

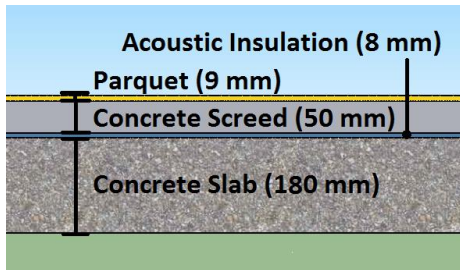


Figure 2.10. Floor section for lobby

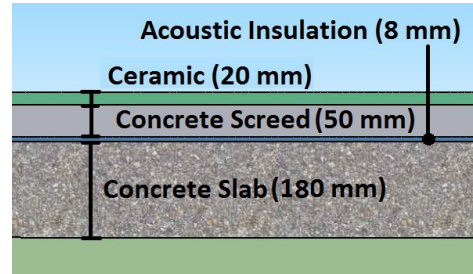


Figure 2.11. Floor section for rooms

In order to know all the load of slab, through the densities of all materials, total weight of the flooring can be determined.

Table 2.2. Design loads of materials in a floor layer

Floor layer	Load (kN/m ²)
Concrete slab	5.0
Concrete screed	0.9
Acoustic insulation layer	0.03
Ceramic layer	0.80
Parquet layer	0.23
Total weight	6.96

2.2.2 Ceiling

According to requirements of hotel buildings, the height of every storey in 5 star category hotels should not be less than 3 meters and 0.3 m of it occupied by ceiling. Ceiling is suspended from the slab with steel system that will hide mechanical duct allowance as it can be seen from Figure 2.12. The loads of a whole ceiling depend on how many mechanical duct allowances will be installed and which type of fiber board will be used in design. However, total load of a ceiling is respectively small.

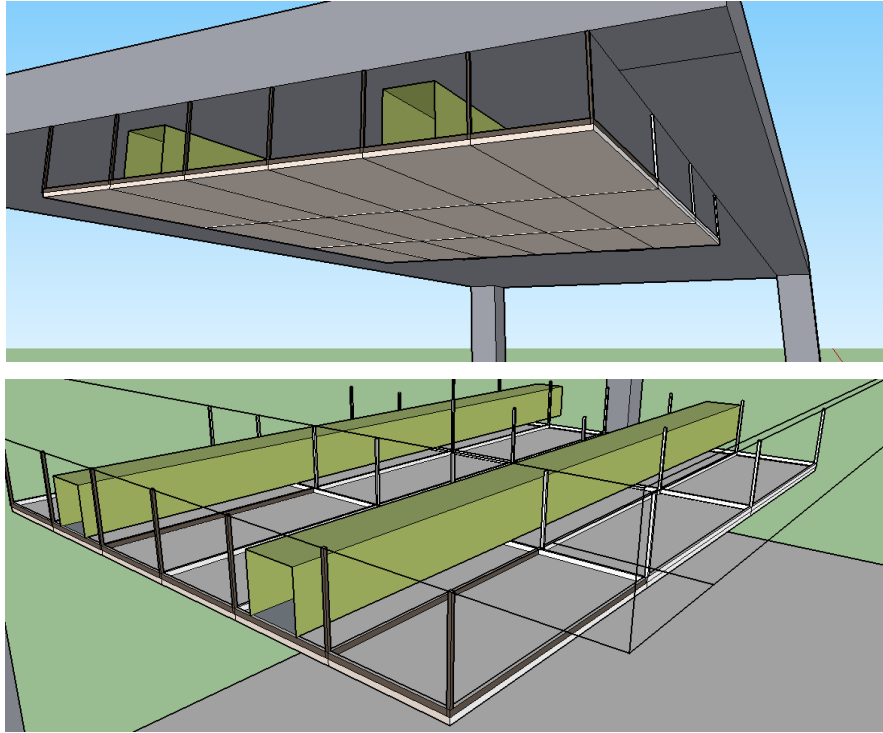


Figure 2.12. Ceiling components (Google Sketch Up 3D model)

2.2.3 Glazing

As the “Utopia Hotel Astana” is initially designed as 5 star hotels with high service and comfort for guests, appearance of a building is significant as it plays main role in attracting guests. Covering hotel building with glass makes it aesthetically pleasing as neighboring buildings and skyscrapers in Astana are mainly covered with glass. Taking into account all these criteria, it was decided to cover the hotel with glass panels. The covering of the building is shown in Figure 2.13 and 2.14.

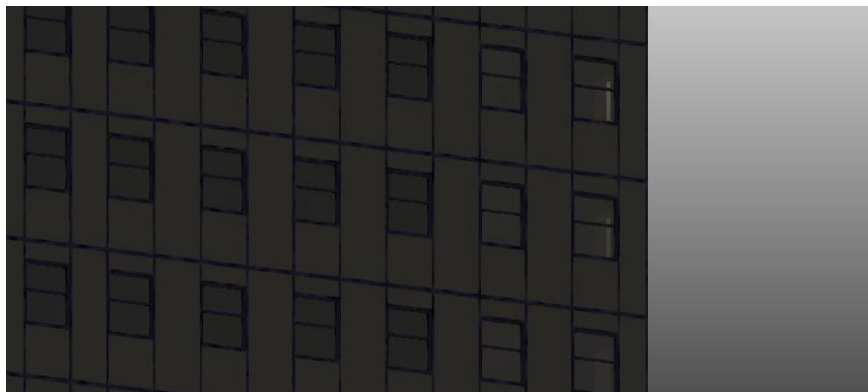


Figure 2.13. Glazing of a hotel (AutoCAD 3D model, Rendering model)

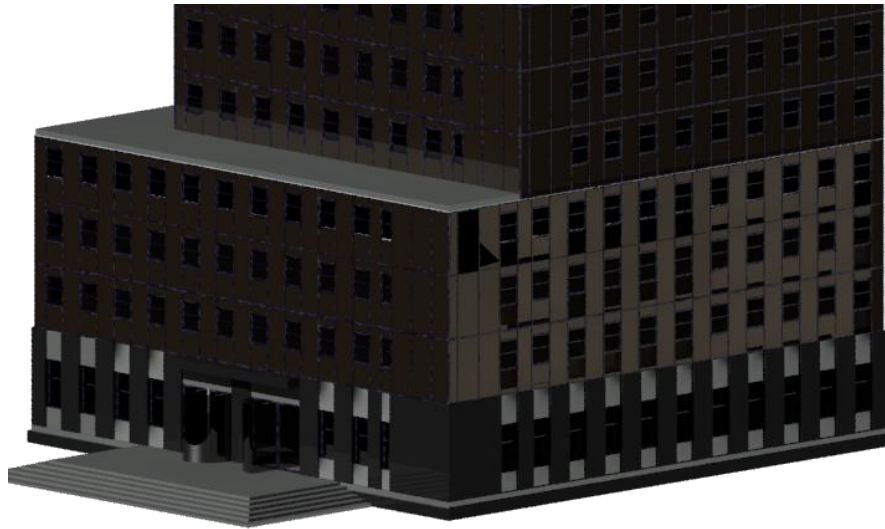


Figure 2.14. Isometric view of a main entrance (AutoCAD 3D model, Rendering model)

2.2.4 Drainage Systems

It is necessary to take the appropriate measures against the ingress of rain, snowmelt, groundwater bearing strata and the building envelope as well as the formation of an unacceptable amount of condensation in the outer protective structures by sufficient sealing structures or ventilation device enclosed spaces and air gaps. Necessary protective compounds must be used and the coating in accordance with applicable regulations.

2.3 Parking Area

Garages and parking lots should be designed according to the requirements of Intergovernmental Building Codes and other existing documents on fire safety (MCH 2.02-05-2000).

Hotels with category *** and above should provide parking lots. The number of parking places depending on the category of hotel should not be less than as specified in Table 2.3. As “Utopia hotel Astana” is designed as a five-star hotel, the percentage of number of rooms should be taken as 30%.

$$\text{Number of places} = \text{Percentage} \times \text{Number of rooms}$$

$$\text{Number of places} = 30 \% \times 154 = 46.2 \approx 47$$

Table 2.3. Percentage of parking places depending on a category of the hotel

Types of hotel	The number of parking places, as a percentage of the number of rooms at the hotel category
----------------	--

	*	**	***	****	*****
Hotels	10	10	20	30	<u>30</u>
Motel	80	80	80	80	-

Furthermore, it should be taken into account that hotels, comprising open to outside visitors catering and trade, should increase the number of parking places up to 20 %.

So, number of places is increased as follows:

$$\text{Number of places} = 47 \times 1.2 = 55.44 \approx 56$$

The dimensions of one parking space should be taken as: for passenger cars – 2.5 m x 5.5 m, for special vehicles designed for disabled people – 3.8 m x 8.0 m. Car parks of hotels with more than 200 residents should provide at least 3 parking spaces for disabled people. Parking spaces for disabled should be located as close to the entrance of the building. Special ramps and lifts should be equipped in accordance with regulations stated in GDC RK 3.01-05-2001.

2.4 Site Layout

As a part of outside zone, following areas must be provided:

- Landscaped area in front of the entrance to public buildings and residential purposes (not less than 0.2 m² per resident);
- Parking space;
- A platform for temporary parking of vehicles and buses;
- Economic area, isolated from the guest area, for trucks entrances (for hotels with capacity more than 100 residents, passage width should be at least 4.5 m with reversal area of at least 12 m x 12 m).
- The outdoor area for short-term parking at the main entrance is designed on the basis of simultaneous placement of 5 vehicles and one bus for every 200 residents.

The site layout and scheme was designed considering all these requirements. In addition to that, in newly constructed hotels, all conditions should be provided for disabled people traveling in wheelchairs. It is required to provide single and double rooms for disabled with all appropriate equipment, including the size of bathrooms, wide aisles and doorways. Devices for free movement of people with disabilities on

horizontal and vertical ways from lobby to the specified numbers should be designed according to GDC RK 3.01-05-2001.

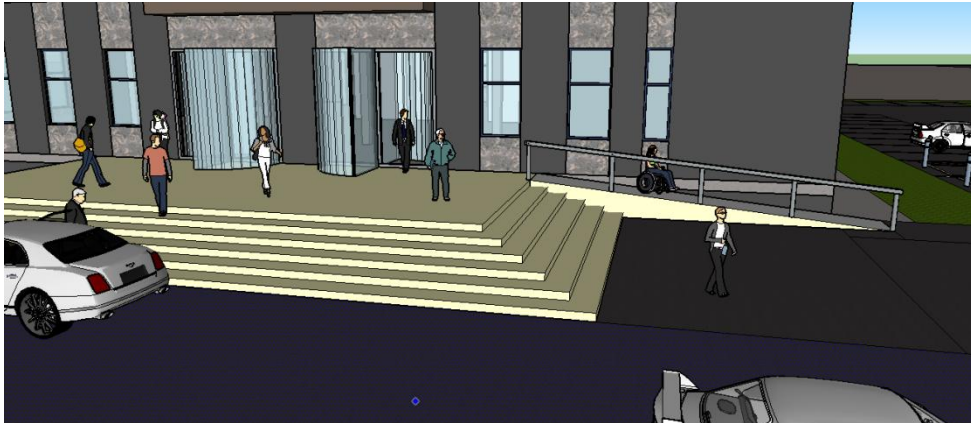


Figure 2.15. Main entrance of a hotel (Google Sketch Up 3D model)



Figure 2.16. Parking lots of a hotel (Google Sketch Up 3D model)

Porch of the main entrances must be equipped with ramps having slope no more than 1:12 and fence with the height of not less than 0.45 meters as shown in Figure 2.15. Furthermore, entrances to the hotel category 4 and 5 star should be equipped with sliding doors or revolving doors with electronic devices as shown in Figure 2.16.

2.5 Safety Requirements

2.5.1 Durability and Maintainability

In accordance with building code of the Republic Kazakhstan, at observance of established rules, supporting structures of a building must retain their properties in agreement with requirements of rules and regulations over estimated useful life, which is established in the design of a project. Components, equipment, elements with lifetime of less than the expected lifetime of the building, must be replaceable in conformity with requirements of a design.

Building structure and assembly parts must be made of materials which have resistance to possible effects of moisture, aggressive environment, low temperatures, chemical and other adverse factors.

2.5.2 Fire-Technical Requirements

2.5.2.1 Fire Barriers

Fire barriers are designed to prevent the spread of fire and combustion products from the room or fire compartment with a fire site to the other rooms. Fire barriers characterized by fire resistance and fire peril. Fire barriers should be determined according to Section 9 of technical regulations in “General requirements for fire safety”. Fire barriers include: walls, partitions, slabs, gaps, curtains and screens. In “Utopia Hotel Astana” building, only walls, slabs, gaps and partitions may be designed. The minimum fire resistance limits and types of fire barriers or elements should be taken according to Table 2.4 obtained from technical regulations. The values indicated in Tables 2.4 and 2.5 are the resistances that fire barriers should have.

Table 2.4. Limits of fire resistance of fire barriers.

Fire Barriers	Type of fire barriers	Limits of fire resistance of fire barriers
Walls	1	REI 150
	2	REI 45
Partitions	1	EI 45
	2	EI 15
Building slabs	1	REI 150
	2	REI 60
	3	REI 45
	4	REI 15

The fire doors, valves, gates, windows and hatches should be installed in the openings of fire barriers. Their characteristics should be used in accordance with Table 2.4.

Table 2.5. Limits of fire resistance of the filling openings in fire barriers

Elements of openings in fire barriers	Type of openings in fire barriers	Fire resistance limits
Doors, gates, hatches, blinds, screens.	1	EI 60
	2	EI 30
	3	EI 15

Doors of elevator shafts	2	EI 30
Windows	1	E 60
	2	E 30
	3	E 15
Fire-retardant valves	1	EI 90
	2	EI30
	3	EI115

2.6 Stairs and Staircases

According to specifications agreed with the state fire services, for accommodations with the capacity of 50 or more people, it is necessary to provide at least one emergency exit through smoke tight stairwells. Fire-technical classification of stairs and staircases, designed for evacuation in accordance with Section 10 from technical regulations “General requirements for fire safety”. For lifting to a height more than 20 meters, fire ladders of type P2 should be used, which are marching steel stairwells with a slope of not more than 6:1, 0.7m wide, starting from a height of 2.5 m above the ground, with platforms not less than 8 m and handrails as it can be seen from Figure 2.17.

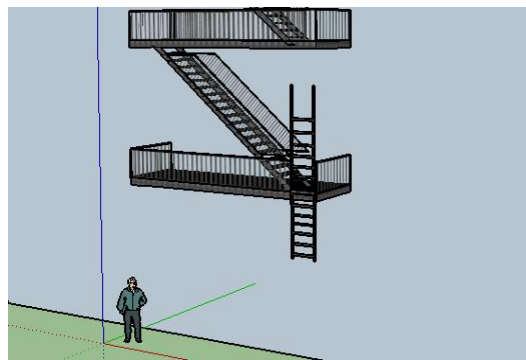


Figure 2.17. Exterior fire escape (Google Sketch Up 3D model)

2.7 Ensuring the Safety of People

2.7.1 Evacuation and Emergency Exits

This section is required to aim at:

- Ensuring a timely and unimpeded evacuation of people;
- Protection of people on the escape routes from the effects of fire hazards.
- Rescuing people, who are not able to evacuate timely due to their age, health conditions or blockings of escape routes.

Fire alarm systems and smoke protection of buildings must be in accordance with the requirements of Section 3.2.2 of technical regulation in “General requirements for fire safety”.

Fire detection systems, warning and evacuation control in case of fire should provide automatic detection of fire and turning on warning systems of notification for fire, in order to organize the evacuation of people in a particular object.

Requirements for evacuation and emergency exits are determined by the technical regulations described in Sections 3.3.5 and 3.3.6 in “General fire safety requirements”.

Exits are evacuative, if they are:

- From premises of the first floor to the outside through the corridor, lobby, stairwell or across the hall and lobby, across recreational area and the stairwell.
- From premises of any floor except the first directly to a staircase or ladder type 3 as illustrated in Figure 2.17.

Floors of buildings classes: F1.1; F1.2; F2.1; F2.2; F3; F4 must have at least two emergency exits. As “Utopia Hotel Astana” building is classified as F1.2, every floor should have two emergency exits. The design of a hotel has two emergency exits to a staircase in every floor plan of a hotel as shown in Figure 2.18. Maximum distance between the farthest apartment and farthest fire staircase is 46 meters.

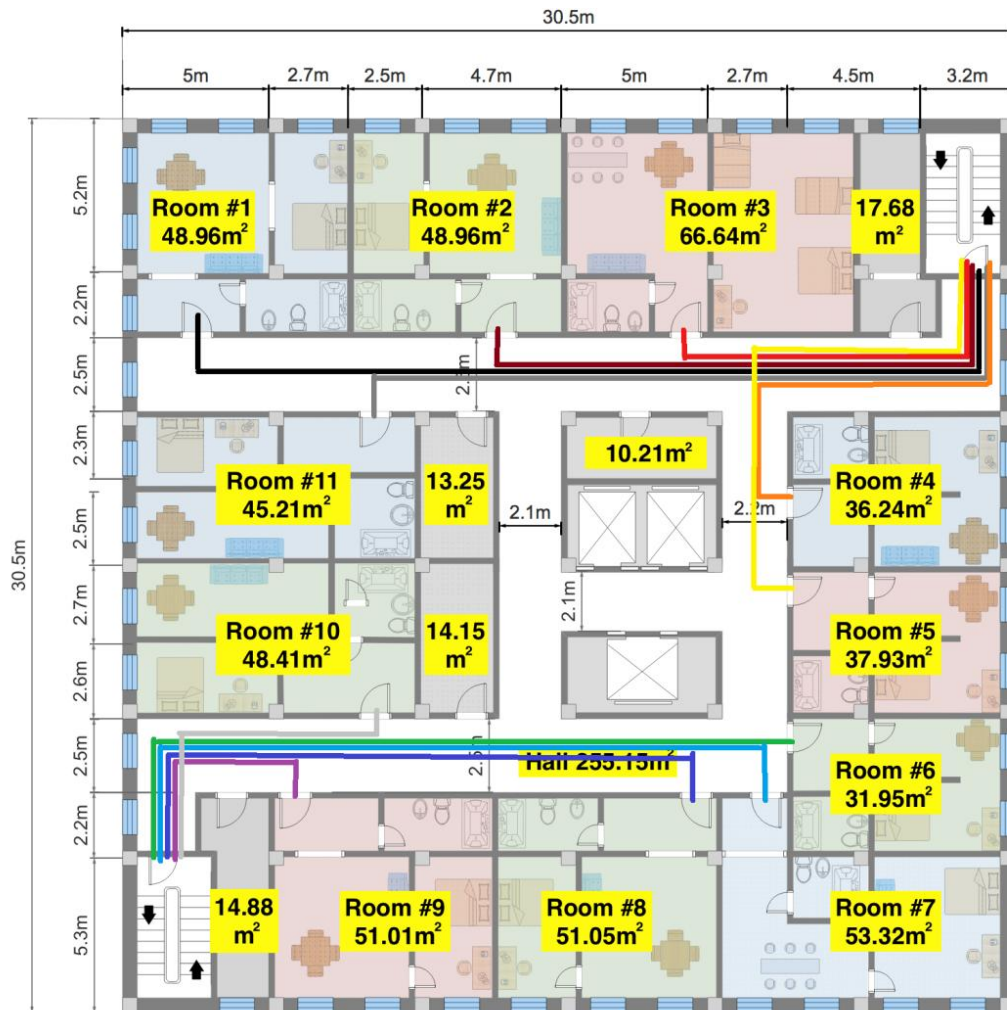


Figure 2.18. Evacuation routes for typical floor of a building

2.7.2 Escape Routes

Design of escape routes should be carried out in accordance with the requirements explained in Paragraph 3.3.5 of technical regulation “General fire safety requirements”. Escape routes should be dealt in accordance with the requirements building regulations. The height of the horizontal sections of evacuation routes should be at least 2 m, width of the horizontal sections of evacuation routes and ramps must be at least:

- 1.2 m – for the common corridors, which can be evacuated from the premises F1 more than 15 people, from the premises of the other classes of functional fire danger – more than 50 people;
- 0.7 m – for passes to single jobs;
- 1.0 m – in all other cases;

In technical regulations of “General requirements for fire safety” do not provide minimum travel distances to exit access. However, this document permits to use other international building codes. So, using International Building Code (IBC), in chapter 10, means of egress, section 1017, minimum exit access travel distances are given as in Table 2.6. According to building occupancy classifications, places providing accommodations for overnight stay such as apartment buildings, hotels, motels and houses are classified as residential (Group R). In addition, design of a hotel includes sprinkler system for every floor. So, minimum travel distance to exit access is 250 feet or 76.2 meters. As it was mentioned in Section 2.6, maximum distance between farthest room and farthest exit is 46 meters, hence the travel distance to exit time is satisfied by international building code.

Table 2.6. Exit access travel distance

Occupancy	Without Sprinkler System (Feet)	With Sprinkler System (Feet)
A, E, F-1, M, R, S-1	200	250
I-1	Not Permitted	250
B	200	300
F-2, S-2, U	300	400
H-1	Not Permitted	75
H-2	Not Permitted	100
H-3	Not Permitted	150
H-4	Not Permitted	175
H-5	Not Permitted	200
I-2, I-2, I-4	Not Permitted	200

2.7.3 Evacuation by Stairs and Staircases

The width of the ladder march intended for the evacuation of people, including those located in the staircase, should to be estimated so that it will not be less than the width of any emergency exit (door) to the ladder, but not less than:

- 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;

- 0.7 m - for the stairs leading to single workplaces;
- 0.9 m - for all other cases.

As the hotel is designed for more than 200 guests, the width of a ladder march should not be less than 1.35m. Tread width should be not less than 0.25 m and the step height not more than 0.22 m. Stairs must meet the established requirements and the length of the escape route stairs should be equal to three times the height of it. The width of staircases must be less than the width of the march. The intermediate pad forward march should be at least 1 m. Staircases must have a way out on the territory adjacent to the building, either directly or through the lobby, separated from the adjacent corridors with doors partitions. When the device of emergency exits of the two staircases through a common lobby for at least one of them, in addition to access to the lobby, should have an exit directly to the outside. Stairwells, except ladder type of A2, should have glazed apertures measuring at least 1,2 m² in exterior walls on each floor. In buildings with more than two floors should provide smoke protection in the event of fire, common corridors, lobbies hallways, lobby and elevator shafts. So, the staircase design with all requirements satisfied is illustrated in Figure 2.19.

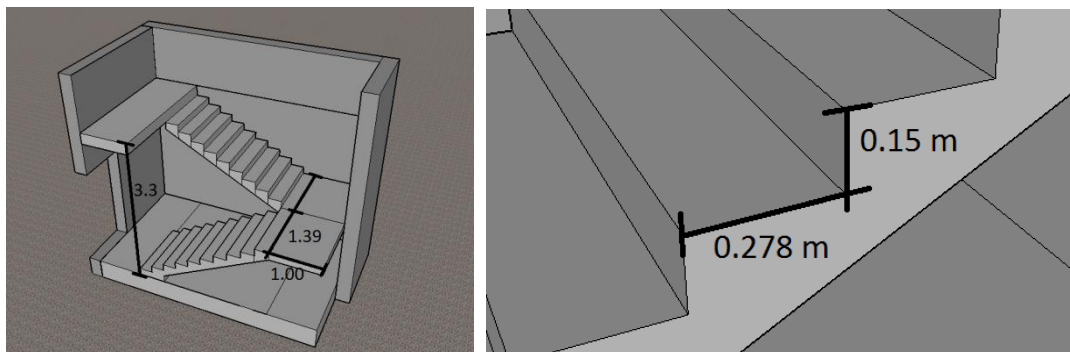


Figure 2.19. Views of staircase (Google Sketch Up 3D model)

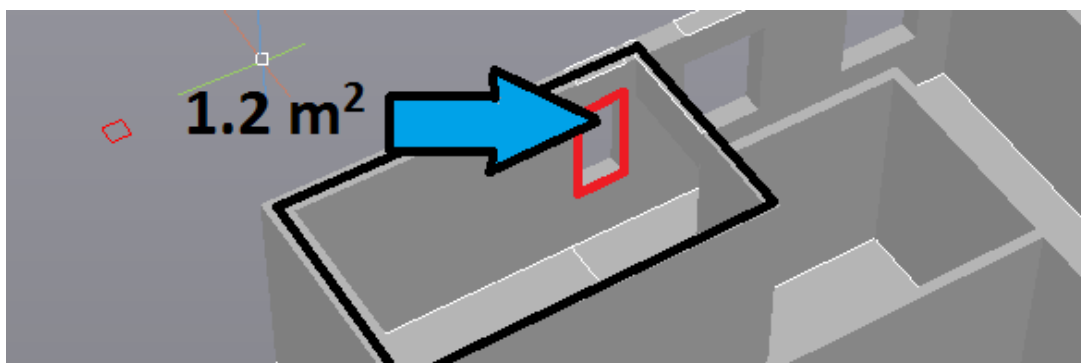


Figure 2.20. Views of staircase (AutoCAD 3D model)

2.7.4 Elevators and elevator shafts

Fire safety requirements for lifts are defined in Section 3.7.4 of technical regulation "General requirements for fire safety." Limits of fire resistance of structures and materials, as well as control systems, alarm systems, telecommunications and electric power supply of lifts must comply with the requirements of "Safety requirements of elevators". Passenger elevators with automatic opening doors with the speed of 1 meter/sec or more must have a mode indicating the fire hazard, activated by signals coming from the systems of automatic fire alarm of a building and should return lift, regardless of the load and direction of movement of an elevator, to the main landing field opening and should open the elevator and hoist way doors. Furthermore, according to regulations, Fire lifts should be provided not less than:

- 2 in. the fire compartment of buildings taller than 16 floors;
- 1 in. a fire compartment of buildings in height from 10 floors to 16 floors, and a multi-storey underground space with two or more floors.

For the 15-storey high hotel, one fire lift will be provided that would satisfy safety requirements of elevators. Fire lifts must have capacity of at least 1000 kg and the ability to accelerate recovery in buildings taller than 16 floors. Cabs of fire lifts must be made of non-combustible materials and fire elevator control system should be capable of operating in elevators normal operation, as well as in the "fire hazard". So, under normal circumstances, fire elevators may be used along with passenger lifts. Power electric receivers must be supplied from two independent transformers and back-up power as a diesel generator. Consequently, 3 elevators, one fire and two passenger lifts, are designed as shown in Figure 2.20 following the safety requirements of elevators.

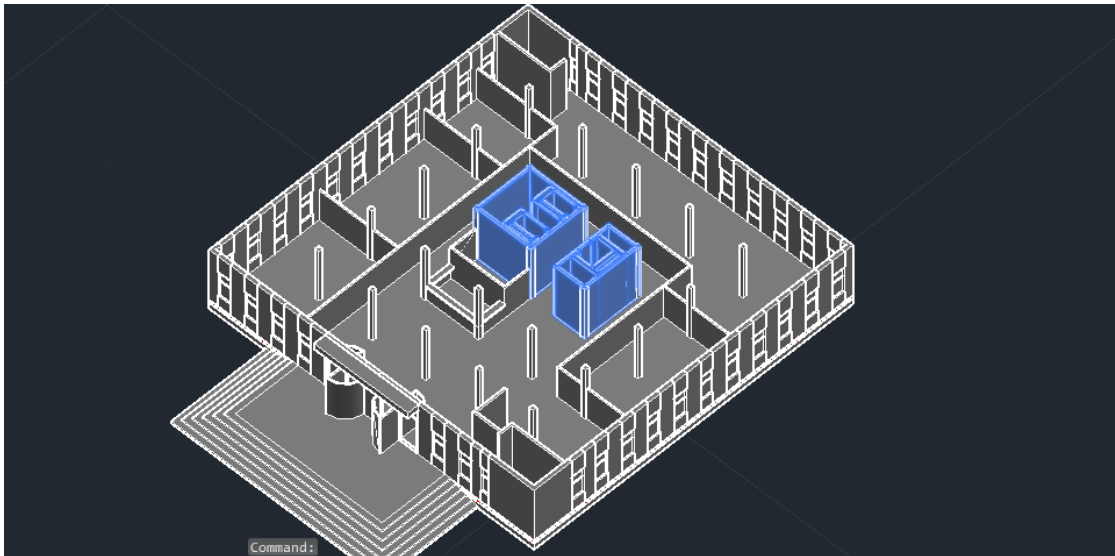


Figure 2.21. Custom view of the first floor with shaded elevator shafts including fire lift's shaft (AutoCAD 3D model)

2.8 Partition

A partition wall is non-load bearing wall which serves for breaking the interior space into several parts. In other words, the main function of this structure is to separate one room or portion of a room from another. The division can be made up of bricks, studding, glass or other such materials. Based on the primary materials used, the partitions can be in different forms such as gypsum board, glass or transforming (HouseUnderConstruction, 2016). The majority of partitioning walls will be designed using the gypsum boards. There are four types of them which are conventional, fire resistant, moisture-proof boards and the combined ones. In the Figure 2.22 below, the gypsum wall with fire resistant characteristics. All these boards can be differentiated by their manufactured colors. For the installation of gypsum boards, the ceiling, angular and leading profiles and their accessories such as suspensions anchor, single or double layered connectors and extenders will be required (OSPANOFF, 2013).



Figure 2.22. Fire resistant gypsum board (OSPANOFF, 2013)



Figure 2.23. Ceiling Profile (OSPANOFF, 2013)

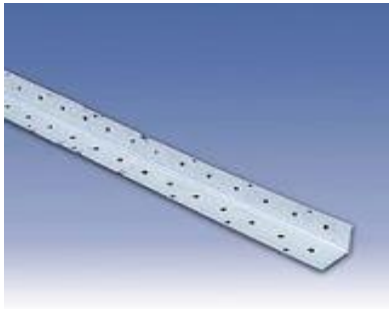


Figure 2.24. Angular profile



Figure 2.25. Double layered profile connector (OSPANOFF, 2013)

- Ceiling Profile Characteristics:
 - Cross section, mm: 60x27
 - Length, mm: 3000
- Leading Profile Characteristics:
 - Cross section, mm: 28x27
 - Length, mm: 3000
- Angular Profile Characteristics:
 - Dimensions, mm: 20x20x3
 - Length, mm: 3000

2.9 Building Envelope Thermophysical Properties

The building envelope is the physical separator between the interior and exterior surroundings of the building. The main components of building envelope include walls, floors and roofs, and they can be characterized in terms of density, roughness, thermal conductivity and specific heat. These thermophysical properties are described in the Table 2.7 below. Since all construction elements are made up of reinforced concrete, their properties are similar.

Table 2.7. Building Envelope Properties (Mastter 2011) and (Matbase n.d.)

	Wall	Floor	Roof
Density	2500 kg/m ³	2500 kg/m ³	2500 kg/m ³
Thermal Conductivity	2.2 W/m.K	2.2 W/m.K	2.2 W/m.K
Specific Heat	840 J/kg.K	840 J/kg.K	840 J/kg.K
Roughness	Medium	Medium	Medium

3 STRUCTURAL DESIGN

3.1 Design Load Calculation

Since Kazakhstani building regulations (SNiP) are not restricted to adopt approved international codes, the Energy Efficient Construction team is applying American agreed standards used by American Society of Civil Engineers (ASCE). To design and analyze building structures as beam, column and slab, estimation of minimum load requirements are needed. Thus, in this section of the report calculation procedure of required dead and live actions will be provided by using standards of ASCE 7-05 for “Minimum design loads for buildings and other structures”.

Since, estimation of wind actions provided by ASCE is limited by United States, we cannot use American codes for estimation of wind loads for building in Astana. Thus, our team is using Eurocode for prediction of wind loading.

3.1.1 Dead Load

Dead load is defined as sum of weight of all materials and equipment incorporated in construction, such as, walls, ceilings, roofs, floors, windows, stairways, cladding, finishes, cranes, plumbing stacks, electrical feeders, other structural and architectural elements.

Table 3.1 Calculation of Dead Loads

Name of element	Quantity	Section Area (m ²)	Height/Length (m)	Volume (m ³)	Unit weight (kN/m ²)	Weight on each floor (kN)
Columns I	49	0.303	3	0.9075	25	1111.6875
Columns II	49	0.203	3	0.6075	25	744.1875
Columns III	49	0.123	3	0.3675	25	450.1875
Slabs	1	900	0.18	162	25	4050
Major beam	84	0.125	5	0.625	25	1312.5
Minor beam	36	0.07	5	0.35	25	315
Walls	1	284.4	0.3	85.32	25	2133
Windows	42	1.8			0.61	46.116

Total weight I typical floor	8968.3035
Total weight II typical floor	8554.6875
Total weight II typical floor	8260.6875

3.1.2 Live Load

Live load is variable free action that produced by utilization and occupancy of the structure, excluding wind load and other environmental actions. Live load should be highest amount of expected load, exceeding minimum uniformly distributed load required (Table 3.2). If position of concentrated live load is not specified, action point should be located as to produce maximum load effect.

Table 3.2 Minimum Uniformly Distributed Live Loads, L_0 (Table 4-1, ASCE 7-05)

Occupancy or Use	Uniform Live Load (kN/m^2)
Lobbies and first floor corridors	4.79
Dining rooms and restaurants	4.79
Fire escapes	4.79
Residential public rooms and corridors	4.79
Ordinary flat roofs	0.96
Storage areas above ceilings	0.96

As type of current hotel building belongs to residential public rooms and corridors, the live load is assumed to be $4.79 kN/m^2$. For typical floor, with dimensions $30m \times 30m$, live load can be computed as $4311 kN$.

3.1.3 Wind Load

Since, ASCE is restricted for United States the values from wind actions decided to be computed according to the regulations of Eurocode (EN 1991-1-4:2005).

The hotel building that exposed to wind actions in Astana has following parameters: $h=51,35 m$, $b=30 m$, $d=30 m$. In order to obtain required values of loading, the fundamental basic wind velocity ($V_{b,0}$) for Astana should be obtained. In other words, the 10-minute mean wind velocity at height of 10 m regardless of direction needed to be used. According to the national meteorological center, the fundamental

wind velocity in Astana equals to the 37.8 m/s with equivalent return period of 50 years (Popov, 2015).

The fundamental value of wind velocity should be modified to the basic wind velocity taking into account seasonal and directional factors. According to the regulations, in our case both values are recommended to be taken as 1, thus $V_b = 37.8 \text{ m/s}$.

$$V_b = c_{dir} \cdot c_{season} \cdot V_{b,0} \quad (3.1.1)$$

Obtained value for basic wind velocity is then used to define mean velocity which depends on the height of the structure (z), roughness factor (c_r) and orography factor (c_o):

$$V_m(z) = c_r(z) \cdot c_o(z) \cdot V_b \quad (3.1.2)$$

Orography factor is influence of topographic relief of mountains and hills. Since Astana is not surround with any elevation the value for $c_o(z)$ is taken as 1.

To compute roughness factor Eurocode provides following equation:

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \quad (3.1.3)$$

where, z_0 is the roughness parameter which in our case equals to 0,3 m (terrain III category), k_r terrain factor depending on roughness length z_0 calculated using

$$k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \quad (3.1.4)$$

$$k_r = 0.19 \cdot \left(\frac{0.3}{0.05}\right)^{0.07} = 0.21539$$

Another factor needed to be computed for determination of wind actions on building is turbulence intensity (I_v).

$$I_v(z) = k_l / (c_o(z) \cdot \ln\left(\frac{z}{z_0}\right)) \quad (3.1.5)$$

where, k_l is the turbulence factor that recommended to be 1, c_o orography factor, z_0 roughness length dependent on specific roughness category.

After the estimation of values above, the peak velocity pressure is ready to be computed.

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot V_m^2(z)$$

where, ρ is air density which is taken as 1.25 kg/m^3 .

To find out final value for wind load, it is needed to take into account the mechanism of wind acting on the building. Since, the building can be exposed by windward and leeward sides of the building the external pressure coefficients used to find total wind action.

Total load from wind actions in terms of force are computed in the table below, where values of mean velocity (V_m), turbulence intensity (I_v), peak pressure (q_p) and external coefficients (coefficients D and E) are used to determine wind loading.

Table 3.3. Characteristic values for each building floor

Floor	Height, <i>m</i>	<i>h/d</i>	Roughness factor, <i>C_r(z)</i>	Mean velocity, <i>V_m(z), m/s</i>	Turbulence intensity, <i>I_v(z)</i>	Peak velocity pressure <i>q_p(z), kN/m²</i>
1	5.15	0.17	0.61	23.15	0.35	1.16
2	8.45	0.28	0.72	27.18	0.30	1.43
3	11.75	0.39	0.79	29.86	0.27	1.62
4	15.05	0.50	0.84	31.88	0.26	1.77
5	18.35	0.61	0.89	33.49	0.24	1.89
6	21.65	0.72	0.92	34.84	0.23	2.00
7	24.95	0.83	0.95	35.99	0.23	2.09
8	28.25	0.94	0.98	37.00	0.22	2.17
9	31.55	1.05	1.00	37.90	0.21	2.25
10	34.85	1.16	1.02	38.71	0.21	2.32
11	38.15	1.27	1.04	39.45	0.21	2.38

12	41.45	1.38	1.06	40.13	0.20	2.44
13	44.75	1.49	1.08	40.75	0.20	2.49
14	48.05	1.60	1.09	41.33	0.20	2.54
15	51.35	1.71	1.11	41.87	0.19	2.59

When the values of peak velocity pressure are known it is possible to compute wind loading by summing load from windward and leeward sides of the building. These values are expressed in Table 3.4, where external pressure coefficients were taken into account in combination with peak velocity pressure.

Table 3.4. Wind loading calculation results

Floor	Coef D	Load distr windward (kN/m)	Load windward (kN)	Coef E	Load distr leeward (kN/m)	Load leeward (kN)	Load total (kN)
1	0,69	4,00	32,52	0,28	1,62	14,03	46,55
2	0,70	5,03	17,92	0,31	2,20	8,16	26,08
3	0,72	5,83	20,33	0,34	2,74	9,88	30,21
4	0,73	6,49	22,41	0,37	3,25	11,56	33,96
5	0,75	7,09	24,28	0,40	3,75	13,22	37,49
6	0,76	7,63	26,00	0,43	4,26	14,88	40,88
7	0,78	8,13	27,63	0,46	4,76	16,54	44,17
8	0,79	8,61	29,17	0,48	5,27	18,22	47,39
9	0,81	9,07	30,66	0,51	5,78	19,90	50,57
10	0,82	9,51	32,10	0,54	6,29	21,61	53,71
11	0,84	9,94	33,50	0,57	6,81	23,32	56,82
12	0,85	10,36	34,87	0,60	7,33	25,05	59,93
13	0,87	10,77	36,22	0,63	7,86	26,80	63,02
14	0,88	11,18	37,54	0,66	8,39	28,56	66,10
15	0,89	11,58	19,10	0,69	8,92	14,72	33,82

3.2 Structural Analysis Methods

The main purpose of the structural design is to provide stability, serviceability and safeness of the building. Several forces may act on a structure that may be result of self-weight, imposed actions and environmental forces. Thus, every structure after preliminary design should be checked for strength and stability requirements. When different types of forces are acting on the structure, several assumptions should be made in order to idealize and simplify distribution of loadings, structure and geometry properties.

Buildings should be analyzed for the resultants of bending moment, deflection, shear stress and axial stress. The analysis is known as evaluation of these actions at critical sections and plotting them for the structural component. Whereas, determination of dimensions for structure components of these moments, shears and stresses is known as design. The structures can be divided into determinate and indeterminate structures. If for the first type we need to apply equilibrium equations, for indeterminate type of structures additional methods should be applied. In real life, most of the structures are statically indeterminate, which means that number of unknown reactions is higher than applied equilibrium equations. Thus, it is required to use structural approximate methods that convert indeterminate structure into determinate one.

3.2.1 Analysis under Dead Load and Wind Load

Structural frames consist of beams that rigidly connected to columns to make building resistive enough. Considering an ordinary beam placed on the column and exposed by vertical loading gives frame with statically indeterminate members to the third degree (6 reactions and 3 equations). In order to make structure statically determinate an approximation methods are required.

In a real case beam supports are neither fixed nor simply supported. The critical difference of real frame between fixed supports is that columns provide some flexibility at supports. Thus, there is an assumption that real frames have point of internal moment at average point of case for fixed supports, $0.21L$ for fixed supports (Figure 3.1).

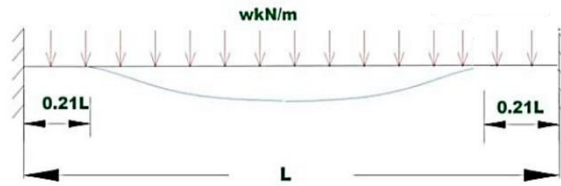


Figure 3.1. Deflected shape and bending moment diagram for fixed beam

However, in practice beams and supports are arranged in a quite different way. Here we apply approximate method to find out points of zero moment provided by columns: $\frac{0.21L}{2} \approx 0.1L$. This is the distance from columns that act in this case as a pin.

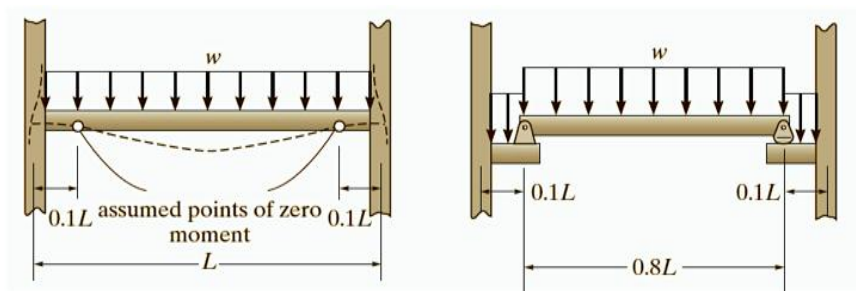


Figure 3.2. Approximation for practical building frame

Overall, following three assumptions convert indeterminate frame into determinate structure:

1. There is an assumed point of zero moment $0.1L$ from the left support
2. There is an assumed point of zero moment $0.1L$ from the right support
3. The beam does not support axial force

3.2.2 Portal Method

In order to design frame of the structure to the lateral loads directed toward it is required to use portal method to the supports at the base of the frame. The approximate analysis of the frame can be investigated through portal method considering following assumptions:

1. First assumption states that under lateral loadings, a frame will deflect in such behavior that a zero moment points will be created at the middle points of the structure members. As it is seen in Figure 3.3, those middle span points are changed to internal hinges (Das et al., 2010).

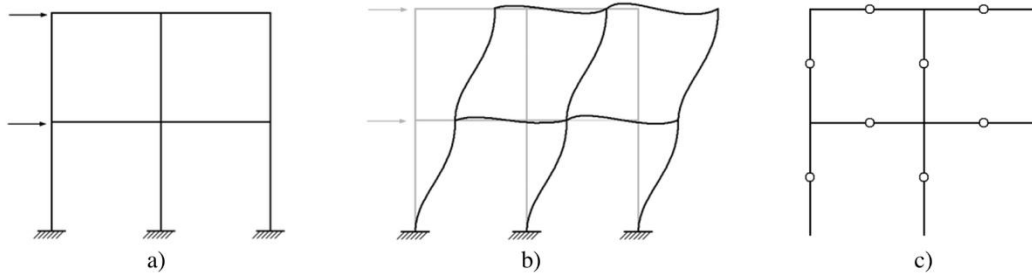


Figure 3.3. Assumed shape of frame under lateral loading and predicted zero moment points

- To make our construction structure determinate, we still need one more assumption. The middle columns carry twice shear base than the reaction force to the two exterior columns (Das et al., 2010).

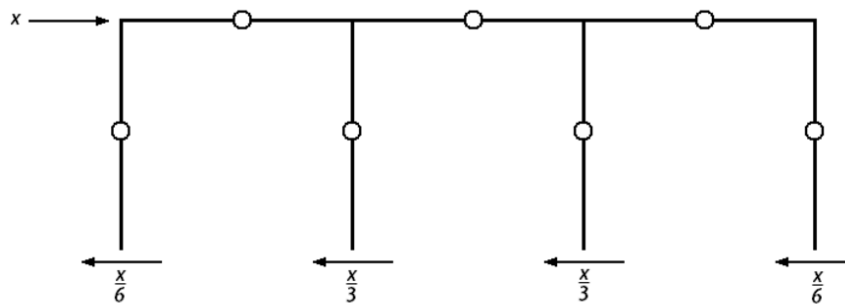


Figure 3.4. The assumed reaction forces of the frame

3.3 Structural Member Dimensioning

3.3.1 Column Sizing

In order to find the size of a column, first of all we have to define all the dead and live loads applied to columns.

Step 1. One way end continuous, $h=180$ mm slab:

The dead load of a typical floor is given as 6.96 kN/m^2 from Table 3.6.

The dead load of roof is 4.5 kN/m^2 from Table 3.7.

Table 3.5. Design loads of materials in a floor layer

Design loads of materials in a floor layer.	
Floor layer	Load (kN/m ²)
Concrete slab	4,32

Concrete screed	0,9
Acoustic insulation layer	0,03
Ceramic layer	0,8
Parquet layer	0,23
Total weight	6,28

Table 3.6. Design load of material in a roofing

Roofing	2500 kg/m ³
depth of a slab	0,180 m
Load of a roof (kg/m ²)	450
Load of a roof (kN/m²)	4,5

So, the total load multiplied by the weight of 14 floors of typical floor and roof gives: $DL\ of\ slabs = 6.28 \times 14 + 4.5 \times 1 = 92.42\ kN/m^2$

Step 2. Dead load of exterior walls. From technical drawing of typical floor plan, it is known that, 10 and 11 windows are located from two sides of a building. Moreover, the height of 14 typical floors is 3 meters and height and width of windows are 1.5 m and 1.2 m respectively. So, area of one window is $1.2 \times 1.5 = 1.8\ m^2$ and area of all windows in each typical floor is: $1.8 \times (10 + 11 + 10 + 11) = 75.6\ m^2$

Total area of four sides of each typical floor is: $4 \times 30 \times 3 = 360\ m^2$. So, by knowing the unit weight of wall and window as a $25\ kN/m^2$ and $0.61\ kN/m^2$, the total load of walls can be found by:

$$Total\ Load\ (walls) = (360 - 75.6) \times 25 \times 0.3 + 75.6 \times 0.61 = 2179.116\ kN$$

It is required to transfer dead load of walls to the load per floor square area:

$$\frac{2179.116}{30 \times 30} = 2.42\ \frac{kN}{m^2}\ for\ each\ floor$$

$$2.42 \times 14 = 33.88\ \frac{kN}{m^2}\ for\ 14\ typical\ floors$$

Step 3. It should be mentioned that dead load of 14 columns needs to be taken into account as 14 columns from above transfers their load to the first-floor columns.

So, initial size of columns was taken as 0.5m x 0.5m column. So, the dead load of columns affecting the first column is:

$$\begin{aligned} \text{Number of columns} \times \text{area} \times \text{height} \times \text{unit weight} &= 14 \times 0.5 \times 0.5 \times 3 \times 25 \\ &= 262.5 \text{ kN} \end{aligned}$$

Step 4. In order to define the total live load of the roof and typical floor plan, Table 3.8 by ACI code was taken. For hotels, minimum distributed live load was taken as 4.8 kN/m² and for ordinary flat roof, minimum uniformly distributed load was taken as 1 kN/m².

So, total live load of all 14 floors and roof: $4.8 \times 14 + 1 = 68.2 \text{ kN/m}^2$

Table 3.7. Minimum uniformly distributed live loads

Occupancy or Use	Live Load (kN/m ²)
Offices	2,4
Corridors above first floor	3,8
Penal institutions	
Cell blocks	1,9
Corridors above first floor	4,8
Residential	
Dwelling (one and two-family)	
Uninhabitable attics without storage	0,5
Uninhabitable attics with storage	1
Habitable attics and sleeping areas	1,4
All other areas except stairs and balconies	1,9
Hotels and multifamily houses	
Private rooms and corridors serving them	1,9
Public rooms and corridors serving them	4,8
Roofs	
Ordinary flat, pitched, and curved roofs	1
Roofs used for promenade purposes	2,9

Roofs used for roof gardens or assembly purpose	4,8
---	-----

Step 5. In order to define the total load applied to first column, we summarize all the loads mentioned above. Tributary area of one column is 25 m² as it shown in Figure 3.5. It should be mentioned that, critical size of columns were estimated, so interior columns were taken due to their maximum tributary area.

$$\begin{aligned}
 N_u &= (1.2D + 1.6L + 0.5 L_r) \times A_{tr} + D_c \\
 &= [1.2 \times (92.42 + 33.88) + 1.6 \times (4.8 \times 14) + 0.5 \times 1] \times 25 + 262.5 \\
 &= 6752 \text{ kN}
 \end{aligned}$$

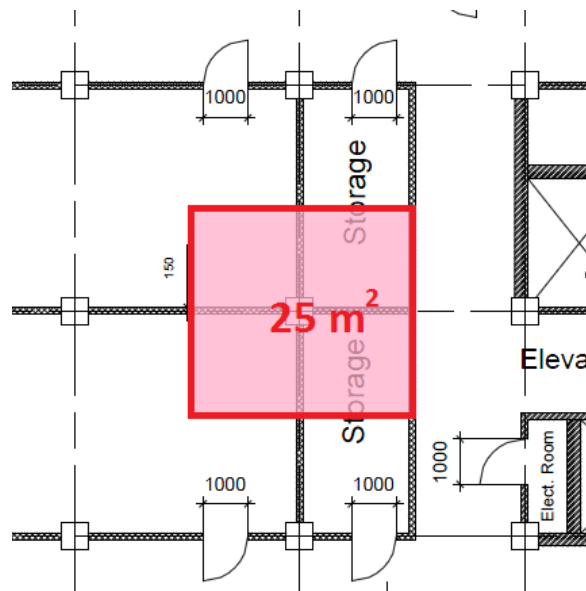


Figure 3.5. Tributary area of one column

Step 6. In order to find the dimensions of column, by using equation from ACI code:

$$N_u = 0.85 \times \left(\frac{\eta f'_c}{\gamma_c} \times A_g + \frac{f'_s}{\gamma_s} \times A_{st} \right) \quad (3.3.1)$$

$$A_{st} = 0.01A_g \quad (3.3.2)$$

where, f'_c is 40 MPa, f'_s 420 MPa, γ_c for flexural load is 1.5 and γ_s for axial load is 1.15 and η , which is compression factor is equal to 1. Substitution of equation (3.3.1) and (3.3.2) gives equation below

$$N_u = 0.85 \times \left(\frac{\eta f'_c}{\gamma_c} \times 0.99 * A_g + \frac{f'_s}{\gamma_s} \times A_g \times 0.01 \right) \quad (3.3.3)$$

$$6752 = 0.85 \times \left(\frac{1 \times 40 \times 10^3}{1.5} \times 0.99 A_g + \frac{420 \times 10^3}{1.15} \times 0.01 A_g \right)$$

$A_g = 0.2648 \text{ m}^2$, consequently, the dimension of square column is $515 \text{ mm} \times 515 \text{ mm}$.

However, by taking into account all other safety considerations, $550 \text{ mm} \times 550 \text{ mm}$ dimensions were taken. This was done due to unknown loads of different type of furniture and different cases when number of residents may over exceed.

3.3.2 Size of Column II

In order to obtain minimum material use and minimum loads applying to columns, the dimensions of columns can be decreased by every stage. However, it is not practical to change the size of column in every stage. So, the dimensions of columns will be changed every 5 floor. This can be obtained by calculating mentioned steps above in order to find the new dimensions of columns for every 5 floor. To find the size of column in stages 6 to 10, loads of ten stories above are considered. So:

$$DL \text{ of slabs (10 floors)} = 6.28 \times 9 + 4.5 \times 1 = 61.01 \text{ kN/m}^2$$

$$DL \text{ of walls} = \frac{2179.116}{30 \times 30} = 2.42 \frac{\text{kN}}{\text{m}^2} \text{ for each floor}$$

$$2.42 \times 9 = 21.78 \frac{\text{kN}}{\text{m}^2} \text{ for 9 typical floor}$$

Dead load of columns affecting the column II is: $9 \times 0.5 \times 0.5 \times 3 \times 25 = 168.75 \text{ kN}$

Total live load of all 14 floors and roof: $4.8 \times 8 + 1 = 44.2 \text{ kN/m}^2$

$$N_u = (1.2D + 1.6L + 0.5L_r) \times A_{tr} + D_c$$

$$= [1.2 \times (61.01 + 21.78) + 1.6 \times (4.8 \times 9) + 0.5 \times 1] \times 25 + 168.75$$

$$= 4393.25 \text{ kN}$$

$$4393.25 = 0.85 \times \left(\frac{1 \times 40 \times 10^3}{1.5} \times 0.99 A_g + \frac{420 \times 10^3}{1.15} \times 0.01 A_g \right)$$

$A_g=0.1723 \text{ m}^2$, consequently, the dimension of square column is 415 mm × 415 mm. However, by taking into account all other safety considerations, 450 mm × 450 mm dimensions were taken.

3.3.3 Size of Column III

To find the size of column in stages 11 to 15, loads of five stories above are considered.

$$DL \text{ of slabs (5 floors)} = 6.28 \times 4 + 4.5 \times 1 = 29.62 \text{ kN/m}^2$$

$$DL \text{ of walls} = \frac{2179.116}{30 \times 30} = 2.42 \frac{\text{kN}}{\text{m}^2} \text{ for each floor}$$

$$2.42 \times 4 = 9.68 \frac{\text{kN}}{\text{m}^2} \text{ for 4 typical floor}$$

Dead load of columns affecting the column III is: $4 \times 0.5 \times 0.5 \times 3 \times 25 = 75 \text{ kN}$

Total live load of all 14 floors and roof: $4.8 \times 4 + 1 = 20.2 \text{ kN/m}^2$

$$N_u = (1.2D + 1.6L + 0.5L_r) \times A_{tr} + D_c$$

$$= [1.2 \times (29.62 + 9.68) + 1.6 \times (4.8 \times 4) + 0.5 \times 1] \times 25 + 75 = 2034.5 \text{ kN}$$

$$2034.5 = 0.85 \times \left(\frac{1 \times 40 \times 10^3}{1.5} \times 0.99A_g + \frac{420 \times 10^3}{1.15} \times 0.01A_g \right)$$

$A_g=0.0798 \text{ m}^2$, consequently, the dimension of square column is 285 mm × 285 mm. However, by taking into account all other safety considerations, 350 mm × 350 mm dimensions were taken.

3.3.4 Beam Sizing

Major beam dimensions: Assume the economic beam depth to be approximately 8 – 10 % of span. In this case it is between 400 – 500 mm, choosing depth to be 500 mm, the typical width to height ratio is 40-60 % of depth, meaning 200 – 300 mm, chose width to be 250 mm, and cross-sectional area $A_{g, \text{major b}} = 0.25 \times 0.5 = 0.125 \text{ m}^2$.

Minor beam dimensions: From the code, beam's depth shall meet the deflection limits. The design does not contain cantilever beam, therefore the most conservative case is simply supported beam design, which has depth $l/16$, meaning $5000/16 = 313$ mm, and the width is $0.6 \times 313 = 188$ mm. Finally, the cross-sectional area $A_{g, \text{minor b}} = 0.188 \times 0.313 = 0.06 \text{ m}^2$.

3.4 Analysis under Wind Load

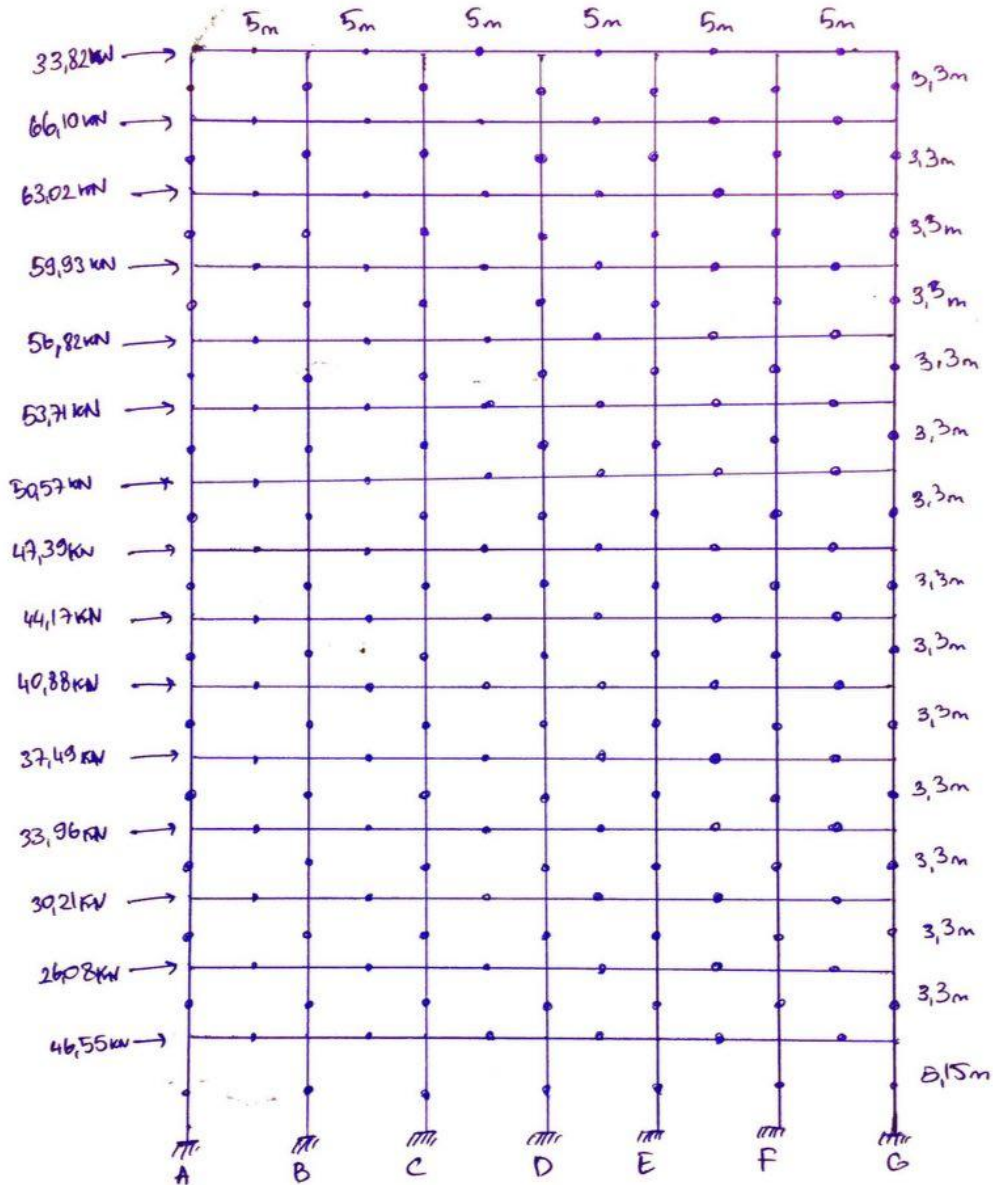
3.4.1 Hand Calculations

Referring to the section 3.1.3 the values for wind loading applied to the structure were found. The values for wind loading were stated in previous sections. In analysis, the wind pressure was considered as a point load applied to the area between middle parts of the floors and span equal to the length of the beam from the side that exposed to the wind. Since the most critical frame of the structure is in the interior part, those loads were analyzed for 2D case.

In order to confirm the correctness of computations obtained from SAP2000, hand calculations were performed. According to the approximation methods described above, the most appropriate method for frame analysis is portal method. Portal method of approximation facilitates calculation of internal forces within acceptable error range. Current method allows to assume values for shear reaction to the horizontal wind load, which is considered as a point load applied to the slab. Additionally, assumption of placement of internal hinges converts indeterminate structure to the determinate one.

In the sheet of hand calculations (Figure 3.6), the value and placement of wind actions on the frame structure are shown. Another hand calculation (Figure 3.7) provides estimation internal forces of 15th floor that was used as an example of the way how values of moment, axial force and shear force were obtained. For further estimation of internal forces same algorithm was used. By using the way of determination of internal loads demonstrated the axial force diagram, shear force diagram and moment diagram (Figure 3.9) were created. It can be seen that shear and axial forces from wind loads transform internal loads from top floors to the bottom. In case of moment, the value of moment gets higher for lower elements of frame structure.

Portal Method of Approximate Analysis for wind loading



Assumptions:

1. The total horizontal shear in all columns of a given story is equal and opposite to the sum of all horizontal loads.
2. The horizontal shear is the same in both exterior columns; the shear in interior column is twice that in an exterior column.
3. The inflection points of all members, columns and girders are located midway between joints.

Figure 3.6. Wind loading condition for portal method analysis

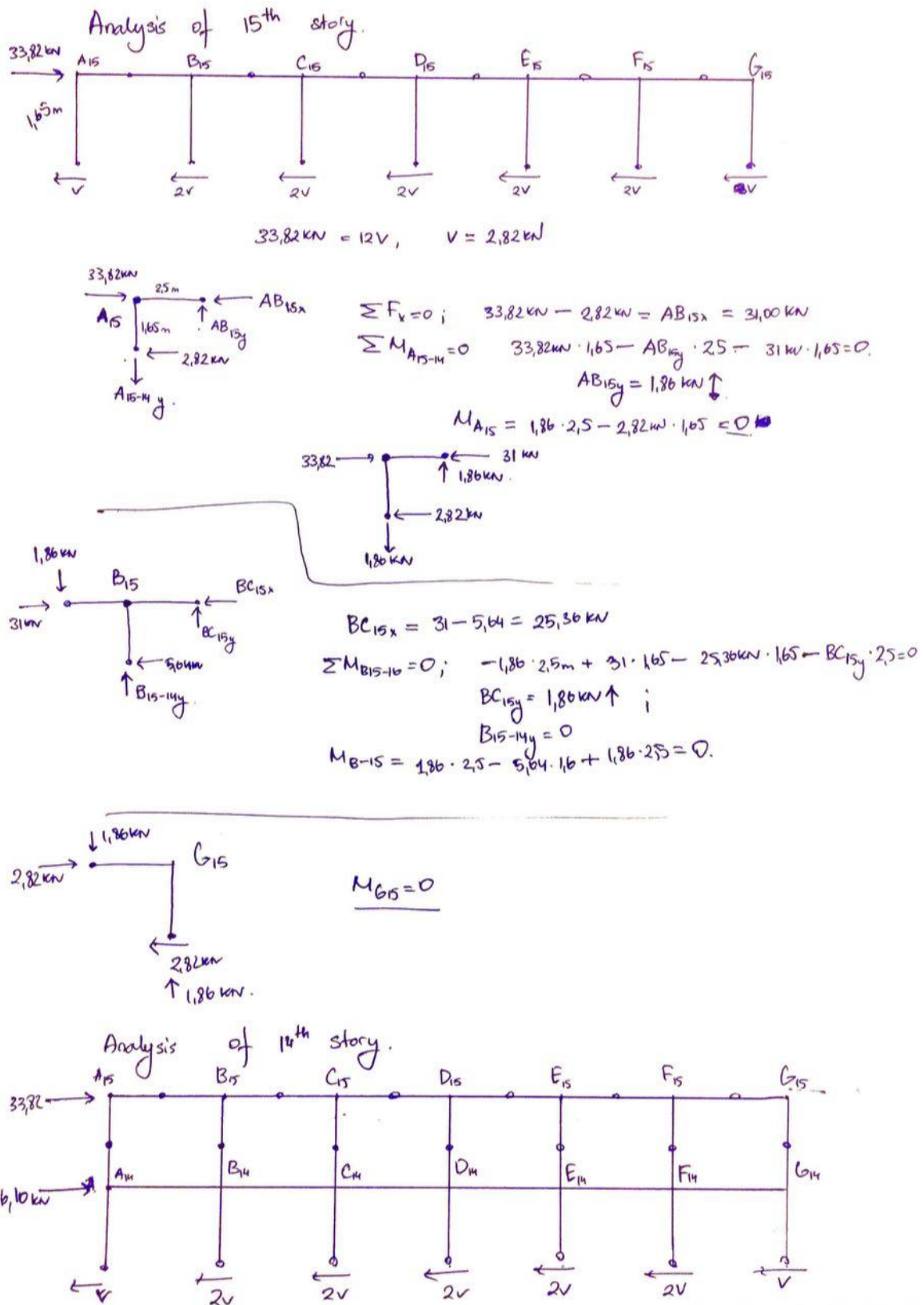


Figure 3.7. Internal forces determination for 15th floor using portal method

In the Figure 3.6, the value and placement of wind actions on the frame structure are shown. The Figure 3.7 describes the hand calculations of internal forces of the 15th floor that was used as an example of the way how values of moment, axial force and shear force were obtained. For further estimation of internal forces same algorithm was used. By using the way of determination of internal loads demonstrated in the Figure 3.7 the axial force diagram, shear force diagram and moment diagram for frame points of first row columns (Figures 3.9) were created.

The Figure 3.8 shows all internal forces present in points from A to O for the first row columns. To estimate these loadings, the method shown in Figure 3.7 was used. As it was stated earlier, assumption that points of internal hinges are located at the middle of structural members was used. Using this algorithm, it is turn for dismember frame at hinges and determine reactions. The estimation is started at the top corner and by applying determined reaction forces the unknown internal loadings for lower spans between hinges can be identified easily.

It can be seen that shear and axial forces from wind loads transform internal loads from top floors to the bottom. In case of moment, the value of moment gets higher for lower elements of frame structure.

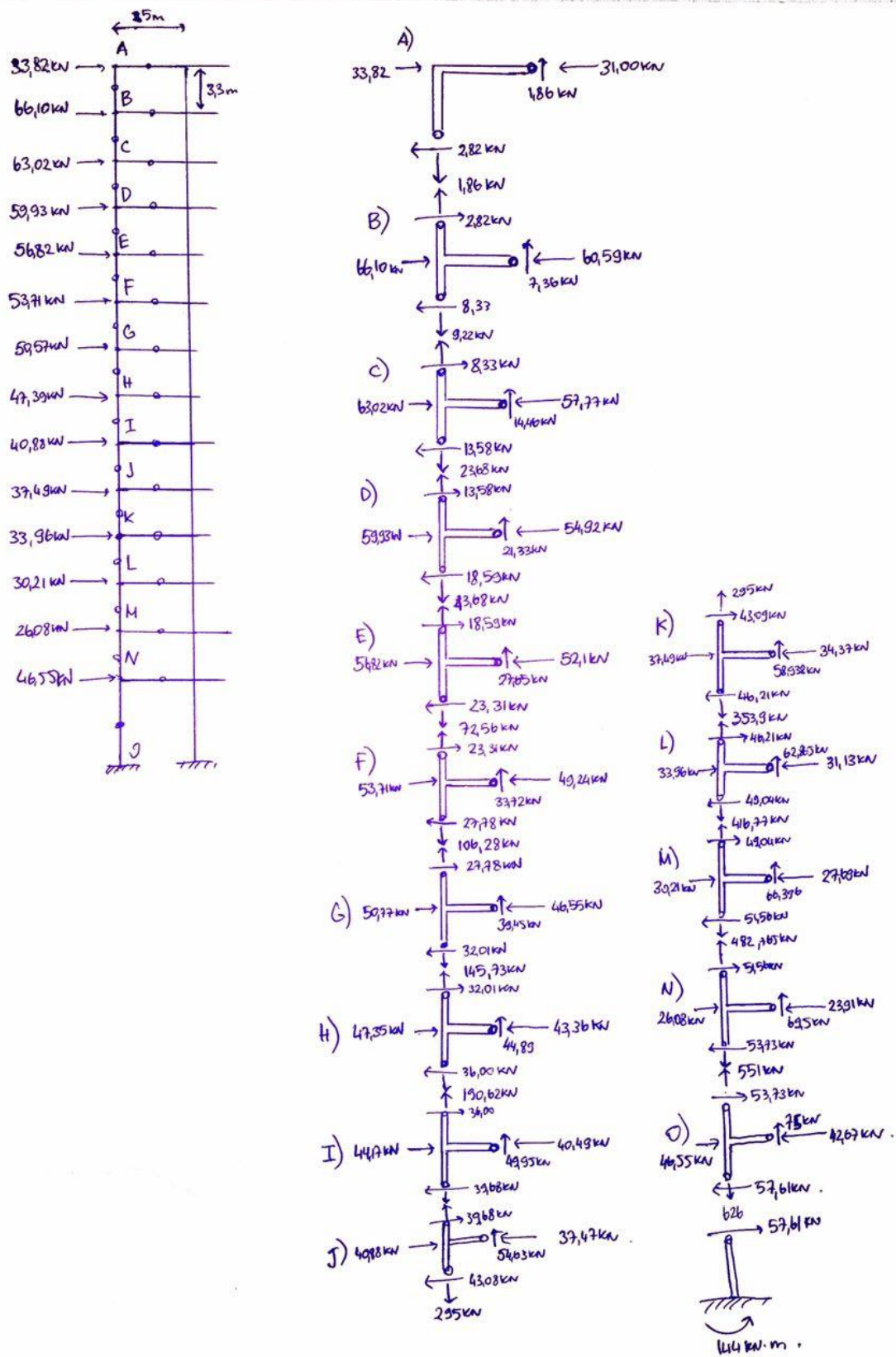


Figure 3.8. Internal forces for points from A to O for the given frame

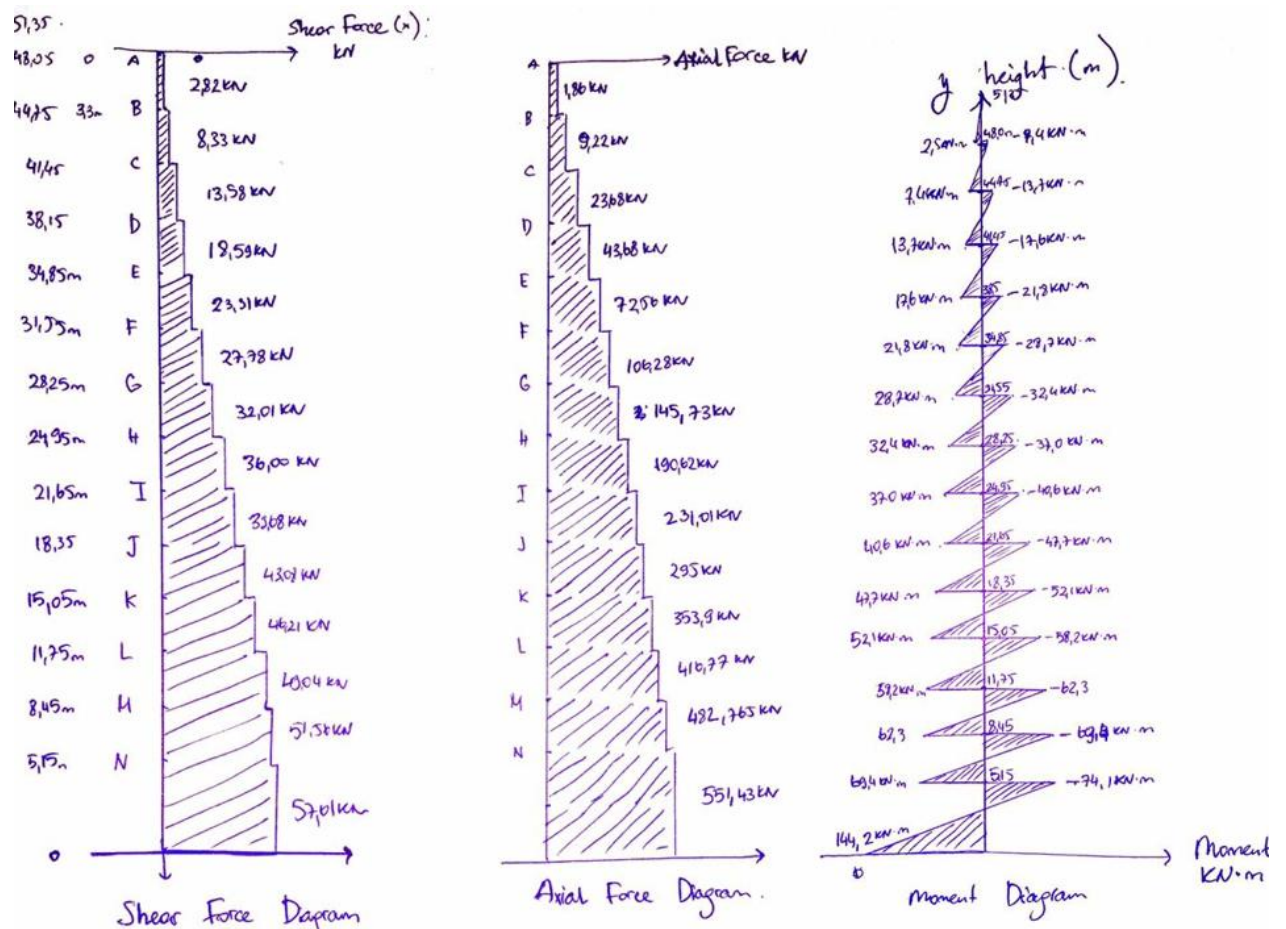


Figure 3.9. Shear, axial and moment diagrams for points from A to O for the given frame.

3.4.2 SAP 2000 Analysis

The analysis of the Utopia Hotel frame was performed via SAP 2000. The virtual 2d frame was created with the help of the software in accordance to design drawings. For this case, the numbers of stories and bays entered for program calculations were 15 and 6, respectively. The length of the girder equals to five meters, consequently the value of bay width is similar. The height of the first story is 5.15 meters and the height of regular stories of 3.3 meters.

Since, the task is to check the frame for wind actions, the self-weight of structural members was neglected. Estimated values of wind loadings from local codes were used as point loads and assigned to the specified points to each storey.

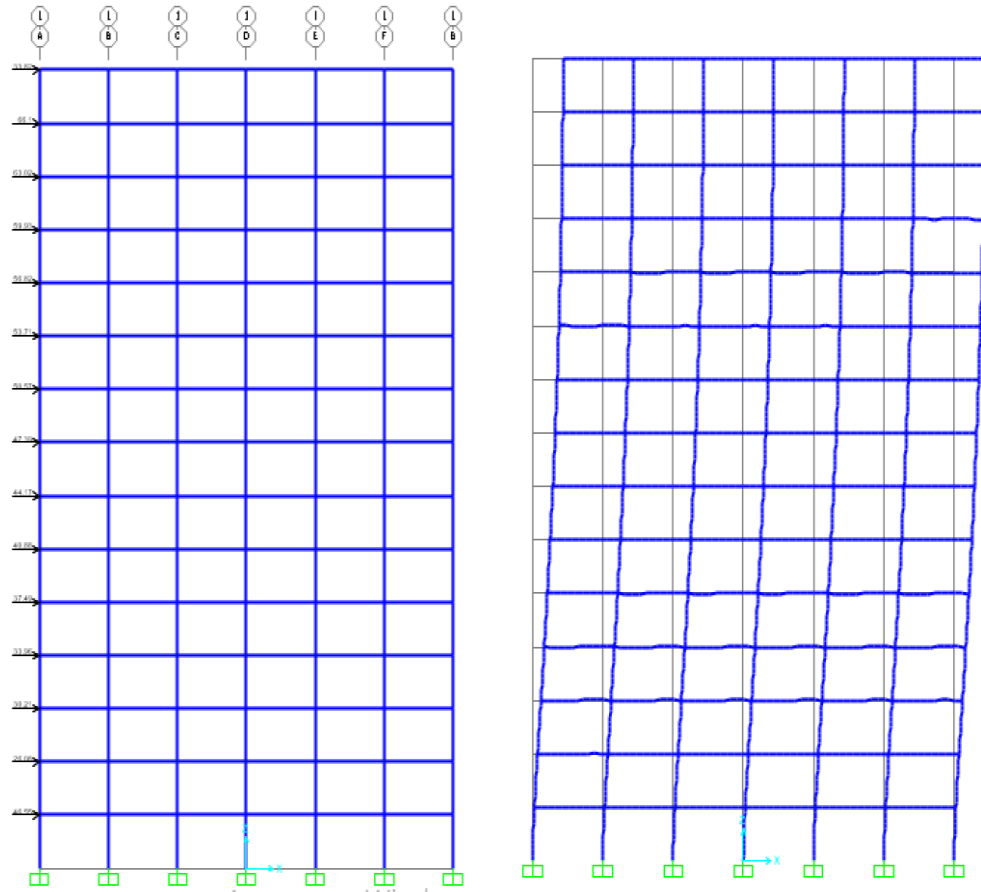


Figure 3.10. Applied wind load to the frame and deformation due to wind load

After entering input data, software outputs results in the form of deflected shape (Figure 3.10) and various diagrams of internal forces. This output data can be used for comparison with hand calculations in order to verify the influence of wind loadings on the whole frame.

It is obvious from software simulation (Figure 3.10) that the structure will deform in direction of load applied. Since there is no any influence of vertical forces and structure is fixed at the bottom, vertical displacement in this case is not observed.

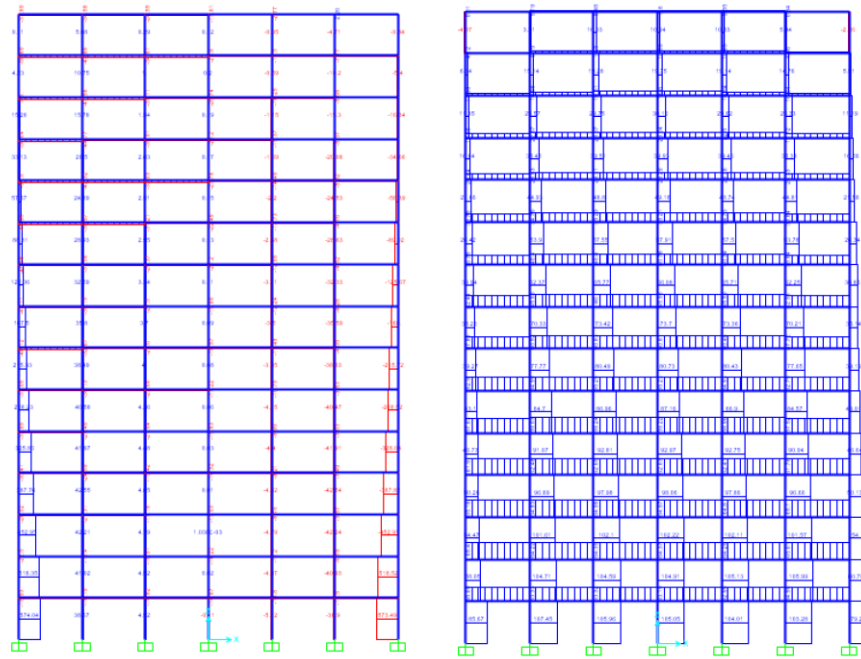


Figure 3.11. Axial force diagram from SAP 2000 and shear force diagram from SAP 2000

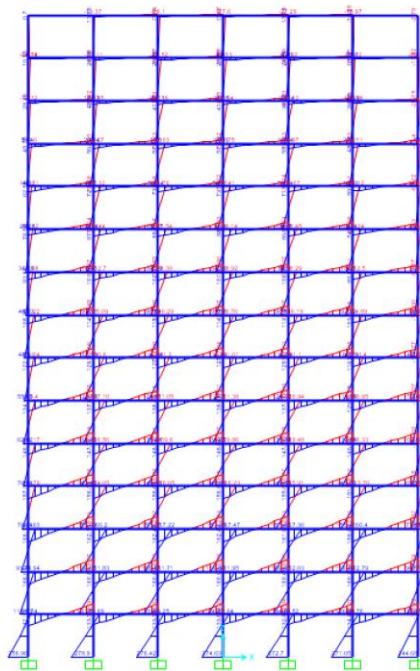


Figure 3.12. Moment diagram from SAP 2000

Comparing the results obtained via SAP 2000 and hand calculations it can be understood that they are almost the same. Since, software is using similar principles for estimation of internal forces the diagrams of moment, axial and shear forces have little

correlation comparing to those that are calculated manually. However, there are some differences that are related to one of the principles of portal method, according to which internal and external columns for same story have same values. In case of software computation, there is some different values of internal forces between same internal and external members. Taking into account how portal method facilitates the analysis of building frame, some slight differences might be considered as insignificant when most of the values are close to the software values. It should be noted that in some cases there is an error present. This owes to the fact that for hand calculations the zero moment points are exactly at the middle of the member, whereas the software uses more accurate points for imaginary internal hinges. Additionally, SAP 2000 does not assume that base of the frame is fully fixed, though it has partial fixity.

Table 3.8. Comparison of SAP2000 simulation and Portal Method results for structural analysis under wind load for points A to O.

Floor	Height (m)	Moment (Hand Calculations) kN-m	Moment (SAP2000) kN-m	Axial (Hand Calculations) kN	Axial (SAP2000) kN	Shear (Hand Calculations) kN	Shear (SAP2000) kN
1	0	144.2	181.7	626.3	594.6	57.6	54.3
	2.58	75.1	93.1	626.3	594.6	57.6	54.3
	5.15	-74.1	-84.3	626.3	594.6	57.6	54.3
2	5.15	69.4	65.9	551.7	543.0	53.8	52.5
	6.8	0.0	-2.3	551.7	543.0	53.8	52.5
	8.45	-69.4	-70.9	551.7	543.0	53.8	52.5
3	8.45	62.3	59.5	482.8	470.4	51.6	50.2
	10.1	0.0	-4.9	482.8	470.4	51.6	50.2
	11.75	-62.3	-58.0	482.8	470.4	51.6	50.2
4	11.75	58.2	54.4	416.8	407.5	49.0	47.5
	13.4	0.0	-5.1	416.8	407.5	49.0	47.5
	15.05	-58.2	-61.3	416.8	407.5	49.0	47.5
5	15.05	52.1	50.0	354.0	347.7	46.2	44.5
	16.7	0.0	-4.4	354.0	347.7	46.2	44.5
	18.35	-52.1	-52.3	354.0	347.7	46.2	44.5
6	18.35	47.7	49.0	295.3	293.5	43.1	42.1
	20	0.0	-1.4	295.3	293.5	43.1	42.1
	21.65	-47.7	-49.9	295.3	293.5	43.1	42.1
7	21.65	40.6	35.3	239.6	234.5	39.7	39.0
	23.3	0.0	-6.2	239.6	234.5	39.7	39.0

	24.95	-40.6	-47.9	239.6	234.5	39.7	39.0
8	24.95	37.0	30.1	190.6	187.6	36.0	35.5
	26.6	0.0	-8.3	190.6	187.6	36.0	35.5
	28.25	-37.0	-38.1	190.6	187.6	36.0	35.5
9	28.25	32.4	26.7	145.7	143.3	32.0	31.1
	29.9	0.0	-7.9	145.7	143.3	32.0	31.1
	31.55	-32.4	-35.2	145.7	143.3	32.0	31.1
10	31.55	28.7	23.3	106.3	104.5	27.8	27.0
	33.2	0.0	-4.3	106.3	104.5	27.8	27.0
	34.85	-28.7	-32.5	106.3	104.5	27.8	27.0
11	34.85	21.8	15.2	72.6	71.0	23.3	21.5
	36.5	0.0	-5.6	72.6	71.0	23.3	21.5
	38.15	-21.8	-28.7	72.6	71.0	23.3	21.5
12	38.15	17.6	14.0	43.7	42.7	18.6	17.5
	39.8	0.0	-4.5	43.7	42.7	18.6	17.5
	41.45	-17.6	-26.4	43.7	42.7	18.6	17.5
13	41.45	13.7	9.3	23.7	22.1	13.6	12.4
	43.1	0.0	-7.4	23.7	22.1	13.6	12.4
	44.75	-13.7	-16.5	23.7	22.1	13.6	12.4
14	44.75	7.4	2.3	9.2	8.6	8.3	8.0
	46.4	0.0	-4.1	9.2	8.6	8.3	8.0
	48.05	-7.4	-12.4	9.2	8.6	8.3	8.0
15	48.05	2.5	-8.3	1.9	1.1	2.8	3.0
	49.7	0.0	-5.7	1.9	1.1	2.8	3.0
	51.35	-2.5	-3.6	1.9	1.1	2.8	3.0

3.5 Analysis Under Dead Load

3.5.1 Hand calculations

There are three basic elements of the construction: beam, column and slab. Some specific features of the slab allow to figure out the influence of the dead load on our construction. Firstly, because one-way slab is applied for our construction, we use two varieties of beam: minor beam and major beam. Secondly, peculiar properties of the beams and distinctions between them compel to use appropriate data for the calculation of the dead load and the way how it affects the construction. Minor beam is placed in the middle of the slab, and it distributes the weight of the slab on itself. Major beam is the one that is placed between columns and that is under the pressure of slab and minor beam. Therefore, in order to find the dead load that acts on the beam, it is necessary to calculate mass of the slab and the weight of that beam. With the help of these

calculations, it is possible to find out the influence of the dead load on the construction. Moreover, this data is transformed into a visual form in terms of figure below.

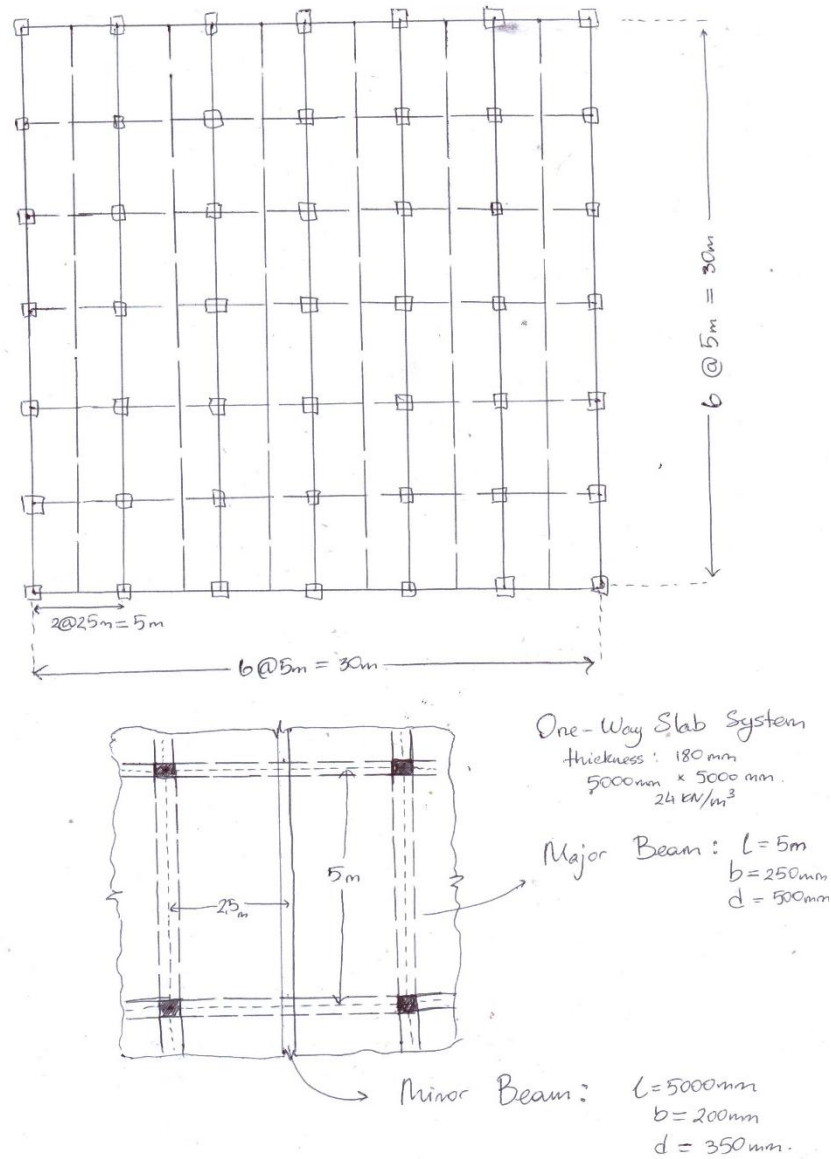


Figure 3.13. Representation of major and minor beams allocation

In order to verify the results from software simulation, hand calculation should be performed. Since there is an assumption that every beam is experiencing same value of dead load, manual estimation of internal forces have been done. Hand calculations below demonstrate the way how vertical forces influence the structural element. As it was stated previously, internal hinge was taken 0.1 fraction of whole span. It means, that 0.5 m away from both sides of the beam, there is the point of zero moment, which

implies to find out shear force at that point. By performing calculations with relevant approximations gives us moment diagram in form of parabola, whose bottom point equals to 27.6 kN-m; and shear force diagram that has linear decrease from 34.5 kN to 34.5 kN.

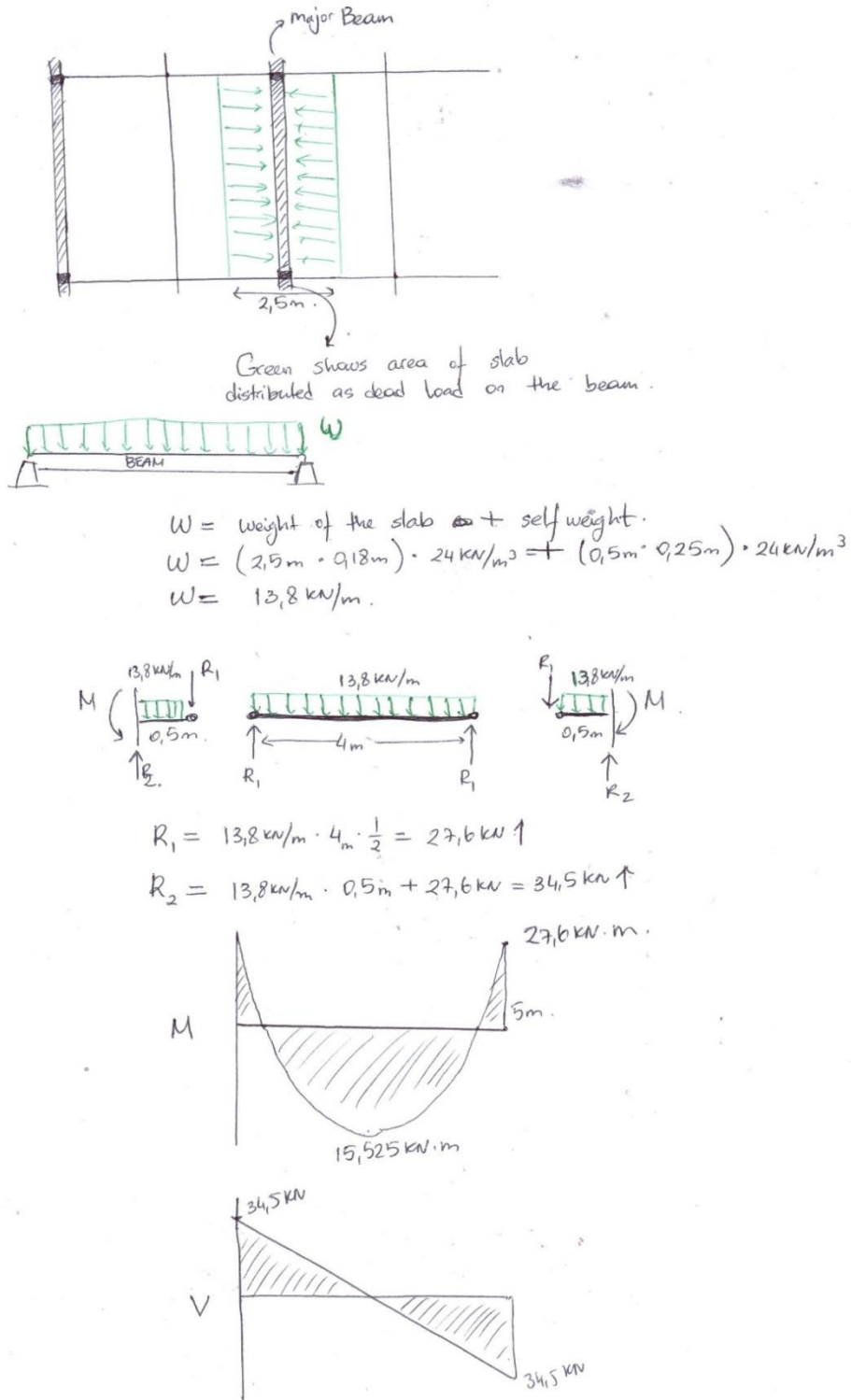


Figure 3.14. Manual calculation of internal forces for typical beam

3.5.2 SAP 2000 analysis

After obtaining results from hand calculations it is necessary to compare them with software simulation in order to verify that software provides correct results. As in case for wind loading analysis, the frame of 15 floors and 6 bays was created. The height between floors was taken 3.3 meters (first floor 5.5 meters) and width between bays 5 meters. Then, material properties for concrete were entered. Input values for weight per unit weight, modulus of elasticity and specified concrete compressive strength were typed ($f_c = 40$ MPa, values expressed in kN and m). Furthermore, section properties as three types of columns ($550mm \times 550mm$, $450mm \times 450mm$, $350mm \times 350mm$) and beam ($500mm \times 250mm$) were applied to the simulated-structure.

Since, we are analyzing frame solely under dead load, the load distribution for each beam was assigned. The application uses self-weight of the elements by default, thus dead load by action of slabs were entered. According to the estimations, structural member experiences 10.4 kN/m of distributed dead load.

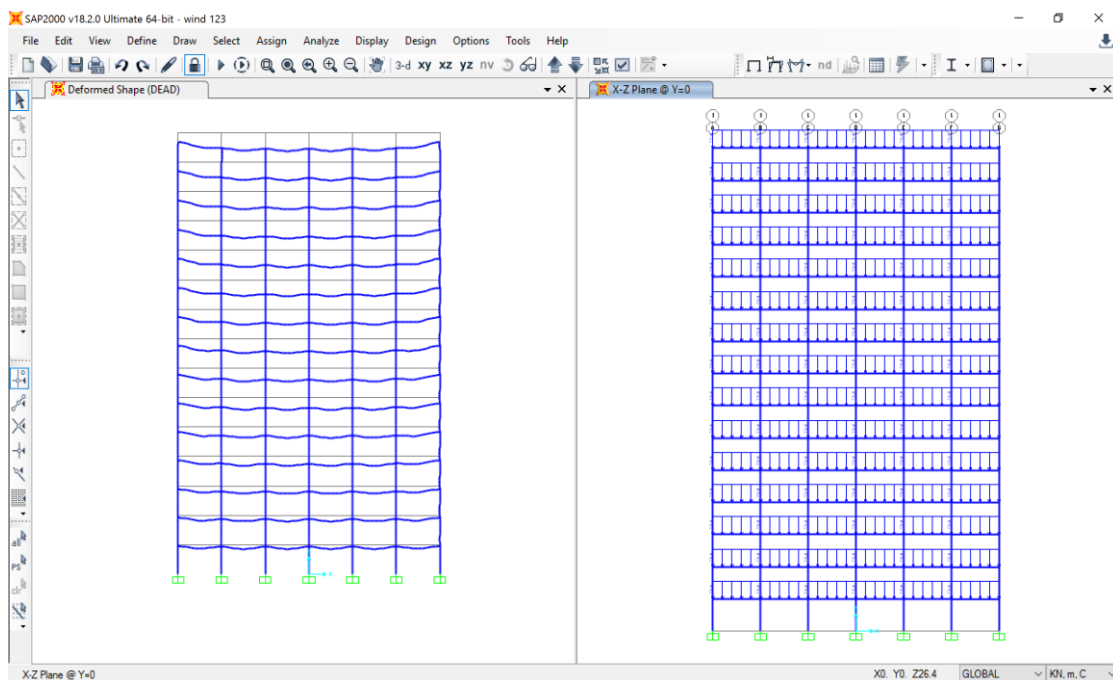


Figure 3.15. Dead Load applied to 2d frame and deflected shape after simulation

After input of all relevant information, software is able to simulate the structure and output results in terms of diagrams of internal forces. From Figure 3.15 we can observe that frame has deflected in vertical direction. Figure 3.18 shows patterns of moment and shear force. As it was stated by hand calculations the shape of diagrams is

same as predicted, but the exact values are still required to check. Therefore, for this case software can compute diagrams with filled values of moment and forces.

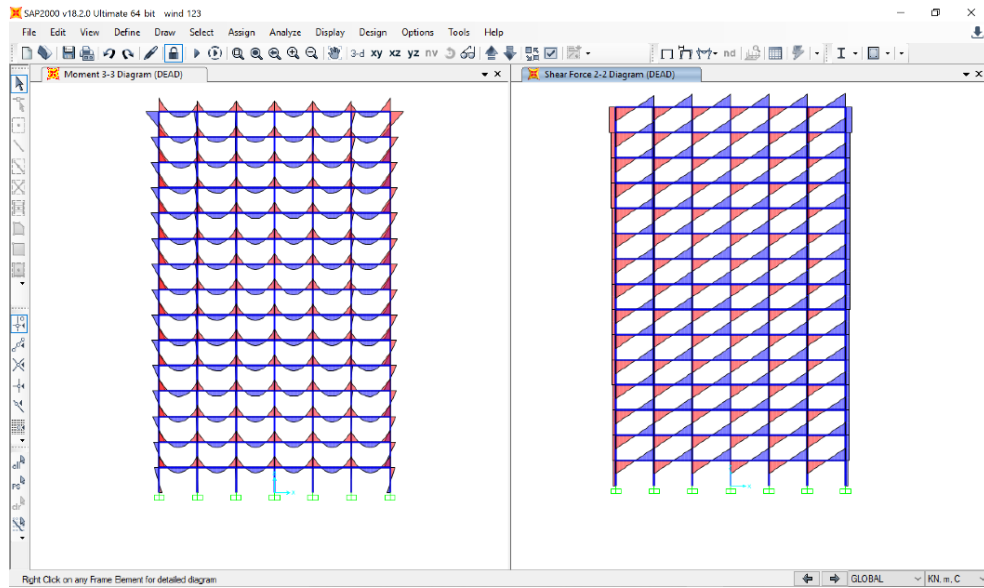


Figure 3.16. Moment diagram and Shear Force diagram by SAP 2000

By comparing the results obtained from hand calculations and software simulation it can be observed that they are close. The value of at end points of the beam is estimated as 15.525 kN-m and at the middle it is 27.6 kN-m. Whereas, according to the figure 3.17 the value provided by SAP 2000 is for end points is 27.96 kN-m and 14.15 kN-m for middle point. Additionally, shear values by SAP 2000 are 33.48 kN and 33.52 kN (Figure 3.17).

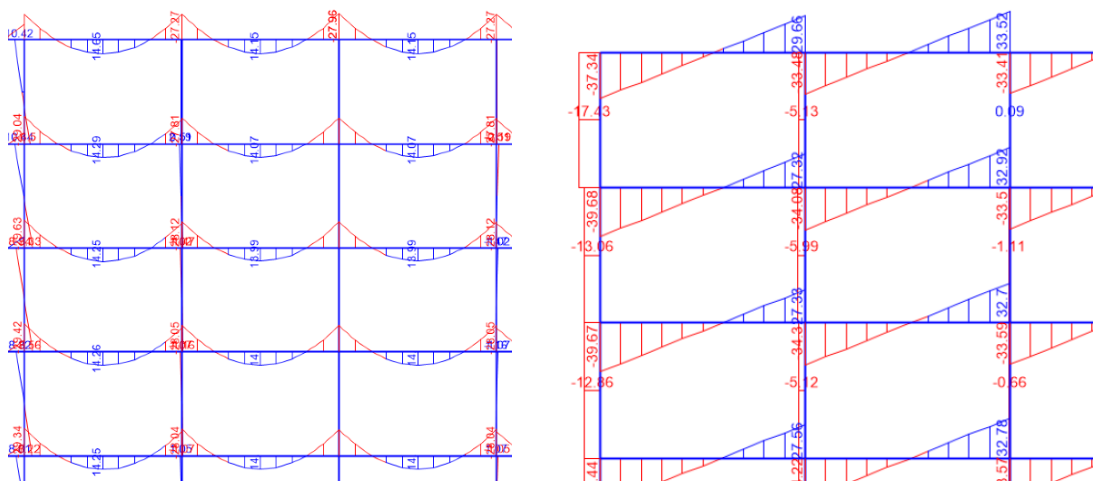


Figure 3.17. Exact values of Moment diagram and Shear Force diagram by SAP 2000

3.6 Structural Member Design

3.6.1 Beam Design

Since SAP2000 computes values for moment under combination of dead, live and wind loads, it is possible to design steel reinforcement by obtaining required steel area. The following procedure according ACI code 7.6.6 shows estimation of required steel area using known material properties and beam dimensions ($b=250\text{ mm}$, $d=500\text{ mm}$; $f'_c=40\text{ MPa}$, $f_y=420\text{ MPa}$) for the lower floor beam with moment value of 300.80 kN-m. The values for remaining beams of a given frame are shown in Appendix B, Table B.8.



Figure 3.18. Resultant moment for a critical beam by SAP2000

Step 1. Identification of the maximum moment that can be resisted by the underreinforced section

$$M_u = 300.80\text{ kN} \cdot \text{m}$$

Step 2. Once beam width b and effective depth d are known, the necessary flexural resistance factor is

$$R = \frac{M_u}{\phi b d^2} = \frac{300.8 \times 10^6}{0.9 \times 250 \times 435^2} = 7.07\text{ MPa} \quad (3.6.1)$$

Step 3. Estimate reinforcement ratio from flexural resistance factor. The reinforcement ratio ρ can be computed from the equation:

$$R = \rho f_y \left(1 - 0.588 \frac{\rho f_y}{f'_c}\right) \quad (3.6.2)$$

$$7.07 = \rho \cdot 420 \left(1 - 0.588 \frac{\rho \cdot 420}{40}\right)$$

Since all needed values are known, the equation gives $\rho=0.0191$. This value needs to be checked against maximum practical reinforcement ratio for beams, which is achieved at a tensile strain $\epsilon_t=0.005$ ($\rho < \rho_{\max}$).

$$\rho_{max} = \rho_{0.005} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \times \frac{40}{420} \times \frac{0.003}{0.003 + 0.005} =$$

$$0.0258 \rho_{max} > \rho$$

Step 4. Calculate required steel area

$$A_s = \rho b d = 0.0191 \times 250 \times 435 = 2077 \text{ mm}^2 \quad (3.6.3)$$

To satisfy spacing and covering regulations, tension steel will be placed 65 mm away from bottom surface ($d = 435 \text{ mm}$). Assume $a = 100 \text{ mm}$ and $\varphi = 0.90$. Calculating A_s , gives

$$A_s = \frac{M_u}{\varphi f_y (d - \frac{a}{2})} = \frac{300.8 \times 10^6}{0.9 \times 420 (435 - \frac{100}{2})} = 2067 \text{ mm}^2 \quad (3.6.4)$$

Choose three No. 25 bars with diameter 25.4 mm and cross sectional area 510 mm^2 , $A_s = 2040 \text{ mm}^2$.

Step 5. Verification of obtained steel area against minimum reinforcement area needs to be controlled. According to ACI code minimum A_s is estimated as follows:

$$A_{s,min} = \frac{0.25 \sqrt{f'_c}}{f_y} b_w d \geq 1.4 b_w d / f_y \quad (3.6.5)$$

$$A_{s,min} = \frac{0.25 \sqrt{40}}{420} \times 250 \times 435 \geq \frac{1.4 \times 250 \times 435}{420} \quad (3.6.6)$$

$$A_{s,min} = 362.5 \text{ mm}^2 \text{ (controlled)}$$

Step 6. Confirm that beam width is sufficient to contain provided reinforcement.

For beam and other elements design it is crucial to consider concrete protection for reinforcement. According to ACI code, for cast in place concrete structural members should be not less 40 mm. By taking into account the cover from the surface, the steel bars should be placed in the range from 65 mm to 75 mm (Nilson, 2004). In general, the minimum number of bars placed is limited by cover and width of the beam. In our case, the width of the beam equals to the 250 mm, which allows placing of three of No.25 bars.

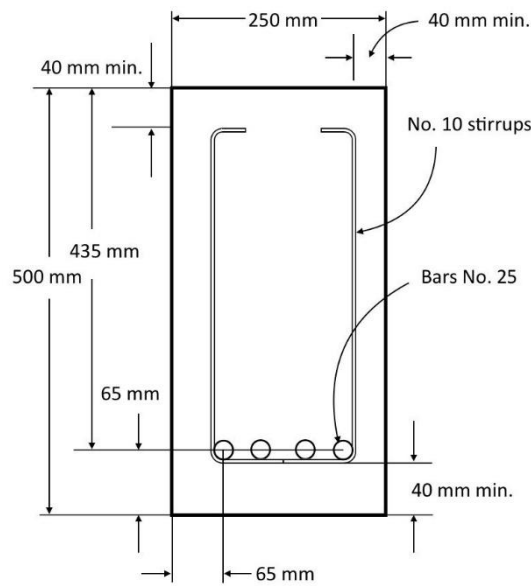


Figure 3.19. Reinforced beam

3.6.2 Column Design

With the aid of column interaction diagram and values for factored axial load and bending moment by SAP2000 we can obtain reinforcement values for column. To find out the steel area required for the building design the critical case for the 1st floor external column was considered. The column is designed for factored axial load P_u of 2621.35 kN and simultaneous factored bending moment of 300.80 kN-m provided by software. For column design ACI code provides basic reduction factor for tied columns $\phi = 0.65$. Structural detailing of columns are given in Appendix B, Table B.9.

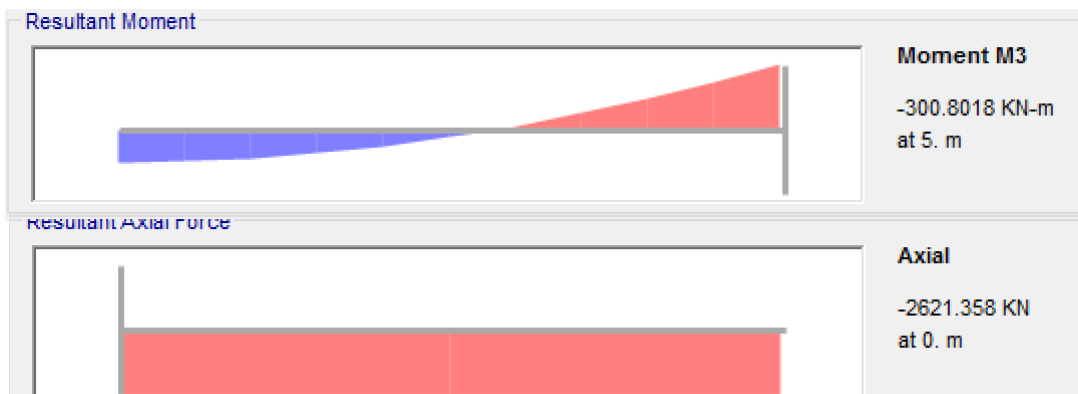


Figure 3.20 Factored axial load and moment provided by SAP2000

Step 1. The cross-sectional dimensions are defined: $b=550 \text{ mm}$, $h=550 \text{ mm}$.

Step 2. The ratio γ based on required cover distances to the bar centroids including trial bar diameter (No. 36) and stirrup dimensions

$$\gamma = \frac{550 - 2 \times 50 - 2 \times 9.5 - 35.8}{550} \approx 0.70 \quad (3.6.7)$$

Step 3. Based on obtained ratio γ in step 3 the specific interaction diagram can be chosen. In our case, it is needed to use column strength interaction diagram for rectangular with bars on four faces and $\gamma = 0.7$.

Step 4. In order to get corresponding reinforcement ratio from interaction diagram the values for K_n and R_n needs to be calculated as follows

$$K_n = \frac{P_u}{\phi f'_c A_g} = \frac{2621.35 \times 10^3}{0.65 \times 40 \times 550^2} = 0.33 \quad (3.6.8)$$

$$R_n = \frac{M_u}{\phi f'_c A_g h} = \frac{300.8 \times 10^6}{0.65 \times 40 \times 550^2 \times 550} = 0.15 \quad (3.6.9)$$

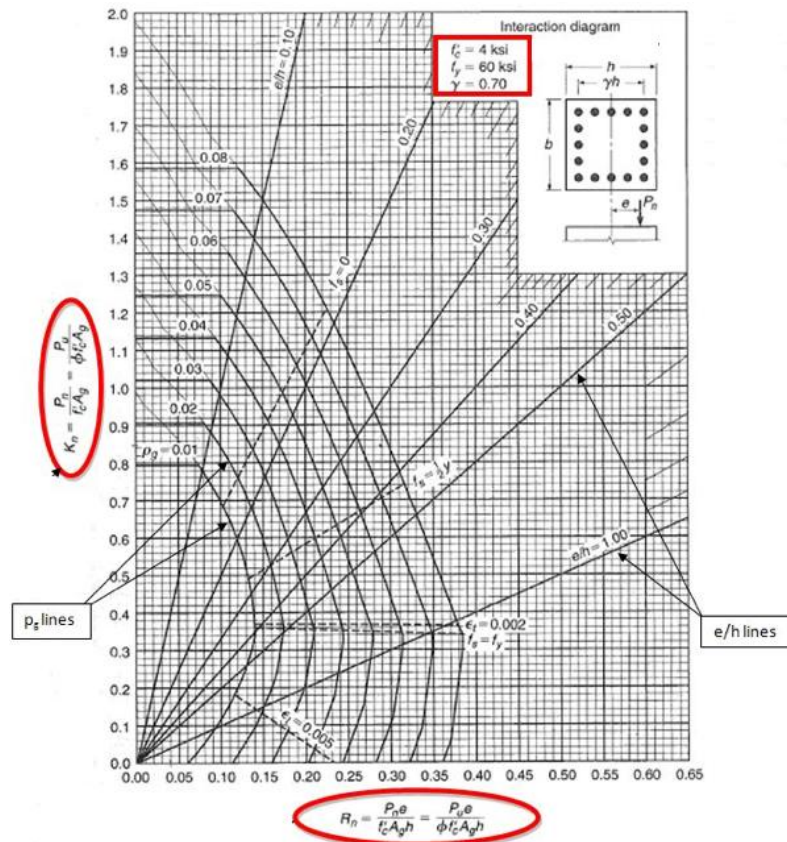


Figure 3.21. Interaction diagram for rectangular column with $\gamma=0.7$

Step 5. From the Figure 3.21, reinforcement ratio ρ_g is obtained from corresponding numbers for values from *Step 4*. The value on abscissa equals to the $K_n=0.33$ and on ordinate is $R_n=0.15$, thus cross-section of these figures shows value for reinforcement ratio $\rho_g=0.02$.

Step 6. When all relevant data is known, it is possible to calculate steel area A_{st} .

$$A_{st} = \rho_g bh = 0.02 \times 550^2 = 6050 \text{ mm}^2 \quad (3.6.10)$$

Twelve No. 25 bars will be used, providing $A_{st}=6050 \text{ mm}^2$, arranged rectangularly. Selecting No. 10 ties, the maximum tie spacing should not exceed $48 \times 9.5 = 456 \text{ mm}$, $16 \times 35.8 = 572.8 \text{ mm}$, or 450 mm.

3.6.3 Slab Design

According to ACI code 9.5.2, Table 3.9 – One-way construction (nonprestressed), the minimum thickness requirements for one-way both ends continuous slab from a table: $L/28=5000/28=178.57 \text{ mm}$. So, thickness of a slab was taken as a 180 mm which means that deflection check is not necessary. All the necessary values such as dead load and live load were computed previously for every floor.

Table 3.9. Minimum thickness of nonprestressed beams or one-way slabs unless deflections are calculated

	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	1/20	1/24	1/28	1/10
Beams or ribbed one-way slabs	1/16	1/18,5	1/21	1/8

Fiver requirements should be met in order to use approximate method.

From equation provided by ACI in order find the total factored load, the critical dead load to typical floor was 8968.3 kN and live load is taken as 4.8 kN/m² as minimum distributed live load. So, $D = 8968.3\text{kN}/900\text{m}^2 = 9.96 \text{ kN/m}^2$:

$$U = 1.2D + 1.6L = 1.2 * 9.96 + 1.6 * 4.8 = 19.632 \text{ kN/m}^2$$

$$l_n = 2.5\text{m} - \frac{0.55}{2}\text{m} = 2.225 \text{ m}$$

From ACI code, in a method of analysis, bending moments found with given equations for different moments and location of a slab.

End span:

Interior face of exterior support for members built with integrally with support:

$$\text{Bending Moment: } \frac{U * l_n^2}{16} = \frac{-19.632 * 2.225^2}{16} = -6.075 \text{ kN} * \text{m} \quad (3.6.11)$$

$$\text{Shear Force: } \frac{U * l_n}{2} = \frac{19.632 * 2.225}{2} = 23.84 \text{ kN} \quad (3.6.12)$$

Midspan:

$$\text{Bending Moment: } \frac{U * l_n^2}{14} = \frac{19.632 * 2.225^2}{14} = 6.94 \text{ kN} * \text{m} \quad (3.6.13)$$

Exterior face of first interior support:

$$\text{Bending Moment: } \frac{U * l_n^2}{10} = \frac{-19.632 * 2.225^2}{10} = -9.72 \text{ kN} * \text{m} \quad (3.6.14)$$

$$\text{Shear Force: } \frac{1.15 * U * l_n}{2} = \frac{1.15 * 19.632 * 2.225}{2} = 25.12 \text{ kN} \quad (3.6.15)$$

Interior span:

Interior face of first interior support:

$$\text{Bending Moment: } \frac{U * l_n^2}{11} = \frac{-19.632 * 2.225^2}{11} = -8.835 \text{ kN} * \text{m} \quad (3.6.16)$$

$$\text{Shear Force: } \frac{U * l_n}{2} = \frac{19.632 * 2.225}{2} = 21.84 \text{ kN} \quad (3.6.17)$$

Midspan:

$$\text{Bending Moment: } \frac{U*ln^2}{16} = \frac{-19.632*2.225^2}{16} = -6.075 \text{ kN} * m \quad (3.6.18)$$

Interior face of interior support:

$$\text{Bending Moment: } \frac{U*ln^2}{11} = \frac{-19.632*2.225^2}{11} = -8.835 \text{ kN} * m \quad (3.6.19)$$

$$\text{Shear Force: } \frac{U*ln}{2} = \frac{19.632*2.225}{2} = 21.84 \text{ kN} \quad (3.6.20)$$

From shorter direction, $ln = 2m$ is used. Since the span length is less than 3m, the negative moment at face of support is found:

$$\text{Bending Moment: } \frac{U*ln^2}{12} = \frac{-19.632*2^2}{12} = -6.54 \text{ kN} * m \text{ per m} \quad (3.6.21)$$

For discontinuous end integral with support as follows:

$$\text{Bending Moment: } \frac{U*ln^2}{14} = \frac{19.632*2^2}{14} = 5.61 \text{ kN} * m \text{ per m} \quad (3.6.22)$$

and shear force at face of all other supports is given as:

$$\text{Shear Force: } \frac{1.15*U*ln}{2} = \frac{1.15*19.632*2}{2} = 22.58 \text{ kN per m} \quad (3.6.23)$$

Design for longer direction span is as follows. Since, the tension-controlled section, which is a cross section in which the net tensile strain in the extreme tension steel at nominal strength could be whenever, the strength reduction factor is taken as $\phi=0.90$.

The minimum reinforcement ratio is given as:

$$\rho_{min} = \frac{0.25*\sqrt{f_c'}}{f_y} = \frac{0.25*\sqrt{40}}{420} = 0.00376 \quad (3.6.24)$$

The maximum reinforcement ratio to produce a selected value of net tensile strain is:

$$\rho = 0.85 * \beta_1 * \frac{f_c'}{f_y} * \frac{\epsilon_u}{\epsilon_u + \epsilon_t} \quad (3.6.25)$$

ACI code 10.3.5 establishes a minimum net tensile strain $\epsilon_t=0.004$ at the nominal strength for members subjected to axial loads. As the concrete stress

parameters are given as $f_c' = 40 \text{ MPa}$, $\beta_1 = 0.85$ and failure criteria: crushing of the concrete is $\epsilon_u = 0.003$.

$$\rho = 0.85 * 0.85 * \frac{40}{420} * \frac{0.003}{0.003+0.004} = 0.0295 \quad (3.6.26)$$

The flexural resistance factor is computed as follows:

$$R_n = \frac{M_u}{\phi b d^2} = \frac{9.72 * 10^6}{0.9 * 1000 * 155^2} = 0.45 \quad (3.6.27)$$

Where $d = 180 \text{ mm} - 25 \text{ mm} = 155 \text{ mm}$ from ACI requirements for slab design and 1 m strip was taken due to code of ACI, $b=100 \text{ mm}$.

Reinforcement ratio ρ should be determined as follows (Fanella, 2009):

$$\rho = \frac{0.85 * f_c'}{f_y} \left[1 - \sqrt{1 - \frac{2R_n}{0.85 * f_c'}} \right] = \frac{0.85 * 40}{420} \left[1 - \sqrt{1 - \frac{2 * 1.8}{0.85 * 40}} \right] \\ = 0.001077$$

Required area of reinforcement in tension A_s is taken as follows:

$$A_s = \rho b d = 0.001077 * 1000 * 155 = 166.94 \text{ mm}^2/\text{m} \quad (3.6.28)$$

According to ACI 10.5.4, the minimum reinforcement required for inspection of temperature cracking and shrinkage is as follows:

$$A_{s,min} = 0.0018 b h = 0.0018 * 1000 * 180 = 324 \text{ mm}^2/\text{m}$$

If required area of reinforcement in tension A_s is smaller than minimum reinforcement required for inspection of temperature cracking and shrinkage, the slab requires only small amount of reinforcing steel A_s . So, $A_{s,min}$ will be used.

In order to calculate the depth of the compression zone, we take $A_s = 324 \frac{\text{mm}^2}{\text{m}}$,

so:

$$a = \frac{A_s * f_y}{0.85 * f_c' * b} = \frac{324 * 420}{0.85 * 40 * 1000} = 3.58 \text{ mm}$$

and the location of the neutral axis is found as:

$$c = \frac{a}{\beta_1} = \frac{3.58}{0.85} = 4.21 \text{ mm}$$

After determination of neutral axis, according to ACI 10.3.4, the net-tensile strain should be more than 0.004, $\epsilon_t > 0.004$.

$$\epsilon_t = 0.003 \left(\frac{d_t}{c} - 1 \right) = 0.003 \left(\frac{155}{4.21} - 1 \right) = 0.1074 > 0.004 \quad (3.6.29)$$

So, the maximum reinforcement requirement is met.

Since the net-tensile strain is $\epsilon_t > 0.004$, the section is tension-controlled.

At end span, the area of steel required **at interior support** is as follows:

$$A_s = \frac{M_u}{\phi * f_y * (d - \frac{a}{2})} = \frac{9.72 * 10^6}{0.9 * 420 * (155 - \frac{3.58}{2})} = 167.84 \text{ mm}^2/\text{m} < 324 \text{ mm}^2/\text{m}$$

Since the difference between earliest and latter values of areas of tension reinforcement is significant, it is more than enough to use minimum reinforcement values. The value of moment of interior face of exterior support is smaller than moment in interior support, so the value of reinforcement in interior face of exterior support is also $324 \text{ mm}^2/\text{m}$. The next step is to find the area of tension reinforcement **at midspan**:

$$A_s = \frac{M_u}{\phi * f_y * (d - \frac{a}{2})} = \frac{6.94 * 10^6}{0.9 * 420 * (155 - \frac{3.58}{2})} = 119.83 \text{ mm}^2/\text{m}$$

$< 324 \text{ mm}^2/\text{m}$ use $A_{s,min}$,

The area of steel required **at interior face of exterior support** is as follows:

$$A_s = \frac{M_u}{\phi * f_y * (d - \frac{a}{2})} = \frac{6.075 * 10^6}{0.9 * 420 * (155 - \frac{3.58}{2})} = 104.9 \text{ mm}^2/\text{m}$$

$< 324 \text{ mm}^2/\text{m}$, use $A_{s,min}$,

Since the areas of tension reinforcement at midspan and exterior support are smaller than $A_{s,min} = 360 \text{ mm}^2/\text{m}$, minimum steel area is used.

At interior span, the moments of interior face of first interior support, midspan and interior face of interior support also small, so that minimum steel area is used,

$$A_{s,min} = \frac{360mm^2}{m} \text{ as follows:}$$

At interior face of first interior support:

$$A_s = \frac{M_u}{\phi * f_y * (d - \frac{a}{2})} = \frac{8.835 * 10^6}{0.9 * 420 * (155 - \frac{3.58}{2})} = 152.6 \text{ mm}^2/m$$

< 324 mm²/m, use $A_{s,min}$,

At midspan:

$$A_s = \frac{M_u}{\phi * f_y * (d - \frac{a}{2})} = \frac{6.075 * 10^6}{0.9 * 420 * (155 - \frac{3.58}{2})} = 105 \text{ mm}^2/m$$

< 324 mm²/m, use $A_{s,min}$,

At interior face of interior support:

$$A_s = \frac{M_u}{\phi * f_y * (d - \frac{a}{2})} = \frac{8.835 * 10^6}{0.9 * 420 * (155 - \frac{3.58}{2})} = 152.6 \text{ mm}^2/m$$

< 324 mm²/m, use $A_{s,min}$,

So, at every location of end span and interior support, required steel area is smaller than minimum required steel area, consequently, area of steel is taken as 324 mm²/m at every location of end span and interior span.

Shear force at distance d from the face of the interior support is computed as follows:

$$V_u = 1.15 * \frac{4.21 * 2.225}{2} - 4.21 * \frac{155}{1000} = 4.734 \text{ kN}$$

and the nominal shear strength that affect the slab is calculated as follows:

$$V_n = 0.17 * \lambda \sqrt{f'_c} b d = 0.17 * 1 * \sqrt{40} * 1000 * 155 * 10^{-3} = 166.65 \text{ kN}$$

Consequently, the design strength of the slab is as follows:

$$\phi V_c = 0.75 * 166.65 = 125 \text{ kN}$$

The value of design strength is greater than required shear strength, so shear reinforcement is not necessary.

Furthermore, the following criteria should be met due to ACI 10.6.4:

$$s = 380 * \left(\frac{280}{f_s}\right) - 2.5 * c_c \leq 380 * \left(\frac{280}{f_s}\right)$$

where c_c , center to center spacing, should be approximately $0.75 \text{ in} \approx 20 \text{ mm}$ and $f_s = \frac{2}{3} * f_y = \frac{2}{3} * 420 = 280 \text{ MPa}$, so:

$$s = 380 * \left(\frac{280}{280}\right) - 2.5 * 20 = 330 \text{ mm} \leq 380 * \left(\frac{280}{280}\right) = 380 \text{ mm}$$

which means that the mentioned above criteria are met.

From Appendix B, Table B.5, table gives values of bar diameter for the area of reinforcement per meter taking into account spacing of bars. So;

For the interior face of exterior support, midspan and interior support of end span and interior face of first interior support, midspan and interior face of interior support is $A_s = 324 \text{ mm}^2/m$, so 10 mm diameter bars are taken with bar spacing 225 mm, $A_{st} = 347 \text{ mm}^2/m$, which is 6.63 % oversized.

The same method is used for other spans on different levels. Results of final design of all slabs for typical floor is given in Appendix B, and construction drawings of slab detail for one typical floor is also shown in Appendix B.

4 GEOTECHNICAL DESIGN

Geotechnical design is an essential part of any building construction process, as well as structural and architectural designs. It is a department of civil engineering, which studies and analyses the earth materials engineering and physical behavior. Generally, geotechnical design includes soil and rock mechanics to analyze surface and subgrade materials, conditions and properties of the materials. Moreover, geotechnical design is necessary to estimate and evaluate required type of the foundation for the structure in order to fulfill the needed conditions for safe construction of the building. Thus, the following project represents the final design of the foundation for Utopia Hotel building and the accompanying calculations are also performed during the Capstone II course.

4.1 Location of the Project

As it has already known, Astana city is generally divided into two parts, which are left and right bank sides of Ishim River, or simply the new and old city respectively. Based on the names (new and old city), it might be noticed that the old city mainly consists of simple- and obsolete-design buildings which are actually designed for population housing and living; whereas, the new city is the growing part of Astana, where the new high-rise buildings are under construction and there is still much space to be built up. Thus, while deciding on the choice of the location, such factors, as people concentration, space availability, design propriety and the building height, were considered. Definitely, the choice fell on to the left side of the river. According to the spaces available on the left side of the city, which are seen from the Google maps, it was found that the space on the intersection of Qabanbay Batyr avenue and Syghanaq Street, near to “Nomad” apartment complex and opposite to “Nur Astana” mosque, as shown on the Figure X below, is the most suitable and appropriate location for our hotel building project, considering all the factors mentioned before.

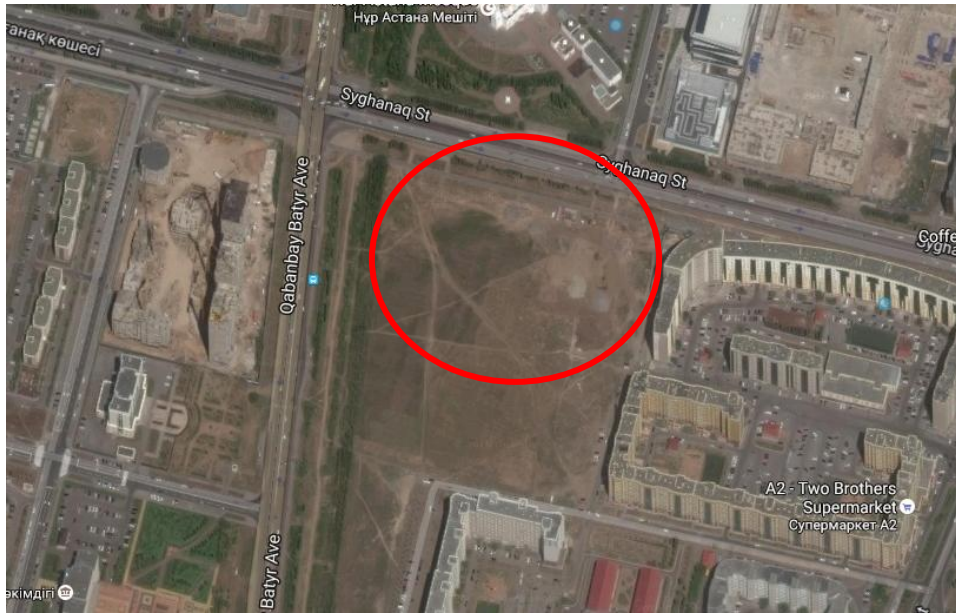


Figure 4.1. Location of the Project

4.2 Soil Profile

The soil properties are based on the data provided by the advisor. This data was obtained while investigation of the ground on Nazarbayev University territory for construction of new blocks of the dormitory. Although, these soil properties are not exactly for our project location, it can be used, since it was assumed that the soil properties are similar to the required ones due to the closeness and the fact that the soil properties are similar along all the left bank of the river. However, basically, it is provided that the whole area of Astana city has the same type of the soil (refer to Appendix C in the end of the report). Thus, the obtained results contain thickness, density, dry unit weight, natural water content, void ratio, frictional angle, cohesion and elastic modulus of the ground layers, which are presented in the Table 4.1 below.

Table 4.1. Soil Characteristics

<i>Soil layer</i>	Backfill (tQ_{IV})	Loam (aQ_{II-IV})	Medium-size sand (aQ_{II-IV})	Coarse sand (aQ_{II-IV})	Gravelly sand (aQ_{II-IV})	Gravel soil (aQ_{II-IV})	Loam (eC₁)
<i>Depth, m</i>	3.0	5.5	2.4	3.2	7.1	1.9	5.9
<i>Unit weight, γ (kN/m³)</i>	18.7	19.7	19.2	20.0	20.0	20.5	19.3
<i>Dry unit weight, γ_{dry} (kN/m³)</i>	15.6	17.3	17.0	17.7	17.7	17.8	17.2

<i>Natural water content, w, %</i>	18.2	20.9	20.5	6.3	7.9	18.7	23.2
<i>Void ratio, e</i>	0.43	0.76	0.67	0.61	0.45	0.71	0.77
<i>Porosity</i>	0.43	0.67	0.5	0.3	0.3	0.4	0.75
<i>Poissons ratio, μ</i>	0.35	0.4	0.3	0.35	0.35	0.3	0.4
<i>Friction angle, ϕ, °</i>	21	22.23	35.0	38.0	38.0	28	32.01
<i>Cohesion, c (kPa)</i>	44	18.22	2.0	1.0	1.0	0	33.78
<i>Elastic Modulus, E, (MPa)</i>	2.8	6.0	17.0	21.0	21.0	23.0	13.5

Regarding the groundwater, the water table is located at the level of 2.0 m below the ground surface. It means that while excavating the earth materials, the groundwater would immediately appear; so that, the soil below the first layer is not appropriate for the construction, and the proper foundation type must be applied.

In addition, according to the Eurocode EN 1990, which is used in the project for the design of the foundations, the actions on this location are classified as G (permanent) with negligible or monotonic actions due to self-weight of structures, fixed equipment and road-surfacing indirect actions caused by shrinkage and uneven settlements (refer to Appendix C).

4.3 Foundation Type

Foundation is the lowest part of architectural structure which links it to the ground and provides the base of the structure with the super-structure, transferring structure loads to the earth including the self-weight of the structure.

Determination of foundation type is one of the most disputable parts in the geotechnical design. The most important factors that should be considered while selecting the type of the foundation are economic and practical feasibility, height and type of the structure (in our case, it is 15-storey high-rise hotel building), availability of materials, equipment, and labor.

This project is concerned with two types of foundation, which are shallow and deep foundations. Both types of foundation are presented and analyzed below.

4.3.1 Analysis Methods

The decision on the foundation type would be based on the literature review and some hand calculations which will be made throughout the report.

Eurocode configuration depends on Principles (general articulations, investigative models, and regulations) and Application rules, which are for the most part perceived standards that follow the Principles and fulfill their needs. A plan is considered to meet the prerequisites of the construction project, if the presumptions on which the Eurocodes are based are fulfilled. The suspensions are that the structures are enough kept up and utilized as a part of conjunction with the outline suppositions; the development materials and items are as determined in ENs 1990-1999; the decision of auxiliary framework is made by work force with fitting capabilities and encounter.

4.3.2 Shallow Foundation

Shallow foundation is a type of a foundation which allows transferring of the building loads to the earth close to the surface layer, rather than to the subsurface and lower layers; so that, in this case it can be said that the soil layer is proper to support a structure at the relatively shallow depth. Hence, the main purpose of the use of the shallow foundations is to distribute the loads from the building structure over the whole horizontal area at a depth below the ground level, but near to the surface.

There are 5 types of shallow foundation can be used for the construction of the building:

1. Strip Footing

A strip footing (Figures 4.2 and 4.3 below) is accommodated a load-bearing divider. A strip footing is likewise accommodated a line of columns which are so firmly separated that their spread footings cover or about touch each other. In that case, it is more efficient to have a strip footing than to have various spread footings in one line. In addition, it is also known as continuous footing (Mishra, 2016).

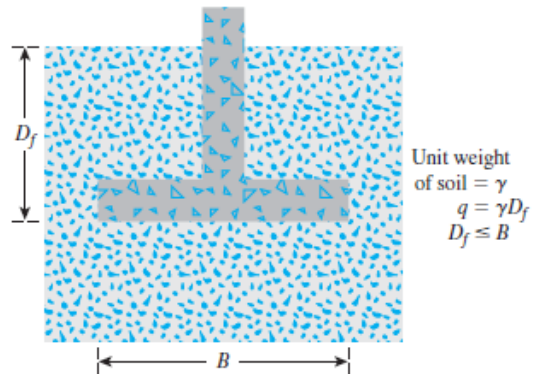
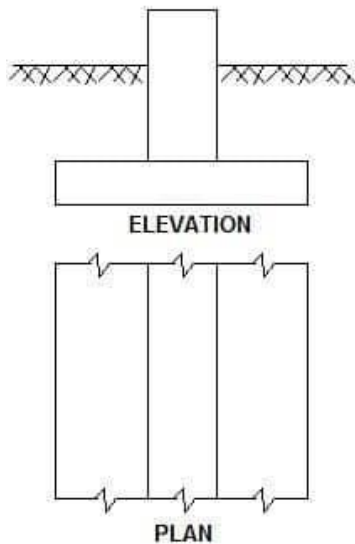


Figure 4.2. Strip Footing (Mishra, 2016) Figure 4.3. Shallow Strip Foundation (Das, 2011)

1. Spread or Isolated Footing (Pad Foundation)

Spread footing (Figure 4.4 below) is designed as it supports an individual column. It is square, rectangular, or circular slab of uniform thickness (Mishra, 2016).

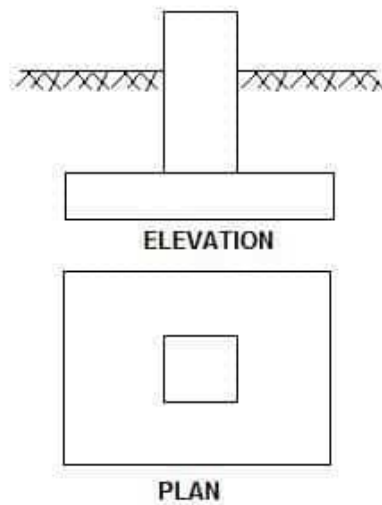


Figure 4.4. Spread Footing (Mishra, 2016)

2. Combined Footing

Combined footing (Figure 4.5 below) supports two columns. In case if the two columns are close to each other to the extent that their individual footings would overlay, then the combined footing is used. It is rectangular or trapezoidal in plan (Mishra, 2016).

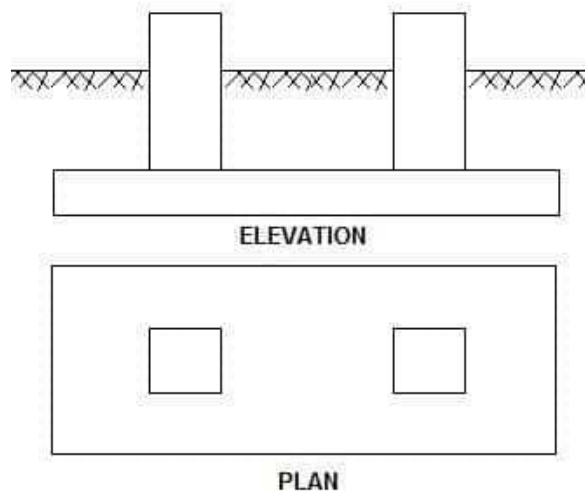


Figure 4.5. Combined Footing (Mishra, 2016)

3. Strap or Cantilever Footing

Cantilever footing (Figure 4.6 below) is a combination of two separate footings connected with a structural strap or a lever. This strap links two footings in the way that they behave as one unit. The strap is a rigid beam (Mishra, 2016).

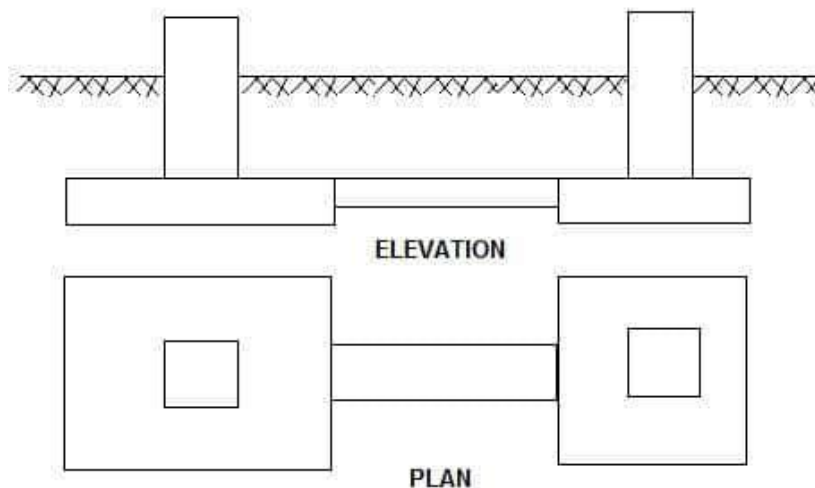


Figure 4.6. Cantilever Footing (Mishra, 2016)

4. Mat (Raft) Foundation

Raft foundation (Figure 4.7 below) is a big slab which supports a number of columns and walls under the big structure part or the whole structure. This type of the foundation is needed in the case of the low value of the allowable pressure of soil or if the columns and walls are close to each other, so that the individual footings may overlay or touch each other. Moreover, raft foundations are also beneficial while decreasing differential settlements on non-homogeneous soils or having significant differences in loads applied on individual columns (Mishra, 2016).

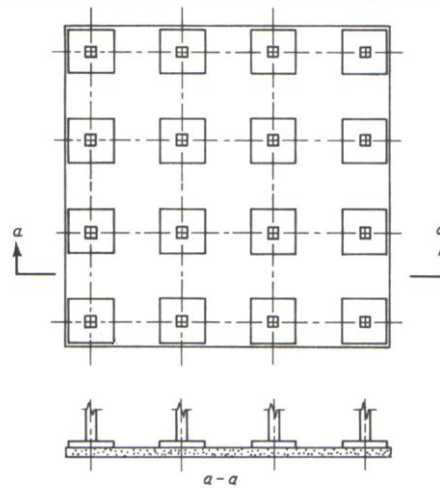


Figure 4.7. Raft Foundation (Mishra, 2016)

For the commercial structures and basements, the most frequently used is the strip footing.

Advantages of shallow foundations:

- It is cost effective;
- Not necessary to hire expert labor to install the foundation;
- Made of concrete material, which is always available;
- Easy and straight forward procedure of construction;

Disadvantages of shallow foundation:

- Limited to structure in soil only;
- It is often subject to pullout, torsions, and moment;
- A major problem regarding the settlement.

One of the responsibilities of geotechnical engineer is to select proper foundation, then calculate its dimension considering internal components, for example, material, like reinforcement. Such properties can be identified according to two main characteristics:

- Ultimate load-carrying capacity of foundation: Ultimate load-carrying describes the ability of soil to support above structure, without undergoing shear failure of soil or failure of the foundation. Thus, it should not be less, than the load applied on the foundation from the structure.
- Excessive displacement or settlement: Settlement stands for displacement of soil as a result of applied load in long or short term period. Instability of structure

and failure of different components of a building could be caused by excessive displacement.

Thus, in order to design a proper foundation and make the foundation perform adequately, both characteristics of foundation should be satisfied and properly checked.

4.3.2.1 Bearing Capacity and Settlement of Shallow Foundation

The General Bearing Capacity equation would be used to find bearing capacity of shallow foundation. The following equation should be used to calculate it:

$$q_u = \left(c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B' N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \right) BL \quad (4.3.1),$$

Where

c' = cohesion;

q = effective stress at foundation bottom level;

γ = unit weight of soil;

B' = width of foundation;

$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors;

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors;

$F_{ci}, F_{qi}, F_{\gamma i}$ = load inclination factors;

N_c, N_q, N_γ = bearing capacity factors.

This formula was suggested by Meyerhof (1963) in order to consider different cases of foundation design, shearing resistance along the surface of failure in soil above foundation bottom, and the likelihood of inclination of the load on the foundation. The deeper explanation of the equation can be found in “Principles of Foundation Engineering” by Braja Das (M., 2014). Effective stress is calculated with simple multiplication of unit weight to depth of foundation. Maximum depth of foundation was taken as one meter below the ground level, because there is no consideration for underground level in design of a building. Width and length of foundation are known as 30.5x30.5 m. It is done for the simplicity for calculation and according to dimensions of the building. Consequently, the equation above is used to calculate them.

Table 4.2 below represents the calculations of the load-bearing capacity of different soil layers on the site and the total allowable bearing capacity, which the foundation may sustain. Using the functions in Microsoft Excel, the necessary parameters for the Equation 4.3.1 were found, applying the following formulae:

$$q = \gamma D_f \quad (4.3.2)$$

$$q = D_1 \gamma_{dry} + D_2 (\gamma - \gamma_w) \text{ (considering water table)} \quad (4.3.3)$$

Bearing capacity factors were found with the usage of Tables 4.2 and 4.3 of the corresponding chapter of the book “Principles of Foundation Engineering” by Braja M. Das (2014) (refer to Appendix C in the end of this report).

In order to obtain the value for the gross allowable load-bearing capacity, it is required to take into account the factor of safety to the gross ultimate load-bearing capacity:

$$q_{all} = \frac{q_u}{FS} \quad (4.3.4)$$

Finalizing the calculations, the total load-bearing capacity became 29708 kN/m², which means that the possible designed shallow foundation may sustain the load of 29708 kN per each square meter, or 29708*30.5*30.5 = 27635796 kN on the whole foundation surface.

To calculate dimensions of foundation, load to the foundation should be determined. The total load, found in the structural part of this report, to overall foundation is equal 250159.5 kN. To design single footing load should be divided by number of columns, because load would transfer load to the foundation through columns in first floor. According to design, hotel building has 49 columns (7x7 for the first floor), which consequently led to 5105 kN of load to each of the footing. Also, moment to the column should be considered. Moment at each internal column was calculated to be 2000 kN/m². By using equation below, eccentrically loaded pressure can be calculated:

$$q_{max} = \frac{Q}{BL} + \frac{6M}{B^2L} = \frac{250.1595}{30.5 \times 30.5} + \frac{6 \times 2}{30.5^2 \times 30.5} = 269 \text{ kN}$$

Finally, with the consideration of vertical loading and moment the load from each column to the foundation is estimated to be 269 kN. It is important to emphasize that bigger types of footings would be considered in case when single footing wouldn't be able to carry applied load or when size of footing is big enough to join them.

Settlement is one of the main problems of shallow foundations for heavy buildings. To calculate settlement of shallow foundation, theory of elasticity would be

used. According to theory of elasticity following formula can be used to find settlement of single footing:

$$S_e = q_0 \times (\alpha \times B') \times \frac{1-\mu_s^2}{E_s} \times I_s \times I_f \quad (4.3.5)$$

Where

q_0 = net applied pressure on the foundation;

μ_s = soil Poisson's ratio;

E_s = Modulus of elasticity;

$B' = \frac{B}{2}$ = width for the center of the foundation;

$I_s = F_1 + \frac{1 - 2\mu_s}{1 - \mu_s} F_2$ = shape factor;

I_f = depth factor.

Table 4.3 below represents the calculations of the settlement of different soil layers on the site at the center of the foundation. Using the functions in Microsoft Excel, the necessary parameters for the Equation 4.3.5 were found, applying the required formulae. However, such parameters, as F_1 , F_2 , and I_f , are found from Tables 7.2-7.4 of the corresponding chapter in the book "Principles of Foundation Engineering" by Braja M. Das (2014) (refer to Appendix C in the end of this report). Those factors were found manually in the way of interpolation, where needed (Figure 4.8).

Values of F_1 with m' and n'

① $m'=1, n'=1.7 \rightarrow F_1=0$
 $m'=1, n'=1.5 \rightarrow F_1=0.224$
 $m'=1, n'=1.75 \rightarrow F_1=0.257$
 $\frac{1.75-1.7}{1.75-1.5} = \frac{0.257-x}{0.257-0.224} \Rightarrow x=0.2504$

② $F_1=? n'=1.84$
 $F_1=0.186, n'=1.25$
 $F_1=0.224, n'=1.6$
 $\frac{0.224-x}{0.224-0.186} = \frac{1.5-1.84}{1.5-1.25} \Rightarrow x=0.1996$

③ $F_1=0, n'=1.19$
 $F_1=0.142, n'=1$
 $F_1=0.186, n'=1.25$
 $\frac{0.186-x}{0.186-0.142} = \frac{1.25-1.19}{1.25-1} \Rightarrow x=0.17544$

④ $F_1=? n'=0.98$
 $F_1=0.095, n'=0.75$
 $F_1=0.142, n'=1$
 $\frac{0.142-x}{0.142-0.095} = \frac{1-0.98}{1-0.75} \Rightarrow x=0.1384$

⑤ $F_1=? n'=0.51$
 $F_1=0.049, n'=0.5$
 $F_1=0.095, n'=0.75$
 $\frac{0.095-x}{0.095-0.049} = \frac{0.75-0.51}{0.75-0.5} \Rightarrow x=0.05084$

⑥ $F_1=? n'=0.39$
 $F_1=0.04, n'=0.25$
 $F_1=0.049, n'=0.5$
 $\frac{0.049-x}{0.049-0.04} = \frac{0.5-0.39}{0.5-0.25} \Rightarrow x=0.0386$

⑦ $F_1=0, n'=0$

Values of F_2 with m' and n'

① $n'=1.7, F_2=?$
 $n'=1.5, F_2=0.075$
 $n'=1.45, F_2=0.069$
 $\frac{1.75-1.7}{1.75-1.5} = \frac{0.069-x}{0.069-0.075} \Rightarrow x=0.0702$

② $n'=1.84, F_2=?$
 $n'=1.25, F_2=0.08$
 $n'=1.5, F_2=0.075$
 $\frac{1.5-1.84}{1.5-1.25} = \frac{0.075-x}{0.075-0.08} \Rightarrow x=0.0712$

③ $n'=1.19, F_2=?$
 $n'=1, F_2=0.083$
 $n'=1.25, F_2=0.08$
 $\frac{1.25-1.19}{1.25-1} = \frac{0.08-x}{0.08-0.083} \Rightarrow x=0.08072$

④ $n'=0.98, F_2=?$
 $n'=0.75, F_2=0.083$
 $n'=1, F_2=0.083$
 $\frac{1-0.98}{1-0.75} = \frac{0.083-x}{0.083-0.083} \Rightarrow x=0.083$

⑤ $n'=0.51, F_2=?$
 $n'=0.5, F_2=0.074$
 $n'=0.75, F_2=0.083$
 $\frac{0.75-0.51}{0.75-0.5} = \frac{0.083-x}{0.083-0.074} \Rightarrow x=0.07436$

⑥ $n'=0.39, F_2=?$
 $n'=0.25, F_2=0.049$
 $n'=0.5, F_2=0.074$
 $\frac{0.5-0.39}{0.5-0.25} = \frac{0.074-x}{0.074-0.049} \Rightarrow x=0.063$

⑦ $n'=0, F_2=0$

Values of Σ_f

① $y=0.35, \Sigma_f=?$
 $y=0.3, \Sigma_f=0.9$
 $y=0.4, \Sigma_f=0.93$
 $\frac{0.4-0.35}{0.4-0.3} = \frac{0.93-x}{0.93-0.9} \Rightarrow \Sigma_f=0.915$

② $y=0.4$
 $\frac{0.4-0.28}{0.4-0.2} = \frac{0.85-x}{0.85-0.93} \Rightarrow \Sigma_f=0.898$

③ $y=0.3$
 $\frac{0.4-0.26}{0.4-0.2} = \frac{0.81-x}{0.81-0.9} \Rightarrow \Sigma_f=0.828$

④ $y=0.35, \Sigma_f=?$
 $y=0.2, \Sigma_f=?$
 $\frac{0.4-0.46}{0.6-0.4} = \frac{0.74-x}{0.74-0.81} \Rightarrow x=0.789$
 $y=0.4, \Sigma_f=?$
 $\frac{0.6-0.46}{0.6-0.4} = \frac{0.78-x}{0.78-0.86} \Rightarrow x=0.829$
 $\frac{0.4-0.35}{0.4-0.3} = \frac{0.829-x}{0.829-0.789} \Rightarrow \Sigma_f=0.802$

⑤ $y=0.5$
 $\frac{0.6-0.695}{1-0.6} = \frac{0.65-x}{0.65-0.74} \Rightarrow x=0.718626$
 $y=0.4, \Sigma_f=?$
 $\frac{1-0.695}{1-0.6} = \frac{0.69-x}{0.69-0.78} \Rightarrow x=0.758625$
 $\frac{0.4-0.35}{0.4-0.3} = \frac{0.758625-x}{0.758625-0.718625} \Rightarrow \Sigma_f=0.738625$

⑥ $y=0.3$
 $\frac{0.6-0.76}{1-0.6} = \frac{0.65-x}{0.65-0.74} \Rightarrow \Sigma_f=0.704$

Figure 4.8. Hand calculations of parameters for the settlement equation

By applying these factors and appropriate parameters of soil, following results are be obtained:

Elastic settlement of backfill layer: 3.85 m.

Elastic settlement of loam layer: 1.59 m.

Elastic settlement of medium-size sand layer: 2.88 m.

Elastic settlement of coarse sand layer: 3.27 m.

Elastic settlement of gravelly sand layer: 1.82 m.

Elastic settlement of gravel soil layer: 0.34 m.

Elastic settlement of clay layer: 0 m.

The settlement results show that the displacement of the soil layers due to the pressure from structure load is excessive, which can lead to the instability of the structure or failure of different components of the building in a short- or a long-term period basis.

Table 4.2. Bearing Capacity of Shallow Foundation

Type of soil	Depth of foundation, D_f , m	Elevation, m	Unit weight of soil, γ , kN/m ³	Dry unit weight of soil, γ_{dry} , kN/m ³	Unit weight of water, γ_w , kN/m ³	Cohesion, c' , kN/m ²	Friction angle, φ , °	Width of foundation, B , m	Length of foundation, L , m	N_c	N_q	N_γ	F_{cs}	F_{qs}	$F_{\gamma s}$	F_{cd}	F_{qd}	$F_{\gamma d}$	$F_{cd} = F_{qd} = F_{\gamma d}$	Bearing capacity, q_{ln} , kN/m ²	Factor of safety	Allowable load-bearing capacity, q_{all} , kN/m ²		
Backfill (sand)	3	3	18,7	15,6	9,81	44	21	30,5	30,5	15,8	7,07	6,2	1,45	1,38	0,6	1,04	1,03	1	1	2333	3	778		
Loam (clay)	5,5	8,5	19,7	17,3	9,81	18	22	30,5	30,5	16,9	7,82	7,13	1,46	1,40	0,6	1,07	1,05	1	1	2756	3	919		
Medium-size sand	2,4	10,9	19,2	17	9,81	2	35	30,5	30,5	46,1	33,3	48,03	1,72	1,70	0,6	1,02	1,02	1	1	14423	3	4808		
Coarse sand	3,2	14,1	20	17,7	9,81	1	38	30,5	30,5	61,4	48,93	78,03	1,80	1,78	0,6	1,02	1,02	1	1	26917	3	8972		
Gravelly sand	7,1	21,2	20	17,7	9,81	1	38	30,5	30,5	61,4	48,93	78,03	1,80	1,78	0,6	1,05	1,05	1	1	33973	3	11325		
Gravel soil	1,9	23,1	20,5	17,8	9,81	0	28	30,5	30,5	25,8	14,72	16,72	1,57	1,53	0,6	1,02	1,02	1	1	8721	3	2907		
Loam (clay)	5,9	29	19,3	17,2	9,81	34	32	30,5	30,5	35,5	23,18	30,22	1,65	1,63	0,6	1,06	1,05	1	1		3			
																				Total bearing capacity		89124	3	29708
																				Total load		82907 387	3	2763579 6

Table 4.3. Settlement of Shallow Foundation

Type of soil	Depth of foundation, D_f , m	Elevation, m	Unit weight of soil, γ , kN/m ³	Dry unit weight of soil, γ_{dry} , kN/m ³	Unit weight of water, γ_w , kN/m ³	Cohesion, c' , kN/m ²	Friction angle, ϕ , °	Width of foundation, B , m	B' , m	Length of foundation, L , m	Poisson's ratio, μ	Elastic modulus, E , kpa	D_f/B	B/L	H , m	a , m'	n'	F_1	F_2	I_s	I_f	Allowable load-bearing capacity, qall, kN/m ²	Settlement, S_e , m	
Backfill (sand)	3	3	18,7	15,6	9,81	44	21	30,5	15,25	30,5	0,35	2800	0,10	1	26	4	1	1,705	0,25	0,07	0,28	0,915	778	3,85
Loam (clay)	5,5	8,5	19,7	17,3	9,81	18,22	22	30,5	15,25	30,5	0,4	6000	0,28	1	20,5	4	1	1,344	0,20	0,08	0,23	0,898	919	1,59
Medium-size sand	2,4	10,9	19,2	17	9,81	2	35	30,5	15,25	30,5	0,3	1700	0,36	1	18,1	4	1	1,187	0,18	0,08	0,22	0,828	4808	2,88
Coarse sand	3,2	14,1	20	17,7	9,81	1	38	30,5	15,25	30,5	0,35	2100	0,46	1	14,9	4	1	0,977	0,14	0,08	0,18	0,809	8972	3,27
Gravelly sand	7,1	21,2	20	17,7	9,81	1	38	30,5	15,25	30,5	0,35	2100	0,70	1	7,8	4	1	0,511	0,05	0,07	0,09	0,738625	11325	1,82
Gravel soil	1,9	23,1	20,5	17,8	9,81	0	28	30,5	15,25	30,5	0,3	2300	0,76	1	5,9	4	1	0,387	0,03	0,06	0,07	0,704	2907	0,34
Loam (clay)	5,9	29	19,3	17,2	9,81	33,78	32	30,5	15,25	30,5	0,4	1350	0,95	1	0	4	1	0	0	0	0	0		0
																					29708			

4.3.3 Deep Foundation

Deep piles have some advantages over other foundations, such as high efficiency in clay and high axial load capacity. These and above demonstrated reason caused to consider pile foundation as main foundation type. In addition to advantages, different types of pile foundation are considered. There are mainly three materials used for pile foundation: timber, steel and concrete. Steel and timber piles are not appropriate due to high corrosion of the soil on the site and non-cost efficiency. Despite this, steel still can be used inside the concrete pile, since steel is protected from corrosion by concrete. As a result, from this research, concrete pile will be used as foundation of the design.

There are two main types of concrete piles: bored pile and driven piles. Based on characteristics of these types and cost estimation, proper pile will be selected. Main function of pile is to support the above structure.

1. Point bearing capacity: Ability of pile to transfer load directly from the structure to the stiff soil or rock layer. To calculate bearing capacity of the pile, Meyerhof's equation will be used. Depending on soil, sand or clay, Meyerhof provided following formulas:

$$Q_p = A_p q_p = A_p q' N_q^* (\text{for sand}) \quad (4.3.3)$$

$$Q_p = A_p q_p = A_p c' N_c^* (\text{for clay}) \quad (4.3.4)$$

In the first equation, the value of q_p should not exceed the value of q_l , which is equal to: $q_l = 0.5 p_a N_q^* \tan \phi'$

2. Frictional resistance: Ability of pile to transfer load to the soil along its length.

For sand frictional resistance will be calculated using following equation:

$$Q_s = \sum p \times \Delta L \times K \times \sigma'_o \times \tan \delta' (\text{for sand}) \quad (4.3.5)$$

Using equations above, load capacity of the pile will be calculated. In the formulae, the main unknown parameters of a pile are length and width of a pile. These factors have effect on capacity of the pile. To find most optimum length and width, some assumptions would be made.

4.3.4 Comparison of Shallow and Deep Foundation Types

Table 4.4. Foundation Characteristics

	Cost Effectiveness	Installation/Equipment
Shallow Foundation	<p>Cost is affordable:</p> <ul style="list-style-type: none"> • No piling is needed • do not require specialized construction equipment or tools • Saving labor resources/manpower 	<p>Construction procedure is simple:</p> <ul style="list-style-type: none"> • Earthworks at smaller volumes • Easy arrangement of formworks • Mostly needed material is concrete • Labor does not need expertise
Deep Foundation	<p>Relatively expensive:</p> <ul style="list-style-type: none"> • Piles are required • Increased consumption of concrete • Increased earthworks • Need for additional techniques 	<p>Construction procedure is more complicated:</p> <ul style="list-style-type: none"> • Need for special plant • Complex structural design • the construction can be carried out in any terrain; • work can be carried out in any weather conditions;

Advantages of shallow foundations:

- It is suitable for foundations with the depth equal or less than the foundation width.
- It can be applied when the bearing capacity of soil is high at shallow depth.
- The settlement can be reduced when the soil is compressive.
- Total cost can be decreased since the piling is not required. At the same time, there is reduction in the amount of labor resources and manpower needed (Civil Engineers Forum 2016).

However, design of shallow foundations has some limitations. The main disadvantages of this foundation type are listed below.

- It cannot be used when the weight of the building is high and distribution of structure load is unequal.
- Top surface soil has a minor bearing capacity.
- If the sub-soil water level is high, it would not be economical to pump out the water from the hole or canal.
- If the structure is near sea or river and scouring might occur, then the shallow foundation is not convenient (Civil Engineers Forum 2016).

Advantages of deep foundation:

- Pier of any length and size can be constructed at the site;
- Construction equipment is normally mobile and construction can proceed rapidly;
- Inspection of drilled holes is possible because of the larger diameter of the shafts;
- Very large loads can be carried by a single drilled pier foundation thus eliminating the necessity of a pile cap;
- The drilled pier is applicable to a wide variety of soil conditions;
- Changes can be made in the design criteria during the progress of a job;
- Ground vibration that is normally associated with driven piles is absent in drilled pier construction;
- Bearing capacity can be increased by under reaming the bottom (in non-caving materials).

Disadvantages of deep foundation:

- Pile driving produces noise and causes disturbance to the adjacent structures and facilities.
- During pile installation it may break and if it is not detected or replaced the whole building can be damaged.

As it was mentioned before, due to the performed calculations, shallow foundation is seen as inappropriate type to be applied because of the excessive distance in the

settlement at the center of the foundation; whereas, the deep foundation (pile foundation) shows good results and can sustain the building load in a proper manner.

Selection of foundation type for a building can be made based on two conditions: ground and load from the building. In terms of ground conditions, the soil about the surface cannot support the structure loads and the hard soil layer is very deep. Therefore, shallow foundation cannot be applied for such cases. Additionally, soil at top layers close to the ground have relatively low bearing capacity, hence at those areas the deep foundation is needed.

On the other hand, in terms of load from the building, for high-rise buildings deep foundation design is preferred. This type is usually considered since the ground is mostly compacted at greater depth (The Constructor 2015).

According to the Table 4.4 and above mentioned pros and cons, shallow foundation design could be selected as it is more cost effective and simple to construct. However, it is not applicable since the soil conditions do not satisfy the needs.

There are mainly three materials used for pile foundation: timber, steel and concrete. Steel piles are commonly used for high-capacity foundations due to their high strength, ductility and easiness to splice, but they are expensive and produce noise while driving them. These piles can be easily subjected to corrosion. Therefore, in order to decrease the cost, avoid corrosion and noise during the installation, steel piles will not be considered in the foundation design. Piles can also be made up of timber. Such piles are the most favorable in terms of cost, however timber piles are mainly suitable for foundations with light driving conditions. As a result, from this research, concrete pile will be used as foundation of the design. There are several advantages of using concrete piles such as that they can be exposed to hard driving, can be readily joined with a concrete superstructure and resistant to corrosion. Moreover, it is better to use concrete piles as friction ones which will not face refusal (refusal is the state when the pile cannot be driven any longer, hence it would be needed to cut off the top portion) while driving. However, difficulties in performing accurate cutoff and transportation can cause some drawbacks. Nevertheless, concrete piles are well known and commonly used since they are less expensive than steel piles having a large load capacity at the same time (Coduto 2014). Due to the weak soil conditions and absence of bedrock or

rocklike material, point bearing piles cannot be used. More economical and effective way is to use friction piles.

4.3.4.1 Design of Pile Foundation

The pile foundation will be performed in a form of a group piles. Before designing the piles, first of all, it is necessary to decide on the cross-sectional shape of the piles. As it can be seen from the Figure 4.9 below, piles may have various shapes in cross-sectional view, such as triangular, square, hexagonal, circular, rectangular, elliptical, etc. Based on the advantages and disadvantages, listed on the figure, the decision laid on the square shape, because of the ease of the installation, good skin-friction perimeter to volume ratio, low manufacturing cost, and good major axes bending strength.





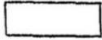


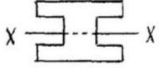
Shape	Advantages	Disadvantages
	Highest ratio of skin-friction perimeter to cross-sectional area. Low manufacturing cost.	Low bending strength.
	Good ratio of skin-friction perimeter to volume. Low manufacturing cost. Good bending strength on major axes.	
	Approximately equal bending strength on all axes. Good penetrating ability. Good column stability (l/r ratio).	Surface defects on top sloping surfaces as cast are hard to avoid.
	Equal bending strength on all axes. No sharp corners aid appearance and durability. Minimum wave and current loads. Good column stability (l/r ratio).	Manufacturing cost generally higher. Surface defects on upper surfaces as cast are hard to avoid.
 or 	May be used where greater bending strength is required around one axis, especially if minimum surface to lateral wave and current forces is desired.	Difficulty in maintaining orientation during driving.
	High bending moment about axes in relation to cross-sectional area.	High cost of manufacture. Difficulty of orientation.
	High bending moment about axis X-X in relation to cross-sectional area.	High cost of manufacture. Difficulty of orientation.

Figure 4.9. Cross Sectional Shape for Piles (Ben C. Gerwick, n.d.)

Moreover, another crucial factor is the choice of material (timber, steel, or concrete). The cheapest one is the timber material; however, it is appropriate for low structural loads only, which is not suitable for Utopia building. Steel piles are the most expensive and noisy in installation, but they would be the most resistant to the tensile

strength. Nevertheless, the usage of steel material for the piles would be really money demanding and increasing the overall construction process cost, which should be avoided. Therefore, the remaining material, which is concrete, becomes the most beneficial material to be used for the pile foundation, since it is not very expensive and can sustain high structural loads. In addition, concrete piles are not really complicated in installation method, since they are driven into the soil.

To select pile length, it is necessary to consider that the pile should not reach the last layer, which is mostly clayey. Thus, the maximum length of the pile is taken as $29 - 5.9 = 23.1$ m.

The piles may be formed into a group in different ways. So that, the pile group may consist of 2, 3, 4, and even 18 piles in one group. In order to be able to resist shear and moment loads on the piles, it would be more beneficial to design 4-piles group. Moreover, from the construction point of view, the 4-piles group is easier to be installed, rather than any other groups, especially asymmetric ones (3-piles, 5-piles, etc.). Eventually, from the economic point of view, the usage of 4-piles group is the most optimum one, because it does not demand high expenses; whereas, the usage of more piles can be not cost effective, where 4 piles may sustain the structural loads in the same way.

After deciding on the piles group, the decision on the pile width of 0.6 m was done. Thus, the width of the piles is 0.6 m, and the maximum length of the piles is 23 m.

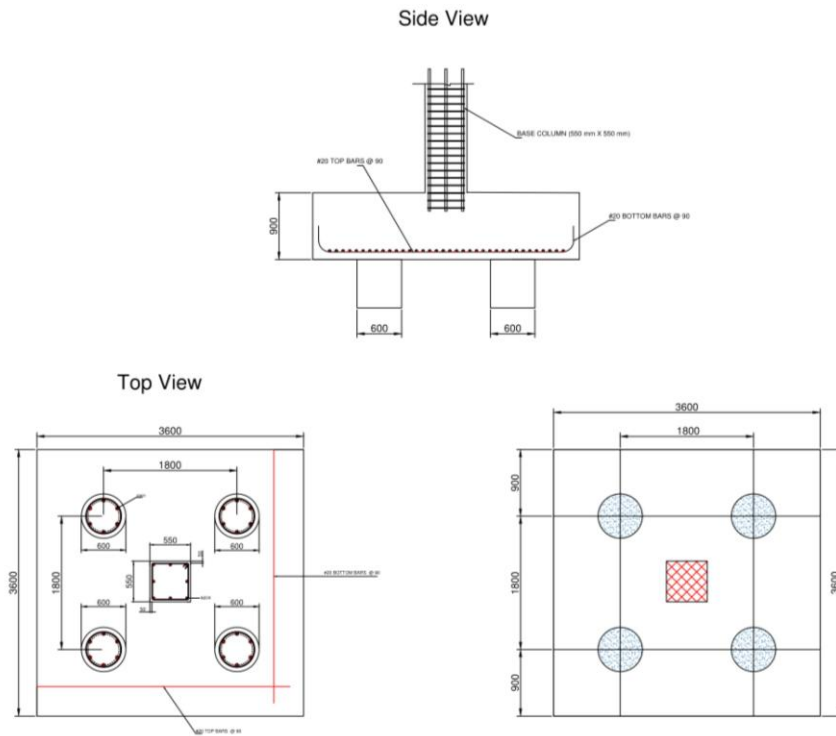


Figure 4.10. Foundation Design

4.3.5 Installation Method

The piles for the hotel building will be driven into the soil by a pile hammer, which is shown in Figure 4.10 below:



Figure 4.11. Pile Driver

Driven piles may be made of timber, steel or concrete. Even though, the timber material seems to be the cheapest option to install the piles, it is used only for lower structural loads, which is not suitable in case of Utopia Hotel. Whereas, the steel piles would be the strongest foundation for the building, it is not beneficial to use that type of material from cost-efficiency perspectives, and moreover, the noise releasing from installation of steel foundations is much higher than from any other type of the piles material. Thus, for our project, foundation would be made from the concrete, because of relatively lower cost and high compressive strength. In that way, if the piles material is concrete, it means that the piles are precast.

While driving the piles into the soil, the equal amount of soil, as the volume of the pile, is displaced, which helps to compact the soil around the pile sides; so that, the soil is further densified. Such compaction and densification of the soil increases its bearing capacity for the pile foundation driven into it (The Constructor 2015).

Since the soil on the location, where the project would be located, is not in the stiff ground conditions, there is no need in pre-augering (breaking up soil and making it from cohesive to non-cohesive condition type). Therefore, pile driver operates in the way that a pile driver hammer of the weight, which is approximately equal to the weight of the pile, is brought an appropriate height up and released to hit the head of the pile. In our case, the piles are driven by groups of 4 piles; thus, the hammer hits a pile cap, which covers and binds four piles, and so drives four piles at the same time into the soil (The Constructor 2015).

4.3.5.1 Static Loading Test

Despite the fact that the hand calculations on determining the settlement and ultimate bearing capacity of the soil are performed and show the ability of the foundation to sustain the load from the hotel building, it is still necessary to conduct the in-situ test to check its capability on practice.

First of all, it should be mentioned that before starting the test, it is required to wait for the minimum period of 7 days since the installation of the last pile group on the site.

The principle of static loading test is that it is necessary to use high weights, usually iron or concrete, onto the pile. In that way, the platform is constructed in order to place the weights on it. The platform is set up onto the pile and puts it under the

loads. After that, the electronic gauges measure the movement of the pile, its resistance to the loads, change in elevation of the top of the pile and other characteristics to find out the properties of the ground. Thus, if the elevation of the pile has changed by more than 5 mm, it means that the ground does not sustain the load, and some considerations must be taken into account.

4.3.5.2 Non-Destructive Test

While driving the piles, there is always a risk of piles concrete being damaged. Since the piles are stroked with the hammer with high power into the soil, there is a big probability of appearing of breaks inside the pile due to hitting or friction of concrete with soil. With that purpose, it is necessary to conduct the test checking the breaks inside the pile.

The most frequently-used method to test the pies is non-destructive test, namely stress-wave method. This method is based on the small impact given to the top of the pile, and then the time necessary for the stress wave, which is produced by the impact, to go down through the pile and to be reflected back to accelerometer. The impact is usually transmitted by the small sledgehammer with electronic trigger.

5 INSULATING MATERIALS

Insulating materials are construction materials which prevent the outflow of indoor heat and inflow of outdoor heat. The main function of these materials is to generate a thermal envelope of a building and reduce the heat transfer (Haimei 2011). Insulating materials play an essential role in energy-efficient building measures. In last decades, the usage of insulating materials has been significantly growing in major countries. Due to their heat-insulating features, they preserve heating and cooling energy and hence facilitate in the reduction of carbon dioxide emissions (Margit et al. 2007). Insulating materials are mainly used for walls, ceilings, floors, roof and thermal equipment/pipelines as well. Insulation can be performed on exterior or interior walls depending on the architectural requirements. Roof can also be insulated in two ways: over the deck or under the deck. However, over deck insulation is mostly preferred to keep away from the absorption and retention of heat by the concrete surface ('Building insulation' 2008).

5.1 Properties of Insulating Materials

A low thermal conductivity is certainly the most essential property that insulating materials should possess. However, there are other important properties which must be considered while selecting insulating materials for a particular application (Haimei 2011). For example, characteristics such as density, thermal resistance (R-value), thermal transmittance (U-value), specific heat capacity, water absorption and reaction to fire are desirable to take into consideration (Margit et al. 2007).

Density has a direct relation to the thermal performance features of the insulating materials. A low density basically indicates a high porosity or a high volume of voids, which consequently implies reduced thermal conductivity. Therefore, the thermal performance of material is as better as the density is lower (Margit et al. 2007).

Thermal conductivity is a measurement of a capacity of a substance to transfer a thermal energy, hence insulating materials are not supposed to have a good ability to conduct heat. In order to prevent heat flows through the material, the thermal conductivity should be low (Margit et al. 2007).

Thermal resistance or R-value is a variable that specifies the resistance to heat flow. The R value of the insulating material depends on the material type, the thermal conductivity and thickness of a material. The higher the thermal resistance, greater is the effectiveness of building insulation ('Building insulation' 2008). Its relationship with thermal conductivity, λ , and material thickness, d , can be displayed as follows (Margit et al. 2007):

$$R = \frac{d}{\lambda}, [\text{m}^2\text{K}/\text{W}] \quad (5.1.1)$$

Thermal transmittance or U-value is the reciprocal of the total thermal resistance. It can be defined as an amount of heat exchanged per second between a surface of 1 m^2 and the surrounding air during permanent heating with a 1K difference in temperature between the air and the surface ('Building insulation' 2008).

$$U = \frac{1}{R}, [\text{W}/\text{m}^2\text{K}] \quad (5.1.2)$$

Specific heat capacity can be defined as a material's ability to absorb heat depending on its mass. This property can be measured with an application of a special device, calorimetry. The amount of heat which can be stored is connected with the microstructure and density of the material. Then, materials with high specific heat capacity can keep more heat (Margit et al. 2007).

Basically, water or moisture absorption is unpleasant for any insulating materials. Ability to absorb water is mainly connected with a growth in the thermal conductivity, since the thermal conductivity of a stationary air is approximately 20 times lower than that of water. However, water can penetrate materials in various ways. For instance, it may happen due to design or construction errors and during the transport or installation procedures. Moreover, it can possibly be as a result of saturation during the storage. Most insulating materials are processed with water-repellent treatment within manufacturing, so that materials do not absorb moisture from the air (Margit et al. 2007). Similarly, insulating materials should not be flammable in order to prevent fire development.

5.2 Types of Insulating Materials

The classification of insulating materials is based on their raw materials, hence they can be divided into two types: organic and inorganic. The insulating materials within these two main groups are further subdivided into natural and synthetic. The inorganic natural materials are ones whose original raw material stays basically unchanged. The synthetic is related to variations in a mineralogical composition of raw materials due to certain treatment processes (Margit et al. 2007). Besides these groups, there are other newly developed materials. The overview of these materials is shown in the Table 5.1 below.

Table 5.1. Classification of insulating materials (Margit et al. 2007)

Inorganic Insulating Materials	
Synthetic	Glass Wool
	Rock wool
	Calcium silicate foam
	Cellular glass
	Foamed glass
Natural	Expanded perlite
	Expanded mica
	Expanded clay
	Exfoliated vermiculite
Organic Insulating Materials	
Synthetic	Polystyrene, expanded
	Polystyrene foam, extruded
	Polyurethane rigid foam
	Polyurethane in situ foam
	Polyethylene foam
	Phenolic foam
Natural	Wood wool
	Wood fibres
	Cotton
	Cellulose fibres
	Insulation cork board
Advanced	
	Transparent thermal insulation (TWD)
	Switchable thermal insulation
	Nanocellular foams
	Vacuum insulation panel (VIP)

Insulating materials described in the table above have different thermal properties and are used in different purposes. The common materials usually used in the market include mineral wools (glass/ rock wool), cellulose, Expanded Polystyrene (EPS),

Extruded Polystyrene (XPS), Polyurethane foam and Vacuum insulation panels. However, in this paper only three types of insulating materials will be evaluated in the further analysis. They are Glass Wool, Polyurethane Rigid Foam and Vacuum Insulation Panel. The reason is that the proposed hotel building is expected to be insulated only in exterior walls. Therefore, these insulating materials were selected among others as the ones most appropriate for wall insulation and which have excellent insulating effects.

5.2.1 Glass Wool

Glass wool is an inorganic synthetic material which consists of quartz sand, dolomite and limestone. It provides an efficient thermal insulation and acoustic insulation as well. Glass wool is light in weight and mainly applied for duct and wall insulation ('Building insulation' 2008). The pleasant feature of glass wools is their flexibility. In other words, they are appropriate as infill materials and can be forced into one position and there will not be a necessity to fix them further; it will stay pinched. Forms of supply of glass wool insulating materials are various. They are commonly prefabricated as rolls, batts, loose in sacks (as caulking material) and boards. Furthermore, the transportation and installation of glass wool materials can be achieved without great effort. They are also capable to recycle and can just be reused as an aggregate in clay brick production (Margit et al. 2007).



Figure 5.1. Glass wool insulating material (Margit et al. 2007)

5.2.2 Polyurethane rigid foam (PUR)

Polyurethane rigid foam (PUR) is organic synthetic material that is now one of the best commercially accessible options. It has relatively well thermal insulating features with low moisture-vapor permeability. Moreover, the density of polyurethane foams is

low and they are highly resistant to water absorption. PUR rigid foams exhibit very low thermal conductivity and high compressive strength. Their reaction to fire is favorable even if the facings are not present. One of the privileges of PUR rigid foams is that they have good resistance to mould and do not rot. However, they can suffer it if there is a permanent exposure to ultraviolet light. PURs are manufactured in the forms of boards and moulded parts. Mainly, PUR rigid foam boards are applied in the metal sandwich elements for roof and façade construction as an insulating core (Margit et al. 2007).

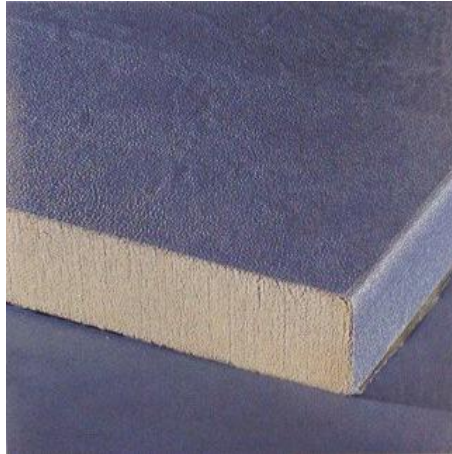


Figure 5.2. Polyurethane rigid foam (Margit et al. 2007)

5.2.3 Vacuum Insulation Panel (VIP)

Vacuum insulating panel (VIP) is an advanced form of thermal insulation. It is made up of a gas-tight enclosure and a rigid core in which there is no air. VIP has an extremely low thermal conductivity in comparison with other conventional insulating materials. Also, these panels reveal high compressive strength. These panels are appropriate for several applications such as insulation of thermal bridges, door cores, rooftop terraces/balconies and internal insulation with inner lining. One of the inconveniences of vacuum panels is an impossibility to cut as thermal properties can be wasted (Margit et al. 2007).

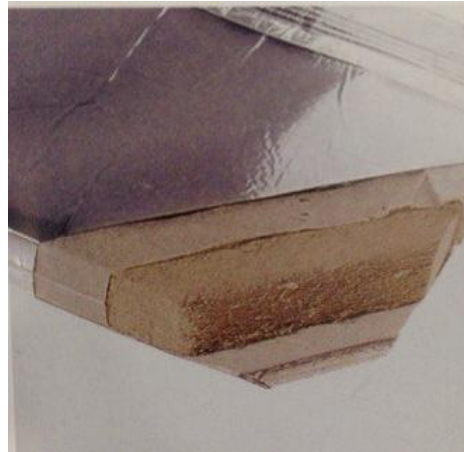


Figure 5.3. Vacuum Insulation panel (Margit et al. 2007)

According to above description and Table 5.2, these three insulating materials have appropriate characteristics that can result in satisfactory wall insulating impacts, although there are some distinctions in thermal properties and are applied differently for certain purposes. Then, in order to come up with the best material, the factors such as their energy performance, economic efficiency and ease of construction will be considered in future works.

Table 5.2. Thermal properties and size formats of insulating materials (Margit et al. 2007).

	Polyurethane (PUR) rigid foam	Glass wool rolls	Vacuum insulation panel (VIP)
Density, kg/m ³	30-100	15-150	-
Thermal Conductivity, W/(mK)	0.024-0.030	0.035-0.045	0.002-0.008
Specific heat capacity, J/kgK	1400-1500	840-1000	-
Available Range of Thickness, mm	20-300	70-240	10-50
Usual formats, mm	Length: 600, 1000, 1200,2400 Width: 500, 600, 800, 1020	Length: up to 9000 Width: 600-200	600×500 1200×500 1000×600

5.3 How Insulation Works

Basically, heat flows where there is a temperature difference: from warmer to cooler side and this heat transfer occurs in three mechanisms as convection, conduction and radiation. In buildings without thermal insulation, large amount of heat may escape making the inside space colder. This consequently will require higher heating energy which in turn increases fuel consumption. However, when building is insulated, insulating materials will perform by lowering the conductive heat flow and radiant heat gain. Most insulating materials have cellular and fibrous structure that reduces convection. Moreover, radiation can be blocked by air gaps with reflective and low-emissivity surfaces, whereas materials with low density and complex solid paths reduce conduction (Shpresa 2011).

5.3.1 Installation Method

Before starting assembling works of insulating materials to any facility, it is required to inspect the exterior walls and select the way of preparation. The base surface must be dry and free from dust and dirt. Additionally, for the installation of materials special glue and plastic dowels with steel rods are used. The insulating boards are installed from bottom up with a compliance of assembling regulations. Also, edges of materials have to be fixed tightly. At the same time, the flatness of materials' surface should be controlled and it can be done by the usage of leveling instrument. The deviation from horizontal and vertical surfaces of assembled materials should not exceed 5mm each one meter distance. The most important thing, there must not be some gaps between the surfaces of base and material or edges of adjacent materials (RITES 2007). Regarding the vacuum insulation panels, during the installation process the materials cannot be cut or swan. The illustrations of assembly works of glass wool and vacuum insulation panels are indicated in Figures 5.4 and 5.5.



Figure 5.4. Exterior wall insulation with Glass wool (RITES 2007)



Figure 5.5. Exterior wall insulation with VIP (RITES 2007)

5.4 Significance of Insulating Materials

There are several benefits of using insulating materials in the construction in terms of both energy and environment. Firstly, building insulation as a new application to enhance the energy-efficiency in buildings assists to considerably reduce the energy consumption. Since insulating materials do not allow inflow and outflow of heat thorough materials, they decrease the energy demands of heating and cooling systems. This, in turn, helps to lower a monthly energy bills. Furthermore, this adds an overall comfort by maintaining a convenient temperature as a result of reduced leakages. Many insulating materials provide thermal as well as an acoustical insulation which is also plays important role in the construction (Shpresa 2011). The most important advantage of insulation is that emission of carbon dioxide (CO₂) can significantly be reduced due to low energy consumption. As it was mentioned above, insulating materials make buildings more energy efficient and hence it reduces amount of fossil fuels required for heating and cooling, thus decrease the quantity of carbon dioxide and sulphur dioxide discharged into the atmosphere. Moreover, based on intensified urbanization throughout the 21st century and increased CO₂ emission as a contributor of global warming and sulfur dioxide as a primary factor of acid rain, frugality of energy consumption is vital on a global scale. Nowadays, thermal insulation has been increasingly used in major countries due to its insulating characteristics and began to be mandatory in many European and Asian countries (Papadopoulos, Karamanos & Avgelis 2002). The overall benefits and drawbacks of using insulating materials are summarized in Table 5.3 below.

Table 5.3. Benefits and Drawbacks of building insulation (Shpresa 2011)

Benefits	Drawbacks
<ul style="list-style-type: none"> ▪ Provides thermal as well as acoustical insulation ▪ Efficient energy savings ▪ Lower energy bills ▪ Reduction in CO2 emission ▪ Reduced heat leakage ▪ Provide comfort temperature 	<ul style="list-style-type: none"> ▪ Installation risks: materials are vulnerable to poor installation ▪ Relatively high cost of many insulating materials with perfect properties (i.e. VIP) ▪ Some materials are not always easily available ▪ Require accurate maintenance for longer lifespan ▪ My lose effectiveness if water is absorbed

6 ENERGY PERFORMANCE

6.1 Methodology

Energy analysis is another important part of this project as it aims to show how Insulating Materials influence on the energy consumption of a typical room during the summer and winter seasons. In order to achieve this goal, a special software program for energy simulations, EnergyPlus (EP), will be applied. Also, additional software, Design Builder, would be used for analysis of energy performance of the building. Final outputs from two software programs results will be compared and based on them contribution of IM integration in energy savings would be evaluated.

6.1.1 EnergyPlus Software

Investigation of how the energy use of the structure behaves after integrating with insulating material can be implemented with the application of EnergyPlus software that provides energy consumption for heating, cooling, ventilation, lighting, equipment influence and hot water use. EnergyPlus (EP) is convenient and useful tool that available for engineers to accurately compute efficiency of the structure. The main advantage of using current program is that software is capable of simulation in time steps of less than an hour. Additionally, the software performs simulation with heat balance-based zone simulation.

The energy simulation in EP goes in five steps such as baseline building, hand calculation, geometry modeling, energy modeling and visualization. However, before these steps, whole information regarding the building energy bills (annual consumption on heating and electricity), floor plans, construction materials and their properties and internal loads schedules should be distinguished and prepared. Moreover, its main distinction from other energy simulating software is that the whole workflow goes within other computer programs. In other words, the EP is not used stand alone, but combination of interconnected tools is employed to make energy performance studies for a baseline model. The overall work sequence is described in the following paragraph.

In the first step, selected baseline building model is created in OpenStudio plug in to Sketch Up neglecting interior floors and walls, leaving only exterior thermal envelope. After, OpenStudio changes the geometry and building information into a

format that EP can read and generates an Intermediate Data Format (IDF) file. This file is imported to IDF editor with climate data within which the simulation takes place. If the simulation is run successfully with no error, final outputs in EP are transferred into an Excel file to display and study the obtained results. Accordingly, by combined operation on Energy Plus and Open Studio energy simulation was generated for two cases: with and without insulation for summer, winter and whole year in order to see the effectiveness of each insulating material. The final decision on the best material will be made within the consideration of cost and energy consumption used.

6.1.2 Design Builder

Besides the tools mentioned in previous part, there is one more useful software known as Design Builder which enables to design energy-efficient building. With the application of this tool, an environmental performance of the building can be easily checked. This software applies the EnergyPlus in simulating processes, so it can obtain any energy performance related data. However, EP tools are installed to Design Builder itself so that it can work stand alone. Throughout the design process primary performance features as energy consumption, carbon emissions, thermal comfort and cost can be provided by rendered outputs. The uniqueness of this tool is the capability to create a baseline model, select required climate conditions and construction details, and then simulate the model to get desired outputs in single software. In comparison with EP (interconnected with Open Studio), Design Builder has its own library where weather data of all countries in the world and variety of construction materials with its properties are included already. Moreover, it is possible to add new materials or change the properties of existing ones for the satisfaction of project needs (DesignBuilder Software 2017). For the energy analysis, a typical corner room located in the fifth floor was chosen as a baseline model. This model was simulated with selected insulating materials: glass wool, PUR and VIP. Obtained results were compared with the simulation results of the model without insulation for the summer, winter and whole year.

6.2 Description of simulated building

In order to see the effectiveness of insulating materials and perform energy simulation one typical room can be selected instead of considering the entire hotel building; hence, a corner typical room at 5th floor was selected as the baseline model.

This room is located on the northwest corner with the total area of 48.96m². From the Figure 6.2 it can be seen that the room has three thermal zones such as Hall area, Bathroom and Bedroom. Also, simulated room has six windows each having a size of 1.2x1.5meters. The walls with windows are exterior ones, while the rest two walls are interior. There are also partitions that divide the room into three zones. This baseline model is created in Design Builder software for further simulation works.

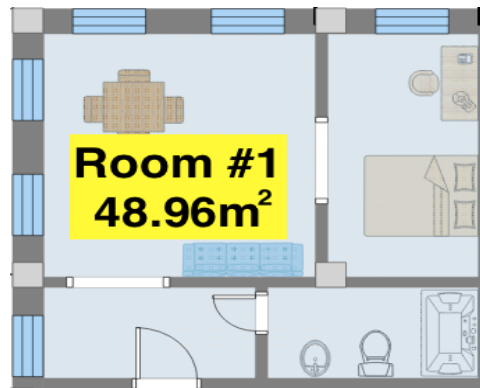


Figure 6.1. Floor plan of typical corner room

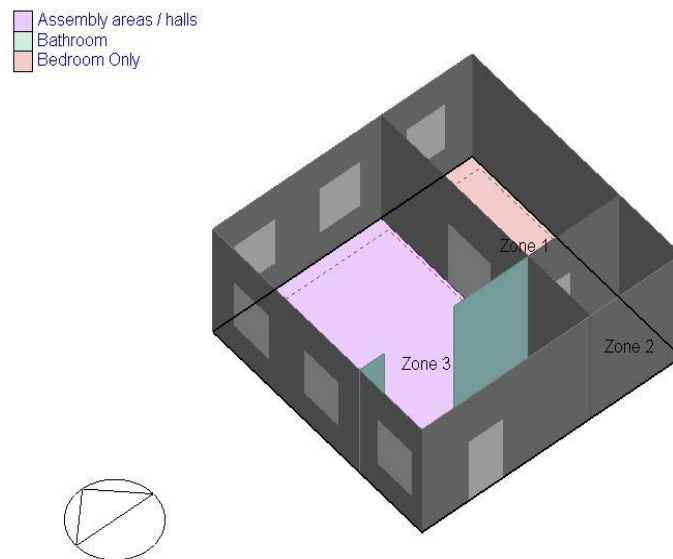


Figure 6.2. Corner typical floor plan created in Design Builder

6.3 Climate in Astana

Astana is located in Central Kazakhstan. In climatic terms, Astana is the third coldest capital in the world, with common temperatures in winter of -35 to -40 °C (Themongolist.com, 2016). Astana is distinct with humid continental climate with extraordinary cold winters and warm summer. A temperature below 0°C starts approximately from the end of October and lasts till the end of March. Average

temperature of warm period in Astana is about 19°C. The hottest month in Astana is stated to be July, which has an average high of 27°C. Another remarkable feature of climate in Astana is that most of days have 50-75% of cloud cover.

The variation of daily high and low temperatures in one year can be observed in the Figure 6.2 below. The illustrated plot is based on the historical weather record for 2015. Referring to this illustration, the hottest day corresponds to 18th of July with the temperature of +38°C, whereas the coldest day matches to the 25th of January with the temperature of -37°C. Therefore, July and January are considered as the hottest and coldest months (Weatherspark.com, 2015).

Since this project includes energy analysis it is essential to take into account climatic characteristics of Astana as it is in direct relation with insulating materials effect. The energy simulation using the EnergyPlus software requires weather data in *.epw* format. Thus, hourly weather information of typical meteorological year (TMY) for Astana was used as an input to the software.

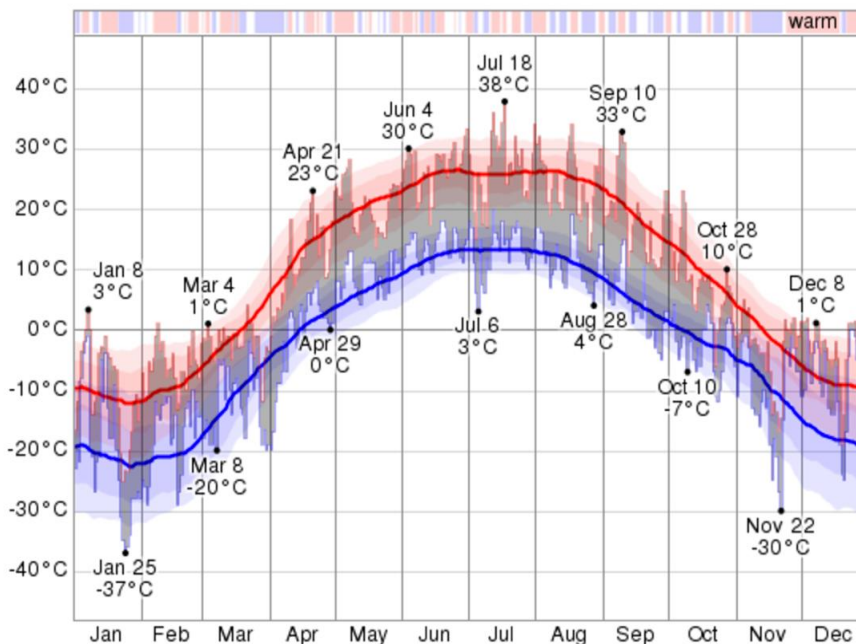


Figure 6.3. Daily low (blue) and high (red) temperatures for Astana, in 2015 (Weatherspark.com, 2015)

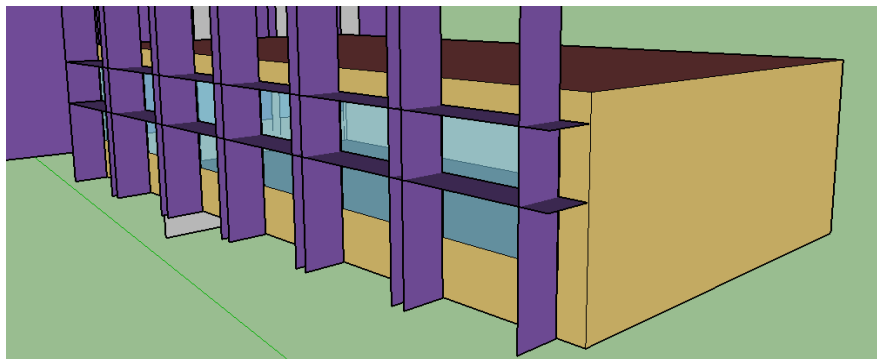
6.4 Analysis and Results Discussion

6.4.1 Simulation on EnergyPlus

In order to choose the best insulating material among mentioned materials, preliminary energy simulation was done in EnergyPlus and Opens Studio software for a typical building with the climatic conditions of Astana. The thermal characteristics of insulating materials such as density, thermal conductivity, specific heat capacity and available thickness are depicted in Table 6.1 that are used as the input parameters for simulating operations. Concerning the baseline building model, it is a beforehand existing layout which is integrated with the simulator. The already created model is the classroom with the total area of 29747 m² (Figure 6.3). This model and all the other required simulation functions were developed by Brendon Levitt, a licensed architect in the state of California and a LEED Accredited Professional, for students use in workshops in UC Berkeley College of Environmental Design. Therefore, it is not possible to create your new model or change the dimensions corresponding to your own case-study. However, material properties and some construction features can be edited.

Table 6.1. Thermal Properties of insulating materials

	Polyurethane rigid foam (PUR)	Glass wool (Rolls)	Vacuum insulation panel (VIP)
Density, kg/m³	50	100	170
Thermal Conductivity, W/(mK)	0.030	0.04	0.005
Specific heat capacity, J/kgK	1500	900	800
Thickness, mm	200	150	40



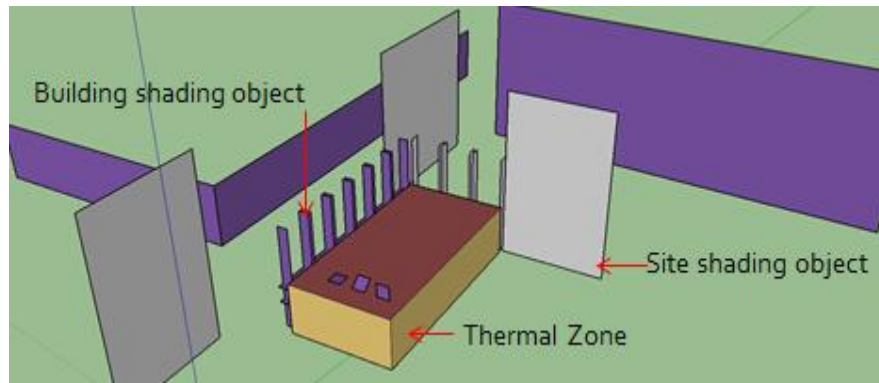


Figure 6.4. Sketchup 3D model integrated with EnergyPlus software

Since insulating materials have various thermal characteristics, each material behaves differently in each period of time. As the presence of insulating materials affects the energy consumption, three different periods of time were examined to find the best material to be covered. All materials were simulated for a whole year, for winter and summer seasons. The obtained modeling results are tabulated in Tables 6.2 – 6.5.

EnergyPlus program itself requires time range when the room is under usage. So, periods of usage for a whole day in summer and winter differs from each other. Furthermore, summer period does not need to be heated due to warm season. So, energy use with insulating materials in summer is much lower than in winter period. This manner can be observed from Tables 6.3 and 6.4. The total energy uses of the model with PUR are 270 756 kWh and 15 384 kWh; with Glass Wool 271 046 kWh and 15 399 kWh; with VIP 270678 kWh and 15399 kWh correspondingly for winter and summer seasons. It should be noted that total energy consumptions with PUR and VIP are almost the same. Moreover, it can be seen from Figure 6.4 that majority of total annual energy use comprises energy consumption during winter season, the heating energy.

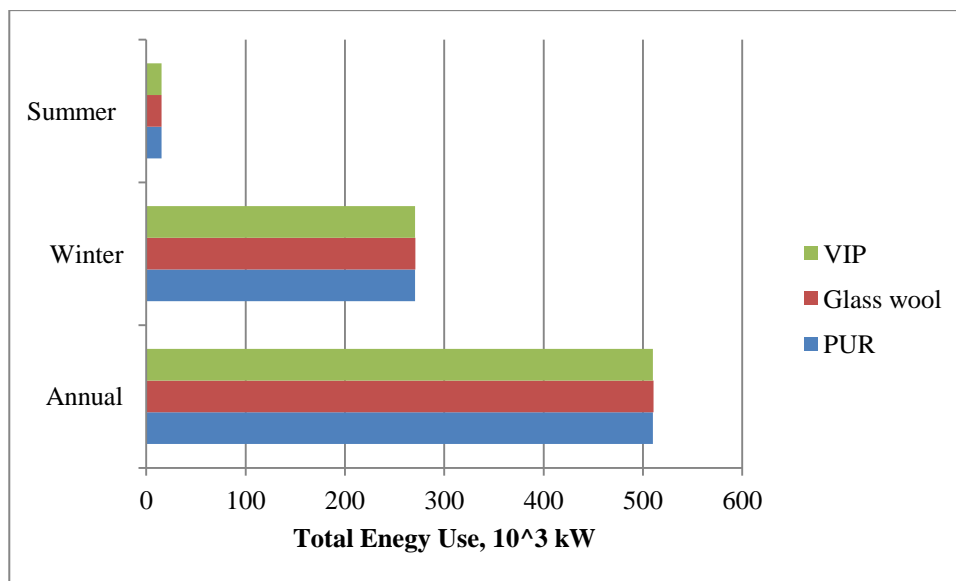


Figure 6.5. Total Energy uses with different IMs by seasons

According to Table 6.2, the most effective insulating material for a whole year is the Vacuum insulation panel with the energy use of 509911 kWh, which is less than Polyurethane rigid foam and Glass wool by 113 kWh and 676 kWh respectively. This numbers seems to be little, however if the energy usage is converted to price sales of electric energy to consumers, energy bills would be 12360242 tenge and 12376625 tenge with VIP and Glass wool accordingly. The difference in bills after using these materials would be 16386.24 tenge (\$52) for a whole year taking the electricity cost as 24.24 tenge/kWh. The price is agreed by the Department of the Committee for Regulation of natural Monopolies and Protection of Competition of the Ministry of national Economy of the Republic of Kazakhstan.

Table 6.2. Energy uses for different insulating materials for a whole year

	PUR (whole year)	Glass wool (whole year)	VIP (whole year)
Heating	476 784	477 344	476 667
Cooling	1 724	1 728	1 729
Ventilation	16 315	16 315	16 315
Lighting	6 230	6 230	6 230
Equipment	8 365	8 365	8 365
Process	418	418	418
Hot Water	188	188	188
Total (kWh)	510 024	510 587	509 911

Table 6.3. Energy Uses for different insulating materials during winter season

	PUR (winter)	Glass wool (winter)	VIP (winter)
Heating	262 075	262 364	261 997
Cooling	-	-	-
Ventilation	4 157	4 157	4 157
Lighting	2 076	2 076	2 076
Equipment	2 286	2 286	2 286
Process	114	114	114
Hot Water	48	48	48
Total (kWh)	270 756	271 046	270 678

Table 6.4. Energy Uses for different insulating materials during for summer season

	PUR (summer)	Glass wool (summer)	VIP (summer)
Heating	6 845	6 857	6 856
Cooling	1 610	1 613	1 613
Ventilation	4 112	4 112	4 112
Lighting	1 002	1 002	1 002
Equipment	1 684	1 684	1 684
Process	84	84	84
Hot Water	47	47	47
Total (kWh)	15 384	15 399	15 399

The classroom model was also simulated without the integration of insulating materials in order to see how the energy consumption changes. The obtained numbers for energy at different internal loads are filled in the Table 6.5.

Table 6.5. Energy use of a room without insulating material

	Whole Year	Winter	Summer
Heating	484 015	266 163	6 705
Cooling	1 680	-	1 575
Ventilation	16 315	4 157	4 112
Lighting	6 230	2 076	1 002
Equipment	8 365	2 286	1 684
Process	418	114	84
Hot Water	188	48	47
Total (kWh)	517 211	274 844	15 210

The difference in total energy consumptions of the model with and without insulating materials is graphically represented in Figure 6.5 below. There is apparent distinction between the amounts of energy use at four cases. It can be observed that the

classroom consumes more energy if it is not constructed with insulating materials. Therefore, it is more preferable to use IMs to make the building energy efficient. According to the results, among three selected materials, the application of VIP can provide in lesser energy use and it can save 7300kW annually. This number is only for a single class room; hence for the entire building it would be a greater quantity.

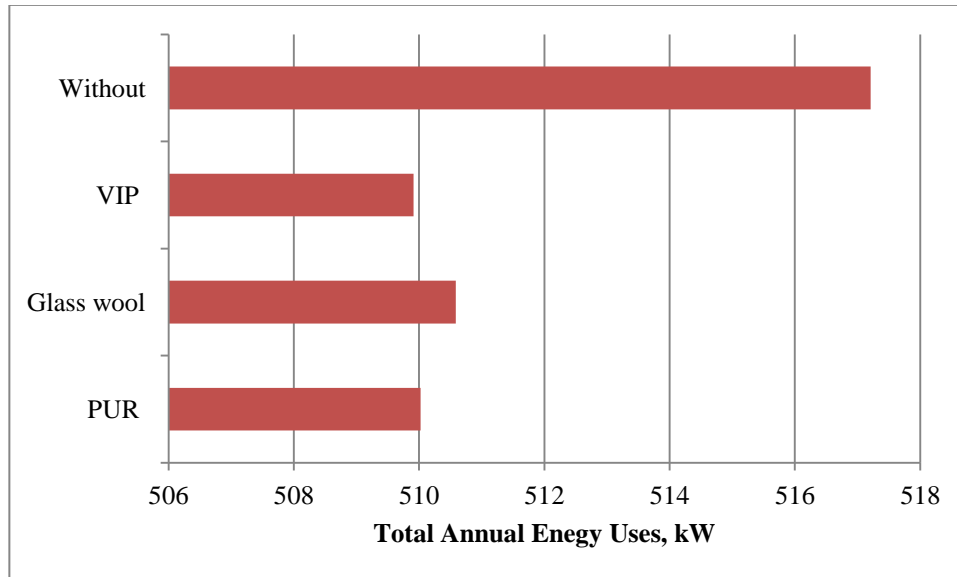


Figure 6.6. Total annual energy consumptions at different cases

If the quantity of energy use that can be saved using VIP is 7300 kW, the amount of money to be saved can also be derived. By multiplying to the sales price of electric energy for consumers, it can be computed that energy cost is $7300\text{kW} * 24.24 \text{ tenge/kW} = 176952 \text{ tenge}$ or \$536.218 per year.

Consequently, it can be concluded that using insulating materials is more efficient and can save more than 7300 kWh for a single class room. So, performing this simulation for the whole building with bigger scale will provide outcomes with the values of saved energy which would be reasonably high.

6.4.2 Simulation on Design Builder

As it was mentioned above, the baseline building model to simulate is the typical corner room at the fifth floor. Firstly, all the project data with Astana climate data was adjusted. Moreover, all external and internal walls, floors, partitions and windows are modeled within their own thermal and construction characteristics. The process of how new project was installed can be seen from the Figures 1 and 6 in Appendix D. Within the fixed project, model of a corner typical room was created which is illustrated in

Figures 6.7 and 6.8. The Design Builder software has its primary functions (dialog) where all model related features can be edited. For example, in the Layout dialog 3D model is created following desired dimensions, whereas the Activity dialog permits to define the usage of the zones including information on occupancy. There is also a Construction dialog where layers and properties of structural elements can be edited.

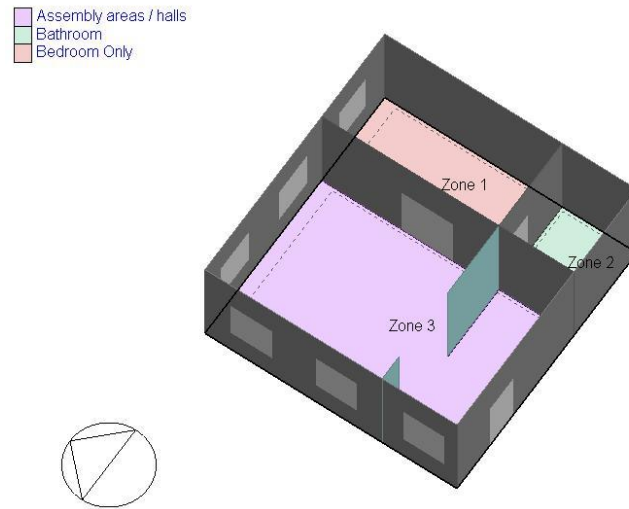


Figure 6.7. 3D model of a typical corner room developed in Design Builder

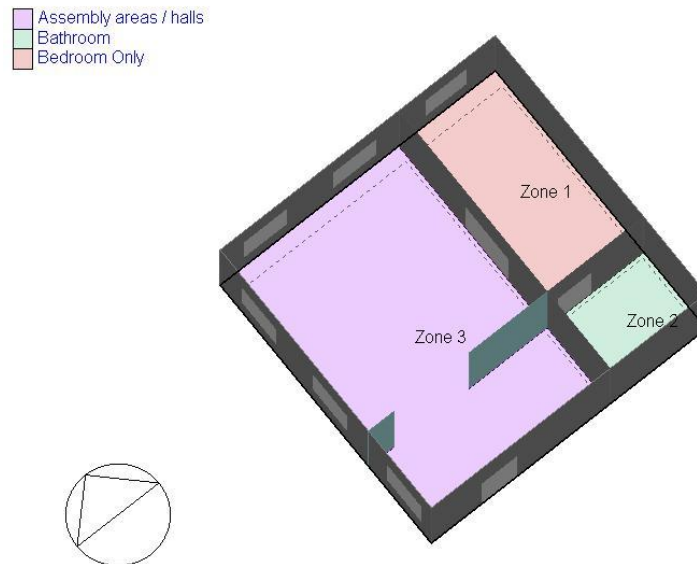


Figure 6.8. Model of a typical room with three zones (Zone1- Bedroom, Zone 2 – Bathroom and Zone 3 – Assembly area)

Since insulating materials are integrated only on the external walls, the layer properties were also adjusted in accordance with material characteristics. The layers of exterior walls insulated with glass wool, PUR, VIP and without insulation are indicated

in the Table 6.6 below. The material layers are ordered from outside to inside of the room.

Table 6.6. Exterior wall details

Wall types	Layers
1	0.4 m Glass wool 0.01m Cement Plaster 0.35 m Reinforced Concrete 0.015 m Gypsum plaster
2	0.2 m PUR board 0.01m Cement Plaster 0.35 m Reinforced Concrete 0.015 m Gypsum plaster
3	0.04 m VIP 0.01m Cement Plaster 0.35 m Reinforced Concrete 0.015 m Gypsum plaster

6.4.3 Model Verification

Energy analysis is performed using two different software programs discussed earlier and simulations in both tools show the effectiveness of insulating materials application. However, only Design Builder will be used in further simulating operations due to several reasons. Firstly, the outputs derived from EP and Open Studio software programs do not correspond to the baseline model of this project. A new satisfying model could not be developed as there are software limitations. Secondly, parametric studies are required for detailed analysis of insulating materials. Therefore, it is possible only with the Design Builder where model of typical room is created and simulation can provide more precise and accurate values. In Figures 6.9-6.11, the total energy uses are represented that are obtained after baseline model simulation by two tools. By comparing these outputs, it can be noticed that in all three cases the energy consumption of the model integrated with VIP is lower than with other materials. Moreover, it should be noted that there is a significant difference between the energy consumptions derived from two software programs. The primary reason is that the simulated models are different in terms of area and usage of zones (activity). The baseline model applied

in EP combined with Open Studio has a larger (29747 m²) are than the one used in Design Builder (48.96 m²).

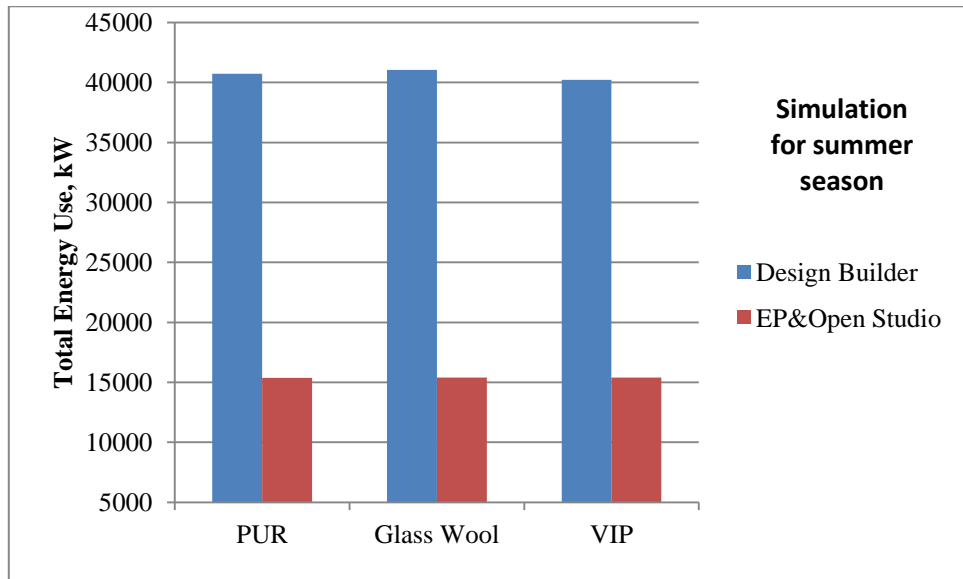


Figure 6.9. Comparison of simulation outputs for summer season

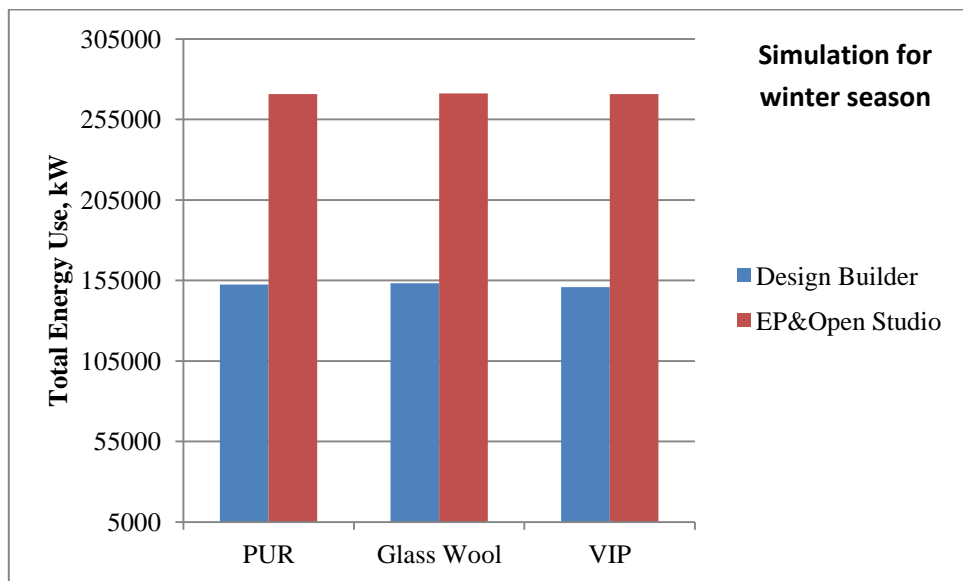


Figure 6.10. Comparison of simulation outputs for winter season

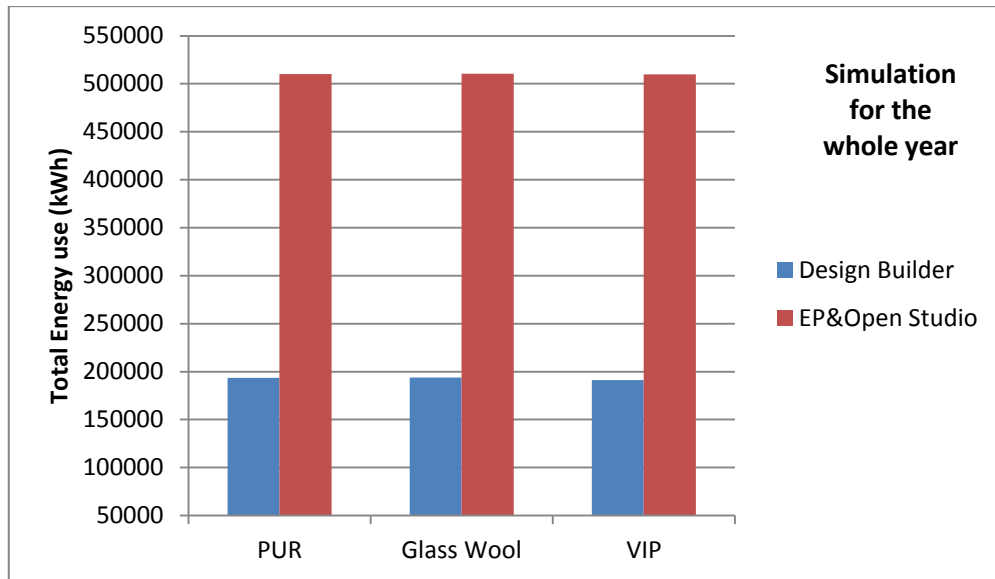


Figure 6.11. Comparison of simulation outputs for whole year

Table 6.7. Amount of energy and related costs of insulating materials

	Total energy consumption	Saved Energy	Electricity cost	Purchase Cost
Glass Wool	194003,98	46142 kW	1118476 tg (\$3550)	\$8737
PUR	191125,09	49020kW	1188261tg (\$3772)	\$102427
VIP	193445,74	46700 kW	1132008tg (\$3594)	\$614563

All above simulation outputs indicate that there is a considerable difference between using and non-using the insulating materials in the construction of building. It was demonstrated that exterior wall insulation can greatly influence on the energy consumption and particular percentage can be saved. Referring to Table 6.7, it can be considered that Polyurethane rigid foam and Vacuum insulation panels are more appropriate ones to reach the project goals. Nevertheless, PUR can save larger amount of energy rather than Glass wool and VIP. However, this reveals bigger electricity cost per one year. With respect to purchase costs, VIP panels are very costly material. Therefore, it is decided that PUR materials will be applied for the exterior wall insulation of the hotel building.

6.4.4 Parametric Optimization

The entire building envelope behaves variously in different climate conditions. For instance, in cold weather conditions it has to prevent outgoing of heat flow and preserve solar radiation. There are various parameters that can affect these features such as

thermal conductivity, specific heat and material thickness. In this part, the thermal properties of insulating materials will remain unchanged, while the most favorable value of thickness will be found out. To achieve this, the room model was simulated with different insulating material thicknesses in the range of 0.02-0.3 meters.

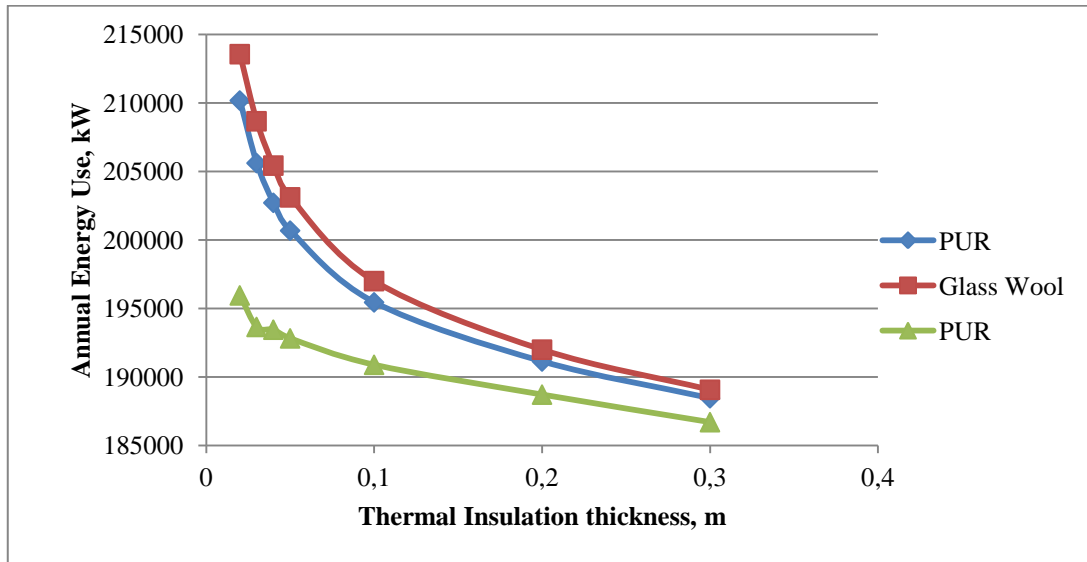


Figure 6.12. Insulation thickness effect on annual energy use

According to Figure 6.12, there is a direct relation between energy use and thermal insulation thickness. It can be observed that, the amount of energy consumed annually decreases as the thermal insulation thickness also goes down. The insulating material thickness can be increased further to lower the energy use, but it is economically inefficient.

Referring to Passive house standard, the recommended value of Polyurethane foam thickness is 0.2 m. For this value, a gradual decrease in energy use is observed from 210174kW to 191125kW. From here, PUR layer with the thickness of 0.2m is viewed as the optimum value.

6.4.5 Model simulation with PUR

The final design provides outputs derived from the model simulation with the optimum thickness of insulating material, PUR. The layer specifications of external wall are represented in Figure 6.13. Heating design and Cooling design calculations were also performed. Heating design calculations are generated to indicate the capacity of heating equipment required to correspond to the coldest winter weather conditions that possible may occur at the site location. Whereas, the Cooling design calculations

determine the size of mechanical cooling equipment needed to confront the hottest summer days.

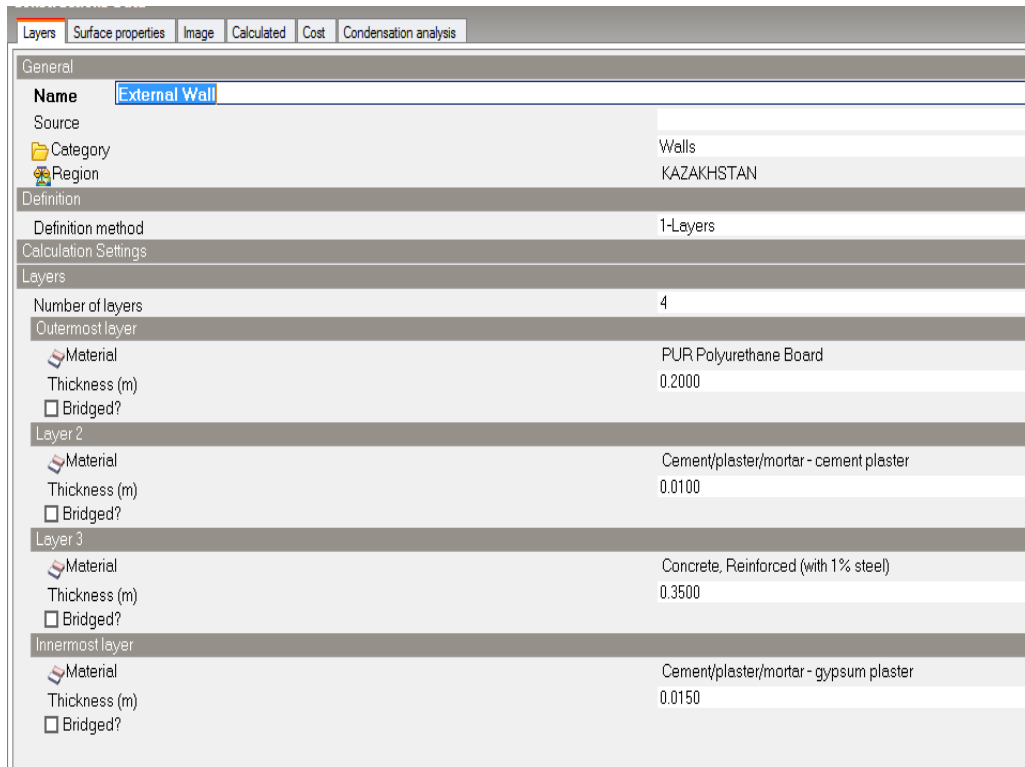


Figure 6.13. External Wall layer specifications with PUR

Figure 6.14 demonstrates the behaviour of heating and cooling energy throughout the year. According to the diagram below, the majority of heating energy is consumed from November to February, during the winter season with cold weather conditions.

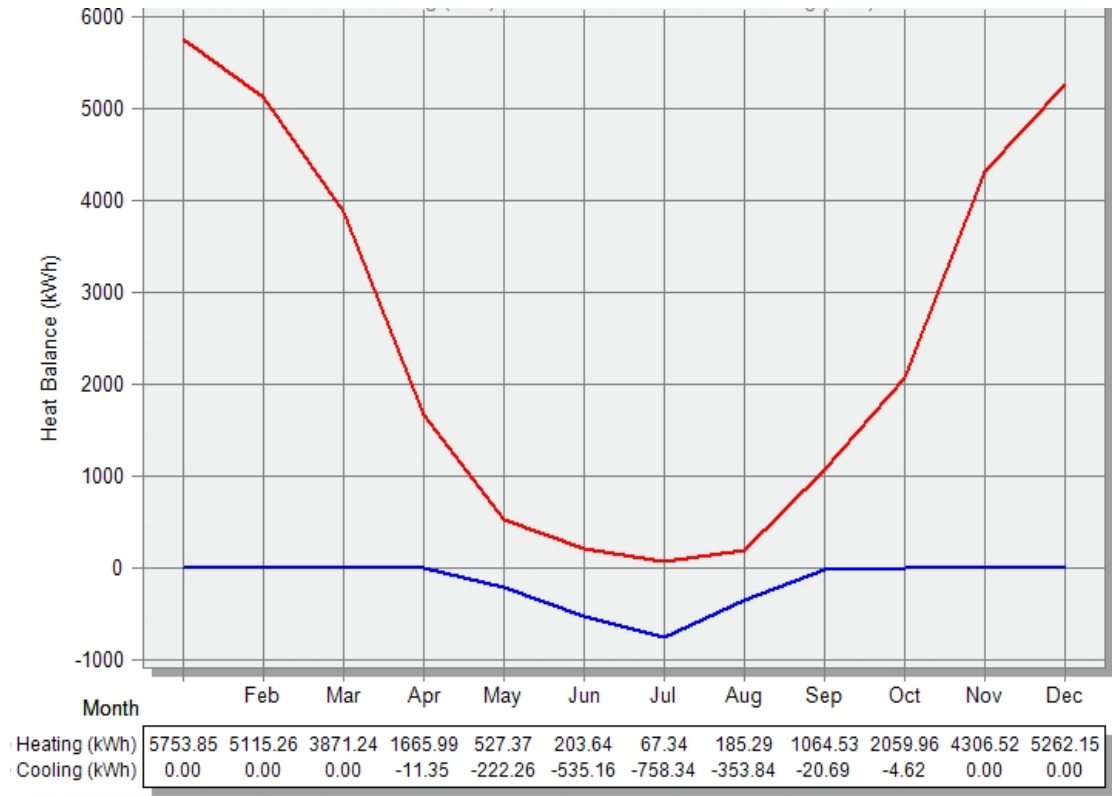


Figure 6.14. Annual Heating (in red) and Cooling (in blue) Energy consumptions

The Design Builder also enables to generate calculation on CO₂ emission. It is apparent that within the decrease in energy consumption, the carbon dioxide emission also reduces. This manner can be observed from Figures 6.15 and 6.16 where monthly CO₂ productions are indicated for the model with PUR and without insulating materials. The room model integrated with PUR has the average gas emission of 1076.25 kg, while the model without IM has the average CO₂ emission of 1288.3 kg. In this way, emission of carbon dioxide can be reduced by 212.05 kg that is 16.5% of one without insulating material.

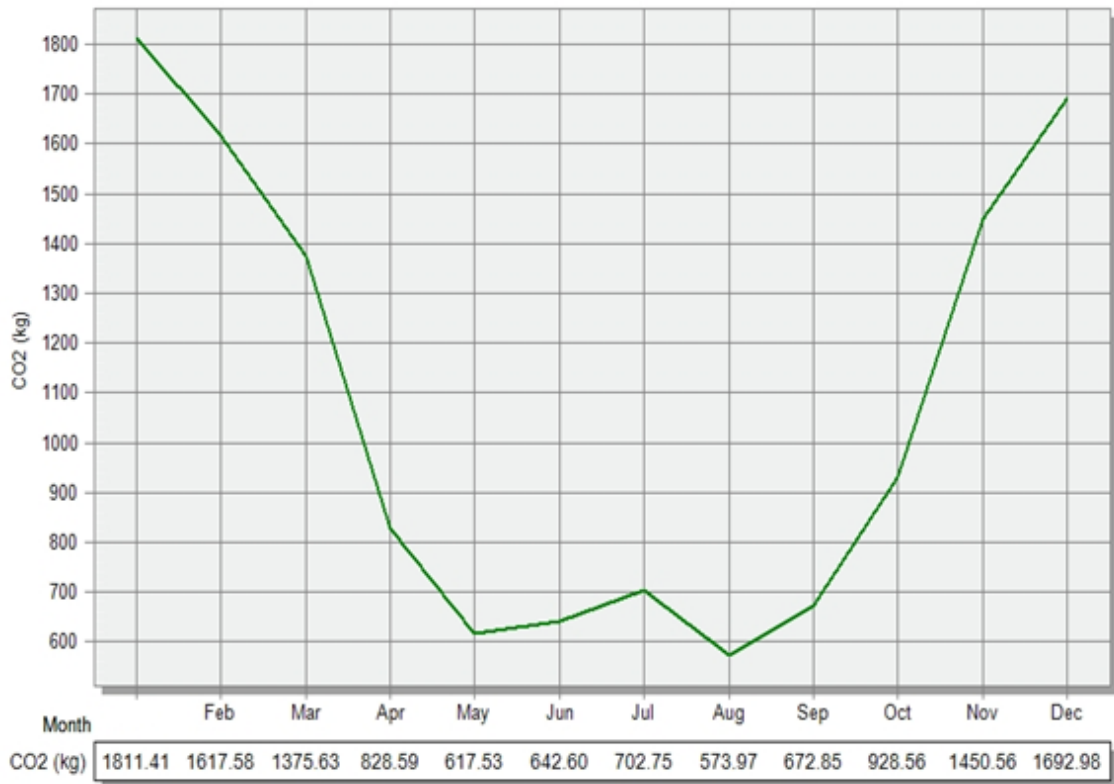


Figure 6.15. Monthly CO2 production with PUR



Figure 6.16. Monthly CO2 production without IM

The total annual energy use of the typical room insulated with PUR is 191125.1 kW. The site energy comprises about 22.4% of total consumption, while the source energy covers 77.6%. The site energy is associated with the amount of heat and electricity required by the building. Whereas, the amount of raw fuel needed to operate the building is represented as the source energy.

Table 6.8. Total annual energy use with PUR, 0.2 m

	Total Energy, kW	Energy Use Intensity, kW/m²
Site Energy	42718,57	940,96
Source Energy	148406,52	3268,95

Finally, in order to make the hotel building as the energy efficient construction, the PUR material was selected for exterior wall insulation purposes with the optimum thickness of 0.2m. As simulation investigations show application of PUR material can result in the energy conservation of 49020.64 kW that correspond to the 20% of energy consumption without insulating material.

Table 6.9. Comparison of total annual energy uses

With PUR	191125,1 kW
Without IM	240145,7 kW
Saved Energy	49020,64 kW

7 CONSTRUCTION PROJECT MANAGEMENT

7.1 Cost Estimation

Estimating project's costs is very essential part of the project management. While the project has being improved, various ways of earning profit from the project would be constantly analyzed and evaluated. Effective project budget management is directly related to how accurate the project cost is estimated. Cost estimation allows to compare expected incomes against expected losses in order to understand whether the project is worthwhile. It also enables to view whether funds required to support the project are available. At the same time, cost estimation is important since it can facilitate in ensuring that the project has sufficient amount of funds for the project completion (Dummies n.d.).

Generally, all costs related to the project are evaluated at its approximate values throughout the project development stage. These costs are reconsidered within each design changes to guarantee that they will not go beyond the available budget (Sears et al, 2008).

During the project planning and design period, preliminary forthcoming expenditures are estimated. As mentioned before, they are necessarily approximate since estimation is generated without completely detailed project. Whereas, the final cost estimate of a project is produced when the definitive specification and drawings are ready. Thereby, the final construction costs will be fairly different (Sears et al, 2008).

Construction projects can be associated by various costs which can be direct and indirect. Even if indirect costs are not straightly connected with construction, they are essential to include in calculations, because they are one of factors that affects project progress. Indirect costs involve expenses that can be undergone to financial agreements, real estate transactions, consultant services, public relations, marketing, government regulations, maintenance and operations (Gould & Joyce 2011). The direct costs include costs such as for material, labor and equipment.

The project price is mainly dependent on the project size, the quality, location, construction techniques used and other market related factors. The ways of performing evaluations can be various by the data available at the time of estimation. There are several methodologies used for the preliminary estimation such as Rough Order of

Magnitude Estimate, Square Foot Estimates and Assemblies Estimates. Besides these methods, there are other techniques known as the Bottom-up method and Top-down method.

7.1.1 Rough Order of Magnitude (ROM) Estimates

ROM estimation method is mainly used in the project's conceptual stage when specified details are insufficient. It sets up the costs per unit of capacity; hence the basic thing needed is the amount of capacity. For example, it can be the number of hotels rooms, parking spaces and number of beds at the hospital. The units are evolved from the past projects and then the costs such as cost per apartment can be obtained. From the product of cost and given number of units, a preliminary price is determined. If the city cost indexes are available, the obtained price may be adjusted. While using the costs from the past projects, it is important to adjust to the current currency. ROM is beneficial since it can be completed in a shorter time than other methods. Accurateness of the obtained results by ROM relies on the correspondence and precision of data employed (Gould & Joyce 2011). This type of estimate is not entirely convenient to this project due to insufficient data of costs per unit of capacity.

7.1.2 Square Foot Estimate

If there are sufficient information about the floor plans, elevations and building sections, the square foot estimate can possibly be used. It allows to calculate the floor areas or building volumes and get the square/cubic foot estimate when they are multiplied to proper unit costs. These unit costs can be acquired from particular data sources or in-house sources with the past similar projects. The output is more precise than the one from ROM. It can take few hours and easy to study. In general the square foot estimate is more preferable since it uses more project-specific information (Gould & Joyce 2011). In the same way as in ROM, this method cannot be used since costs from similar projects are not available. There are no open sources that could provide required information.

7.1.3 Assemblies Estimates

This type of estimation is brought into usage when a relatively more data is provided. The main distinction from other methods is that it applies the assembly or system units of the project. The units can be foundation, the roofing, the electrical system and so on. The units are divided into systems, therefore estimate is more

adaptable, but demands more design inputs and can take several hours. This method is more suitable for the projects with smaller systems and subsystems (Gould & Joyce 2011).

7.1.4 Bottom-up Method

In this technique the costs of individual work packages are evaluated at the bottom level and then are put together to determine the total project cost. This method is more accurate since it considers separate work activities which are small components of the whole project. Bottom-up method can be used when project is relatively detailed and have all required specifications. It is costly and time-consuming, but it provides most reliable and precise output (PM Study Circle 2016).

7.1.5 Top-down Method

In comparison with the bottom-up method, in the top-down approach the total cost is broken down into smaller work packages. Substantially, this method relies on the studies of costs from past projects and their specific parameters. The calculation through this approach is fairly simple and it can be generated using only available data. That is the advantage of this method. However, the completed estimation would be approximate since the some details might be missed. Nevertheless, the results can be updated when the specific details are improved later (Cabinet Office 2016).

7.2 Project cost estimation

The cost estimation for this project will be completed with the application of top-down method. Referring to the descriptions provided above, it is one of the most appropriate techniques for this project. At this stage of project development, there are fairly enough specifications about the project components. However, with the help of top-down method and using cost from past project and statistical data the following cost estimation was developed. In the calculation of project cost, indirect costs will not be included and the estimation will be limited to direct costs only. Furthermore, only the major components related to the whole project cost will be considered such as the material cost, equipment, labor cost and construction costs.

According to the investigations of “Verniy Capital” JSC, the average construction cost of hotel areas for 2015 year constitutes \$2.5–3 thousands per 1m² (Forbes Kazakhstan 2011). For this project, the construction cost per square meters can be

considered as \$3000/ m². Then, for the total area of 20725.2m² the construction cost would be 2072.5m²×\$3000/ m²= \$6217500. This cost includes the overall material, equipment and labor costs.

Referring to the final calculations of structural design, the dimensions of columns are available; hence the material cost of concrete can be determined. Related prices per cubic meters are indicated in Table 7.1. The total cost for concrete is 4918376 tg which corresponds to \$15714. (\$1= 313tg (Kazfin 2016))

Table 7.1. Calculation of costs of concrete for each floor (Satu 2016)

	Number of units/floor	Area, m ²	Price, tg/m ³	Cost, tg
1 floor				
Column I	49	550x550 mm	11000	673750
2-5 floors				
Column II	49x4=196	550x550 mm	11000	1617000
6-10 floors				
Column III	49x5= 245	450x450 mm	11000	1637213
11-15 floors				
Column IV	49x5=245	350x350 mm	11000	990413

To determine the procurement cost for insulating materials, the total surface area of external walls is required. The total surface area to be insulated is 4916.5 m². Through energy analysis, the PUR insulating material was selected and its price square meters is indicated in the Table 7.2. Having the price of \$15/ m² the total costs for the procurement of PUR is \$1024207 (see Figure 7.2).

Table 7.2. Procurement costs of insulating materials (TIU 2016) and (Satu 2016)

	Cost per 1 m ² / cost per roll	Manufactured dimension, mm	Required area/ amount of rolls	Purchase cost
PUR	\$15/ m ²	1200×600× 200	6828m ²	\$102427

Taking into account the procurement of PUR, the total project cost would be as follows:

$$\text{Total Project Cost: } \$6217500 + \$102427 = \$6319927$$

Over the 50 % of profit in hotels comes from the room renting, while the rest percentage would include the income from visiting the restaurant, holding events or celebrations, renting the conference halls (Forbes Kazakhstan 2011).

There are 154 rooms in total that comprises 70 standard rooms, 56 triple or family rooms and 28 suit apartments. In order to estimate the approximate rate per night, four hotels, addressed as five star hotels, were examined which are located in Astana. The number of rooms and the mean prices per one night for different categories are illustrated above in Table 7.3. Then, by calculating the average of prices the following results were obtained.

- Standard Room: 46250 tg per night
- Family Room: 81250 tg per night
- Suit Room: 122625 tg per night

The income from renting rooms depend on the occupancy, hence for further calculation the average of rates was found which is 83375 tg. According to the statistics provided by Committee on Ministry of National Economy, the moderate occupancy of hotels is close to 45 percent. Then, for total 154 rooms with an average rate of 83375 tg per night, the monthly income would be 179×10^6 tg which is about \$572250

Table 7.3. Amount of rooms and rates per night of five star hotels in Astana (Booking 2016)

	Number of apartments	Price for Standard Roms, tg	Price for Family Roms, tg	Price for Suit Roms, tg
Jumbaktas Astana Hotel	61	45000	72000	62500
Grand Park Esil Hotel	126	35000	59000	83000
Beijing Palace Soluxe Hotel Astana	151	55000	102000	215000
Diplomat Hotel	77	50000	92000	130000

According to Forbes investigation on top hotels by revenue in Astana, the average annual income from bar and restaurant services and renting banquet/conference and

rooms halls is 282 million tenge. This value was determined by taking the mean value of annual incomes of hotels with close number of rooms. (282 million tenge = \$902846)

Table 7.4. Amount of rooms and annual estimated incomes of top hotels in Astana (Forbes 2016)

	Number of rooms	Annual estimated incomes
Rixos Khadisha Shymkent	177	375 million tenge
Royal Tulip Almaty	165	254 million tenge
Soluxe Hotel Astana	151	442 million tenge

With approximate values of total project cost and annual income, the payback periods can be estimated for three possible integrations of insulating materials. As presented in Table 7.5, the calculated payback periods are rounded and they are same for three cases.

$$\text{Payback period} = \frac{\text{Total project cost}}{\text{Total monthly income}}$$

Table 7.5. Calculation of Payback period

	PUR
Total Project cost, \$	\$6319927
Total Annual Income, \$/year	\$902846
Payback Period, years	7

The important thing to note is that the above estimations are not exact, provided calculations are approximate. For example, the monthly income found for this project includes only the profit from renting rooms. If the earnings from restaurant, renting conference halls and undertaking nutrition are taken into consideration, the income rate will be higher. This in turn will lower the payback period, which means that the investment can be recovered in a shorter time.

7.3 Feasibility Analysis

A feasibility study is another essential part of management which is required to define the viability of project concept. The main function of this section is to make certain that the project is enforceable from the perspective of several standpoints. For analysis, technical, economical, market and operational aspects should be evaluated. The performed studies will show whether the investment is worth to be committed (Gould and Joyce 2011).

7.3.1 Market Demand

The location of the hotel building is expected to be constructed in Astana, the capital city of Kazakhstan, where the hotel market is one of the dynamically evolving sectors. Within the development economy, industry, business and tourism, there is a considerably growing demand in the hospitality sector. It also should be noted that there is the increasing interest of international network companies in emerging at Kazakh hotel market. It is one of the signs of prospecting of this sector. According to Statistics Agency, over 72% of Astana visitors come for business purposes that is the result of growth in business –activity (Forbes Kazakhstan 2011). Moreover, majority of constantly conducted international events are hold in Astana as a primary city of Kazakhstan. It also should be pointed out that the upcoming EXPO 2017 exhibition will positively influence the hotels development considering that this event will attract about 5 million guests (Forbes Kazakhstan 2011). Furthermore, location of the hotel is very suitable, since it is near all facilities that might be needed and it is located in the left coast where all major buildings (i.e. Khan shatyr, Keruen, Baiterek and so on) are constructed. Hence, it would be easier for guests to get some particular destinations.

7.3.2 Economic Feasibility

The expenditures and profits associated with the project will facilitate in evaluating the viability of the project. As it was obtained above, the approximate project cost is \$6319927 and the expected monthly income from hotel operation is \$902846 with a probable payback period of 7 years. Nowadays, the typical payback period for hotel business in Kazakhstan is between 10 and 15 years (Forbes Kazakhstan 2011). Referring to this statistic, it can be inferred that this project can be proceeded from an economic view.

7.3.3 Operational Feasibility

This study analyzes whether the established goals and requirements are executed by using the proposed approach. Moreover, it is important to see to what extent the selected system solves the issues and brings advantages to the project (Simply Learn 2016). As it was mentioned at the beginning, one of the main purposes of this project is to construct an energy efficient building with an integration of insulating materials. By applying insulating materials in the construction, around 20% of energy can be reduced. Due to their insulating characteristics as discussed earlier, it is possible to save heating and cooling energy and make the hotel energy efficient. Except its heat insulating effect, these materials contribute in the reduction of carbon dioxide emissions. So, there will be no negative influence on the ecology or environment. Therefore, this project is environmentally friendly. At the same time, this project aims to build a safe structure. It can be surely claimed that the hotel will be safe building since all governmental regulations and other related codes and standards were strictly followed.

According to the analyses above, it can be summarized that the project is reliable and can be undertaken. From the market demand, economic, environmental and operational aspects, this project is judged as feasible and the investments can be placed.

7.4 Project Planning and Scheduling

Project planning and scheduling take an important role in construction management, since they facilitate in controlling the project progress step by step. This documentation is required in order to provide successful project completion. It is usually conducted before the resources are committed to the project, so that each activity and their relationship to the ones that go after or before it can be appropriately arranged (Gould and Joyce 2011). Therefore, for successful execution of this project the Work Breakdown Structure (WBS) and Gantt Chart were created which are the typical tools used by many engineers. These tools were implemented in order to provide a clear view of how the project is carried out, monitored, controlled and closed. The project implementation process is represented through WBS (see Figure 7.1). As it is shown, there are main seven tasks which are defining scope, literature review, data collection and analysis, preliminary design, interim report, detailed analysis and project completion phase. All these tasks are completed in series. Moreover, each task has their corresponding sub-tasks. In Figures 7.2-7.3, the Gantt charts for Capstone I and

Capstone II Implementation are illustrated which describes the timeline of tasks characterized in Project Implementation WBS. Additionally, the work breakdown structure for the hotel construction was performed in order to show the general scope of the project. It defines the primary deliverables that affect the overall advancement of construction process. Included deliverables are Project Management, Groundwork, Project Procurement, Building Structure, Interiors and Landscaping. The sub-deliverables are also indicated in WBS (see Figure 7.4).

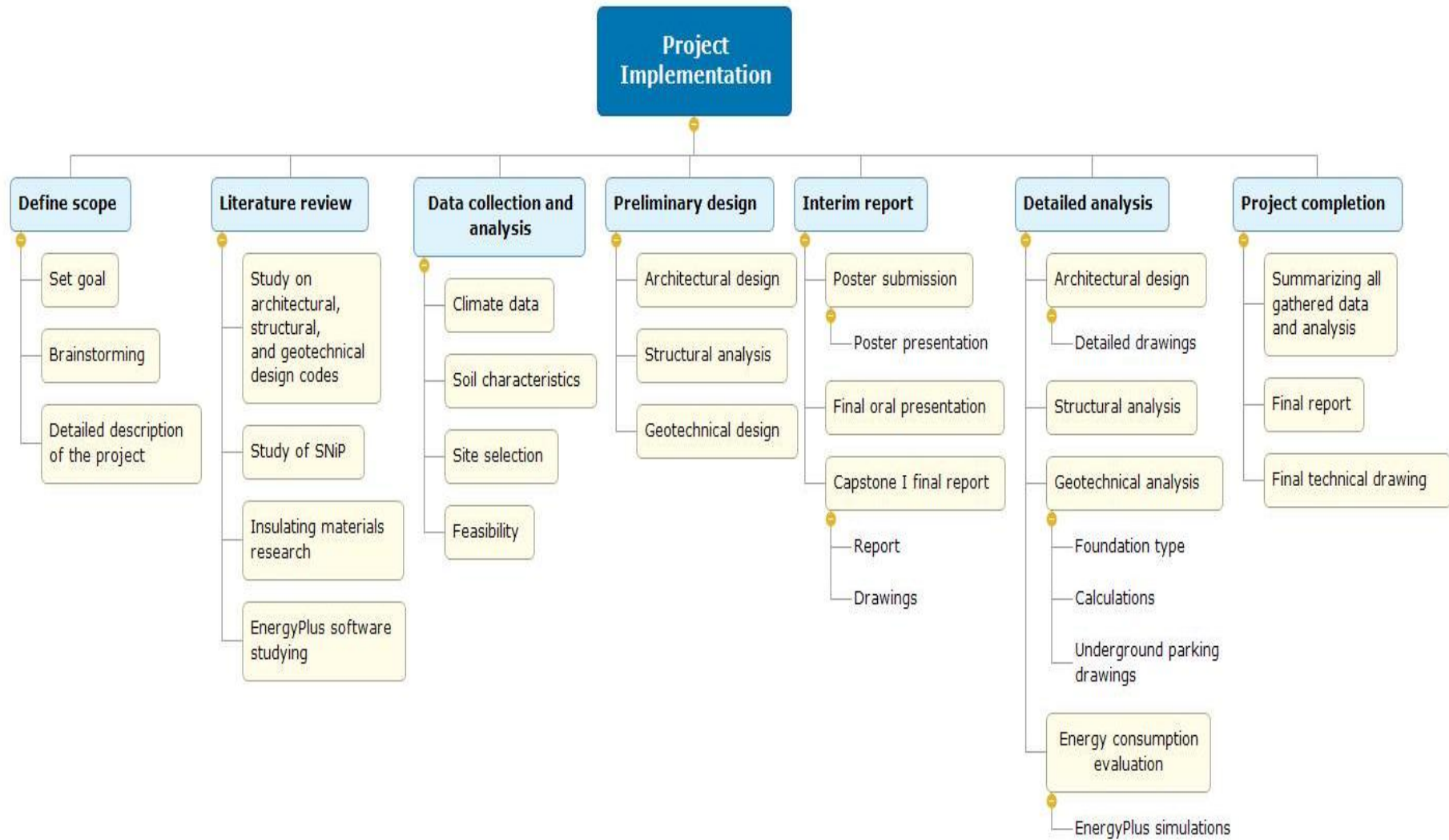


Figure 7.1. Work Breakdown Structure for the project implementation

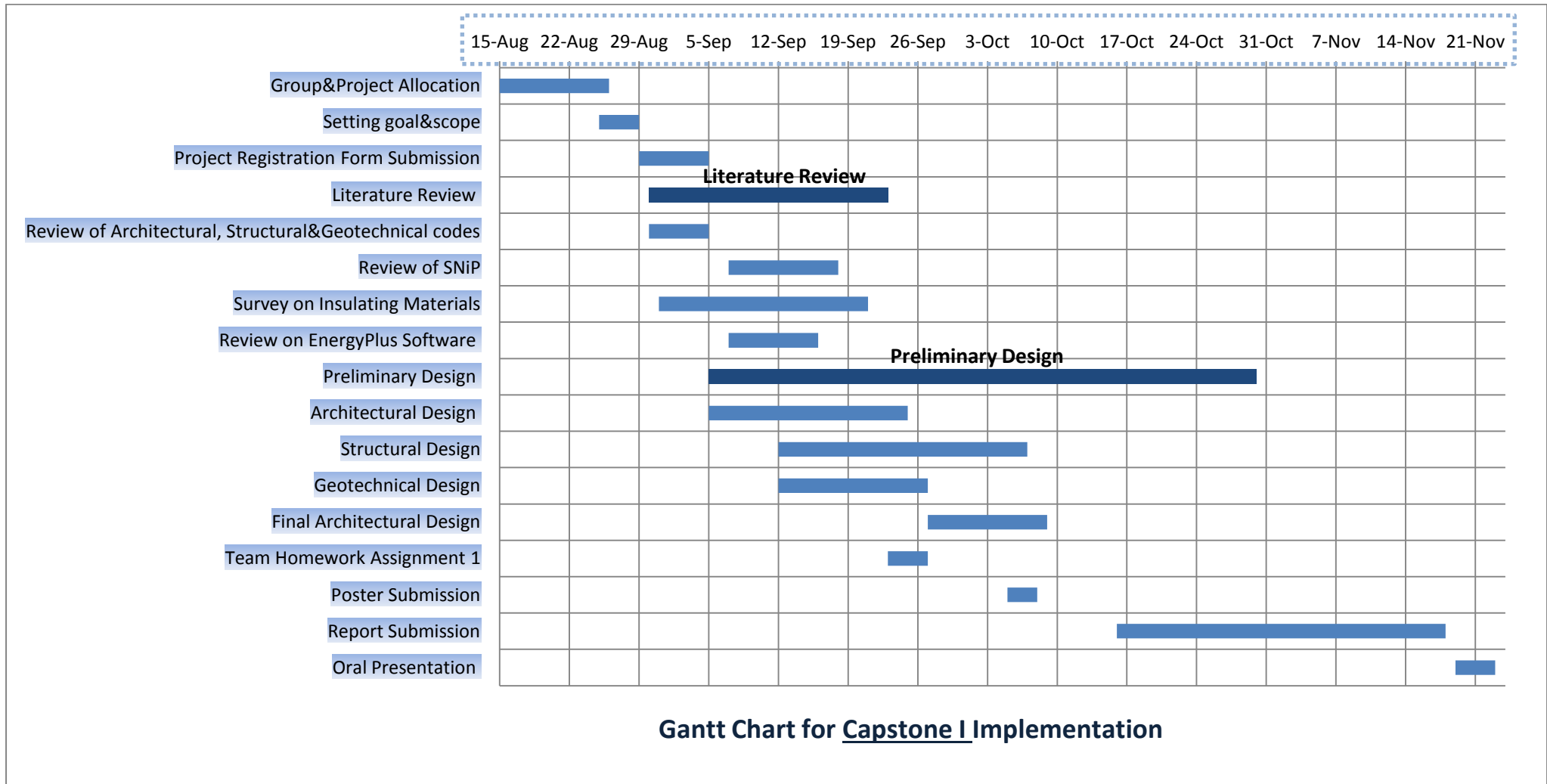


Figure 7.2. Gantt Chart for the Capstone I implementation

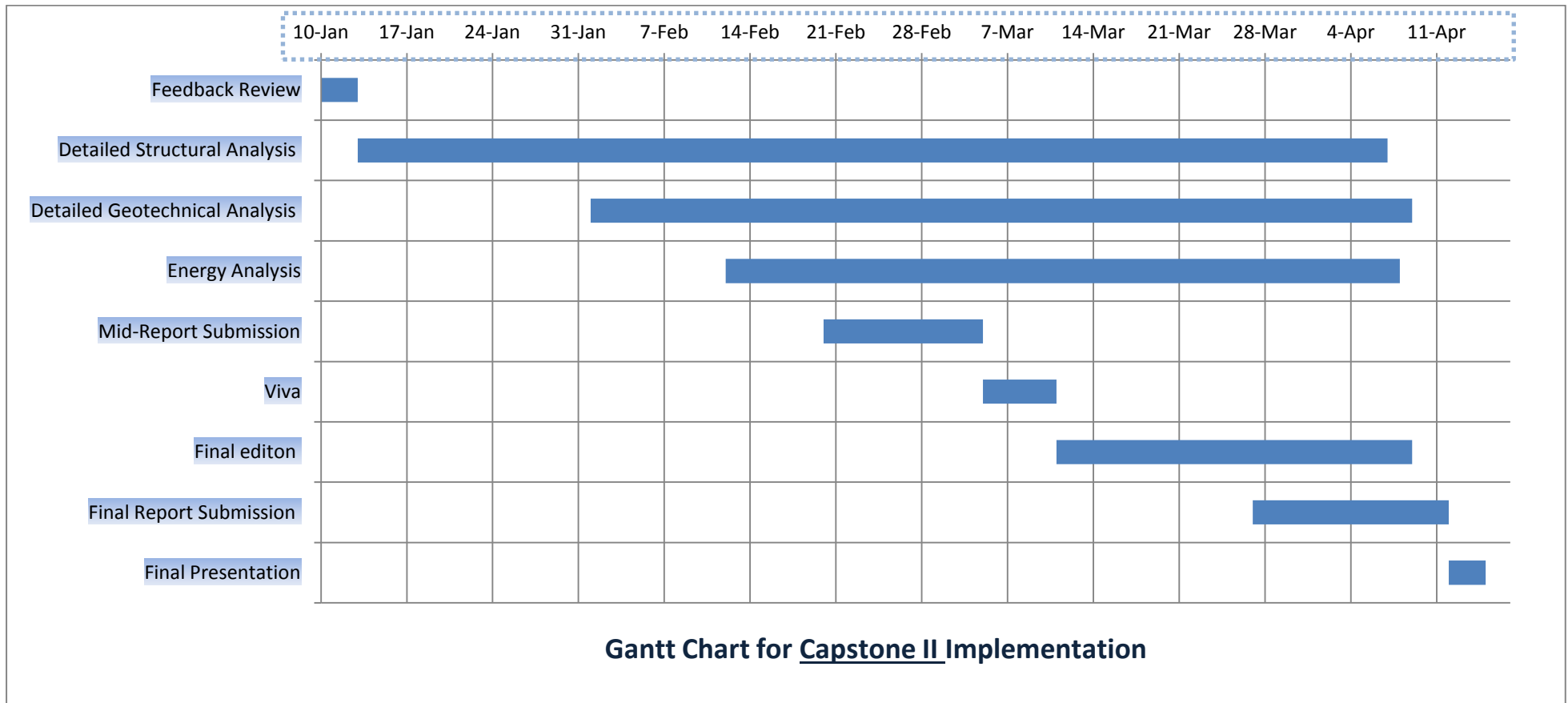


Figure 7.3. Gantt Chart for the Capstone I implementation

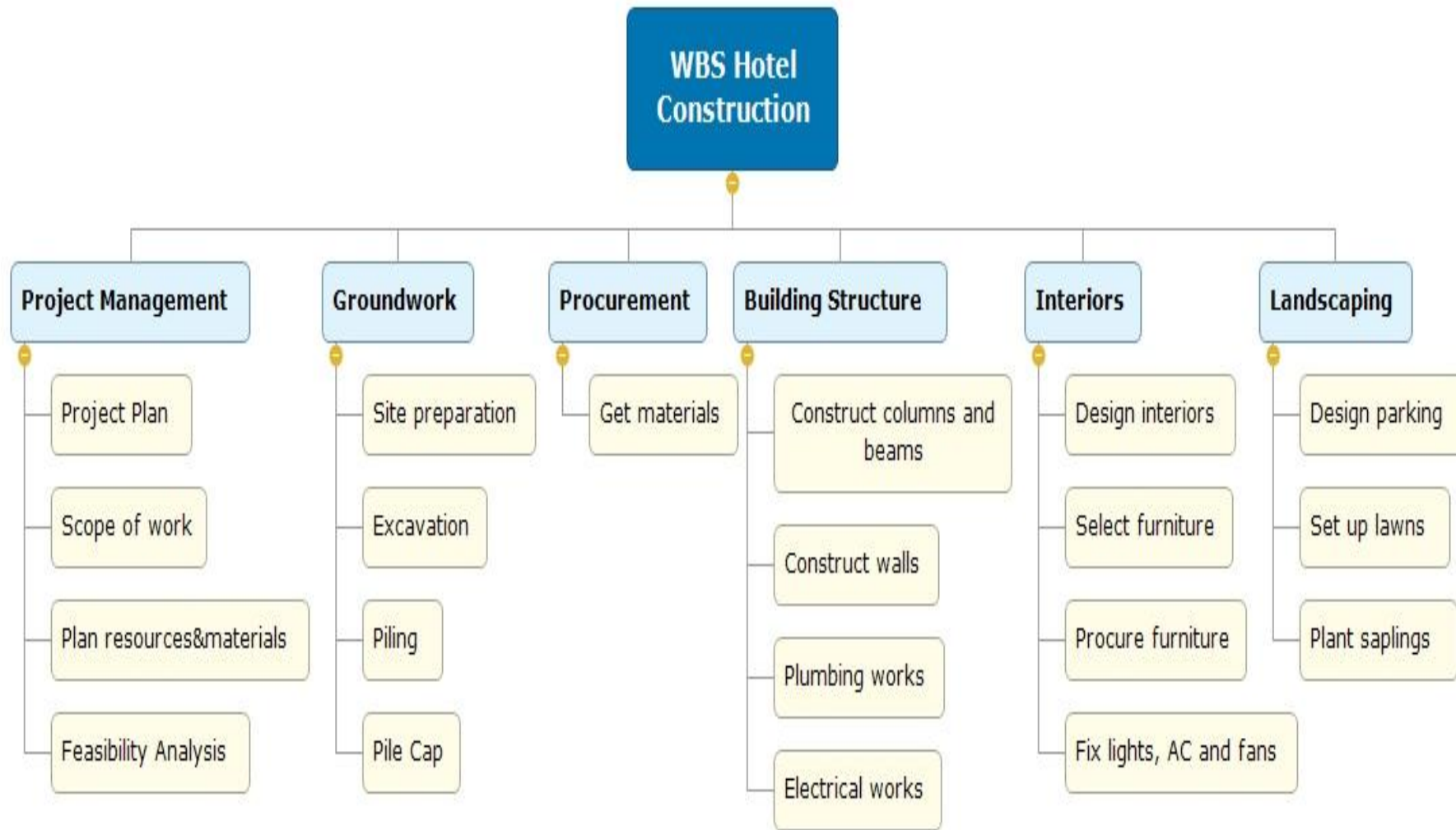


Figure 7.4. Work Breakdown Structure for the hotel construction

8 CONCLUSIONS

To conclude, Energy Efficient Construction Company provided design of 15-storey hotel building in Astana, Kazakhstan. The entire architectural design is developed by fulfilling all the state and governmental regulations (SNiP). Also, 3D models of the building created in AutoCAD and Sketch Up are provided to give visual representation.

Moreover, using ACI codes estimation of load induced on the structure and distribution of columns, dimensions of columns and slab thickness are computed. Since, in the base floor of the Utopia Hotel Astana 49 columns are arranged in 5 meters from each other, their approximate dimensions are estimated to be 550x550mm. As the columns of higher floors experience less load, the dimensions of column of floors range 2-5, 6-10 and 11-15 are 550x550 mm, 450x450mm and 350x350mm respectively. Regarding the final slab thickness, it was calculated to be 180 mm. Moreover, the beams were designed so that major ones are 250x500mm, while the minor beams are 188x313mm. These values may change as more detailed calculations will be performed further and analyzed via SAP2000. Besides, the ACI code provisions and procedure of design of beams, columns, slabs and reinforcement was provided for more accurate and safe estimation of the structure.

Furthermore, it was proved that the application of insulating materials is a good approach to make a building energy efficient due to their heat insulating features. For the hotel construction, the insulating materials will cover the exterior wall of the building. For energy analysis, glass wool, polyurethane rigid foam and vacuum insulation panels were selected which have appropriate thermal characteristics and are famous in the global market.

As a part of construction management the project cost estimation was completed taking into account relevant assumptions and rough calculations. According to obtained results, the total project cost for the hotel integrated with PUR is \$6319927 with the payback period of 7 years.

From the perspectives of market, economic and operational aspects, the whole project was evaluated as feasible.

Furthermore, one of the main parts of the project is to analyze performance of the structure in terms of energy efficiency. The main task was to reduce energy consumptions of the building to reduce environmental effect of the structure. The review of how this task should come true was described and various effects of insulation materials were discussed. According to the thermal characteristics three insulation materials (PUR, glass wool and vacuum panel) were chosen to analyze them in EnergyPlus and Design Builder software. For the preliminary design the typical building in Astana with typical climatic conditions was analyzed for energy performance for summer and winter seasons. As the result of the analysis, more than 7300 kWh for standard room was saved which computes \$ 536.218 per year. These results are illustration of positive effect of the insulation material on the building efficiency. The effectiveness of insulating materials was also proved with the application of Design Builder software which provided simulation outputs demonstrating that use insulating materials can greatly decrease the energy consumption of the building. As results indicate, with the application of PUR, the typical room can save about 49020.64 kW of energy (20% of energy consumption without insulating materials).

In conclusion, it should be mentioned that the architectural, structural and geotechnical designs were performed in accordance with related codes, insulating materials and its application in buildings were investigated and Energy analysis were performed with the use of two software programs. Moreover, learning about the software, such as EnergyPlus and SAP2000 were also showed. Furthermore, in this paper, the architectural part also included the site layout (landscaping, parking and traffic flow), 1st and typical floor design, choice of structural and non-structural materials, climate data of Astana city and the choice of insulating materials to be integrated. Moreover, energy performance of the hotel building was evaluated through simulations in EnergyPlus and Design Builder.

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10 APPENDICES

Appendix A

Table A1. Compliance with degree of fire resistance and fire resistance the structures of buildings, structures and fire compartments

The degree of fire resistance of buildings	The minimum limit of fire resistance of building structures, mines (above the line), and the maximum limits the spread of fire, see (below the line)								
	Walls				Columns	landings, stringers, steps, beams and staircases marches	boards, decking (including insulation) and other supporting structures	Covering elements	
	bearing and stairwells	self-supporting	exterior curtain (including the hinged panels)	interior non-load bearing (partition)				slabs, flooring (including insulation) and runs	beams, trusses, arches, frames
I	$\frac{150}{0}$	$\frac{90}{0}$	$\frac{30}{0}$	$\frac{30}{0}$	$\frac{150}{0}$	$\frac{60}{0}$	$\frac{60}{0}$	$\frac{30}{0}$	$\frac{30}{0}$
II	$\frac{120}{0}$	$\frac{60}{0}$	$\frac{15}{0}$	$\frac{15}{0}$	$\frac{120}{0}$	$\frac{60}{0}$	$\frac{45}{0}$	$\frac{15}{0}$	$\frac{15}{0}$
III	$\frac{120}{0}$	$\frac{60}{0}$	$\frac{15;}{30}$ $\frac{0}{40}$	$\frac{15}{40}$	$\frac{120}{0}$	$\frac{60}{0}$	$\frac{45}{25}$	-	
IIIa	$\frac{60}{0}$	$\frac{20}{0}$	$\frac{15}{40}$	$\frac{15}{40}$	$\frac{15}{0}$	$\frac{60}{0}$	$\frac{15}{0}$	$\frac{15}{25}$	$\frac{15}{0}$
IIIб	$\frac{60}{40}$	$\frac{30}{40}$	$\frac{15;}{30}$ $\frac{0}{40}$	$\frac{15}{40}$	$\frac{60}{40}$	$\frac{45}{0}$	$\frac{45}{25}$	$\frac{15;}{0}$ $\frac{30}{25}$	$\frac{45}{2}$
IV	$\frac{30}{40}$	$\frac{15}{40}$	$\frac{15}{40}$	$\frac{15}{40}$	$\frac{15}{40}$	$\frac{15}{25}$	$\frac{15}{25}$	-	
IVa	$\frac{15}{40}$	$\frac{15}{40}$	$\frac{15}{40}$ H. H	$\frac{15}{40}$	$\frac{15}{0}$	$\frac{15}{0}$	$\frac{15}{0}$	$\frac{15}{0}$	$\frac{15}{0}$
V	-								
Notes:									
- means that the indicator is not standardized.									



Figure A1. Site layout of a hotel from perspective view



Figure A2. Site layout of a hotel from frontal view



Figure A3. Site layout of a hotel from frontal view

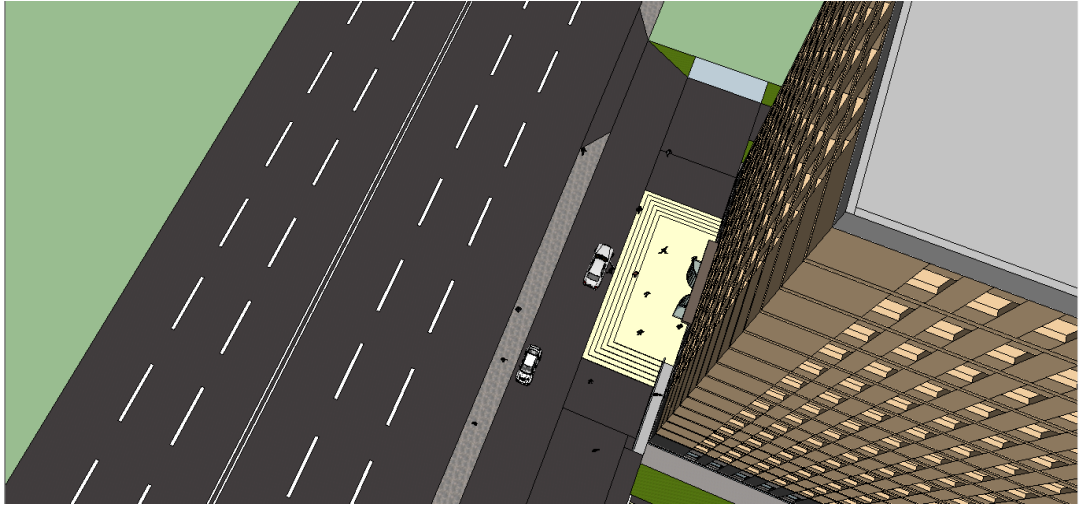


Figure A4. Site layout of a hotel from top view

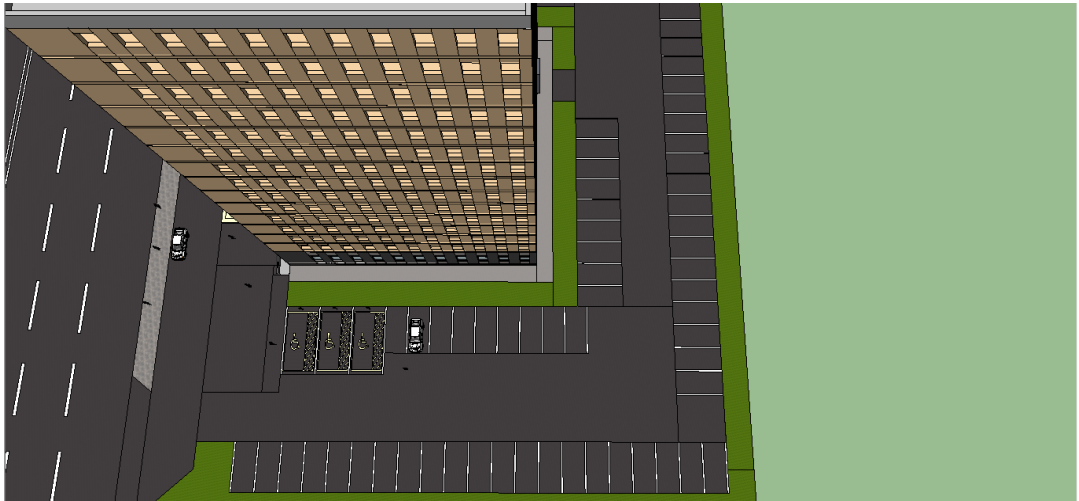


Figure A5. Site layout of a hotel from top view



Figure A6. Site layout of a hotel with surrounding facilities



Figure A7. Site layout of a hotel with surrounding facilities



Figure A8. Site layout of a hotel with main road



Figure A9. Site layout of a hotel with main road

Appendix B

Table B. 1

	Cross-sectional Area (mm ² /m)								
	Bar Diameter (mm)								
Bar Spacing(mm)	10	12	16	20	24	28	32	36	40
100	780	1130	2010	3140	4520	6160	8040	10200	12600
125	624	904	1608	2512	3616	4928	6432	8160	10080
150	520	753	1340	2093	3013	4107	5360	6800	8400
175	446	646	1149	1794	2583	3520	4594	5829	7200
200	390	565	1005	1570	2260	3080	4020	5100	6300
225	347	502	893	1396	2009	2738	3573	4533	5600
250	312	452	804	1256	1808	2464	3216	4080	5040
275	284	411	731	1142	1644	2240	2924	3709	4582
300	260	377	670	1047	1507	2053	2680	3400	4200

Cross Sectional area for different bar spacings

Table B. 2 Slab design at interior face of exterior support, at end span

Interior face of exterior support														
Level	Dead Load (kN/m ²)	Live Load (kN/m ²)	wu (kN/m ²)	f'c (MPa)	fy (MPa)	L,m	h slab (mm)	b (mm)	d (mm)	Column width (mm)	Clear span (mm)	Minimum reinfor Cement ratio	Bending moment	Minim required steel area, mm ² /m
1	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
2	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
3	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
4	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
5	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
6	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
7	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
8	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
9	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
10	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655

11	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
12	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
13	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
14	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
15	4,5	1	7	40	420	2,5	180	1000	155	350	2,325	0,00376	2,364961	40,83618

Table B. 3 Slab design at midspan, at end span.

Midspan, end span															
Level	Dead Load (kN/m ²)	Live Load (kN/m ²)	wu (kN/m ²)	f'c (MPa)	fy (MPa)	L,m	h slab (mm)	b (mm)	d (mm)	Column width (mm)	Clear span (mm)	Minimum reinfor Cement ratio	Bending moment	Minim required steel area, mm ² /m	
1	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,942191	119,872	
2	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,942191	119,872	
3	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,942191	119,872	
4	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,942191	119,872	

5	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,942191	119,872
6	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	7,257705	125,32
7	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	7,257705	125,32
8	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	7,257705	125,32
9	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	7,257705	125,32
10	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	7,257705	125,32
11	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	7,580231	130,8891
12	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	7,580231	130,8891
13	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	7,580231	130,8891
14	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	7,580231	130,8891
15	4,5	1	7	40	420	2,5	180	1000	155	350	2,325	0,00376	2,702813	46,66991

Table B. 4 Slab design at interior support, at end span

Interior support, end span															
Level	Dead Load (kN/m ²)	Live Load (kN/m ²)	wu (kN/m ²)	f'c (MPa)	fy (MPa)	L,m	h slab (mm)	b (mm)	d (mm)	Column width	Clear span (mm)	Minimum reinfor	Bending moment	Minim required	

										(mm)		Cement ratio		steel area, mm ² /m
1	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	9,719067	167,8208
2	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	9,719067	167,8208
3	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	9,719067	167,8208
4	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	9,719067	167,8208
5	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	9,719067	167,8208
6	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	10,16079	175,448
7	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	10,16079	175,448
8	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	10,16079	175,448
9	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	10,16079	175,448
10	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	10,16079	175,448
11	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	10,61232	183,2448
12	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	10,61232	183,2448
13	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	10,61232	183,2448
14	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	10,61232	183,2448

15	4,5	1	7	40	420	2,5	180	1000	155	350	2,325	0,00376	3,783938	65,33788
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Table B. 5 Slab design at interior face of first interior support, at interior span.

Interior face of first interior support, interior span														
Level	Dead Load (kN/m ²)	Live Load (kN/m ²)	wu (kN/m ²)	f _c (MPa)	f _y (MPa)	L, m	h slab (mm)	b (mm)	d (mm)	Column width (mm)	Clear span (mm)	Minimum reinfor Cement ratio	Bending moment	Minim required steel area, mm ² /m
1	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
2	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
3	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
4	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
5	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
6	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
7	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
8	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982

9	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
10	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
11	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
12	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
13	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
14	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
15	4,5	1	7	40	420	2,5	180	1000	155	350	2,325	0,00376	3,439943	59,39807

Table B. 6 Slab design at midspan, at interior span.

Midspan, interior span														
Level	Dead Load (kN/m ²)	Live Load (kN/m ²)	wu (kN/m ²)	f'c (MPa)	fy (MPa)	L,m	h slab (mm)	b (mm)	d (mm)	Column width (mm)	Clear span (mm)	Minimum reinfor Cement ratio	Bending moment	Minim required steel area, mm ² /m
1	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
2	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888

3	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
4	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
5	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	6,074417	104,888
6	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
7	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
8	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
9	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
10	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	6,350492	109,655
11	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
12	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
13	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
14	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	6,632702	114,528
15	4,5	1	7	40	420	2,5	180	1000	155	350	2,325	0,00376	2,364961	40,83618

Table B. 7 Slab design at interior face of interior support, at interior span.

Interior face of first interior support, interior span														
Level	Dead Load (kN/m ²)	Live Load (kN/m ²)	wu (kN/m ²)	f _c (MPa)	f _y (MPa)	L,m	h slab (mm)	b (mm)	d (mm)	Column width (mm)	Clear span (mm)	Minimum reinfor Cement ratio	Bending moment	Minim required steel area, mm ² /m
1	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
2	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
3	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
4	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
5	9,96	4,8	19,632	40	420	2,5	180	1000	155	550	2,225	0,00376	8,835515	152,5643
6	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
7	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
8	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
9	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982
10	9,96	4,8	19,632	40	420	2,5	180	1000	155	450	2,275	0,00376	9,237079	159,4982

11	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
12	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
13	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
14	9,96	4,8	19,632	40	420	2,5	180	1000	155	350	2,325	0,00376	9,647566	166,5861
15	4,5	1	7	40	420	2,5	180	1000	155	350	2,325	0,00376	3,439943	59,39807

Table B. 8 Beam reinforcement detailing

A

Member ID	Type	Location (mm)	Top Area Required (mm ²)	detail	Area Provided (mm ²)	error %	Bot Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Beam 500x250	0	2882.75	8@22	3096	7.40	1381.29	4@22	1548	12.07
1	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
1	Beam 500x250	5000	2744.86	8@22	3096	12.79	1318.16	4@22	1548	17.44
2	Beam	0	3317.67	8@22	3096	6.68	1578.29	4@22	1548	1.92

	500x250									
2	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
2	Beam 500x250	5000	3004.47	8@22	3096	3.05	1522.66	4@22	1548	1.66
3	Beam 500x250	0	3369.03	6@25	3060	9.17	1601.34	4@22	1548	3.33
3	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
3	Beam 500x250	5000	2903.05	8@22	3096	6.65	1544.31	4@22	1548	0.24
4	Beam 500x250	0	3313.66	6@25	3060	7.65	1576.49	4@22	1548	1.81
4	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
4	Beam 500x250	5000	2721.23	8@22	3096	13.77	1476.00	4@22	1548	4.88
5	Beam 500x250	0	3213.67	6@25	3060	4.78	1531.48	4@22	1548	1.08

5	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
5	Beam 500x250	5000	2513.25	5@25	2550	1.46	1372.01	4@22	1548	12.83
6	Beam 500x250	0	3091.15	8@22	3096	0.16	1476.09	4@22	1548	4.87
6	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
6	Beam 500x250	5000	2298.24	5@25	2550	10.95	1250.41	4@22	1548	23.80
7	Beam 500x250	0	2951.17	8@22	3096	4.91	1412.50	4@22	1548	9.59
7	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
7	Beam 500x250	5000	2080.39	4@25	2040	1.94	1115.59	3@22	1161	4.07
8	Beam 500x250	0	2795.33	8@22	3096	10.76	1341.31	4@22	1548	15.41
8	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01

8	Beam 500x250	5000	1860.47	4@25	2040	9.65	968.75	4@19	1136	17.26
9	Beam 500x250	0	2624.42	5@25	2550	2.84	1262.76	4@22	1548	22.59
9	Beam 500x250	2500	827.14	3@19	852	3.01	844.35	3@19	852	0.91
9	Beam 500x250	5000	1638.48	4@22	1548	5.52	852.06	3@19	852	0.01
10	Beam 500x250	0	2438.95	5@25	2550	4.55	1176.99	3@22	1161	1.36
10	Beam 500x250	2500	771.91	3@19	852	10.37	844.28	3@19	852	0.91
10	Beam 500x250	5000	1414.31	4@22	1548	9.45	852.06	3@19	852	0.01
11	Beam 500x250	0	2239.54	4@25	2040	8.91	1084.15	3@22	1161	7.09
11	Beam 500x250	2500	711.98	3@19	852	19.67	844.24	3@19	852	0.92
11	Beam 500x250	5000	1188.01	3@22	1161	2.27	779.02	3@19	852	9.37

12	Beam 500x250	0	2028.94	4@25	2040	0.55	985.40	4@19	1136	15.28
12	Beam 500x250	2500	648.04	3@19	852	31.47	844.05	3@19	852	0.94
12	Beam 500x250	5000	959.60	4@19	1136	18.38	631.31	3@16	597	5.44
13	Beam 500x250	0	1813.88	4@25	2040	12.47	883.84	3@19	852	3.60
13	Beam 500x250	2500	582.10	3@16	597	2.56	844.46	3@19	852	0.89
13	Beam 500x250	5000	852.06	3@19	852	0.01	479.67	2@16	398	17.03
14	Beam 500x250	0	1580.74	4@22	1548	2.07	852.06	3@19	852	0.01
14	Beam 500x250	2500	509.86	3@16	597	17.09	838.61	3@19	852	1.60
14	Beam 500x250	5000	691.12	3@19	852	23.28	343.13	2@16	398	15.99
15	Beam 500x250	0	1297.22	3@22	1161	10.50	849.28	3@19	852	0.32

15	Beam 500x250	2500	420.96	2@16	398	5.45	852.06	3@19	852	0.01
15	Beam 500x250	5000	790.70	3@19	852	7.75	392.16	2@16	398	1.49

B

Member ID	Type	Location (mm)	Top Area Required (mm ²)	detail	Area Provided (mm ²)	error %	Bot Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Beam 500x250	0	2761.21	8@22	3096	12.12	1325.67	4@22	1548	16.77
1	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
1	Beam 500x250	5000	2738.60	8@22	3096	13.05	1315.29	4@22	1548	17.69
2	Beam 500x250	0	3145.63	8@22	3096	1.58	1500.75	4@22	1548	3.15
2	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01

2	Beam 500x250	5000	3121.05	8@22	3096	0.80	1489.63	4@22	1548	3.92
3	Beam 500x250	0	3163.03	8@22	3096	2.12	1508.61	4@22	1548	2.61
3	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
3	Beam 500x250	5000	3118.51	8@22	3096	0.72	1488.48	4@22	1548	4.00
4	Beam 500x250	0	3080.68	8@22	3096	0.50	1471.35	4@22	1548	5.21
4	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
4	Beam 500x250	5000	3021.17	8@22	3096	2.48	1444.34	4@22	1548	7.18
5	Beam 500x250	0	2960.40	8@22	3096	4.58	1416.70	4@22	1548	9.27
5	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
5	Beam 500x250	5000	2887.15	8@22	3096	7.23	1383.30	5@19	1420	2.65

6	Beam 500x250	0	2820.33	8@22	3096	9.77	1352.76	5@19	1420	4.97
6	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
6	Beam 500x250	5000	2734.28	8@22	3096	13.23	1313.30	5@19	1420	8.12
7	Beam 500x250	0	2665.66	5@25	2550	4.34	1281.76	5@19	1420	10.78
7	Beam 500x250	2500	839.35	3@19	852	1.51	839.35	3@19	852	1.51
7	Beam 500x250	5000	2567.93	5@25	2550	0.70	1236.70	3@22	1161	6.12
8	Beam 500x250	0	2498.11	5@25	2550	2.08	1204.41	3@22	1161	3.60
8	Beam 500x250	2500	789.59	3@19	852	7.90	833.61	3@19	852	2.21
8	Beam 500x250	5000	2390.03	5@25	2550	6.69	1154.27	3@22	1161	0.58
9	Beam 500x250	0	2318.65	5@25	2550	9.98	1121.06	3@22	1161	3.56

9	Beam 500x250	2500	735.83	3@19	852	15.79	834.31	3@19	852	2.12
9	Beam 500x250	5000	2201.68	4@25	2040	7.34	1066.44	3@22	1161	8.87
10	Beam 500x250	0	2128.06	4@25	2040	4.14	1031.96	3@22	1161	12.50
10	Beam 500x250	2500	678.22	3@16	597	11.97	834.92	3@19	852	2.05
10	Beam 500x250	5000	2003.77	4@25	2040	1.81	973.55	4@19	1136	16.69
11	Beam 500x250	0	1927.26	4@25	2040	5.85	937.47	4@19	1136	21.18
11	Beam 500x250	2500	616.95	3@16	597	3.23	835.41	3@19	852	1.99
11	Beam 500x250	5000	1797.33	6@19	1704	5.19	876.00	3@19	852	2.74
12	Beam 500x250	0	1718.49	6@19	1704	0.84	852.06	3@19	852	0.01
12	Beam 500x250	2500	552.63	3@16	597	8.03	835.81	3@19	852	1.94

12	Beam 500x250	5000	1583.48	4@22	1548	2.24	852.06	3@19	852	0.01
13	Beam 500x250	0	1509.36	4@22	1548	2.56	852.06	3@19	852	0.01
13	Beam 500x250	2500	487.58	2@19	568	16.49	835.47	3@19	852	1.98
13	Beam 500x250	5000	1361.60	5@19	1420	4.29	852.06	3@19	852	0.01
14	Beam 500x250	0	1280.16	5@19	1420	10.92	838.32	3@19	852	1.63
14	Beam 500x250	2500	415.58	2@16	398	4.23	837.20	3@19	852	1.77
14	Beam 500x250	5000	1174.19	3@22	1161	1.12	770.11	3@19	852	10.63
15	Beam 500x250	0	1071.17	4@19	1136	6.05	703.59	2@22	774	10.01
15	Beam 500x250	2500	349.28	2@16	398	13.95	850.99	3@19	852	0.12
15	Beam 500x250	5000	1059.12	4@19	1136	7.26	695.79	2@22	774	11.24

C

Member ID	Type	Location (mm)	Top Area Required (mm ²)	detail	Area Provided (mm ²)	error %	Bot Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Beam 500x250	0	1321.170114	5@19	1420	7.48	2751.41	5@25	2550	7.32
1	Beam 500x250	2500	852.0563362	3@19	852	0.01	852.06	3@19	852	0.01
1	Beam 500x250	5000	1318.868501	5@19	1420	7.67	2746.40	5@25	2550	7.15
2	Beam 500x250	0	1498.811826	5@19	1420	5.26	3141.34	8@22	3096	1.44
2	Beam 500x250	2500	852.0563362	3@19	852	0.01	852.06	3@19	852	0.01
2	Beam 500x250	5000	1498.099139	5@19	1420	5.21	3139.77	8@22	3096	1.39
3	Beam 500x250	0	1504.103183	4@22	1548	2.92	3153.05	8@22	3096	1.81
3	Beam 500x250	2500	852.0563362	3@19	852	0.01	852.06	3@19	852	0.01

3	Beam 500x250	5000	1502.117277	4@22	1548	3.05	3148.65	8@22	3096	1.67
4	Beam 500x250	0	1464.205238	5@19	1420	3.02	3064.94	8@22	3096	1.01
4	Beam 500x250	2500	852.0563362	3@19	852	0.01	852.06	3@19	852	0.01
4	Beam 500x250	5000	1461.31403	5@19	1420	2.83	3058.56	8@22	3096	1.22
5	Beam 500x250	0	1407.763387	5@19	1420	0.87	2940.78	8@22	3096	5.28
5	Beam 500x250	2500	852.0563362	3@19	852	0.01	852.06	3@19	852	0.01
5	Beam 500x250	5000	1404.263712	5@19	1420	1.12	2933.10	8@22	3096	5.55
6	Beam 500x250	0	1342.2214	5@19	1420	5.79	2797.33	8@22	3096	10.68
6	Beam 500x250	2500	852.0563362	3@19	852	0.01	852.06	3@19	852	0.01
6	Beam 500x250	5000	1338.152985	5@19	1420	6.12	2788.45	8@22	3096	11.03

7	Beam 500x250	0	1269.773116	3@22	1161	8.57	2639.63	5@25	2550	3.40
7	Beam 500x250	2500	831.6448865	3@19	852	2.45	831.64	3@19	852	2.45
7	Beam 500x250	5000	1265.173463	3@22	1161	8.23	2629.64	5@25	2550	3.03
8	Beam 500x250	0	1191.143958	3@22	1161	2.53	2469.47	5@25	2550	3.26
8	Beam 500x250	2500	825.7440463	3@19	852	3.18	781.04	2@22	774	0.90
8	Beam 500x250	5000	1186.063839	3@22	1161	2.11	2458.51	5@25	2550	3.72
9	Beam 500x250	0	1106.696239	3@22	1161	4.91	2287.85	4@25	2040	10.83
9	Beam 500x250	2500	825.8670243	3@19	852	3.16	726.55	2@22	774	6.53
9	Beam 500x250	5000	1101.194824	3@22	1161	5.43	2276.06	4@25	2040	10.37
10	Beam 500x250	0	1016.704948	3@22	1161	14.19	2095.55	4@25	2040	2.65

10	Beam 500x250	2500	825.9771041	3@19	852	3.15	668.33	3@16	597	10.67
10	Beam 500x250	5000	1010.853227	3@22	1161	14.85	2083.09	4@25	2040	2.07
11	Beam 500x250	0	921.4927905	3@19	852	7.54	1893.43	6@19	1704	10.00
11	Beam 500x250	2500	826.0824871	3@19	852	3.14	606.57	3@16	597	1.58
11	Beam 500x250	5000	915.3886478	3@19	852	6.92	1880.52	6@19	1704	9.39
12	Beam 500x250	0	852.0563362	3@19	852	0.01	1683.37	6@19	1704	1.23
12	Beam 500x250	2500	826.0796504	3@19	852	3.14	541.75	3@16	597	10.20
12	Beam 500x250	5000	852.0563362	3@19	852	0.01	1669.60	6@19	1704	2.06
13	Beam 500x250	0	852.0563362	3@19	852	0.01	1470.66	5@19	1420	3.44
13	Beam 500x250	2500	825.4923364	3@19	852	3.21	475.48	2@16	398	16.29

13	Beam 500x250	5000	852.0563362	3@19	852	0.01	1453.55	5@19	1420	2.31
14	Beam 500x250	0	824.122043	3@19	852	3.38	1258.07	3@22	1161	7.72
14	Beam 500x250	2500	828.5973249	3@19	852	2.82	408.60	2@16	398	2.59
14	Beam 500x250	5000	824.0819529	3@19	852	3.39	1258.01	3@22	1161	7.71
15	Beam 500x250	0	711.6453085	2@22	774	8.76	1083.63	3@22	1161	7.14
15	Beam 500x250	2500	831.7476419	3@19	852	2.43	358.40	2@16	398	11.05
15	Beam 500x250	5000	722.1102609	2@22	774	7.19	1099.82	3@22	1161	5.56

D

Member ID	Type	Location (mm)	Top Area Required (mm ²)	detail	Area Provided (mm ²)	error %	Bot Area Required (mm ²)	detail	Area Provided (mm ²)	error %
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1	Beam 500x250	0	2742.76	5@25	2550	7.03	1317.20	5@19	1420	7.80
1	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
1	Beam 500x250	5000	2743.98	5@25	2550	7.07	1317.76	5@19	1420	7.76
2	Beam 500x250	0	3140.28	8@22	3096	1.41	1498.33	5@19	1420	5.23
2	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
2	Beam 500x250	5000	3142.38	8@22	3096	1.48	1499.28	5@19	1420	5.29
3	Beam 500x250	0	3149.04	8@22	3096	1.68	1502.29	4@22	1548	3.04
3	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
3	Beam 500x250	5000	3153.82	8@22	3096	1.83	1504.45	4@22	1548	2.89
4	Beam 500x250	0	3058.09	8@22	3096	1.24	1461.10	5@19	1420	2.81

4	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
4	Beam 500x250	5000	3063.97	8@22	3096	1.05	1463.77	5@19	1420	2.99
5	Beam 500x250	0	2932.58	8@22	3096	5.57	1404.02	5@19	1420	1.14
5	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
5	Beam 500x250	5000	2939.71	8@22	3096	5.32	1407.27	5@19	1420	0.90
6	Beam 500x250	0	2787.98	8@22	3096	11.05	1337.94	5@19	1420	6.13
6	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
6	Beam 500x250	5000	2796.37	8@22	3096	10.72	1341.78	5@19	1420	5.83
7	Beam 500x250	0	2629.20	5@25	2550	3.01	1264.97	3@22	1161	8.22
7	Beam 500x250	2500	831.37	3@19	852	2.48	831.37	3@19	852	2.48

7	Beam 500x250	5000	2638.71	5@25	2550	3.36	1269.35	3@22	1161	8.54
8	Beam 500x250	0	2458.08	5@25	2550	3.74	1185.86	3@22	1161	2.10
8	Beam 500x250	2500	780.77	2@22	774	0.87	825.74	3@19	852	3.18
8	Beam 500x250	5000	2468.58	5@25	2550	3.30	1190.73	3@22	1161	2.50
9	Beam 500x250	0	2275.63	4@25	2040	10.35	1101.00	3@22	1161	5.45
9	Beam 500x250	2500	726.29	2@22	774	6.57	825.87	3@19	852	3.16
9	Beam 500x250	5000	2286.98	4@25	2040	10.80	1106.29	3@22	1161	4.95
10	Beam 500x250	0	2082.67	4@25	2040	2.05	1010.66	3@22	1161	14.88
10	Beam 500x250	2500	668.08	2@22	774	15.85	825.98	3@19	852	3.15
10	Beam 500x250	5000	2094.70	4@25	2040	2.61	1016.31	3@22	1161	14.24

11	Beam 500x250	0	1880.11	6@19	1704	9.37	915.19	3@19	852	6.90
11	Beam 500x250	2500	606.31	3@16	597	1.54	826.08	3@19	852	3.14
11	Beam 500x250	5000	1892.58	4@25	2040	7.79	921.09	3@19	852	7.50
12	Beam 500x250	0	1669.10	6@19	1704	2.09	852.06	3@19	852	0.01
12	Beam 500x250	2500	541.43	3@16	597	10.26	826.08	3@19	852	3.14
12	Beam 500x250	5000	1682.34	6@19	1704	1.29	852.06	3@19	852	0.01
13	Beam 500x250	0	1453.30	5@19	1420	2.29	852.06	3@19	852	0.01
13	Beam 500x250	2500	475.32	2@16	398	16.27	825.49	3@19	852	3.21
13	Beam 500x250	5000	1470.16	4@22	1548	5.29	852.06	3@19	852	0.01
14	Beam 500x250	0	1259.83	3@22	1161	7.84	825.25	3@19	852	3.24

14	Beam 500x250	2500	409.77	2@16	398	2.87	828.60	3@19	852	2.82
14	Beam 500x250	5000	1261.78	3@22	1161	7.99	826.51	3@19	852	3.08
15	Beam 500x250	0	1102.35	3@22	1161	5.32	723.75	2@22	774	6.94
15	Beam 500x250	2500	359.21	2@16	398	10.80	831.75	3@19	852	2.43
15	Beam 500x250	5000	1088.79	3@22	1161	6.63	714.98	2@22	774	8.25

E

Member ID	Type	Location (mm)	Top Area Required (mm ²)	detail	Area Provided (mm ²)	error %	Bot Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Beam 500x250	0	2723.97	5@25	2550	6.39	1308.57	5@19	1420	8.52
1	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01

1	Beam 500x250	5000	2742.85	5@25	2550	7.03	1317.24	5@19	1420	7.80
2	Beam 500x250	0	3123.28	8@22	3096	0.87	1490.64	5@19	1420	4.74
2	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
2	Beam 500x250	5000	3148.59	8@22	3096	1.67	1502.09	4@22	1548	3.06
3	Beam 500x250	0	3120.00	8@22	3096	0.77	1489.16	5@19	1420	4.64
3	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
3	Beam 500x250	5000	3164.90	8@22	3096	2.18	1509.46	4@22	1548	2.55
4	Beam 500x250	0	3019.23	8@22	3096	2.54	1443.46	5@19	1420	1.62
4	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
4	Beam 500x250	5000	3078.21	8@22	3096	0.58	1470.22	5@19	1420	3.42

5	Beam 500x250	0	2885.03	8@22	3096	7.31	1382.33	5@19	1420	2.72
5	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
5	Beam 500x250	5000	2957.73	8@22	3096	4.67	1415.48	5@19	1420	0.32
6	Beam 500x250	0	2732.38	8@22	3096	13.31	1312.43	5@19	1420	8.20
6	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
6	Beam 500x250	5000	2817.94	8@22	3096	9.87	1351.66	5@19	1420	5.06
7	Beam 500x250	0	2566.11	5@25	2550	0.63	1235.86	5@19	1420	14.90
7	Beam 500x250	2500	838.67	3@19	852	1.59	838.67	3@19	852	1.59
7	Beam 500x250	5000	2663.37	5@25	2550	4.26	1280.71	3@22	1161	9.35
8	Beam 500x250	0	2388.27	5@25	2550	6.77	1153.45	3@22	1161	0.65

8	Beam 500x250	2500	788.92	2@22	774	1.89	833.61	3@19	852	2.21
8	Beam 500x250	5000	2495.88	5@25	2550	2.17	1203.38	3@22	1161	3.52
9	Beam 500x250	0	2199.96	4@25	2040	7.27	1065.64	3@22	1161	8.95
9	Beam 500x250	2500	735.17	2@22	774	5.28	834.31	3@19	852	2.12
9	Beam 500x250	5000	2316.49	5@25	2550	10.08	1120.05	3@22	1161	3.66
10	Beam 500x250	0	2002.11	4@25	2040	1.89	972.77	3@22	1161	19.35
10	Beam 500x250	2500	677.58	3@16	597	11.89	834.92	3@19	852	2.05
10	Beam 500x250	5000	2125.96	4@25	2040	4.04	1030.98	3@22	1161	12.61
11	Beam 500x250	0	1795.66	6@19	1704	5.10	875.20	3@19	852	2.65
11	Beam 500x250	2500	616.30	3@16	597	3.13	835.41	3@19	852	1.99

11	Beam 500x250	5000	1925.15	4@25	2040	5.97	936.48	3@22	1161	23.98
12	Beam 500x250	0	1581.49	4@22	1548	2.12	852.06	3@19	852	0.01
12	Beam 500x250	2500	551.86	3@16	597	8.18	835.81	3@19	852	1.94
12	Beam 500x250	5000	1715.99	6@19	1704	0.70	852.06	3@19	852	0.01
13	Beam 500x250	0	1360.55	5@19	1420	4.37	852.06	3@19	852	0.01
13	Beam 500x250	2500	487.15	3@16	597	22.55	835.47	3@19	852	1.98
13	Beam 500x250	5000	1507.98	5@19	1420	5.83	852.06	3@19	852	0.01
14	Beam 500x250	0	1181.60	3@22	1161	1.74	774.89	2@22	774	0.11
14	Beam 500x250	2500	418.56	2@16	398	4.91	837.20	3@19	852	1.77
14	Beam 500x250	5000	1289.61	5@19	1420	10.11	844.39	3@19	852	0.90

15	Beam 500x250	0	1069.39	3@22	1161	8.57	702.44	2@22	774	10.19
15	Beam 500x250	2500	353.41	2@16	398	12.62	850.99	3@19	852	0.12
15	Beam 500x250	5000	1084.13	3@22	1161	7.09	711.97	2@22	774	8.71

F

Member ID	Type	Location (mm)	Top Area Required (mm ²)	detail	Area Provided (mm ²)	error %	Bot Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Beam 500x250	0	2717.04	5@25	2550	6.15	1305.39	5@19	1420	8.78
1	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
1	Beam 500x250	5000	2848.37	8@22	3096	8.69	1365.58	5@19	1420	3.99
2	Beam 500x250	0	3010.51	8@22	3096	2.84	1528.18	4@22	1548	1.30

2	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
2	Beam 500x250	5000	3326.39	8@22	3096	6.93	1582.20	4@22	1548	2.16
3	Beam 500x250	0	2905.45	8@22	3096	6.56	1546.52	4@22	1548	0.10
3	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
3	Beam 500x250	5000	3371.72	8@22	3096	8.18	1602.54	4@22	1548	3.40
4	Beam 500x250	0	2717.55	5@25	2550	6.17	1472.58	5@19	1420	3.57
4	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
4	Beam 500x250	5000	3308.97	8@22	3096	6.44	1574.38	4@22	1548	1.68
5	Beam 500x250	0	2509.48	5@25	2550	1.61	1368.50	5@19	1420	3.76
5	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01

5	Beam 500x250	5000	3209.00	8@22	3096	3.52	1529.37	4@22	1548	1.22
6	Beam 500x250	0	2294.81	4@25	2040	11.10	1247.19	5@19	1420	13.86
6	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
6	Beam 500x250	5000	3086.86	8@22	3096	0.30	1474.14	5@19	1420	3.67
7	Beam 500x250	0	2077.07	4@25	2040	1.78	1112.45	3@22	1161	4.36
7	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
7	Beam 500x250	5000	2947.00	8@22	3096	5.06	1410.60	5@19	1420	0.67
8	Beam 500x250	0	1857.18	4@25	2040	9.84	965.63	3@19	852	11.77
8	Beam 500x250	2500	852.06	3@19	852	0.01	852.06	3@19	852	0.01
8	Beam 500x250	5000	2791.21	8@22	3096	10.92	1339.42	5@19	1420	6.02

9	Beam 500x250	0	1635.20	4@22	1548	5.33	852.06	3@19	852	0.01
9	Beam 500x250	2500	825.92	3@19	852	3.16	844.35	3@19	852	0.91
9	Beam 500x250	5000	2620.31	5@25	2550	2.68	1260.87	5@19	1420	12.62
10	Beam 500x250	0	1411.04	5@19	1420	0.63	852.06	3@19	852	0.01
10	Beam 500x250	2500	770.69	2@22	774	0.43	844.28	3@19	852	0.91
10	Beam 500x250	5000	2434.86	5@25	2550	4.73	1175.09	3@22	1161	1.20
11	Beam 500x250	0	1184.65	3@22	1161	2.00	776.85	2@22	774	0.37
11	Beam 500x250	2500	710.71	2@22	774	8.90	844.24	3@19	852	0.92
11	Beam 500x250	5000	2235.36	4@25	2040	8.74	1082.19	3@22	1161	7.28
12	Beam 500x250	0	955.68	3@22	1161	21.48	628.77	3@16	597	5.05

12	Beam 500x250	2500	646.58	2@22	774	19.71	844.05	3@19	852	0.94
12	Beam 500x250	5000	2024.13	4@25	2040	0.78	983.14	3@19	852	13.34
13	Beam 500x250	0	852.06	3@19	852	0.01	477.63	3@16	597	24.99
13	Beam 500x250	2500	580.79	3@16	597	2.79	844.46	3@19	852	0.89
13	Beam 500x250	5000	1809.63	6@19	1704	5.84	881.83	3@19	852	3.38
14	Beam 500x250	0	707.66	2@22	774	9.37	351.28	2@16	398	13.30
14	Beam 500x250	2500	514.76	3@16	597	15.98	838.61	3@19	852	1.60
14	Beam 500x250	5000	1596.50	4@22	1548	3.04	852.06	3@19	852	0.01
15	Beam 500x250	0	790.70	2@22	774	2.11	392.16	2@16	398	1.49
15	Beam 500x250	2500	420.96	2@16	398	5.45	852.06	3@19	852	0.01

15	Beam 500x250	5000	1297.22	<u>5@19</u>	1420	9.47	849.28	<u>3@19</u>	852	0.32
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Table B. 9 Column reinforcement detailing

Columns A

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Column 550x550	0	4916.572	8@29	5160	4.95
1	Column 550x550	2575	4225	8@25	4080	3.43
1	Column 550x550	5150	4225	8@25	4080	3.43
2	Column 550x550	0	4225	8@25	4080	3.43
2	Column 550x550	1650	4225	8@25	4080	3.43
2	Column 550x550	3300	4225	8@25	4080	3.43
3	Column 550x550	0	3600	8@25	4080	13.33
3	Column 550x550	1650	3600	8@25	4080	13.33
3	Column 550x550	3300	3600	8@25	4080	13.33
4	Column 550x550	0	3600	8@25	4080	13.33
4	Column 550x550	1650	3600	8@25	4080	13.33
4	Column 550x550	3300	3600	8@25	4080	13.33
5	Column 500x500	0	3600	8@25	4080	13.33
5	Column 500x500	1650	3600	8@25	4080	13.33

5	Column 500x500	3300	3600	8@25	4080	13.33
6	Column 450x450	0	3025	8@22	3096	2.35
6	Column 450x450	1650	3025	8@22	3096	2.35
6	Column 450x450	3300	3025	8@22	3096	2.35
7	Column 450x450	0	3025	8@22	3096	2.35
7	Column 450x450	1650	3025	8@22	3096	2.35
7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	3025	8@22	3096	2.35
10	Column 450x450	1650	3025	8@22	3096	2.35
10	Column 450x450	3300	3025	8@22	3096	2.35

11	Column 350x350	0	2500	8@22	3096	23.84
11	Column 350x350	1650	2500	8@22	3096	23.84
11	Column 350x350	3300	2500	8@22	3096	23.84
12	Column 350x350	0	2500	8@22	3096	23.84
12	Column 350x350	1650	2500	8@22	3096	23.84
12	Column 350x350	3300	2500	8@22	3096	23.84
13	Column 350x350	0	2500	8@22	3096	23.84
13	Column 350x350	1650	2500	8@22	3096	23.84
13	Column 350x350	3300	2500	8@22	3096	23.84
14	Column 350x350	0	2500	8@22	3096	23.84
14	Column 350x350	1650	2500	8@22	3096	23.84
14	Column 350x350	3300	2500	8@22	3096	23.84
15	Column 350x350	0	3600	8@25	4080	13.33
15	Column 350x350	1650	3600	8@25	4080	13.33
15	Column 350x350	3300	3600	8@25	4080	13.33

Columns A-B

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Column 550x550	0	5475.17	4@43	5808	6.08
1	Column 550x550	2575	5527.63	4@43	5808	5.07
1	Column 550x550	5150	5579.78	4@43	5808	4.09
2	Column 550x550	0	4273.69	8@25	4080	4.53
2	Column 550x550	1650	4306.48	8@25	4080	5.26
2	Column 550x550	3300	4339.08	8@25	4080	5.97
3	Column 550x550	0	4225	8@25	4080	3.43
3	Column 550x550	1650	4225	8@25	4080	3.43
3	Column 550x550	3300	4225	8@25	4080	3.43
4	Column 550x550	0	3600	8@25	4080	13.33
4	Column 550x550	1650	3600	8@25	4080	13.33
4	Column 550x550	3300	3600	8@25	4080	13.33
5	Column 500x500	0	3600	8@25	4080	13.33
5	Column 500x500	1650	3600	8@25	4080	13.33
5	Column 500x500	3300	3600	8@25	4080	13.33

6	Column 450x450	0	3025	8@22	3096	2.35
6	Column 450x450	1650	3025	8@22	3096	2.35
6	Column 450x450	3300	3025	8@22	3096	2.35
7	Column 450x450	0	3025	8@22	3096	2.35
7	Column 450x450	1650	3025	8@22	3096	2.35
7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	3002.45	8@22	3096	3.12
10	Column 450x450	1650	3000.12	8@22	3096	3.20
10	Column 450x450	3300	2997.79	8@22	3096	3.28
11	Column 350x350	0	2620	8@22	3096	18.17

11	Column 350x350	1650	2620	8@22	3096	18.17
11	Column 350x350	3300	2620	8@22	3096	18.17
12	Column 350x350	0	2500	8@22	3096	23.84
12	Column 350x350	1650	2500	8@22	3096	23.84
12	Column 350x350	3300	2500	8@22	3096	23.84
13	Column 350x350	0	2500	8@22	3096	23.84
13	Column 350x350	1650	2500	8@22	3096	23.84
13	Column 350x350	3300	2500	8@22	3096	23.84
14	Column 350x350	0	2500	8@22	3096	23.84
14	Column 350x350	1650	2500	8@22	3096	23.84
14	Column 350x350	3300	2500	8@22	3096	23.84
15	Column 350x350	0	3600	8@25	4080	13.33
15	Column 350x350	1650	3600	8@25	4080	13.33
15	Column 350x350	3300	3600	8@25	4080	13.33

Columns B-C

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
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1	Column 550x550	0	5550.19	4@43	5808	4.65
1	Column 550x550	2575	5602.22	4@43	5808	3.67
1	Column 550x550	5150	5653.96	4@43	5808	2.72
2	Column 550x550	0	4373.69	8@25	4080	6.71
2	Column 550x550	1650	4306.48	8@25	4080	5.26
2	Column 550x550	3300	4339.08	8@25	4080	5.97
3	Column 550x550	0	4306.48	8@25	4080	5.26
3	Column 550x550	1650	4339.08	8@25	4080	5.97
3	Column 550x550	3300	4225	8@25	4080	3.43
4	Column 550x550	0	3878.23	8@25	4080	5.20
4	Column 550x550	1650	3681.12	8@25	4080	10.84
4	Column 550x550	3300	3412.53	8@25	4080	19.56
5	Column 500x500	0	3600	8@25	4080	13.33
5	Column 500x500	1650	3600	8@25	4080	13.33
5	Column 500x500	3300	3600	8@25	4080	13.33
6	Column 450x450	0	3025	8@22	3096	2.35

6	Column 450x450	1650	3025	8@22	3096	2.35
6	Column 450x450	3300	3025	8@22	3096	2.35
7	Column 450x450	0	3025	8@22	3096	2.35
7	Column 450x450	1650	3025	8@22	3096	2.35
7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	3002.45	8@22	3096	3.12
10	Column 450x450	1650	3000.12	8@22	3096	3.20
10	Column 450x450	3300	2997.79	8@22	3096	3.28
11	Column 350x350	0	2620	8@22	3096	18.17
11	Column 350x350	1650	2620	8@22	3096	18.17

11	Column 350x350	3300	2620	<u>8@22</u>	3096	18.17
12	Column 350x350	0	2601.22	<u>8@22</u>	3096	19.02
12	Column 350x350	1650	2601.22	<u>8@22</u>	3096	19.02
12	Column 350x350	3300	2567.48	<u>8@22</u>	3096	20.59
13	Column 350x350	0	2500	<u>8@22</u>	3096	23.84
13	Column 350x350	1650	2500	<u>8@22</u>	3096	23.84
13	Column 350x350	3300	2500	<u>8@22</u>	3096	23.84
14	Column 350x350	0	2500	<u>8@22</u>	3096	23.84
14	Column 350x350	1650	2500	<u>8@22</u>	3096	23.84
14	Column 350x350	3300	2500	<u>8@22</u>	3096	23.84
15	Column 350x350	0	3600	<u>8@25</u>	4080	13.33
15	Column 350x350	1650	3600	<u>8@25</u>	4080	13.33
15	Column 350x350	3300	3600	<u>8@25</u>	4080	13.33

Columns C-D

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Column 550x550	0	6130.98	<u>12@25</u>	6120	0.18

1	Column 550x550	2575	6005.12	12@25	6120	1.91
1	Column 550x550	5150	5989.61	12@25	6120	2.18
2	Column 550x550	0	4811.06	8@29	5160	7.25
2	Column 550x550	1650	4737.13	8@29	5160	8.93
2	Column 550x550	3300	4772.98	8@29	5160	8.11
3	Column 550x550	0	4737.13	8@29	5160	8.93
3	Column 550x550	1650	4772.98	8@29	5160	8.11
3	Column 550x550	3300	4647.50	8@29	5160	11.03
4	Column 550x550	0	4266.05	8@25	4080	4.36
4	Column 550x550	1650	4049.23	8@25	4080	0.76
4	Column 550x550	3300	3753.78	8@25	4080	8.69
5	Column 500x500	0	3600	8@25	4080	13.33
5	Column 500x500	1650	3600	8@25	4080	13.33
5	Column 500x500	3300	3600	8@25	4080	13.33
6	Column 450x450	0	3025	8@22	3096	2.35
6	Column 450x450	1650	3025	8@22	3096	2.35

6	Column 450x450	3300	3025	8@22	3096	2.35
7	Column 450x450	0	3025	8@22	3096	2.35
7	Column 450x450	1650	3025	8@22	3096	2.35
7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	3602.94	8@25	4080	13.24
10	Column 450x450	1650	3600.144	8@25	4080	13.33
10	Column 450x450	3300	3597.348	8@25	4080	13.42
11	Column 350x350	0	3144	8@22	3096	1.53
11	Column 350x350	1650	3144	8@22	3096	1.53
11	Column 350x350	3300	3144	8@22	3096	1.53

12	Column 350x350	0	3121.464	8@22	3096	0.82
12	Column 350x350	1650	3121.464	8@22	3096	0.82
12	Column 350x350	3300	3080.976	8@22	3096	0.49
13	Column 350x350	0	2620	8@22	3096	18.17
13	Column 350x350	1650	2620	8@22	3096	18.17
13	Column 350x350	3300	2620	8@22	3096	18.17
14	Column 350x350	0	2601.22	8@22	3096	19.02
14	Column 350x350	1650	2601.22	8@22	3096	19.02
14	Column 350x350	3300	2567.48	8@22	3096	20.59
15	Column 350x350	0	3600	8@25	4080	13.33
15	Column 350x350	1650	3600	8@25	4080	13.33
15	Column 350x350	3300	3600	8@25	4080	13.33

Columns D-E

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Column 550x550	0	5716.69	4@43	5808	1.60
1	Column 550x550	2575	5770.29	4@43	5808	0.65

1	Column 550x550	5150	5823.57	4@43	5808	0.27
2	Column 550x550	0	4504.90	4@43	5808	28.93
2	Column 550x550	1650	4435.68	4@43	5808	30.94
2	Column 550x550	3300	4469.25	4@43	5808	29.95
3	Column 550x550	0	4435.68	8@25	4080	8.02
3	Column 550x550	1650	4469.25	8@25	4080	8.71
3	Column 550x550	3300	4351.75	8@25	4080	6.24
4	Column 550x550	0	3994.58	8@25	4080	2.14
4	Column 550x550	1650	3791.55	12@19	3408	10.12
4	Column 550x550	3300	3514.91	12@19	3408	3.04
5	Column 500x500	0	3600	8@25	4080	13.33
5	Column 500x500	1650	3600	8@25	4080	13.33
5	Column 500x500	3300	3600	8@25	4080	13.33
6	Column 450x450	0	3025	8@22	3096	2.35
6	Column 450x450	1650	3025	8@22	3096	2.35
6	Column 450x450	3300	3025	8@22	3096	2.35

7	Column 450x450	0	3025	8@22	3096	2.35
7	Column 450x450	1650	3025	8@22	3096	2.35
7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	3182.597	8@22	3096	2.72
10	Column 450x450	1650	3180.1272	8@22	3096	2.65
10	Column 450x450	3300	3177.6574	8@22	3096	2.57
11	Column 350x350	0	2777.2	8@22	3096	11.48
11	Column 350x350	1650	2620	8@22	3096	18.17
11	Column 350x350	3300	2620	8@22	3096	18.17
12	Column 350x350	0	2705.2688	8@22	3096	14.44

12	Column 350x350	1650	2705.2688	8@22	3096	14.44
12	Column 350x350	3300	2670.1792	8@22	3096	15.95
13	Column 350x350	0	2500	8@22	3096	23.84
13	Column 350x350	1650	2500	8@22	3096	23.84
13	Column 350x350	3300	2500	8@22	3096	23.84
14	Column 350x350	0	2500	8@22	3096	23.84
14	Column 350x350	1650	2500	8@22	3096	23.84
14	Column 350x350	3300	2500	8@22	3096	23.84
15	Column 350x350	0	3600	8@25	4080	13.33
15	Column 350x350	1650	3600	8@25	4080	13.33
15	Column 350x350	3300	3600	8@25	4080	13.33

Columns E-F

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Column 550x550	0	5365.66	12@25	6120	14.06
1	Column 550x550	2575	5417.07	12@25	6120	12.98
1	Column 550x550	5150	5468.19	12@25	6120	11.92

2	Column 550x550	0	4188.22	8@25	4080	2.58
2	Column 550x550	1650	4220.35	8@25	4080	3.33
2	Column 550x550	3300	4252.29	8@25	4080	4.05
3	Column 550x550	0	4140.50	8@25	4080	1.46
3	Column 550x550	1650	4225	8@25	4080	3.43
3	Column 550x550	3300	4225	8@25	4080	3.43
4	Column 550x550	0	3600	8@25	4080	13.33
4	Column 550x550	1650	3600	8@25	4080	13.33
4	Column 550x550	3300	3600	8@25	4080	13.33
5	Column 500x500	0	3600	8@25	4080	13.33
5	Column 500x500	1650	3600	8@25	4080	13.33
5	Column 500x500	3300	3600	8@25	4080	13.33
6	Column 450x450	0	3025	8@22	3096	2.35
6	Column 450x450	1650	3025	8@22	3096	2.35
6	Column 450x450	3300	3025	8@22	3096	2.35
7	Column 450x450	0	3025	8@22	3096	2.35

7	Column 450x450	1650	3025	8@22	3096	2.35
7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	2942.401	8@22	3096	5.22
10	Column 450x450	1650	2940.1176	8@22	3096	5.30
10	Column 450x450	3300	2937.8342	8@22	3096	5.38
11	Column 350x350	0	2567.6	8@22	3096	20.58
11	Column 350x350	1650	2620	8@22	3096	18.17
11	Column 350x350	3300	2620	8@22	3096	18.17
12	Column 350x350	0	2500	8@22	3096	23.84
12	Column 350x350	1650	2500	8@22	3096	23.84

12	Column 350x350	3300	2500	8@22	3096	23.84
13	Column 350x350	0	2500	8@22	3096	23.84
13	Column 350x350	1650	2500	8@22	3096	23.84
13	Column 350x350	3300	2500	8@22	3096	23.84
14	Column 350x350	0	2500	8@22	3096	23.84
14	Column 350x350	1650	2500	8@22	3096	23.84
14	Column 350x350	3300	2500	8@22	3096	23.84
15	Column 350x350	0	3600	8@25	4080	13.33
15	Column 350x350	1650	3600	8@25	4080	13.33
15	Column 350x350	3300	3600	8@25	4080	13.33

Columns F

Member ID	Type	Location (mm)	Area Required (mm ²)	detail	Area Provided (mm ²)	error %
1	Column 550x550	0	5916.572	12@25	6120	3.44
1	Column 550x550	2575	4225	8@25	4080	3.43
1	Column 550x550	5150	4225	8@25	4080	3.43
2	Column 550x550	0	4225	8@25	4080	3.43

2	Column 550x550	1650	4225	8@25	4080	3.43
2	Column 550x550	3300	4225	8@25	4080	3.43
3	Column 550x550	0	3600	8@25	4080	13.33
3	Column 550x550	1650	3600	8@25	4080	13.33
3	Column 550x550	3300	3600	8@25	4080	13.33
4	Column 550x550	0	3600	12@19	3408	5.33
4	Column 550x550	1650	3600	12@19	3408	5.33
4	Column 550x550	3300	3600	12@19	3408	5.33
5	Column 500x500	0	3600	12@19	3408	5.33
5	Column 500x500	1650	3600	12@19	3408	5.33
5	Column 500x500	3300	3600	12@19	3408	5.33
6	Column 450x450	0	3025	8@22	3096	2.35
6	Column 450x450	1650	3025	8@22	3096	2.35
6	Column 450x450	3300	3025	8@22	3096	2.35
7	Column 450x450	0	3025	8@22	3096	2.35
7	Column 450x450	1650	3025	8@22	3096	2.35

7	Column 450x450	3300	3025	8@22	3096	2.35
8	Column 450x450	0	3025	8@22	3096	2.35
8	Column 450x450	1650	3025	8@22	3096	2.35
8	Column 450x450	3300	3025	8@22	3096	2.35
9	Column 450x450	0	3025	8@22	3096	2.35
9	Column 450x450	1650	3025	8@22	3096	2.35
9	Column 450x450	3300	3025	8@22	3096	2.35
10	Column 450x450	0	3025	8@22	3096	2.35
10	Column 450x450	1650	3025	8@22	3096	2.35
10	Column 450x450	3300	3025	8@22	3096	2.35
11	Column 350x350	0	2500	8@22	3096	23.84
11	Column 350x350	1650	2500	8@22	3096	23.84
11	Column 350x350	3300	2500	8@22	3096	23.84
12	Column 350x350	0	2500	8@22	3096	23.84
12	Column 350x350	1650	2500	8@22	3096	23.84
12	Column 350x350	3300	2500	8@22	3096	23.84

13	Column 350x350	0	2500	8@22	3096	23.84
13	Column 350x350	1650	2500	8@22	3096	23.84
13	Column 350x350	3300	2500	8@22	3096	23.84
14	Column 350x350	0	2500	8@22	3096	23.84
14	Column 350x350	1650	2500	8@22	3096	23.84
14	Column 350x350	3300	2500	8@22	3096	23.84
15	Column 350x350	0	3600	8@25	4080	13.33
15	Column 350x350	1650	3600	8@25	4080	13.33
15	Column 350x350	3300	3600	8@25	4080	13.33

Table B. 10 Shear reinforcement detail

A			B			C		
	detail	spacing		detail	spacing		detail	spacing
15	2#10	180	15	2#10	130	15	2#10	130
14	2#10	190	14	2#10	200	14	2#10	200
13	2#10	200	13	2#10	200	13	2#10	300
12	2#10	150	12	2#10	300	12	2#10	300
11	2#10	300	11	2#10	300	11	2#10	300
10	2#10	300	10	2#10	300	10	2#10	300
9	2#10	300	9	2#10	300	9	2#10	300
8	4#10	300	8	4#10	300	8	4#10	300
7	4#10	300	7	4#10	300	7	4#10	300
6	4#10	300	6	4#10	300	6	4#10	300
5	4#10	310	5	4#10	310	5	4#10	310
4	4#10	310	4	4#10	310	4	4#10	310
3	4#8	310	3	4#8	310	3	4#10	310
2	4#8	325	2	4#8	325	2	2#10	325
1	4#8	325	1	4#8	325	1	4#8	325

D			E			F		
	detail	spacing		detail	spacing		detail	spacing
15	2#10	130	15	2#10	130	15	2#10	180
14	2#10	200	14	2#10	200	14	2#10	190
13	2#10	300	13	2#10	200	13	2#10	200
12	2#10	300	12	2#10	150	12	2#10	150
11	2#10	300	11	2#10	300	11	2#10	300
10	2#10	300	10	2#10	300	10	2#10	300

9	2#10	300	9	2#10	300	9	2#10	300
8	4#10	300	8	4#10	300	8	4#10	300
7	4#10	300	7	4#10	300	7	4#10	300
6	4#10	300	6	4#10	300	6	4#10	300
5	4#10	310	5	4#10	310	5	4#10	310
4	4#10	310	4	4#10	310	4	4#10	310
3	4#10	310	3	4#8	310	3	4#8	310
2	2#10	325	2	4#8	325	2	4#8	325
1	4#8	325	1	4#8	325	1	4#8	325

Appendix C

Soil map of Northern Kazakhstan

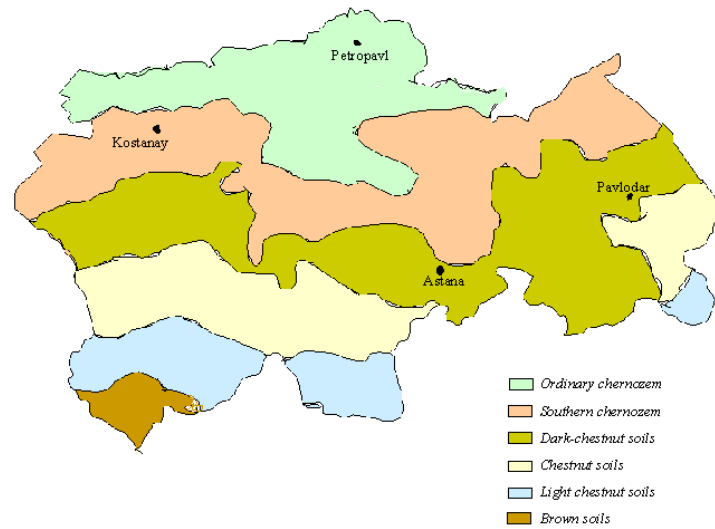


Figure C1. Soil Map of Northern Kazakhstan

Table 2.2.1 Geotechnical categories

GC	Includes...	Design requirements	Design procedure
1	Small and relatively simple structures... with negligible risk	Negligible risk of instability or ground movements Ground conditions known to be straightforward No excavation below water table (or such excavation is straightforward)	Routine design & construction methods
2	Conventional types of structure & foundation with no exceptional risk or difficult soil or loading conditions	Quantitative geotechnical data & analysis to ensure fundamental requirements are satisfied	Routine field & lab testing Routine design & execution
3	Structures or parts of structures not covered above	Include alternative provisions and rules to those in <i>Eurocode 7</i>	

Table 2.3.1 Classification of actions

Action	Duration	Variation with time	Examples
Permanent	G	Likely to act throughout reference period	Negligible or monotonic Self-weight of structures, fixed equipment and road-surfacing; indirect actions [§] caused by shrinkage and uneven settlements
Variable	Q		Neither negligible nor monotonic Imposed loads on building floors, beams and roofs; wind*; snow*
Accidental	A	Usually short	Significant magnitude Explosions, vehicle impact*, seismic* (AE, due to earthquake ground motions)

*may be variable or accidental depending on statistical distribution
[§] may be permanent or variable

НАСЫПНЫЕ ГРУНТЫ tQ_{IV} – мощностью 0,30-3,0м, представлены суглинком, дресвой, щебнем, несслежавшиеся.

СУГЛИНКИ aQ_{IV} вскрыты на глубине 0,30-3,0м, мощность суглинков составила 3,00-5,50м. По полевому описанию суглинки черного, коричневого цвета, от твердых до текучих, с прослойками супеси и линзами песка, содержание гумуса составляет, в основном 0,3-0,9%, только в скважине 222-15, интервал опробования 0,4-1,3м составляет 3,9%.

ПЕСКИ СРЕДНЕЙ КРУПНОСТИ aQ_{IV} вскрыты на глубине 4,60-5,50м, мощность слоя составила 0,80-2,40м. По полевому описанию пески средней крупности коричневого цвета, средней плотности, полимиктовые, водонасыщенные, в скважине 222-15 с линзами песка мелкого.

ПЕСКИ КРУПНЫЕ aQ_{IV} вскрыты на глубине 5,30-6,30м, мощность слоя составила 1,10-3,20м. По полевому описанию пески крупные коричневого, серого, бурого цвета, средней плотности, полимиктовые, водонасыщенные.

ПЕСКИ ГРАВЕЛИСТЫЕ aQ_{IV} вскрыты на глубине 5,30-8,50м, мощность слоя составила 1,80-7,10м. По полевому описанию пески гравелистые коричневого, серого, бурого цвета, средней плотности, полимиктовые, водонасыщенные, с прослойками суглинка.

ГРАВИЙНЫЕ ГРУНТЫ aQ_{IV} вскрыты на глубине 10,30-11,30м, мощность слоя составила 0,40-1,90м. По полевому описанию гравийные грунты бурого цвета, полимиктовые, водонасыщенные.

СУГЛИНКИ e(C₁) вскрыты на глубине 11,10-14,10м, вскрытая мощность слоя составила 1,30-5,90м. По полевому описанию суглинки красноватого, желтоватого, красновато-фиолетового цвета, твердые, с маломощными прослойками глины, ожезленные, омарганцованные, местами с незначительным содержанием щебня.

Figure C2. Soil Characteristics

ИГЭ-2. СУГЛИНКИ аQ_{II-IV} по данным лабораторных определений и материалам изученности характеризуются следующими показателями физических свойств, приведенными в таблице №9.

Таблица №9

№ п. п	Наименование показателей	Единица измерения	Количество определений	Предельные значения		Средние (нормативные) значения
				миним.	максим.	
1	2	3	4	5	6	7
1	Природная влажность	%	27	13,4	26,6	20,9
2	Влажность на пределе текучести	%	46	17	41	26
3	Влажность на пределе раскатывания	%	46	13	25	16
4	Число пластичности	%	46	4	16	10
5	Плотность грунта	г/см ³	21	1,80	2,10	1,97
6	Плотность частиц грунта	--/--	21	2,70	2,74	2,72
7	Коэффициент пористости	доли ед.	21	0,50	0,91	0,67
8	Степень влажности	--/--	21	0,72	1,02	0,83
9	Консистенция		27	<0	1,30	

Figure C3. Soil Parameters

ИГЭ-3. ПЕСКИ СРЕДНЕЙ КРУПНОСТИ аQ_{III-V} характеризуются содержанием определяющей фракции (частиц крупнее 0,25 мм) в пределах 50,6 - 72,5%, среднее содержание 59,0%.

Угол естественного откоса для песков средней крупности составил в сухом состоянии 34 - 41 градус, среднее значение 38 градусов, под водой 22 - 35 градусов, среднее значение 29 градусов.

Нормативные и расчетные значения характеристик для песков средней крупности рекомендуется принять по материалам изученности:

Нормативные значения:
 удельное сцепление – 2 кПа;
 угол внутреннего трения — 35 градусов;
 модуль деформации — 17,0 МПа;
 плотность грунта – 1,92 г/см³.

За расчетные значения характеристик по деформациям рекомендуется принять их нормативные значения с коэффициентом надежности по грунту, равном 1:

удельное сцепление — 2 кПа;
 угол внутреннего трения — 35 градусов;
 плотность грунта – 1,92 г/см³.

Figure C4. Soil Parameters

ИГЭ-4. ПЕСКИ КРУПНЫЕ И ГРАВЕЛИСТЫЕ аQ_{IV-V} характеризуются содержанием определяющей фракции для крупных песков - (частиц крупнее 0,50 мм) 69,3 - 76,8%, среднее значение 71,1%, для песков гравелистых (частиц крупнее 2 мм) — 25,7 - 48,7%, среднее значение — 40,5%. Угол естественного откоса составляет: для сухого грунта — 33 - 36 градусов, среднее значение 34 градуса, под водой — 28 — 31 градус, среднее 30 градусов.

Нормативные и расчетные значения характеристик для песков крупных и гравелистых рекомендуется принять по материалам изученности:

Нормативные значения:
 удельное сцепление — 1 кПа;
 угол внутреннего трения — 38 градусов;
 модуль деформации — 21,0 МПа;
 плотность грунта — 2,00 г/см³.

Figure C5. Soil Parameters

ИГЭ-5. ГРАВИЙНЫЕ ГРУНТЫ $a_{Q_{II,IV}}$ характеризуются содержанием определяющей фракции (частиц крупнее 2 мм) — 54,9-77,8%, среднее значение 66,3%.

Нормативные значения характеристик для гравийных грунтов рекомендуется принять по материалам изученности с учетом действующих на территории РК требований нормативных документов:

расчетное сопротивление – 300 кПа;
плотность грунта – 2,05 г/см³.

Модуль деформации для гравийных грунтов рекомендуется принять по материалам изученности равным 23,0 МПа.

Figure C6. Soil Parameters

ИГЭ-6. ГЛИНИСТЫЕ ГРУНТЫ $e(C_I)$ по данным лабораторных определений и материалам изученности характеризуются следующими показателями физических свойств, приведенными в таблице №12.

Таблица №12

№ п. п.	Наименование показателей	Единица измерения	Количество определений	Предельные значения		Средние (нормативные) значения
				миним.	максим.	
1	2	3	4	5	6	7
1	Природная влажность	%	20	14,4	31,2	23,2
2	Влажность на пределе текучести	%	23	32	47	40
3	Влажность на пределе раскатывания	%	23	21	32	25
4	Число пластичности	%	23	11	19	14
5	Плотность грунта	г/см ³	20	1,82	2,03	1,93
6	Плотность частиц грунта	--/--	20	2,73	2,75	2,74
7	Коэффициент пористости	доли ед.	20	0,61	0,96	0,75
8	Степень влажности	--/--	20	0,64	0,94	0,85
9	Консистенция		20	<0	<0	<0

Figure C7. Soil Parameters

Table C1. Methods for determination of frictional resistance of a pile.

	α - method	λ - method	β - method
<u>Unit skin resistance</u>	$f = \alpha c_u$	$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u)$	$f = \beta \sigma'^o$
<u>Value of factor</u>	$\alpha = C \left(\frac{\bar{\sigma}'_o}{c_u}\right)^{0.45}$ $C \sim 0.4$ to 0.5 for bored piles and ≥ 0.5 for driven piles	The value of λ is given in the Appendix	$\beta = K \tan \phi'_R$ $K_1 = 1 - \sin \phi'_R$ $K_2 = (1 - \sin \phi'_R) OCR^{0.5}$
<u>Meanings of components</u>	$\bar{\sigma}'_o$ = average vertical effective stress c_u = mean undrained shear strength		ϕ'_R = drained friction angle of remolded clay K_1 = Normally consolidated clay K_1 = Over consolidated clay
<u>Final formula</u>	$Q_s = \sum \alpha * c_u * p * \Delta L$	$Q_s = p * \Delta L * f_{av}$	$Q_s = \sum p * \Delta L * f$

Table 4.2 Bearing Capacity Factors

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	16	11.63	4.34	3.06
1	5.38	1.09	0.07	17	12.34	4.77	3.53
2	5.63	1.20	0.15	18	13.10	5.26	4.07
3	5.90	1.31	0.24	19	13.93	5.80	4.68
4	6.19	1.43	0.34	20	14.83	6.40	5.39
5	6.49	1.57	0.45	21	15.82	7.07	6.20
6	6.81	1.72	0.57	22	16.88	7.82	7.13
7	7.16	1.88	0.71	23	18.05	8.66	8.20
8	7.53	2.06	0.86	24	19.32	9.60	9.44
9	7.92	2.25	1.03	25	20.72	10.66	10.88
10	8.35	2.47	1.22	26	22.25	11.85	12.54
11	8.80	2.71	1.44	27	23.94	13.20	14.47
12	9.28	2.97	1.69	28	25.80	14.72	16.72
13	9.81	3.26	1.97	29	27.86	16.44	19.34
14	10.37	3.59	2.29	30	30.14	18.40	22.40
15	10.98	3.94	2.65	31	32.67	20.63	25.99

(continued)

Table 4.2 Bearing Capacity Factors (Continued)

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
32	35.49	23.18	30.22	42	93.71	85.38	155.55
33	38.64	26.09	35.19	43	105.11	99.02	186.54
34	42.16	29.44	41.06	44	118.37	115.31	224.64
35	46.12	33.30	48.03	45	133.88	134.88	271.76
36	50.59	37.75	56.31	46	152.10	158.51	330.35
37	55.63	42.92	66.19	47	173.64	187.21	403.67
38	61.35	48.93	78.03	48	199.26	222.31	496.01
39	67.87	55.96	92.25	49	229.93	265.51	613.16
40	75.31	64.20	109.41	50	266.89	319.07	762.89
41	83.86	73.90	130.22				

Shape, Depth, and Inclination Factors

Commonly used shape, depth, and inclination factors are given in Table 4.3.

Table 4.3 Shape, Depth and Inclination Factors [DeBeer (1970); Hansen (1970); Meyerhof (1963); Meyerhof and Hanna (1981)]

Factor	Relationship	Reference
Shape	$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	DeBeer (1970)
	$F_{qs} = 1 + \left(\frac{B}{L}\right) \tan \phi'$	
	$F_{\gamma s} = 1 - 0.4 \left(\frac{B}{L}\right)$	
Depth	$\frac{D_f}{B} \leq 1$	Hansen (1970)
	For $\phi = 0$:	
	$F_{cd} = 1 + 0.4 \left(\frac{D_f}{B}\right)$	
	$F_{qd} = 1$	
	$F_{\gamma d} = 1$	
	For $\phi' > 0$:	
	$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$	
	$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \left(\frac{D_f}{B}\right)$	
	$F_{\gamma d} = 1$	
	$\frac{D_f}{B} > 1$	

Table 4.3 Shape, Depth and Inclination Factors [DeBeer (1970); Hansen (1970); Meyerhof (1963); Meyerhof and Hanna (1981)] (Continued)

Factor	Relationship	Reference
	For $\phi = 0$:	
	$F_{cd} = 1 + 0.4 \tan^{-1} \left(\frac{D_f}{B} \right)$	
	$F_{qd} = 1$	
	$F_{\gamma d} = 1$	
	For $\phi' > 0$:	
	$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$	
	$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \tan^{-1} \left(\frac{D_f}{B} \right)$	
	$F_{\gamma d} = 1$	
Inclination	$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90^\circ} \right)^2$	Meyerhof (1963); Hanna and Meyerhof (1981)
	$F_{\gamma i} = \left(1 - \frac{\beta^\circ}{\phi'} \right)^2$	
	β = inclination of the load on the foundation with respect to the vertical	

Table 7.2 Variation of F_1 with m' and n'

n'	m'									
	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.014	0.013	0.012	0.011	0.011	0.011	0.010	0.010	0.010	0.010
0.50	0.049	0.046	0.044	0.042	0.041	0.040	0.038	0.038	0.037	0.037
0.75	0.095	0.090	0.087	0.084	0.082	0.080	0.077	0.076	0.074	0.074
1.00	0.142	0.138	0.134	0.130	0.127	0.125	0.121	0.118	0.116	0.115
1.25	0.186	0.183	0.179	0.176	0.173	0.170	0.165	0.161	0.158	0.157
1.50	0.224	0.224	0.222	0.219	0.216	0.213	0.207	0.203	0.199	0.197
1.75	0.257	0.259	0.259	0.258	0.255	0.253	0.247	0.242	0.238	0.235
2.00	0.285	0.290	0.292	0.292	0.291	0.289	0.284	0.279	0.275	0.271
2.25	0.309	0.317	0.321	0.323	0.323	0.322	0.317	0.313	0.308	0.305
2.50	0.330	0.341	0.347	0.350	0.351	0.351	0.348	0.344	0.340	0.336
2.75	0.348	0.361	0.369	0.374	0.377	0.378	0.377	0.373	0.369	0.365
3.00	0.363	0.379	0.389	0.396	0.400	0.402	0.402	0.400	0.396	0.392
3.25	0.376	0.394	0.406	0.415	0.420	0.423	0.426	0.424	0.421	0.418
3.50	0.388	0.408	0.422	0.431	0.438	0.442	0.447	0.447	0.444	0.441
3.75	0.399	0.420	0.436	0.447	0.454	0.460	0.467	0.458	0.466	0.464
4.00	0.408	0.431	0.448	0.460	0.469	0.476	0.484	0.487	0.486	0.484
4.25	0.417	0.440	0.458	0.472	0.481	0.484	0.495	0.514	0.515	0.515
4.50	0.424	0.450	0.469	0.484	0.495	0.503	0.516	0.521	0.522	0.522
4.75	0.431	0.458	0.478	0.494	0.506	0.515	0.530	0.536	0.539	0.539
5.00	0.437	0.465	0.487	0.503	0.516	0.526	0.543	0.551	0.554	0.554
5.25	0.443	0.472	0.494	0.512	0.526	0.537	0.555	0.564	0.568	0.569
5.50	0.448	0.478	0.501	0.520	0.534	0.546	0.566	0.576	0.581	0.584
5.75	0.453	0.483	0.508	0.527	0.542	0.555	0.576	0.588	0.594	0.597
6.00	0.457	0.489	0.514	0.534	0.550	0.563	0.585	0.598	0.606	0.609
6.25	0.461	0.493	0.519	0.540	0.557	0.570	0.594	0.609	0.617	0.621
6.50	0.465	0.498	0.524	0.546	0.563	0.577	0.603	0.618	0.627	0.632
6.75	0.468	0.502	0.529	0.551	0.569	0.584	0.610	0.627	0.637	0.643
7.00	0.471	0.506	0.533	0.556	0.575	0.590	0.618	0.635	0.646	0.653
7.25	0.474	0.509	0.538	0.561	0.580	0.596	0.625	0.643	0.655	0.662
7.50	0.477	0.513	0.541	0.565	0.585	0.601	0.631	0.650	0.663	0.671
7.75	0.480	0.516	0.545	0.569	0.589	0.606	0.637	0.658	0.671	0.680
8.00	0.482	0.519	0.549	0.573	0.594	0.611	0.643	0.664	0.678	0.688
8.25	0.485	0.522	0.552	0.577	0.598	0.615	0.648	0.670	0.685	0.695
8.50	0.487	0.524	0.555	0.580	0.601	0.619	0.653	0.676	0.692	0.703
8.75	0.489	0.527	0.558	0.583	0.605	0.623	0.658	0.682	0.698	0.710
9.00	0.491	0.529	0.560	0.587	0.609	0.627	0.663	0.687	0.705	0.716
9.25	0.493	0.531	0.563	0.589	0.612	0.631	0.667	0.693	0.710	0.723
9.50	0.495	0.533	0.565	0.592	0.615	0.634	0.671	0.697	0.716	0.719
9.75	0.496	0.536	0.568	0.595	0.618	0.638	0.675	0.702	0.721	0.735
10.00	0.498	0.537	0.570	0.597	0.621	0.641	0.679	0.707	0.726	0.740
20.00	0.529	0.575	0.614	0.647	0.677	0.702	0.756	0.797	0.830	0.858
50.00	0.548	0.598	0.640	0.678	0.711	0.740	0.803	0.853	0.895	0.931
100.00	0.555	0.605	0.649	0.688	0.722	0.753	0.819	0.872	0.918	0.956

(Continued)

Table 7.3 (Continued)

n'	m'									
	4.5	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.25	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053
0.50	0.087	0.087	0.088	0.088	0.088	0.088	0.088	0.088	0.088	0.088
0.75	0.109	0.109	0.109	0.110	0.110	0.110	0.110	0.111	0.111	0.111
1.00	0.121	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.25	0.128	0.130	0.131	0.132	0.132	0.133	0.133	0.134	0.134	0.134
1.50	0.132	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
1.75	0.134	0.136	0.138	0.140	0.141	0.142	0.142	0.144	0.144	0.145
2.00	0.134	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
2.25	0.133	0.136	0.140	0.142	0.144	0.145	0.146	0.149	0.150	0.150
2.50	0.132	0.135	0.139	0.142	0.144	0.146	0.147	0.151	0.151	0.151
2.75	0.130	0.133	0.138	0.142	0.144	0.146	0.147	0.152	0.152	0.153
3.00	0.127	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
3.25	0.125	0.129	0.135	0.140	0.143	0.145	0.147	0.153	0.154	0.154
3.50	0.122	0.126	0.133	0.138	0.142	0.144	0.146	0.153	0.155	0.155
3.75	0.119	0.124	0.131	0.137	0.141	0.143	0.145	0.154	0.155	0.155
4.00	0.116	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
4.25	0.113	0.119	0.127	0.133	0.138	0.141	0.144	0.154	0.156	0.156
4.50	0.110	0.116	0.125	0.131	0.136	0.140	0.143	0.154	0.156	0.156
4.75	0.107	0.113	0.123	0.130	0.136	0.140	0.143	0.154	0.156	0.157
5.00	0.105	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
5.25	0.102	0.108	0.118	0.126	0.131	0.136	0.139	0.154	0.156	0.157
5.50	0.099	0.106	0.116	0.124	0.130	0.134	0.138	0.154	0.156	0.157
5.75	0.097	0.103	0.113	0.122	0.128	0.133	0.136	0.154	0.157	0.157
6.00	0.094	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
6.25	0.092	0.098	0.109	0.118	0.124	0.129	0.134	0.153	0.157	0.158
6.50	0.090	0.096	0.107	0.116	0.122	0.128	0.132	0.153	0.157	0.158
6.75	0.087	0.094	0.105	0.114	0.121	0.126	0.131	0.153	0.157	0.158
7.00	0.085	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
7.25	0.083	0.090	0.101	0.110	0.117	0.123	0.128	0.152	0.157	0.158
7.50	0.081	0.088	0.099	0.108	0.115	0.121	0.126	0.152	0.156	0.158
7.75	0.079	0.086	0.097	0.106	0.114	0.120	0.125	0.151	0.156	0.158
8.00	0.077	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
8.25	0.076	0.082	0.093	0.102	0.110	0.117	0.122	0.150	0.156	0.158
8.50	0.074	0.080	0.091	0.101	0.108	0.115	0.121	0.150	0.156	0.158
8.75	0.072	0.078	0.089	0.099	0.107	0.114	0.119	0.150	0.156	0.158
9.00	0.071	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
9.25	0.069	0.075	0.086	0.096	0.104	0.110	0.116	0.149	0.156	0.158
9.50	0.068	0.074	0.085	0.094	0.102	0.109	0.115	0.148	0.156	0.158
9.75	0.066	0.072	0.083	0.092	0.100	0.107	0.113	0.148	0.156	0.158
10.00	0.065	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.00	0.035	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
50.00	0.014	0.016	0.019	0.022	0.025	0.028	0.031	0.071	0.113	0.142
100.00	0.007	0.008	0.010	0.011	0.013	0.014	0.016	0.039	0.071	0.113

Table 7.4 Variation of I_f with D_f/B , B/L , and μ_s

μ_s	D_f/B	B/L		
		0.2	0.5	1.0
0.3	0.2	0.95	0.93	0.90
	0.4	0.90	0.86	0.81
	0.6	0.85	0.80	0.74
0.4	1.0	0.78	0.71	0.65
	0.2	0.97	0.96	0.93
	0.4	0.93	0.89	0.85
0.5	0.6	0.89	0.84	0.78
	1.0	0.82	0.75	0.69
	0.2	0.99	0.98	0.96
0.5	0.4	0.95	0.93	0.89
	0.6	0.92	0.87	0.82
	1.0	0.85	0.79	0.72

Appendix D

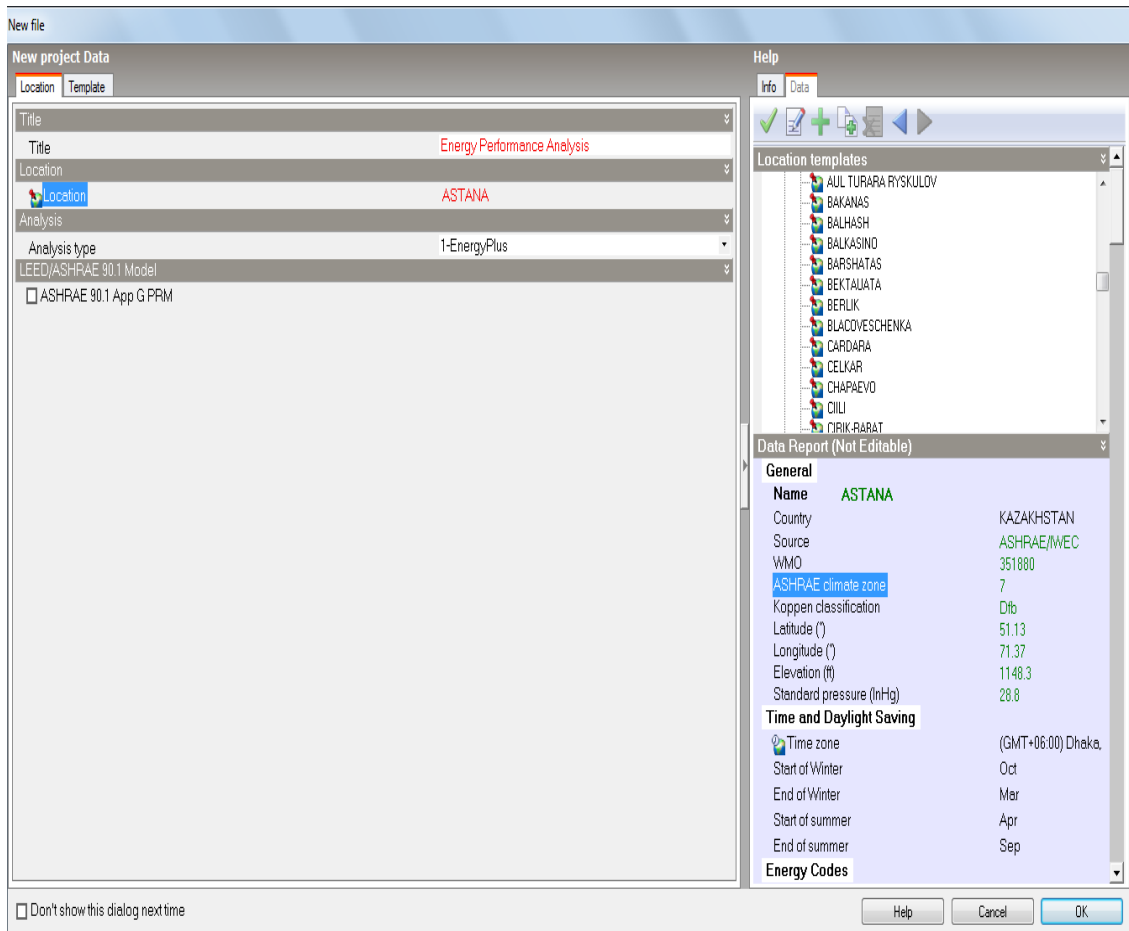


Figure D.1. Setting new project with Astana weather conditions

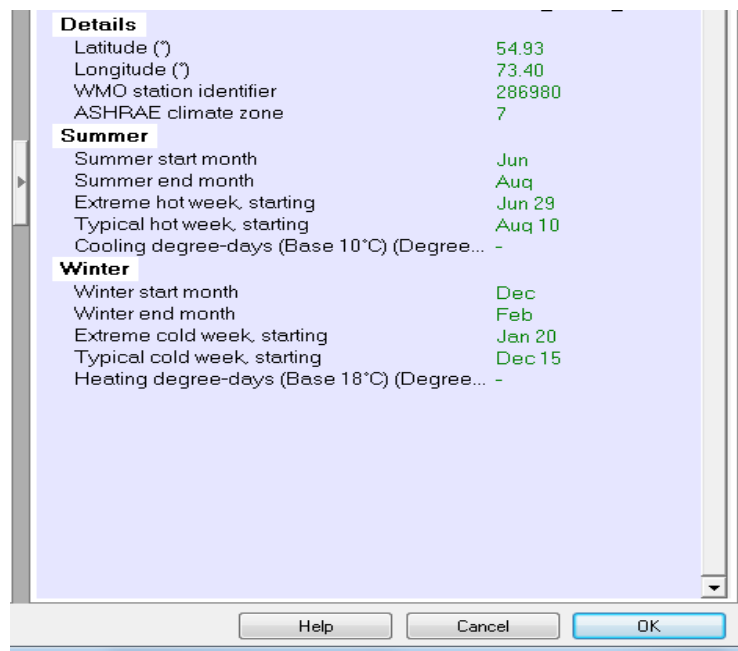


Figure D.2. Details on Astana weather data

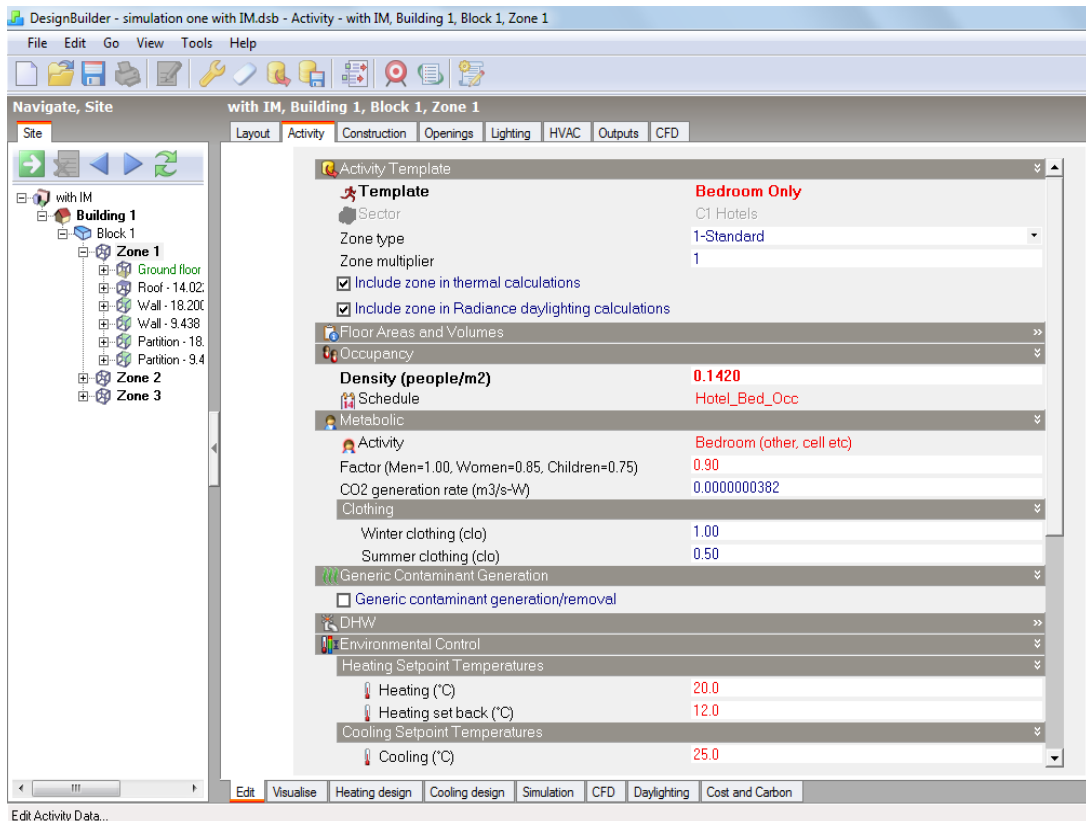


Figure D.3. Editing Activity template of the project

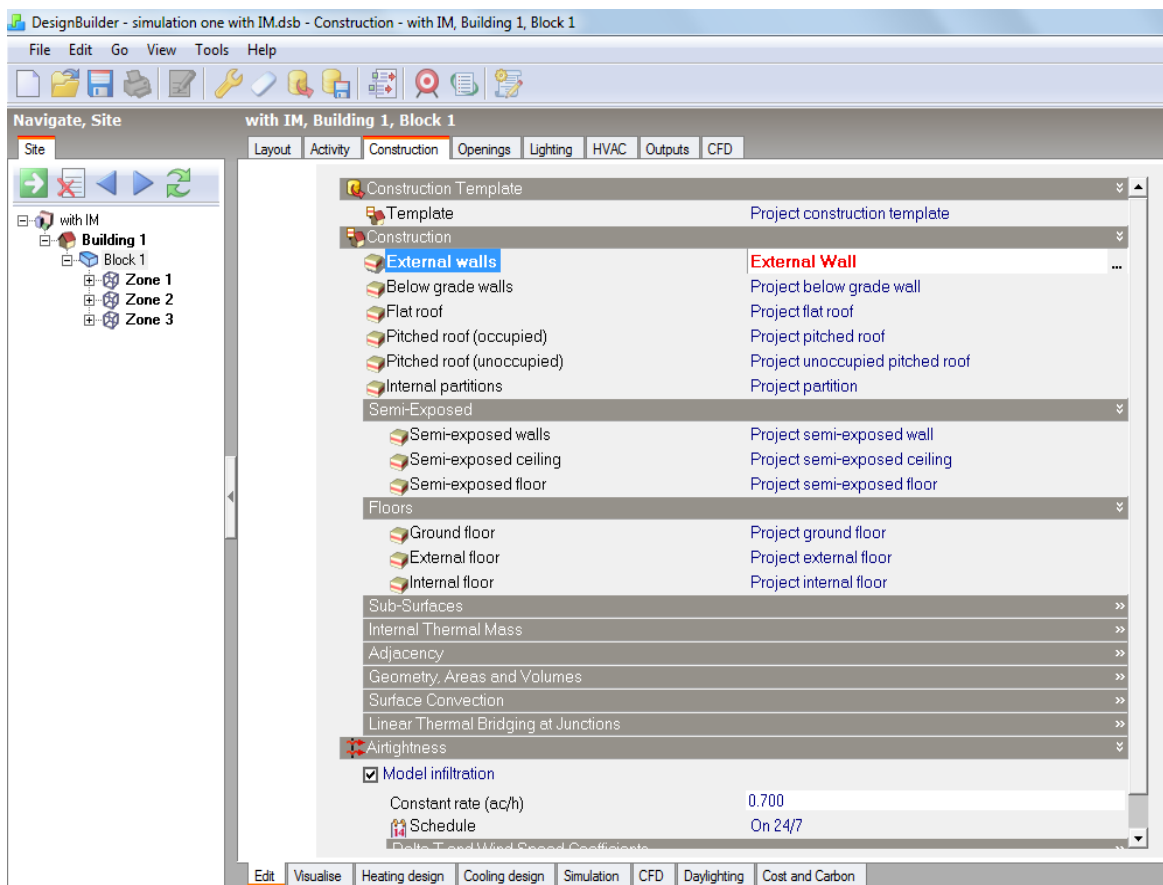


Figure D.4. Editing Construction Characteristics

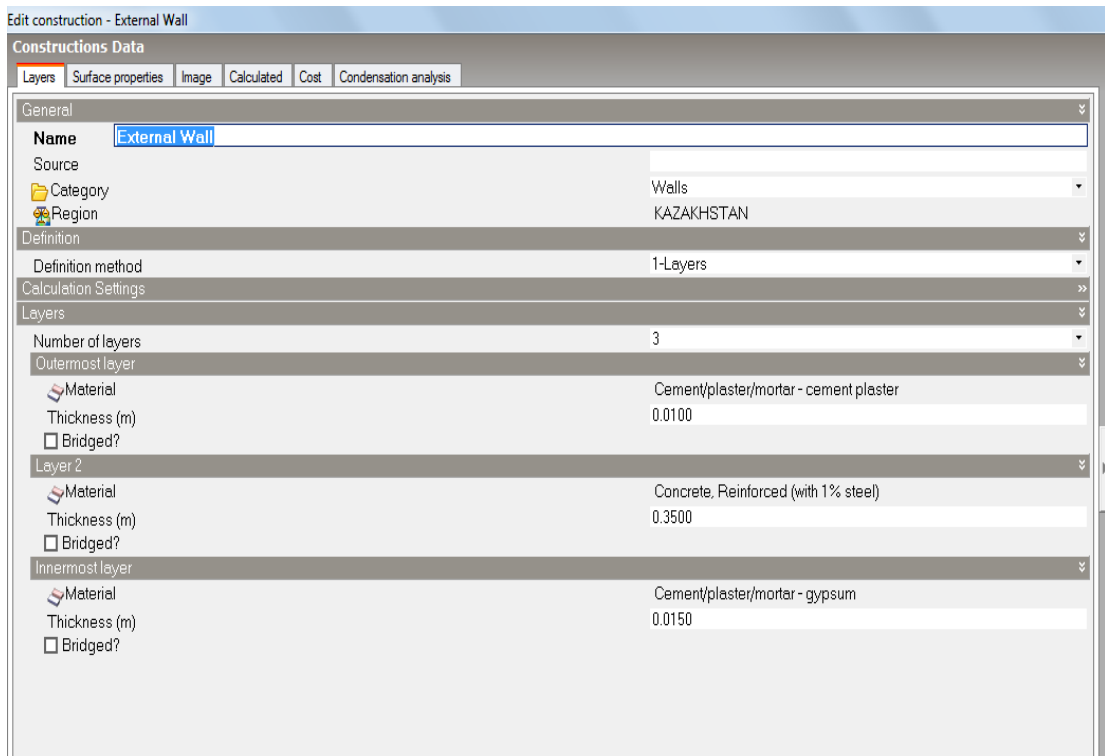


Figure D.5. Editing External Wall Construction Data

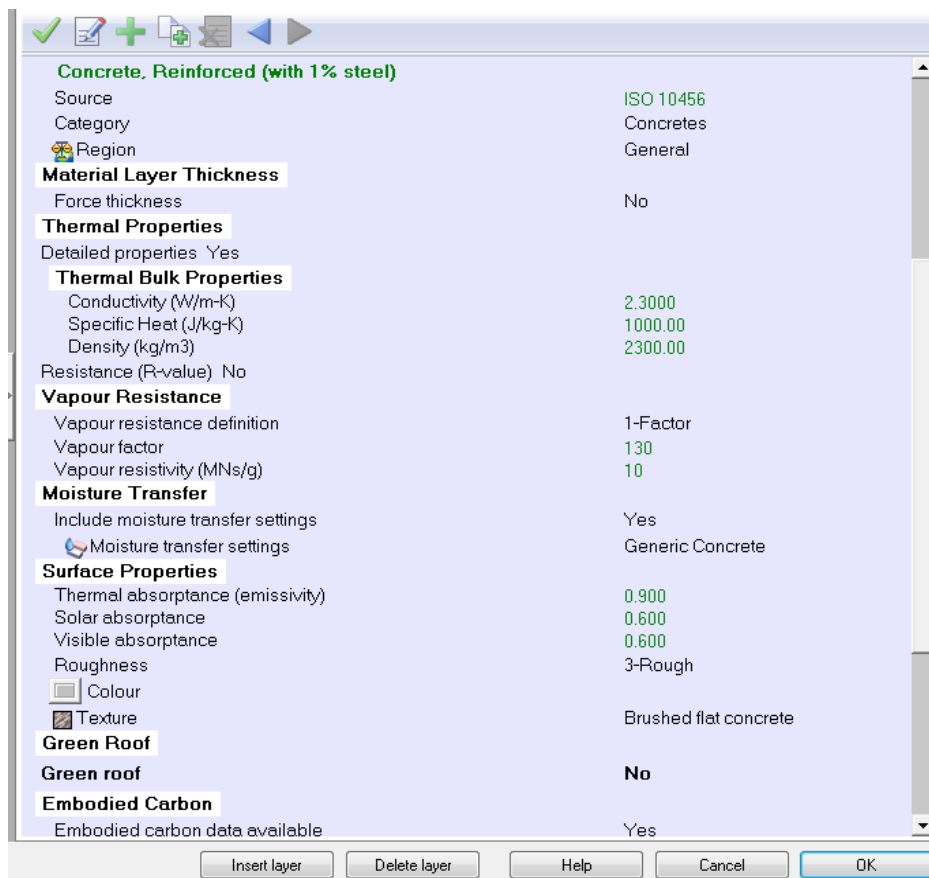


Figure D.6. Construction data on concrete layer of external wall

Table D.1. Design Builder simulation output for summer season

	PUR (summer)	Glass wool (summer)	VIP (summer)
Heating	4842	4453	4401
Cooling	1869	1893	1942
Ventilation	1406	1377	1425
Lighting	1108	1059	1100
Equipment	226	218	225
Hot Water	38	37	38
Total (kWh)	9490	9036	9131

Table D.2. Design Builder simulation output for winter season

	PUR (winter)	Glass wool (winter)	VIP (winter)
Heating	32593	29711	29410
Cooling	-	-	-
Ventilation	5448	5249	5417
Lighting	1102	1053	1094
Equipment	225	216	224
Hot Water	38	37	38
Total (kWh)	6813	6556	6773

Table D.3. Design Builder simulation output for the whole year

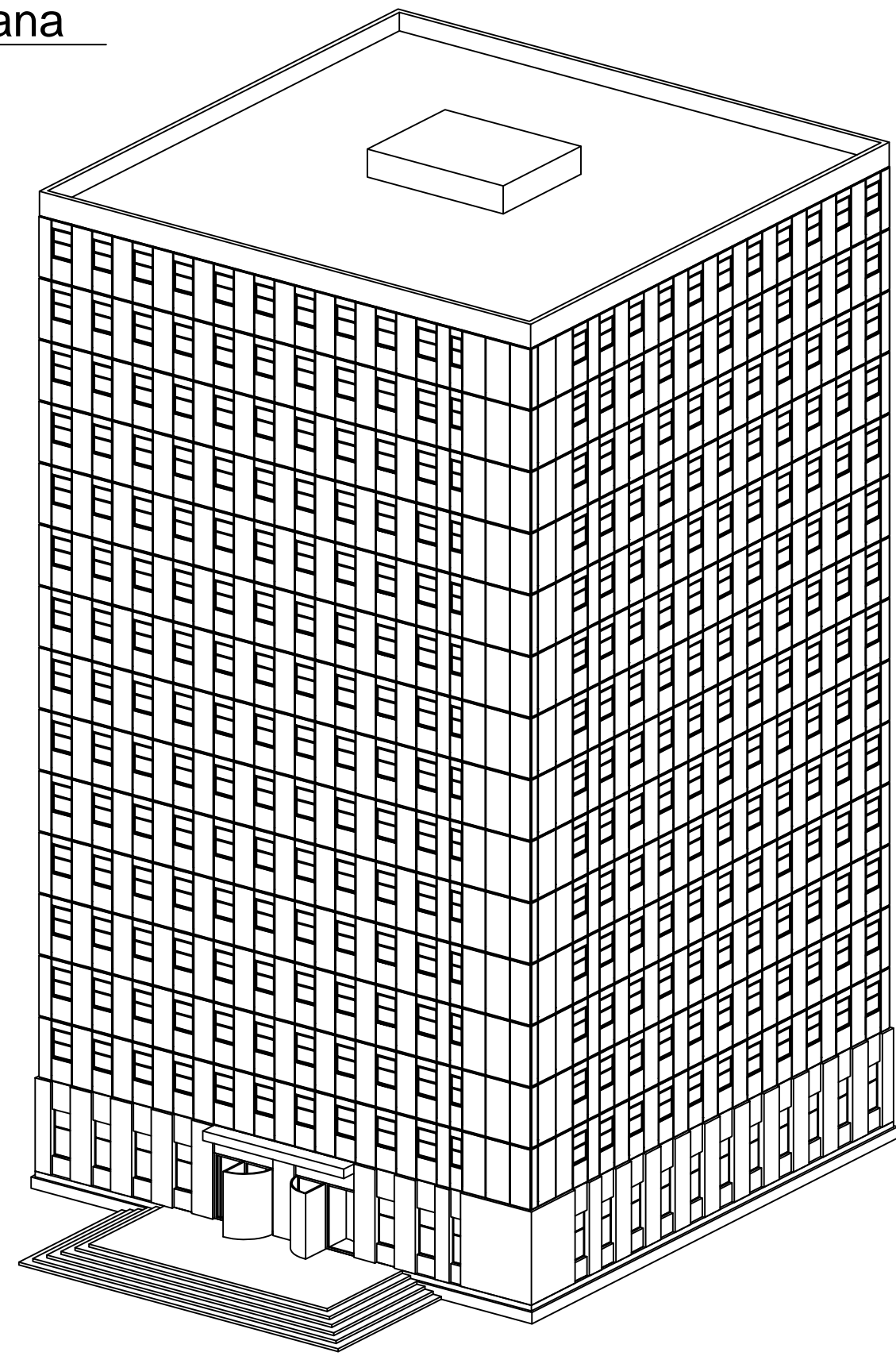
	PUR (whole year)	Glass wool (whole year)	VIP (whole year)
Heating	37441	34173	33821
Cooling	1867	1891	1940

Ventilation	6846	6625	6841
Lighting	2210	2111	2194
Equipment	451	434	448
Hot Water	76	74	76
Total (kWh)	48891	45309	45321

A0

Utopia Hotel Astana

Scale 1:250



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Hotel Building

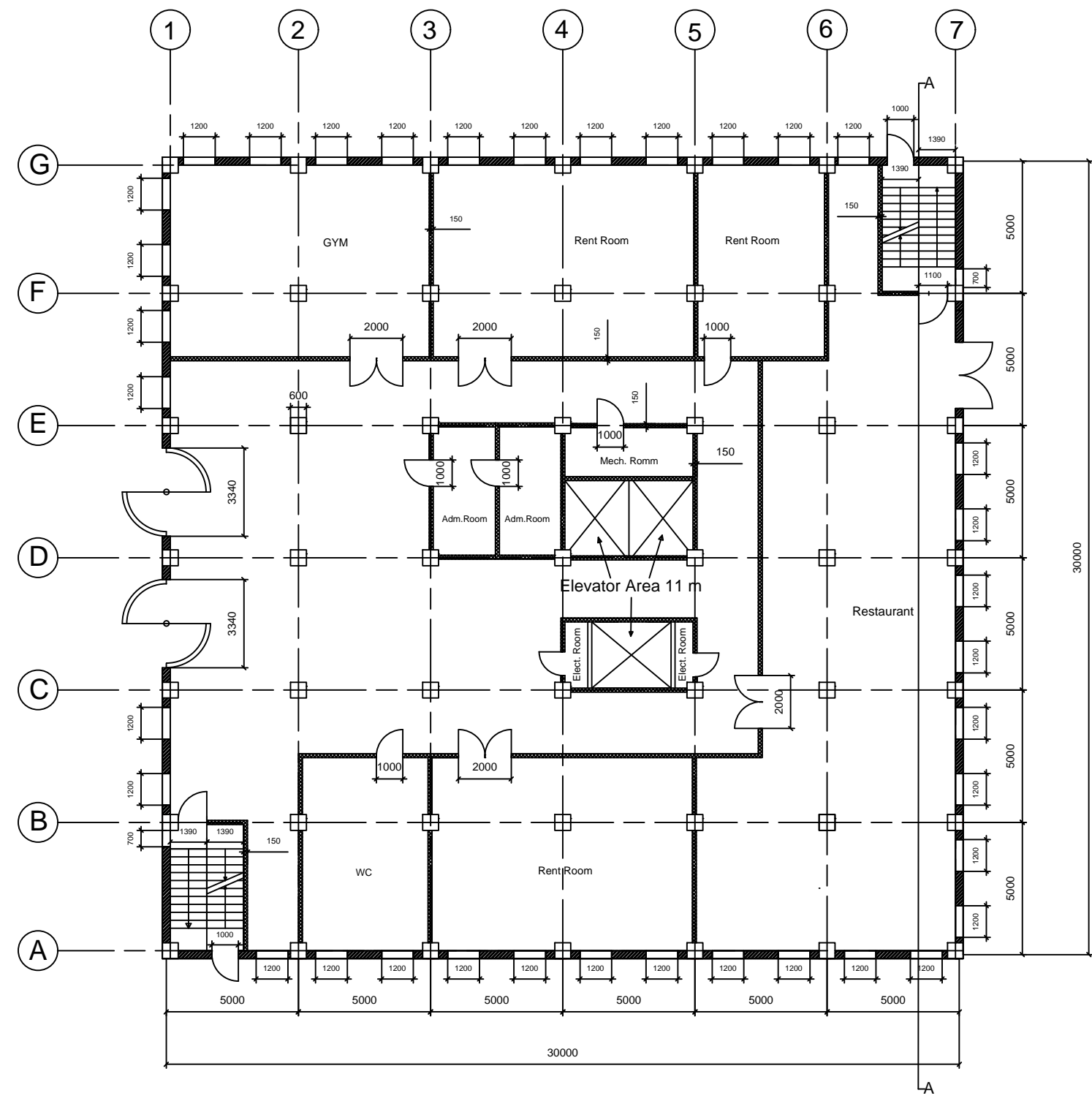
A - 100

Sheet 0 of 8

A1

First Floor Plan

Scale 1:175



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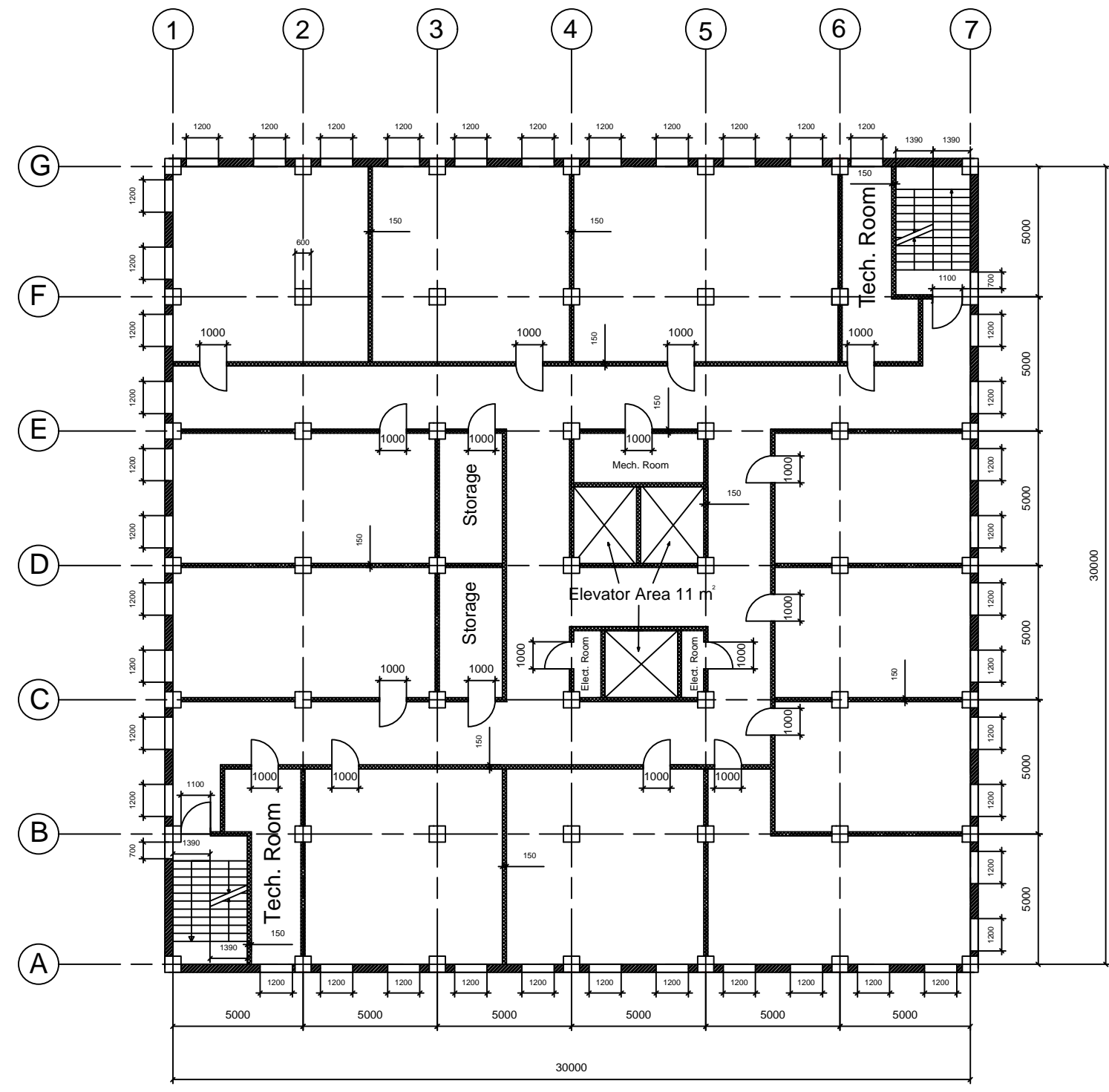
Sheet Title

First Floor Plan

A - 101

Sheet 1 of 8

A2 Typical Floor Plan
Scale 1:175



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Typical Floor Plan

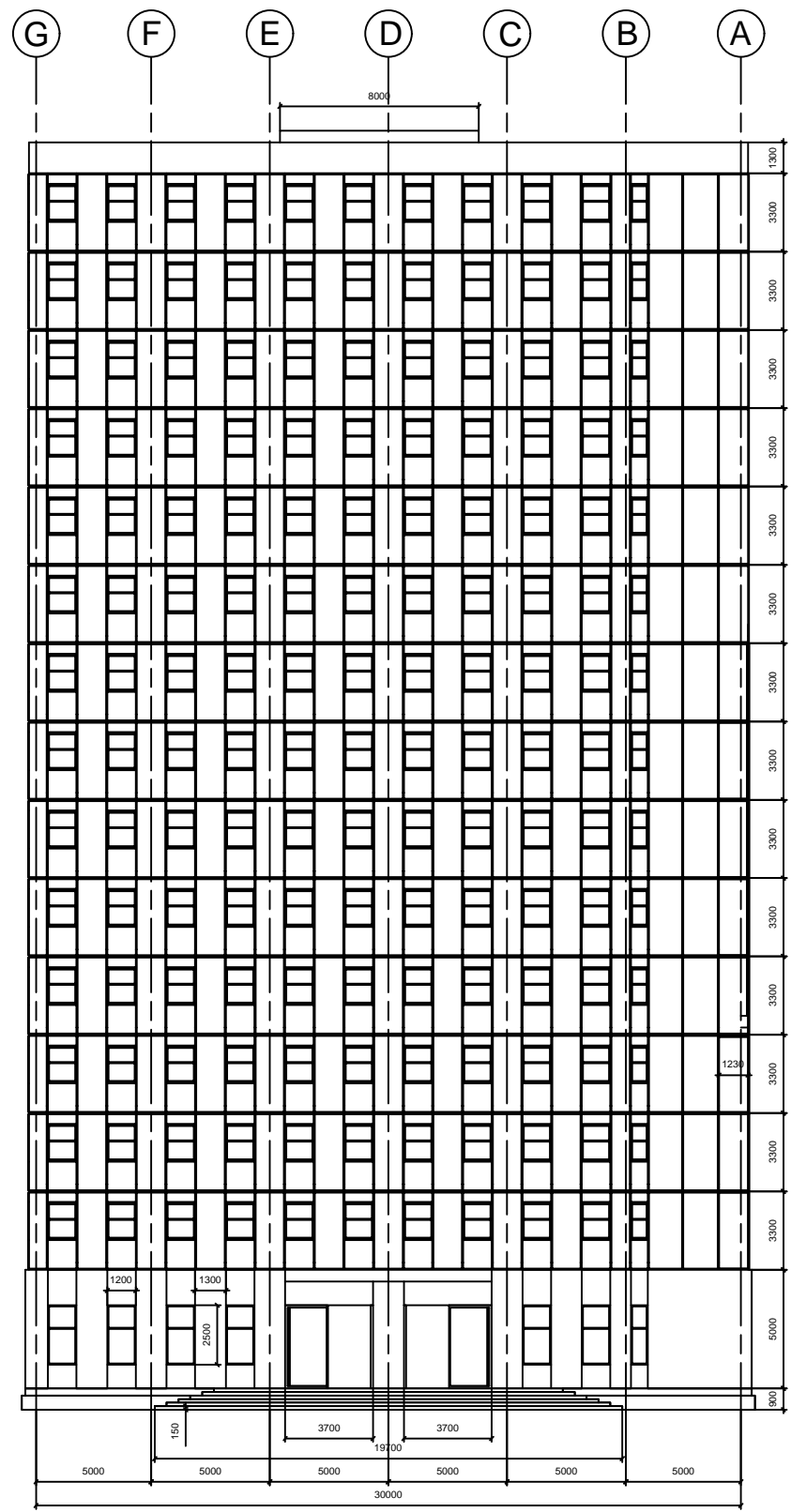
A - 102

Sheet 2 of 8

A3

Front View

Scale 1:250



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Front View

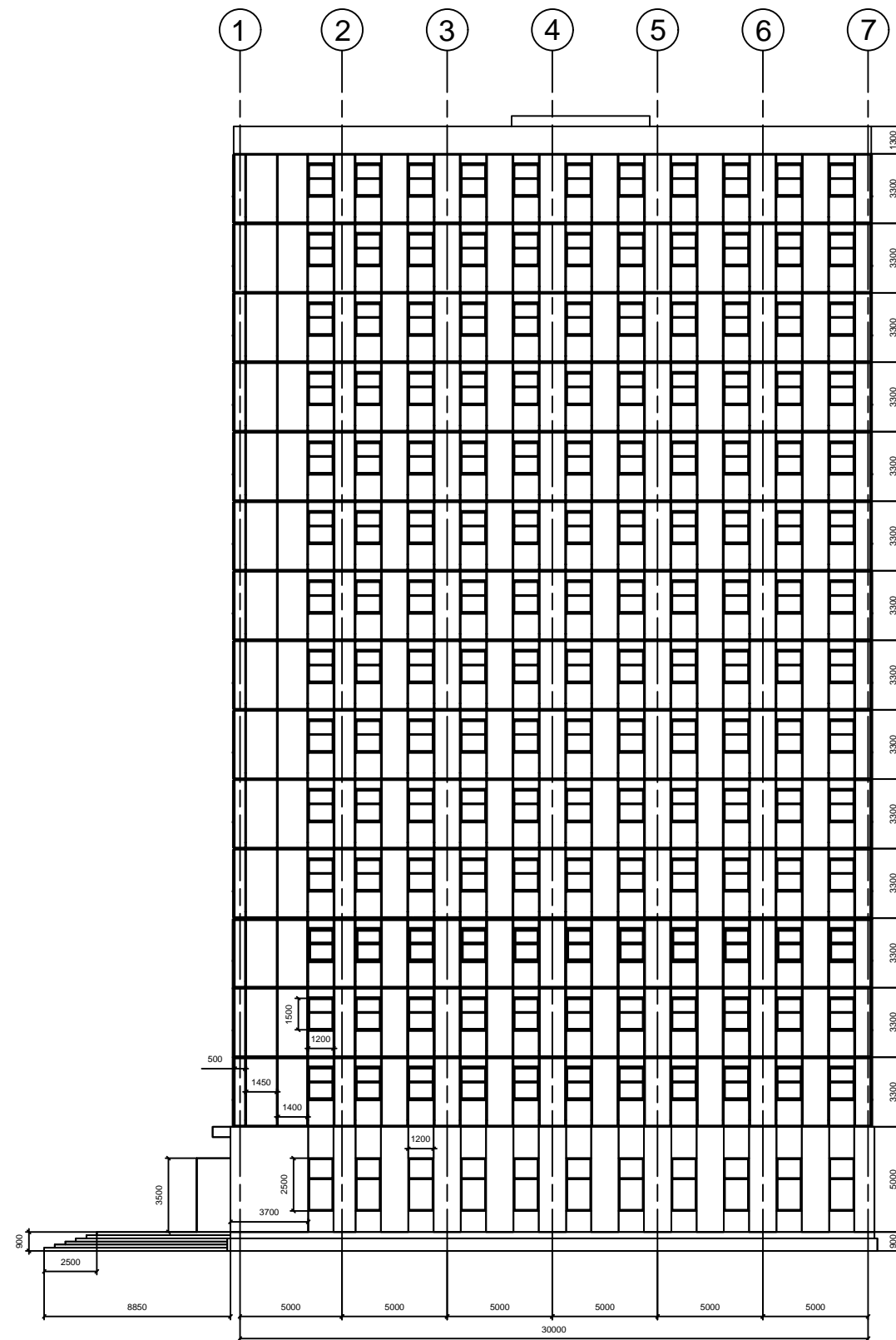
A - 103

Sheet 3 of 8

A4

Right View

Scale 1:250



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Right View

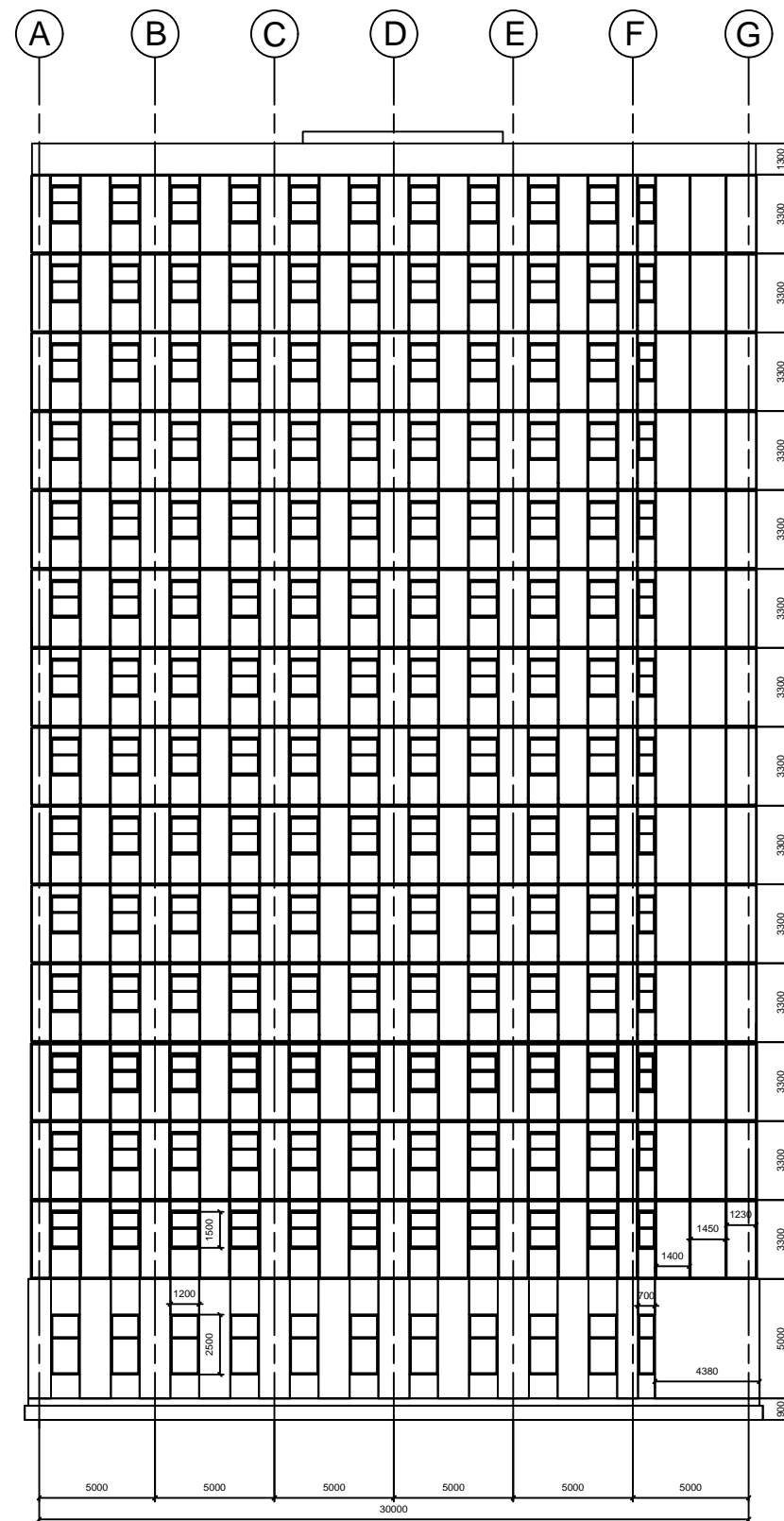
A - 104

Sheet 4 of 8

A5

Back View

Scale 1:250



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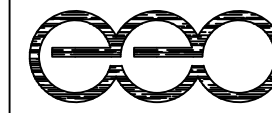
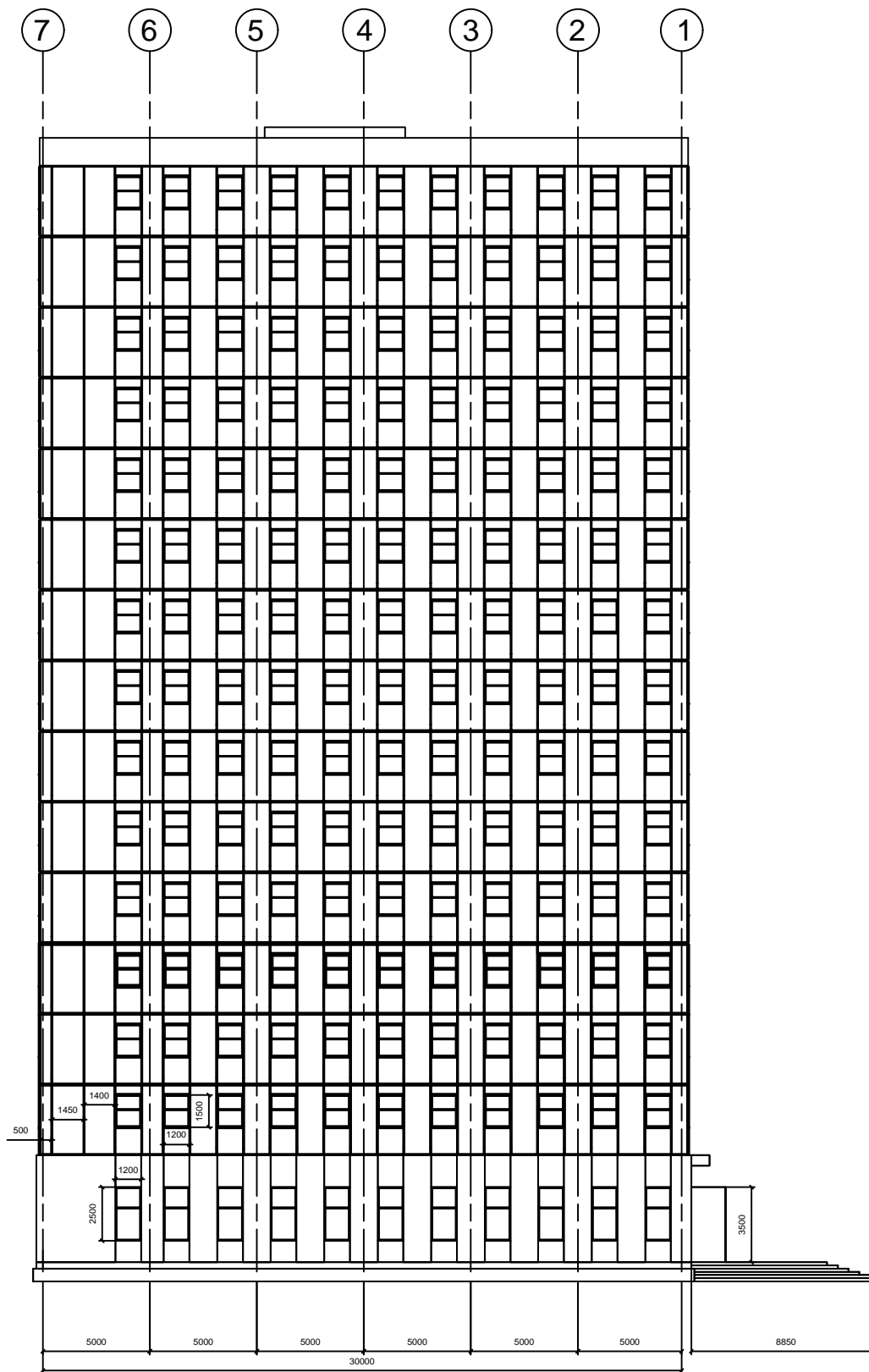
A - 105

Sheet 5 of 8

A6

Left View

Scale 1:250



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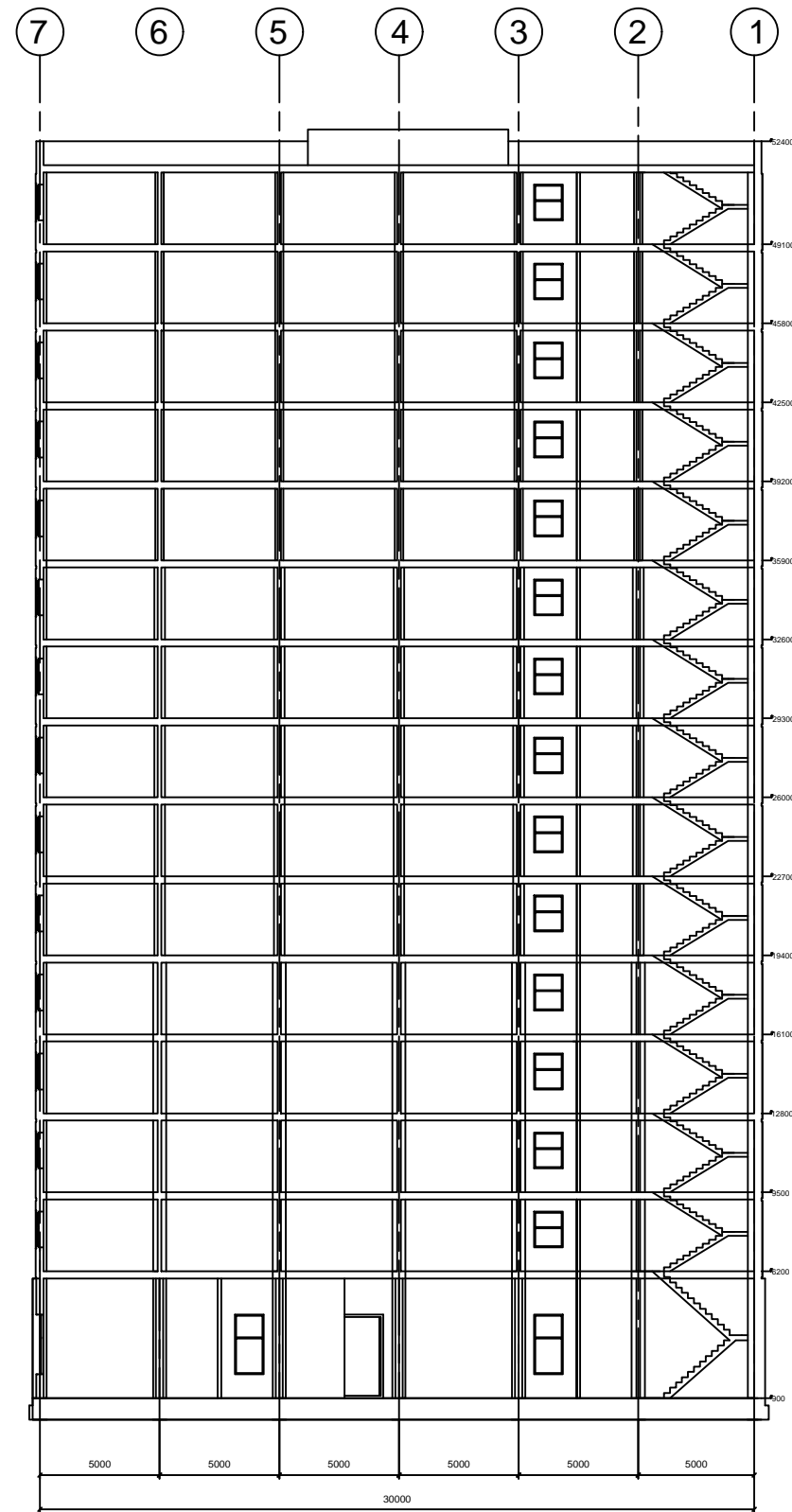
A - 106

Sheet 6 of 8

A7

A-A Section

Scale 1:250



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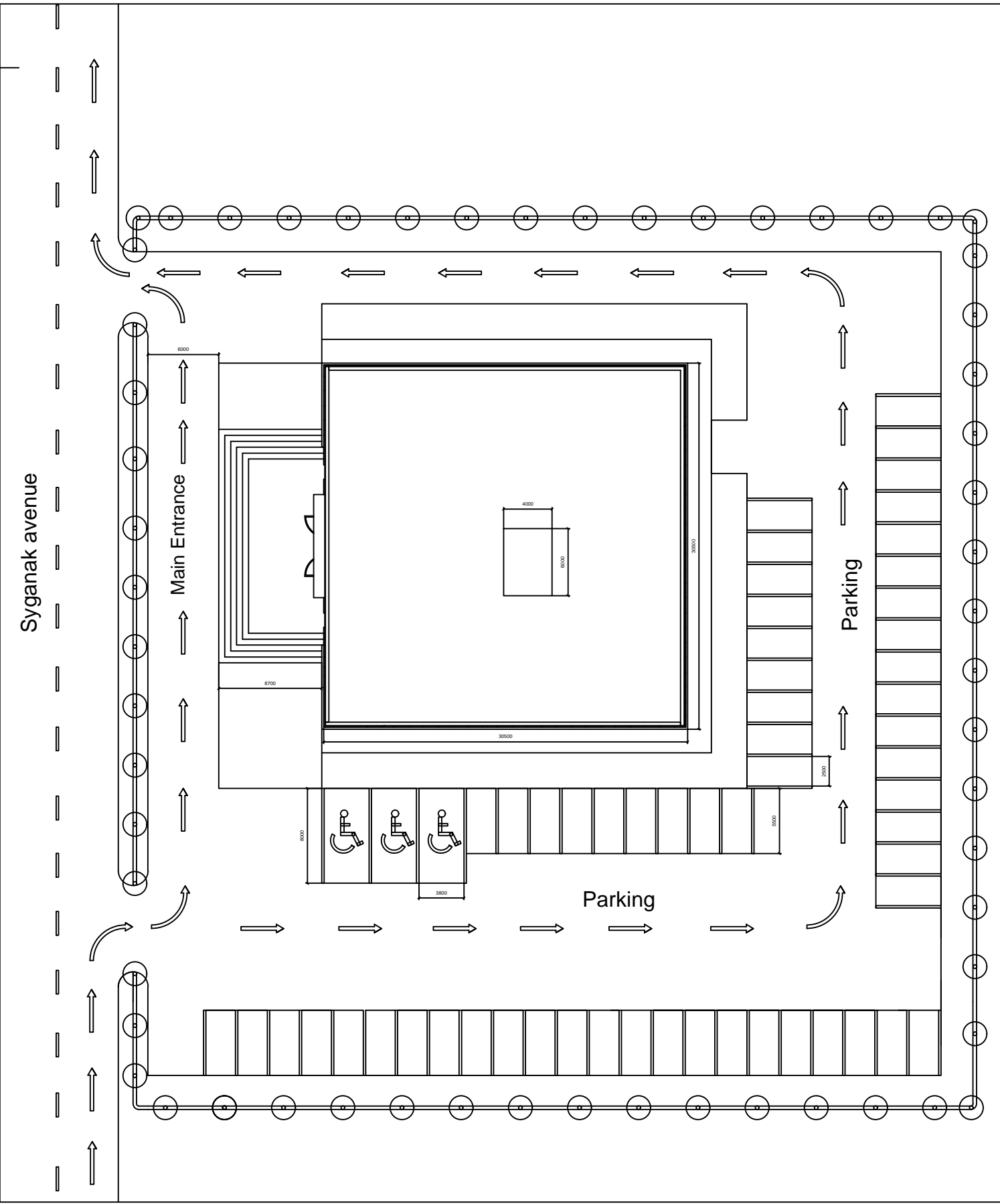
Sheet Title

A-A Section

A - 107

Sheet 7 of 8

A2 **Site Layout**
Scale 1:400



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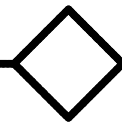
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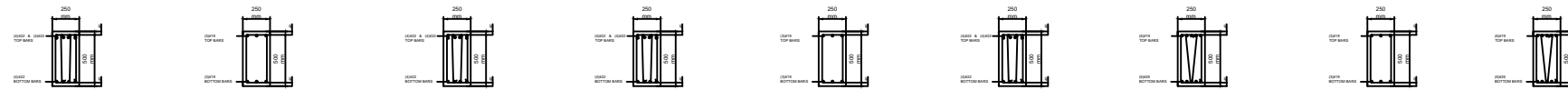
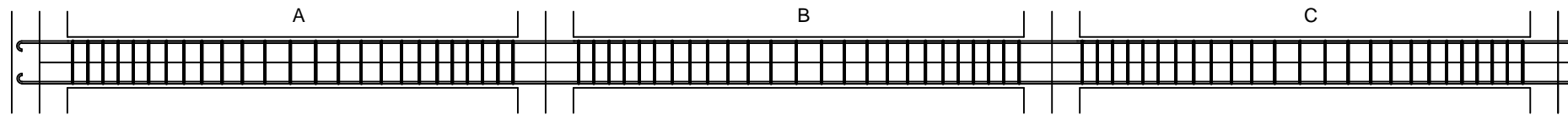
Site Layout

A - 108

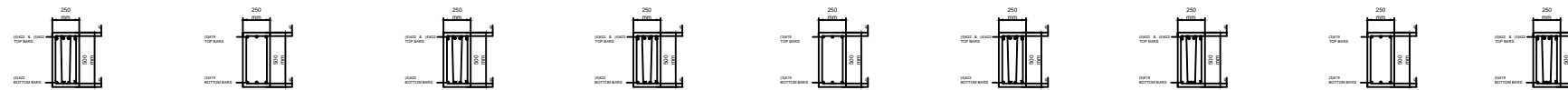
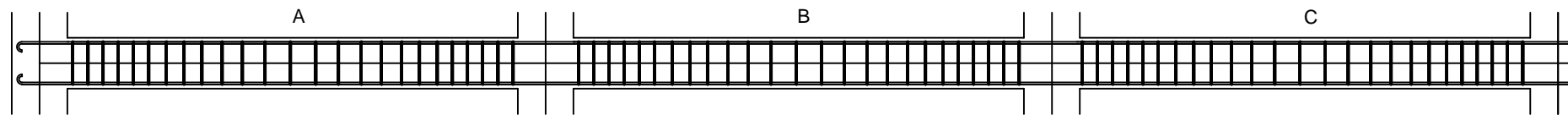
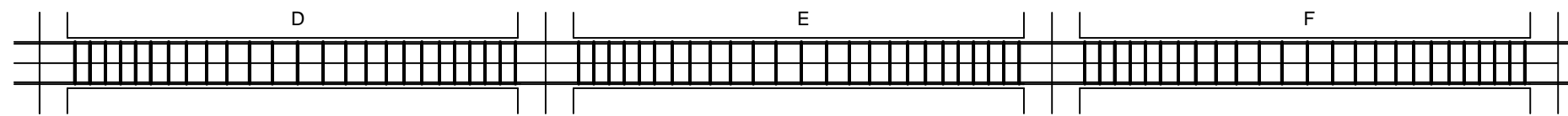
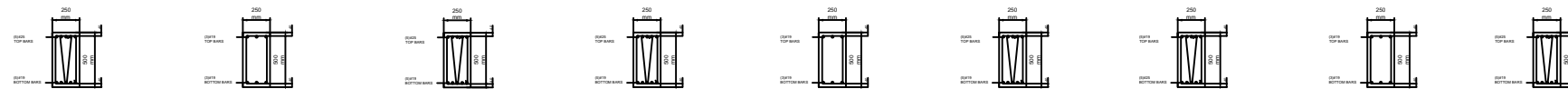
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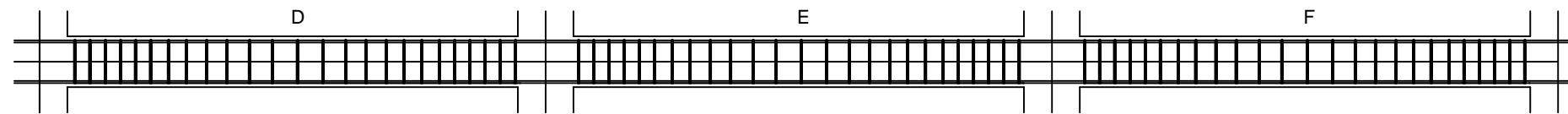
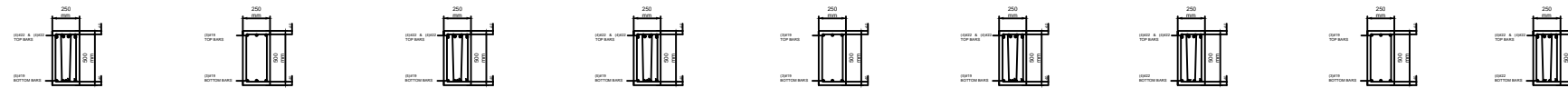
Reinforced Beam Design



LEVEL 1



LEVEL 2



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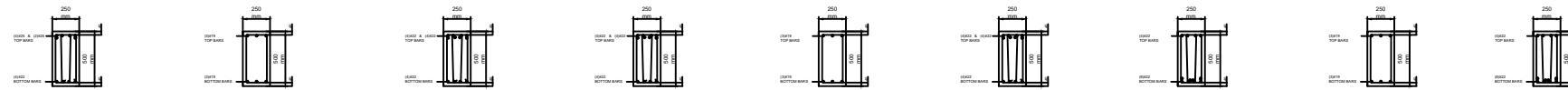
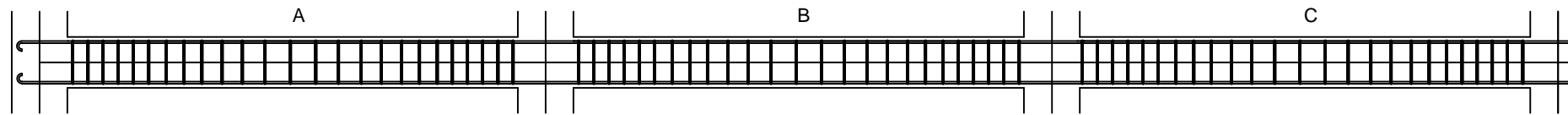
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Reinforced beam
design of levels 1 and 2

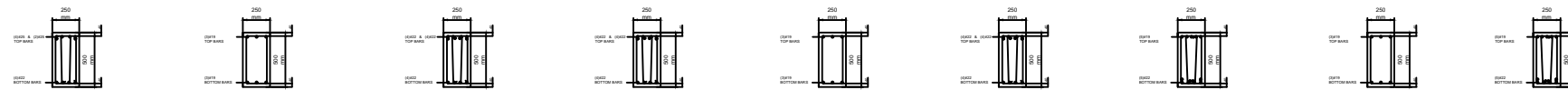
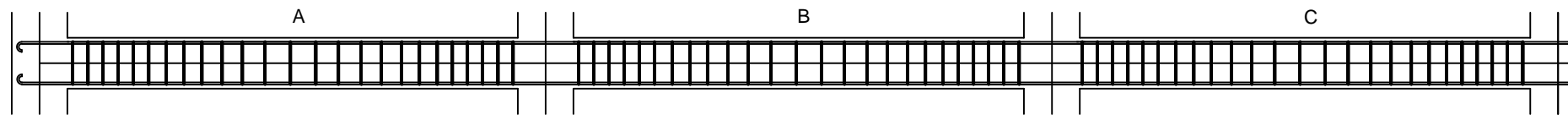
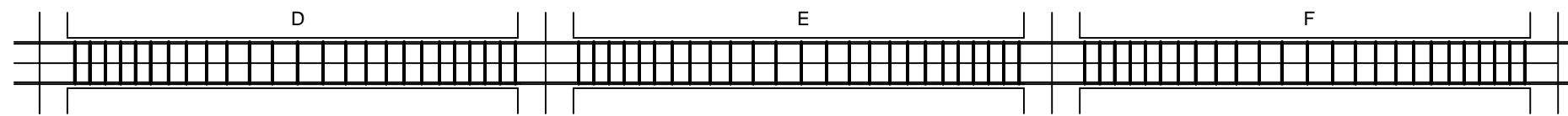
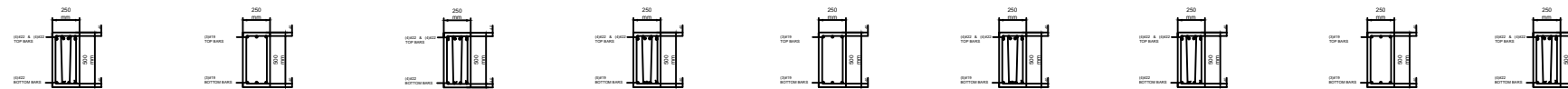
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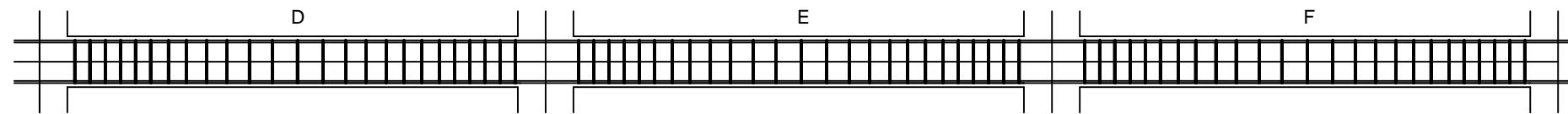
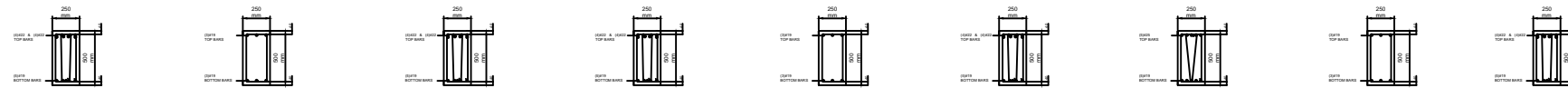
Reinforced Beam Design



LEVEL 3



LEVEL 4



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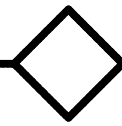
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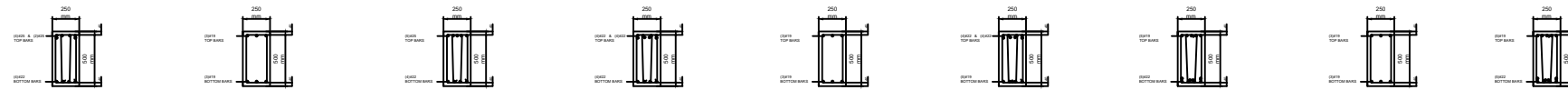
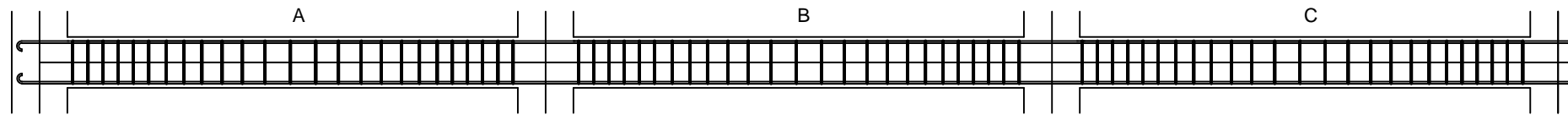
Reinforced beam
design of levels 3 and 4

A - 101

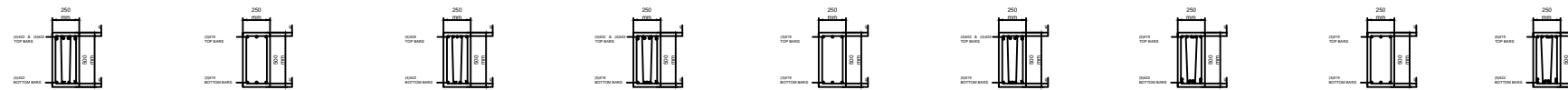
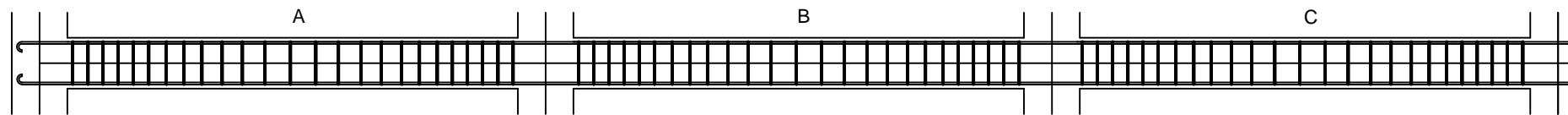
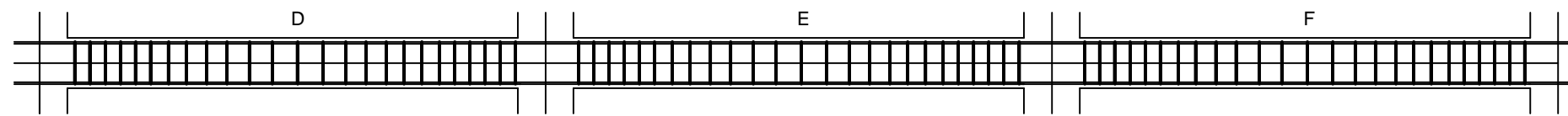
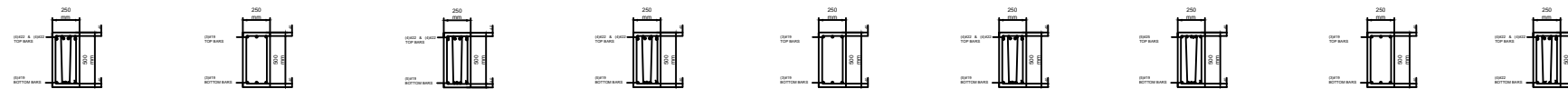
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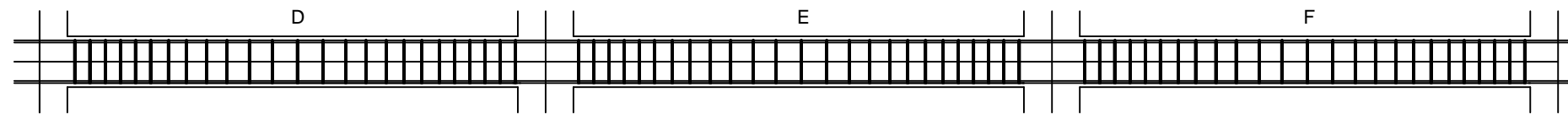
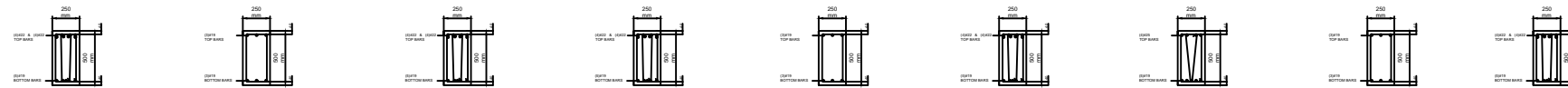
Reinforced Beam Design



LEVEL 5



LEVEL 6



Energy Efficient Construction

Civil Engineering Department

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Shazim Ali Memon
Adviser

Hau Leung
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Jong Kim
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Zauzat Zeken
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Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

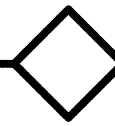
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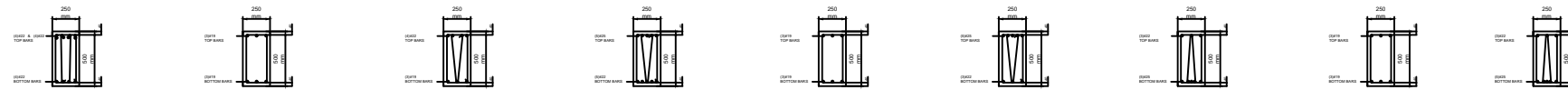
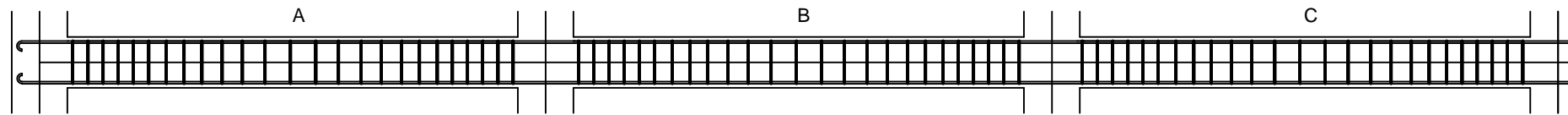
Reinforced beam
design of levels 5 and 6

A - 101

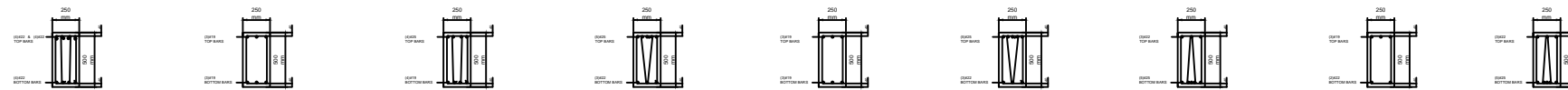
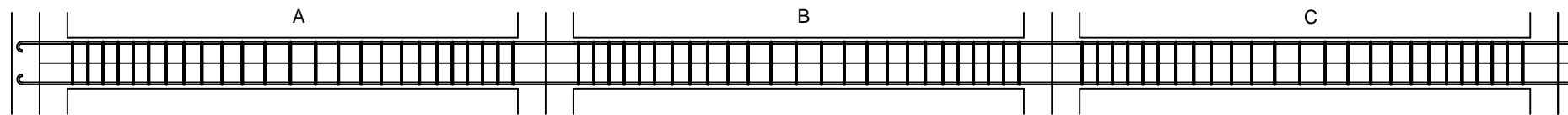
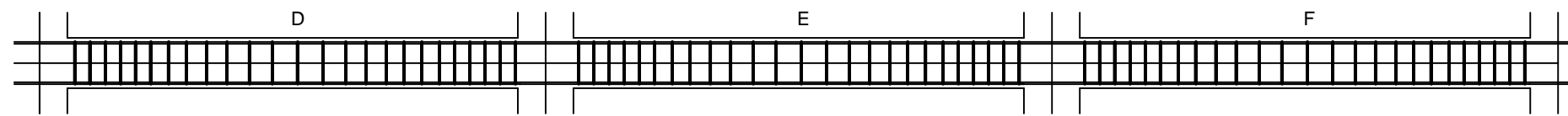
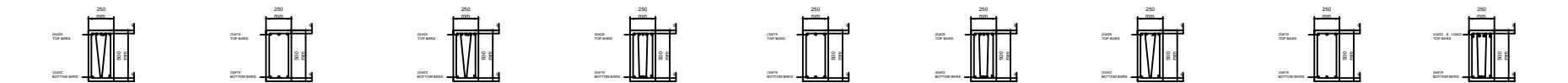
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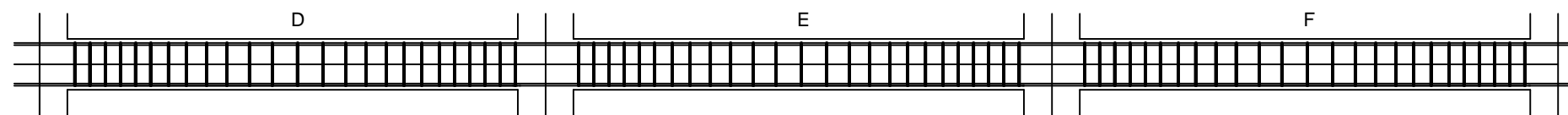
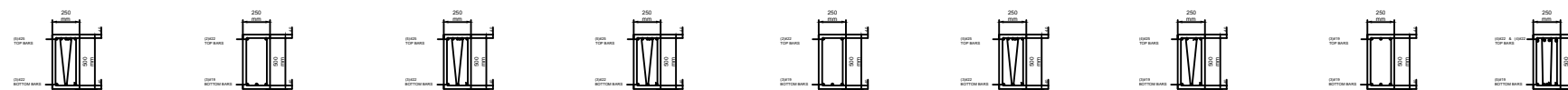
Reinforced Beam Design



LEVEL 7



LEVEL 8



Energy Efficient Construction

Civil Engineering Department

Consultants

Shazim Ali Memon
Adviser

Hau Leung
Marker

Jong Kim
Marker

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Zauzat Zeken
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Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

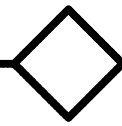
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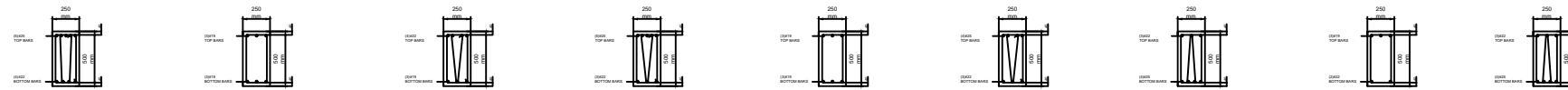
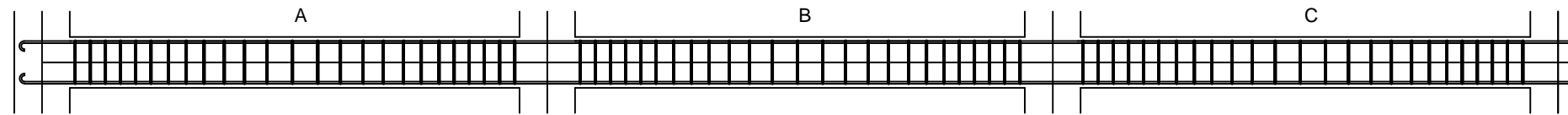
Reinforced beam
design of levels 7 and 8

A - 101

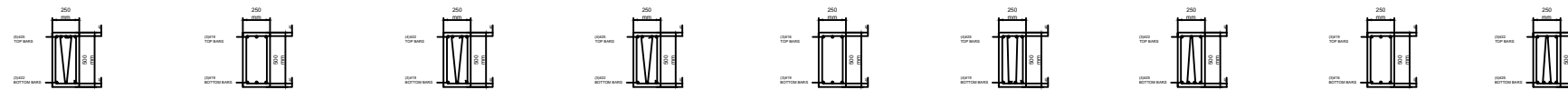
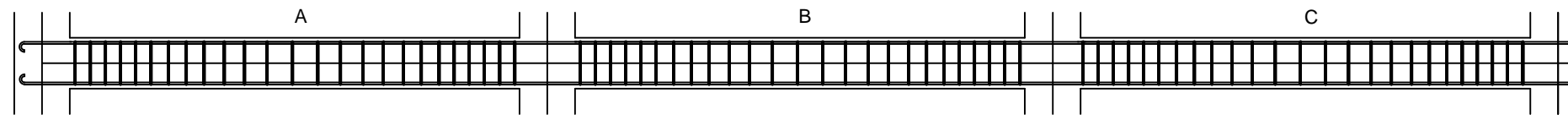
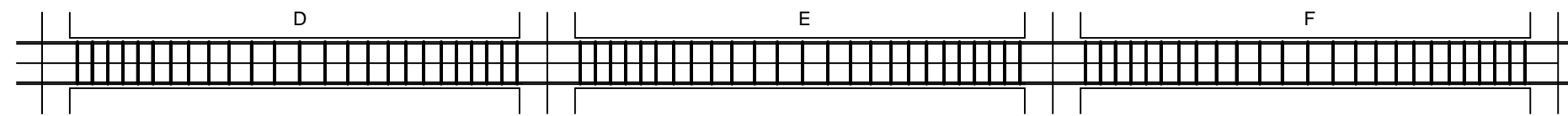
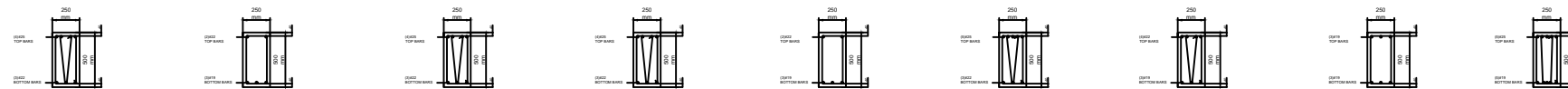
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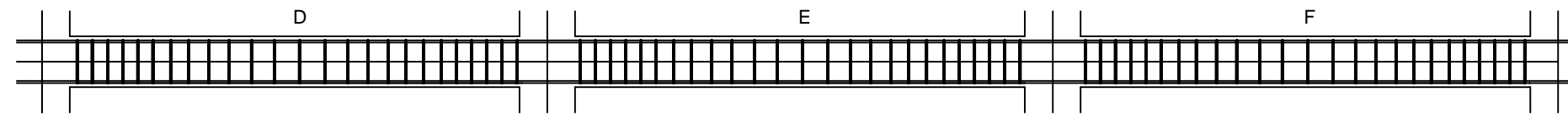
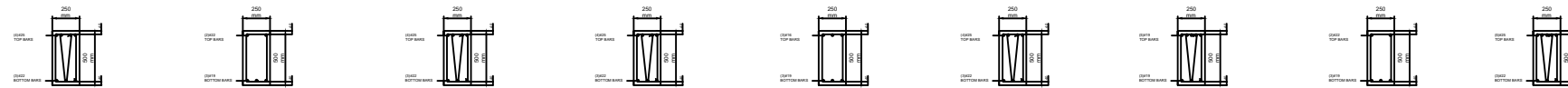
Reinforced Beam Design



LEVEL 9



LEVEL 10



Energy Efficient Construction

Civil Engineering Department

Consultants

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Adviser

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Marker

Jong Kim
Marker

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201201930

Zauzat Zeken
201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

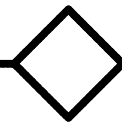
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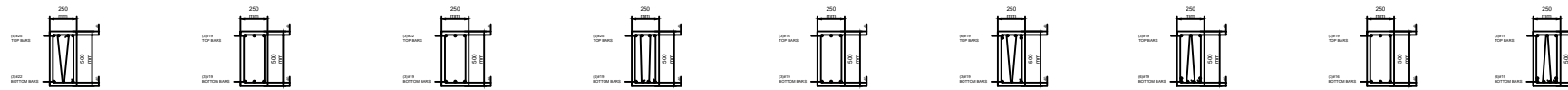
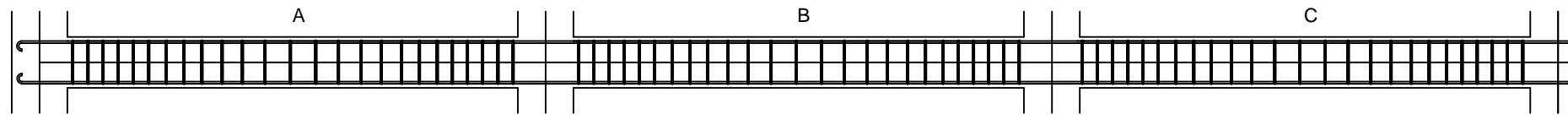
Reinforced beam design
of levels 9 and 10

A - 101

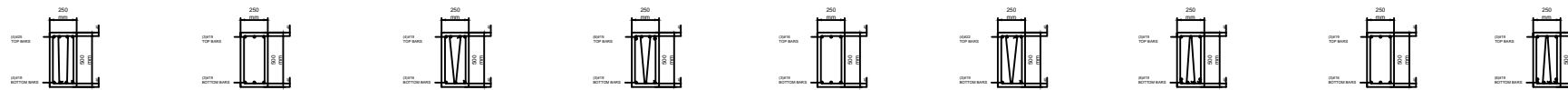
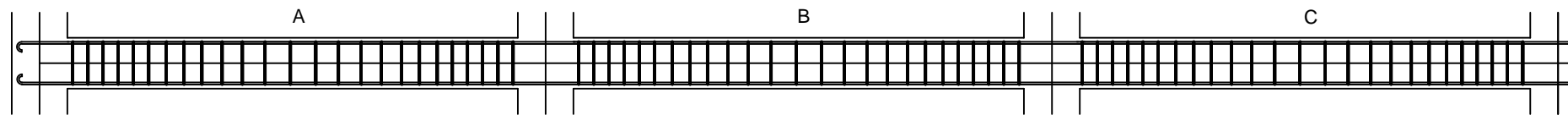
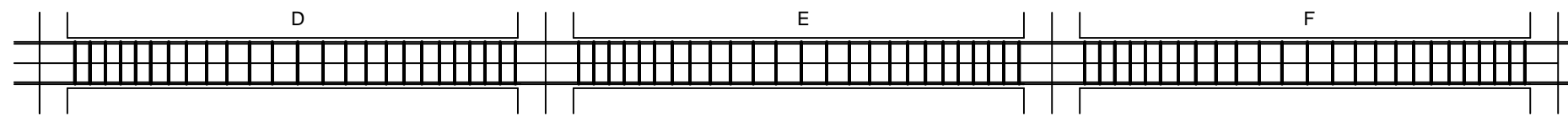
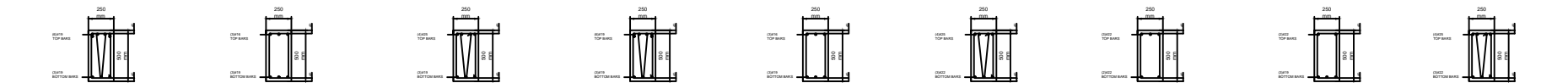
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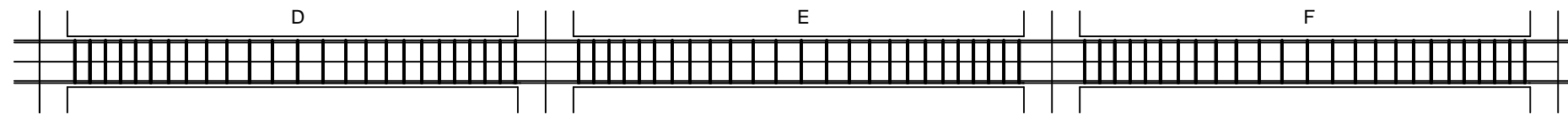
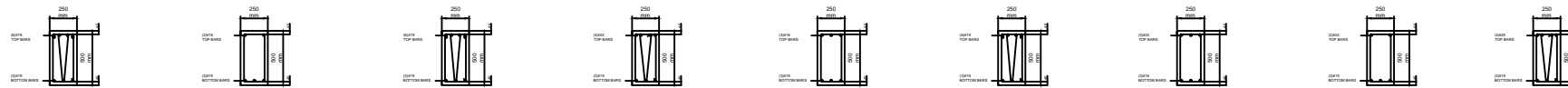
Reinforced Beam Design



LEVEL 11



LEVEL 12



Energy Efficient Construction

Civil Engineering Department

Consultants

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Jong Kim
Marker

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Zauzat Zeken
201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

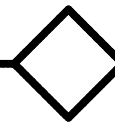
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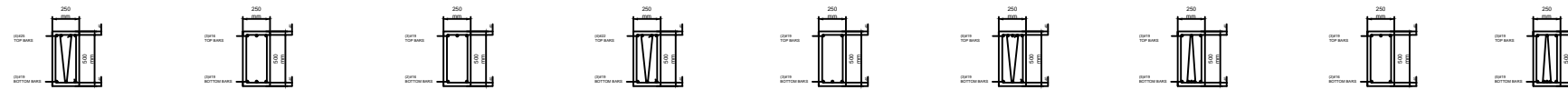
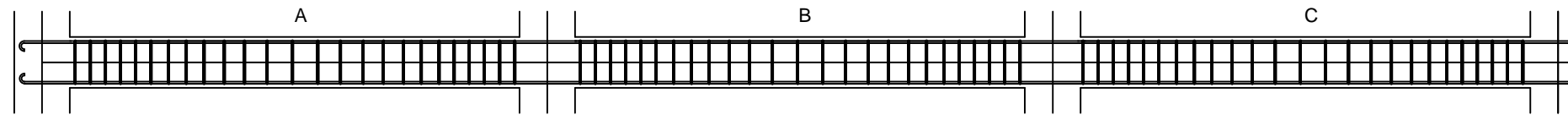
Reinforced beam design
of levels 11 and 12

A - 101

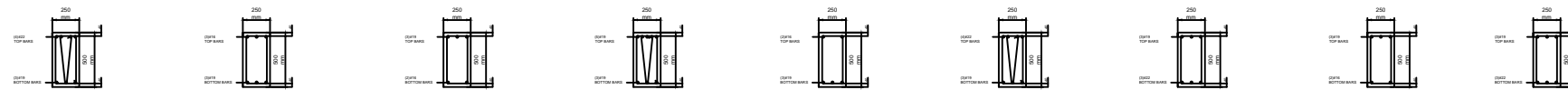
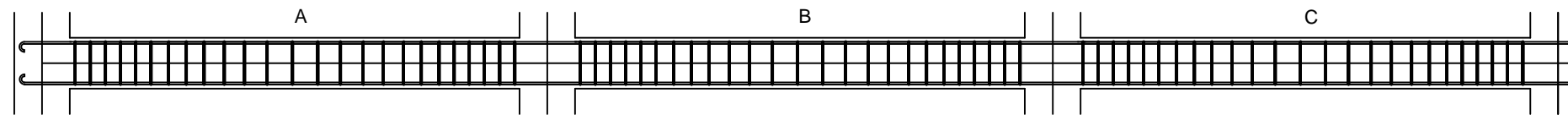
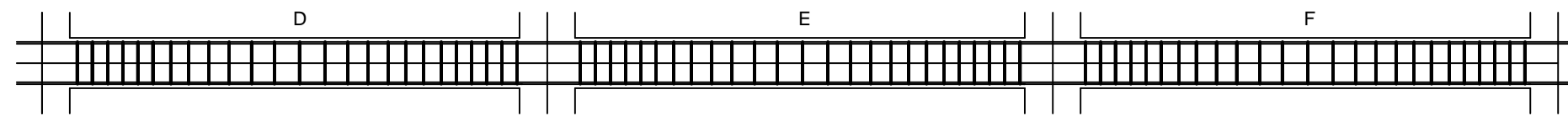
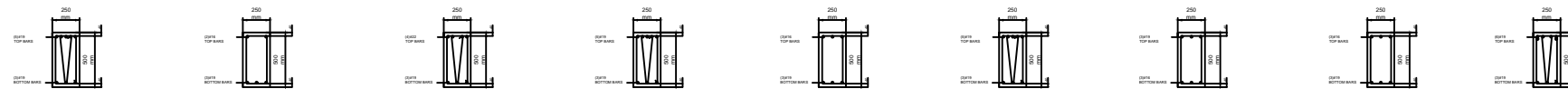
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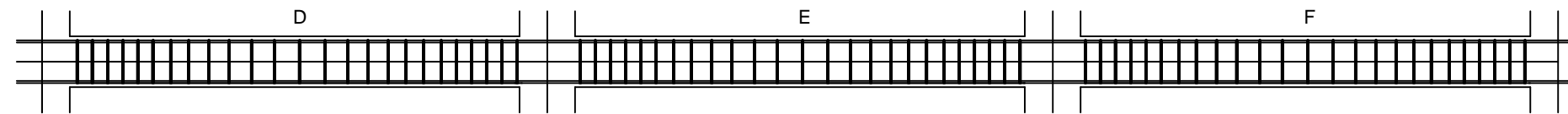
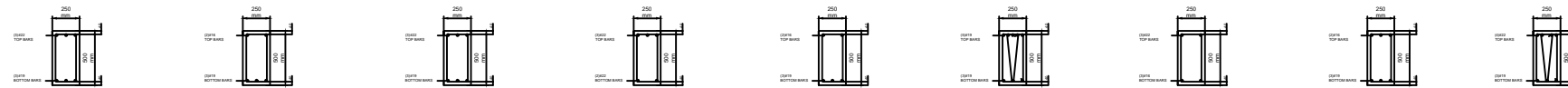
Reinforced Beam Design



LEVEL 13



LEVEL 14



Energy Efficient Construction

Civil Engineering Department

Consultants

Shazim Ali Memon
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Marker

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201201930

Zauzat Zeken
201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

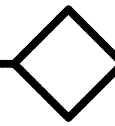
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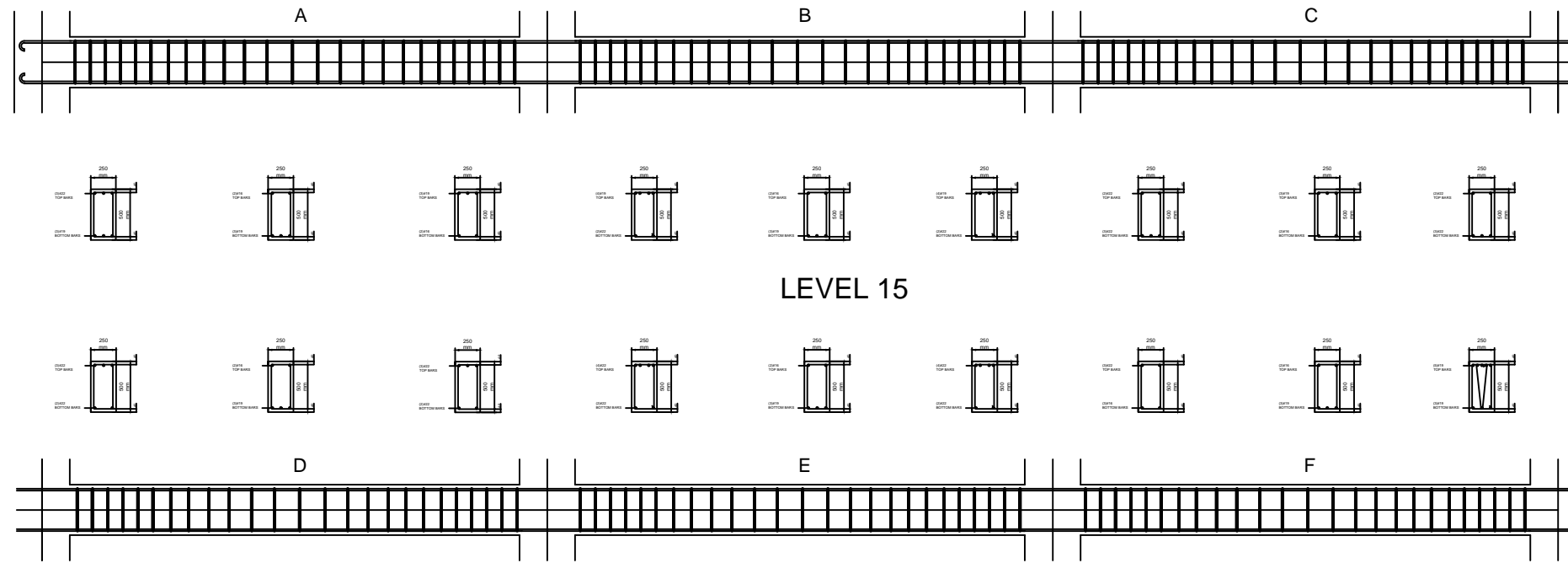
Reinforced beam design
of levels 13 and 14

A - 101

Sheet 7



Reinforced Beam Design



LEVEL 15



Energy Efficient Construction

Civil Engineering Department

Consultants

Shazim Ali Memon
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201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

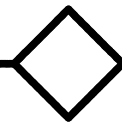
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Sheet Title

Reinforced beam design
of levels 15

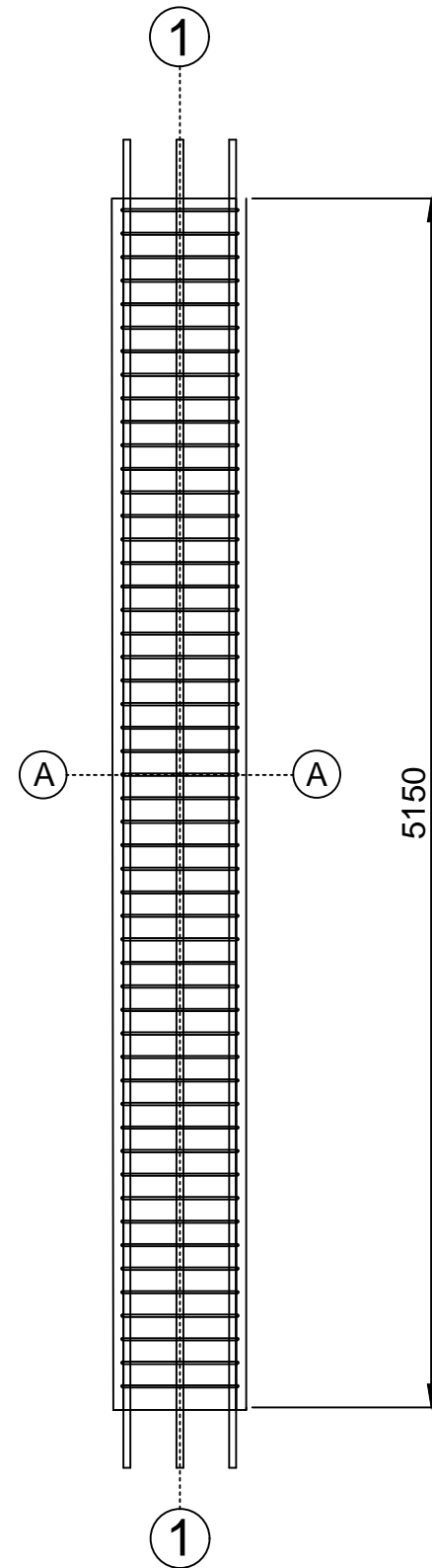
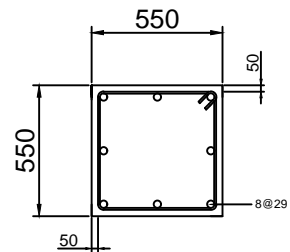
A - 101

Sheet 8



Column Design C1-1

Section A-A



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Marker

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Zauzat Zeken
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Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

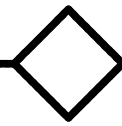
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Column design 1

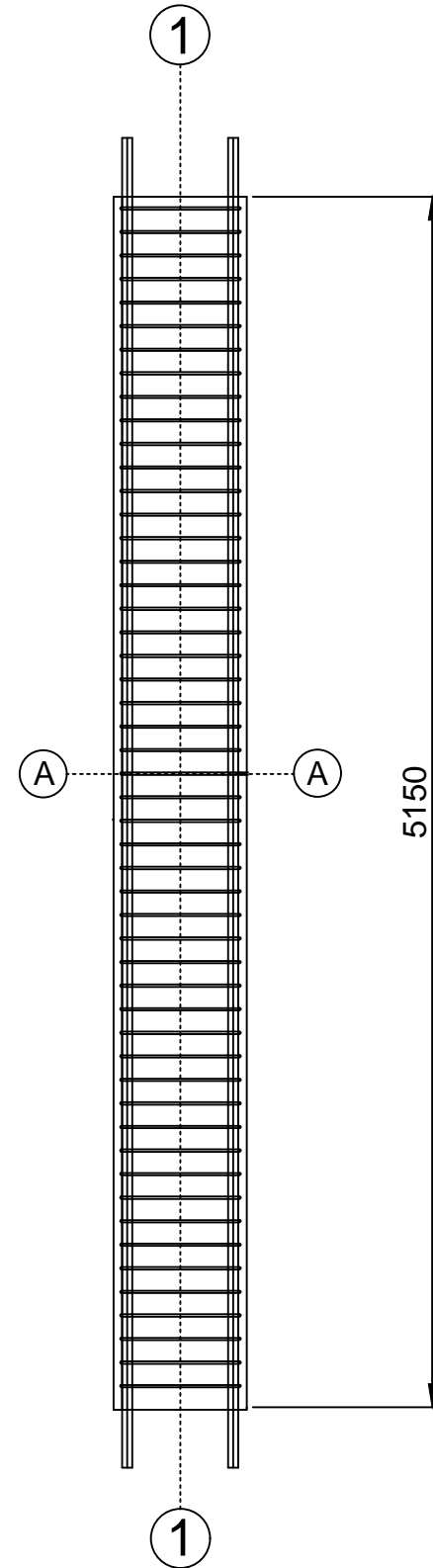
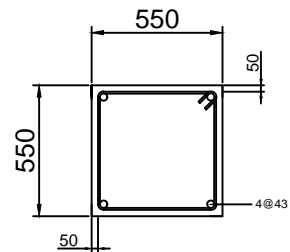
A - 101

Sheet 1



Column Design C1-2, C1-3, C1-5

Section A-A



Energy Efficient Construction

Civil Engineering Department

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201201930

Zauzat Zeken
201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

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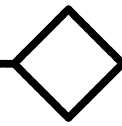
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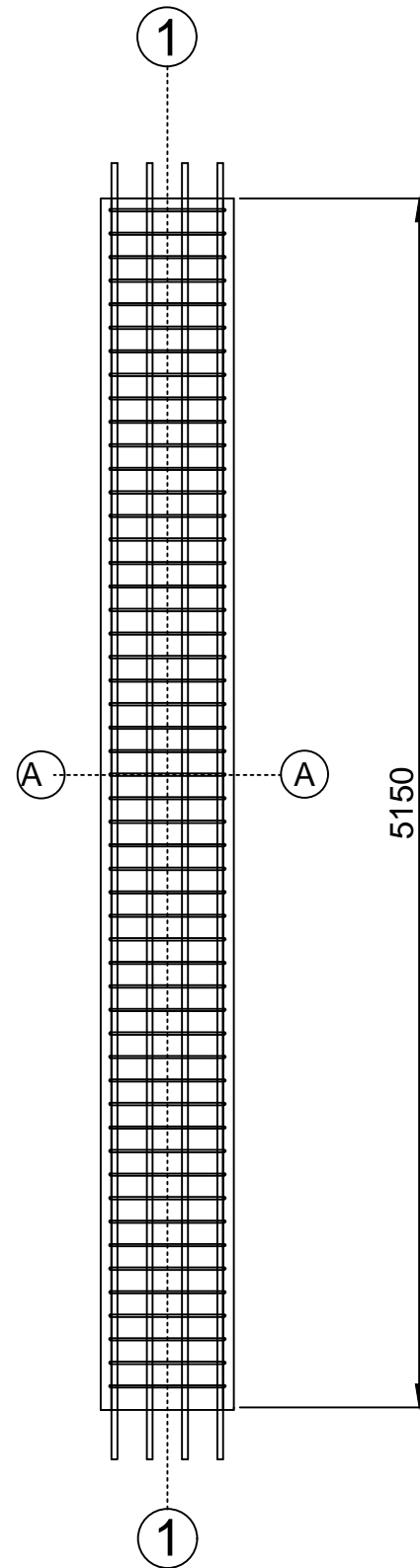
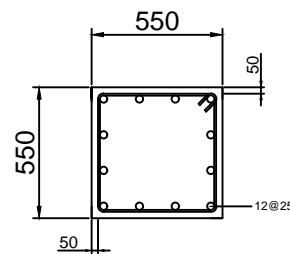
A - 101

Sheet 2



Column Design C1-4, C1-6, C1-7

Section A-A



Energy Efficient Construction

Civil Engineering Department

Consultants

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Marker

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Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

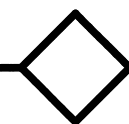
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Sheet Title

Column design 3

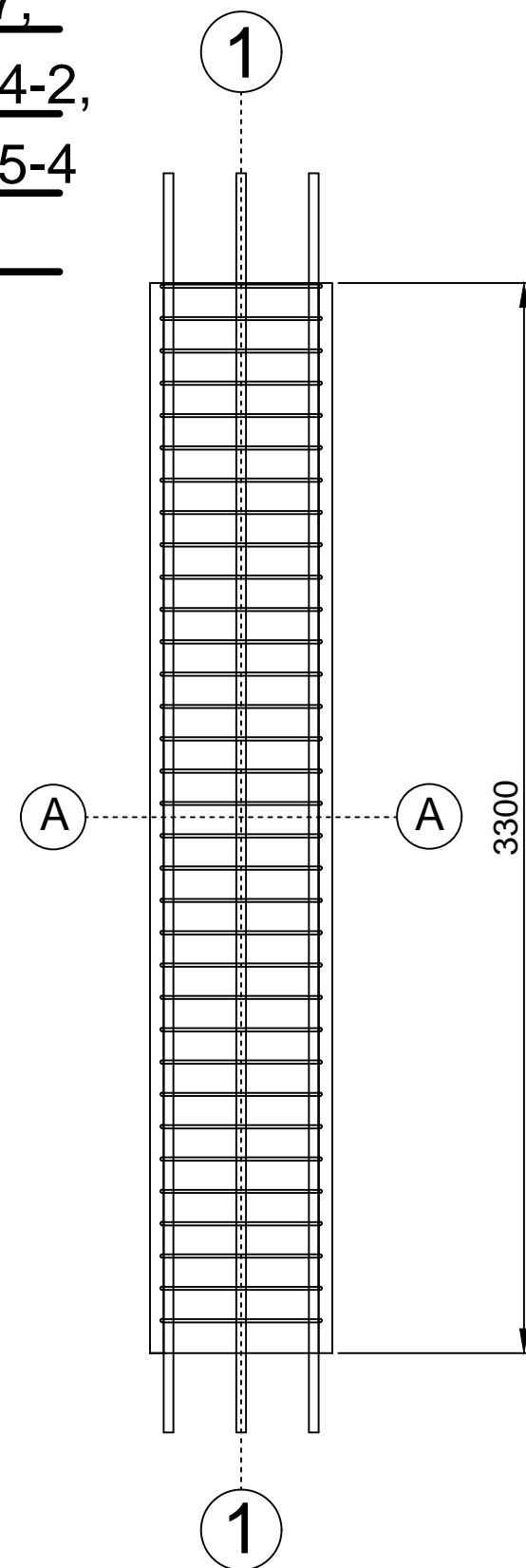
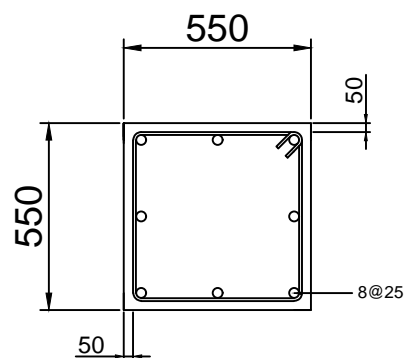
A - 101

Sheet 3



Column Design C2-1, C2-2, C2-3, C2-6, C2-7,
C3-1, C3-2, C3-3, C3-5, C3-6, C3-7, C4-1, C4-2,
C4-3, C4-4, C4-5, C4-6, C5-1, C5-2, C5-3, C5-4
C5-5, C5-6

Section A-A



Energy Efficient Construction

Civil Engineering Department

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Marker

Jong Kim
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201201930

Zauzat Zeken
201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

Checked by: Amire Anuarbek

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Energy Efficient Compacy

Sheet Title

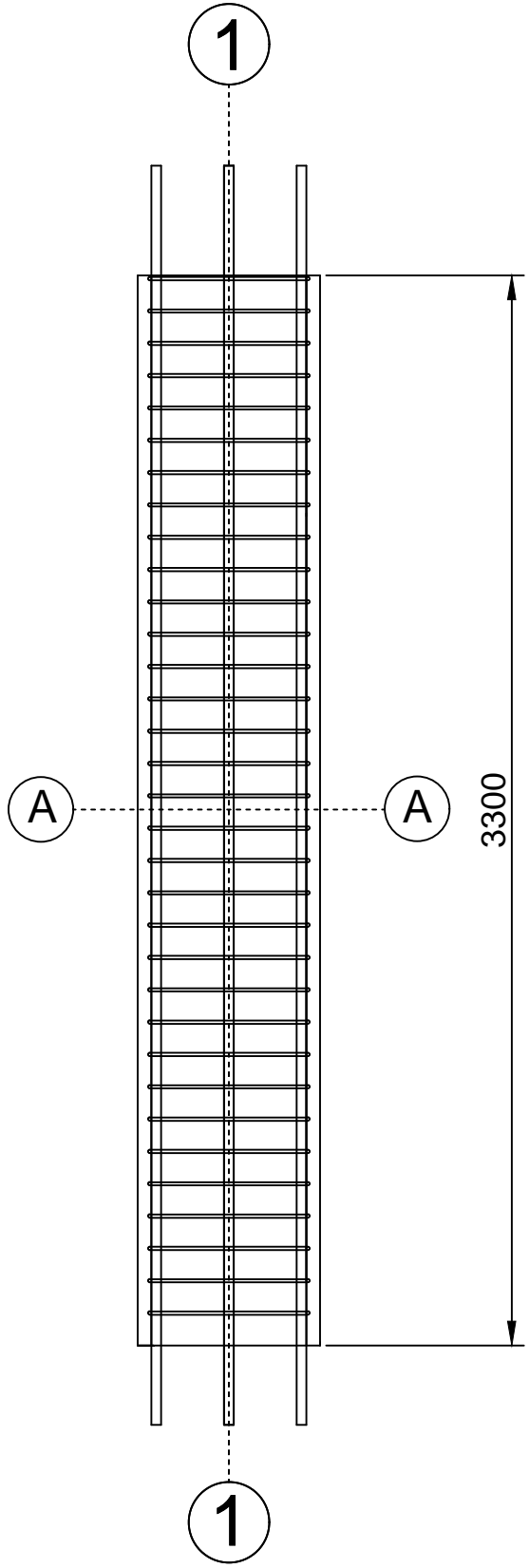
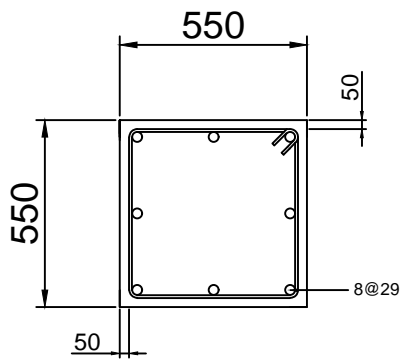
Column design 4

A - 101

Sheet 4

Column Design C2-4, C3-4,

Section A-A



Energy Efficient Construction

Civil Engineering Department

Consultants

Shazim Ali Memon
Adviser

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Marker

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Marker

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Zarina Zharaspayeva
201201930

Zauzat Zeken
201202909

Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

CAT dwg File: first floor.dwg

Drawn by: Nurassyl Battalgazy

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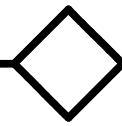
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Energy Efficient Compacy

Sheet Title

Column design 5

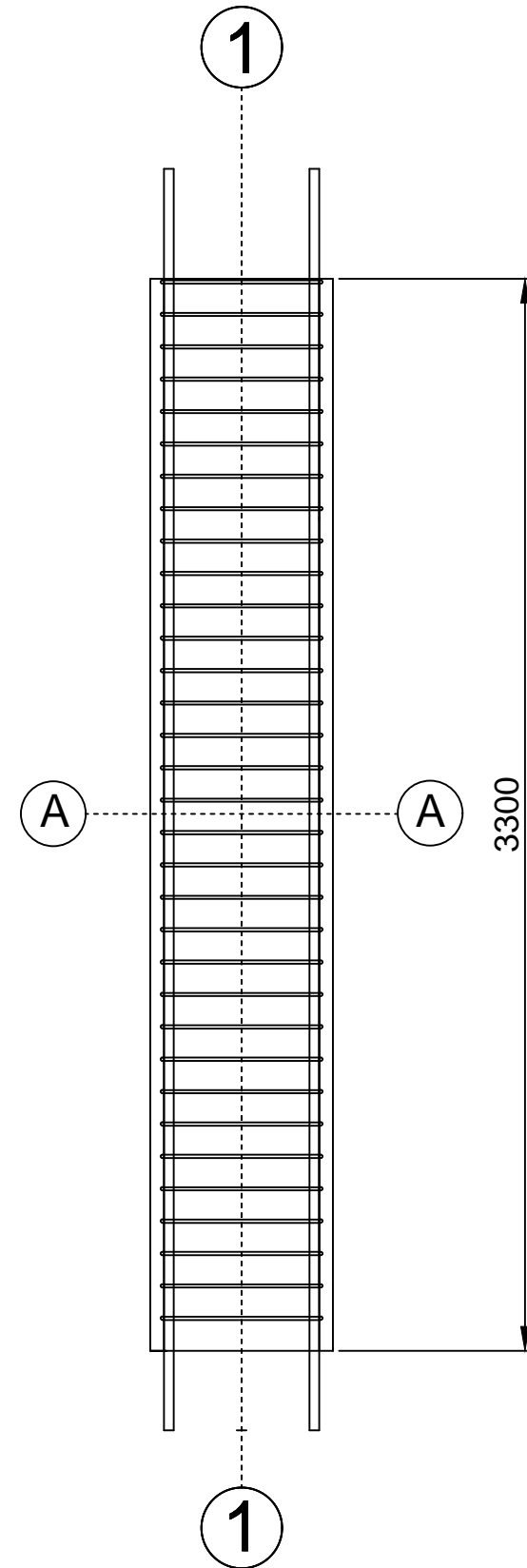
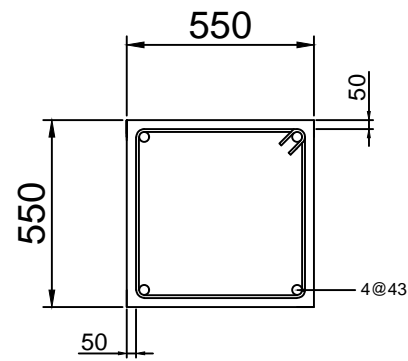
A - 101

Sheet 5



Column Design C2-5,

Section A-A



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Project

Utopia Hotel Astana
15-story hotel building

Astana, Kazakhstan

Project NO: 001

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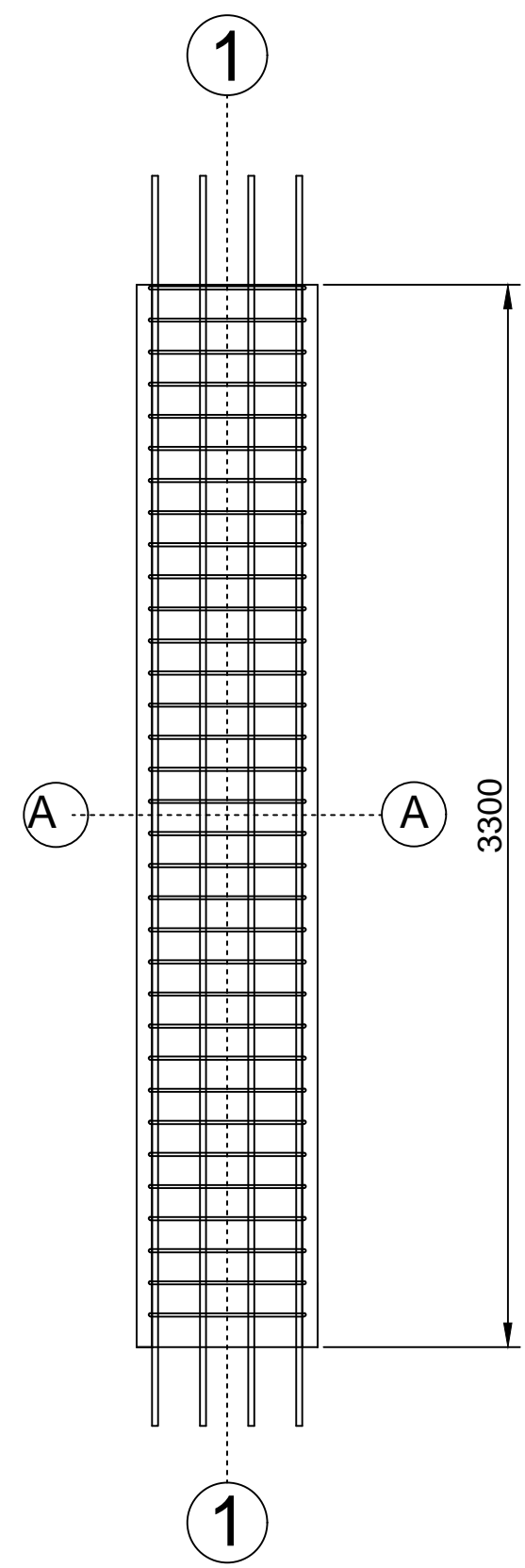
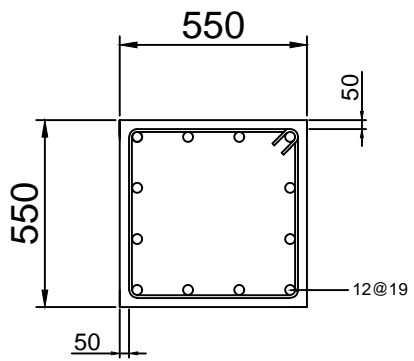
Column design 6

A - 101

Sheet 6

Column Design C4-7, C5-7,

Section A-A



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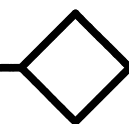
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Column design 7

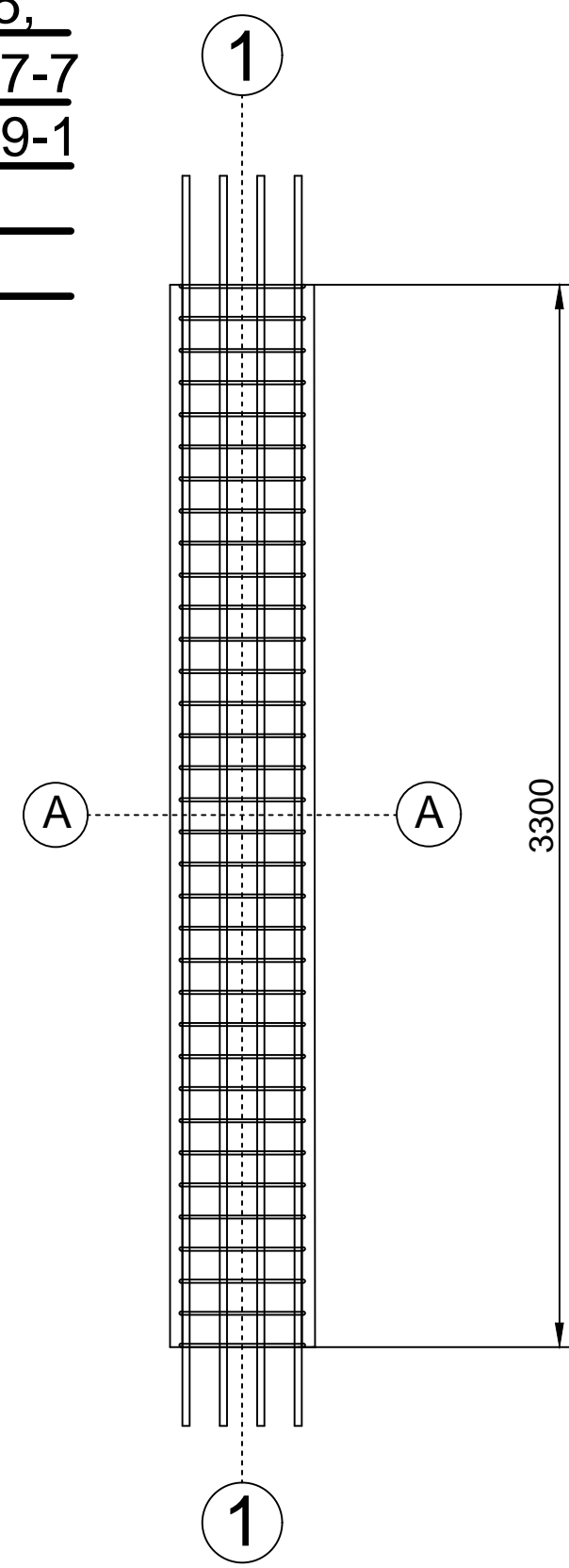
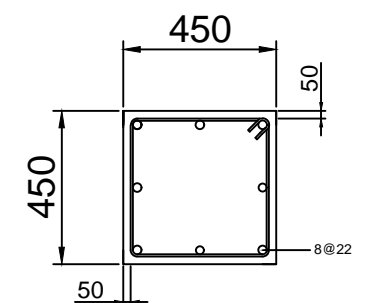
A - 101

Sheet 7



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C8-1, C8-2, C8-3, C8-4, C8-5, C8-6, C8-7, C9-1
C9-2, C9-3, C9-4, C9-5, C9-6, C9-7, C10-1,
C10-2, C10-3, C10-5, C10-6, C10-7

Section A-A



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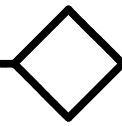
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Column design 8

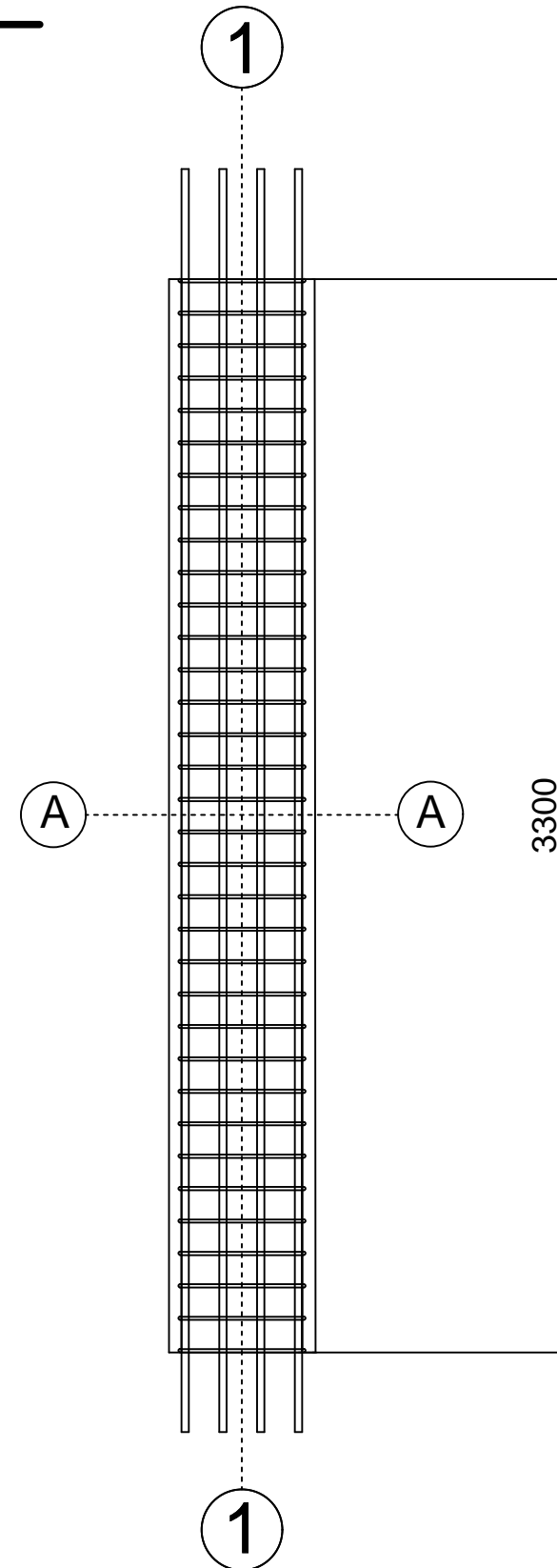
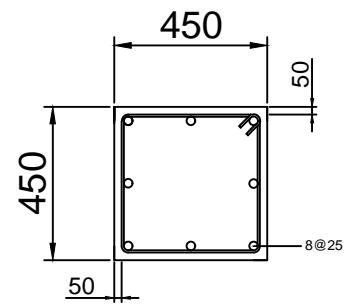
A - 101

Sheet 8



Column Design C10-4,

Section A-A



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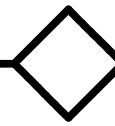
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Sheet Title

Column design 9

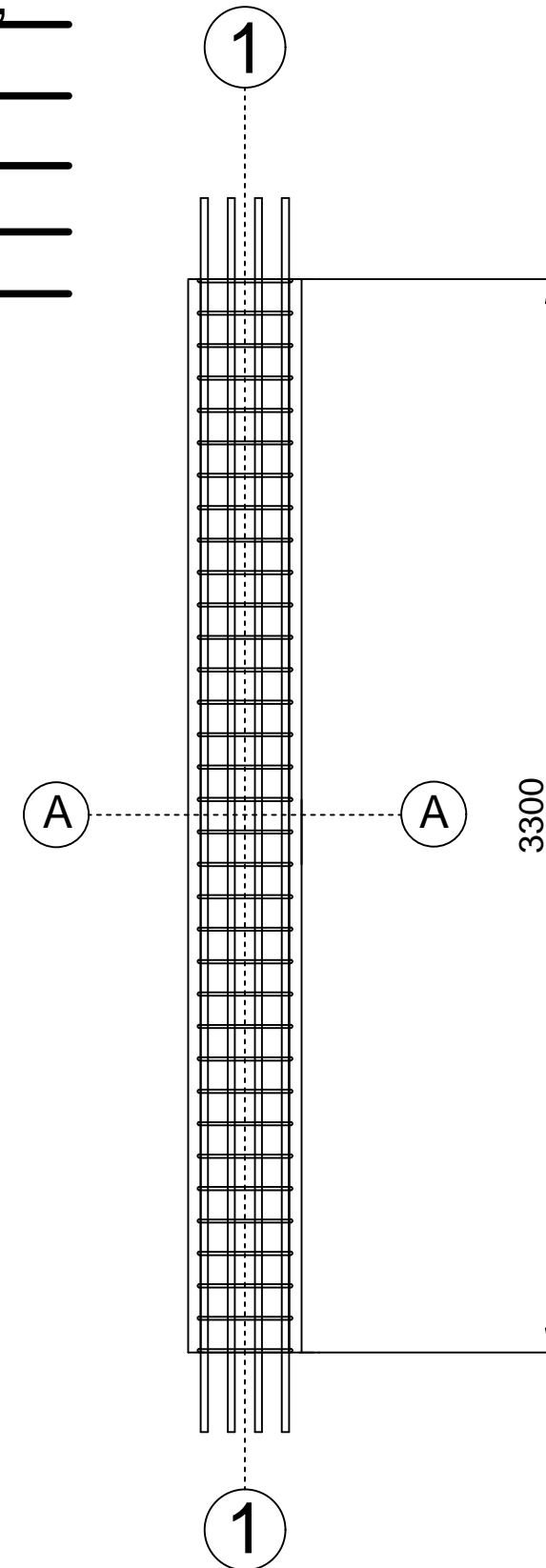
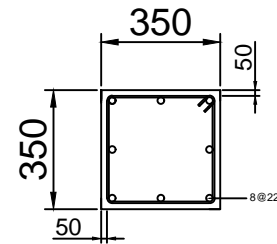
A - 101

Sheet 9



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 C14-2, C14-3, C14-4, C14-5, C14-6, C14-7,

Section A-A



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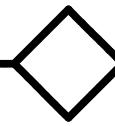
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Column design 10

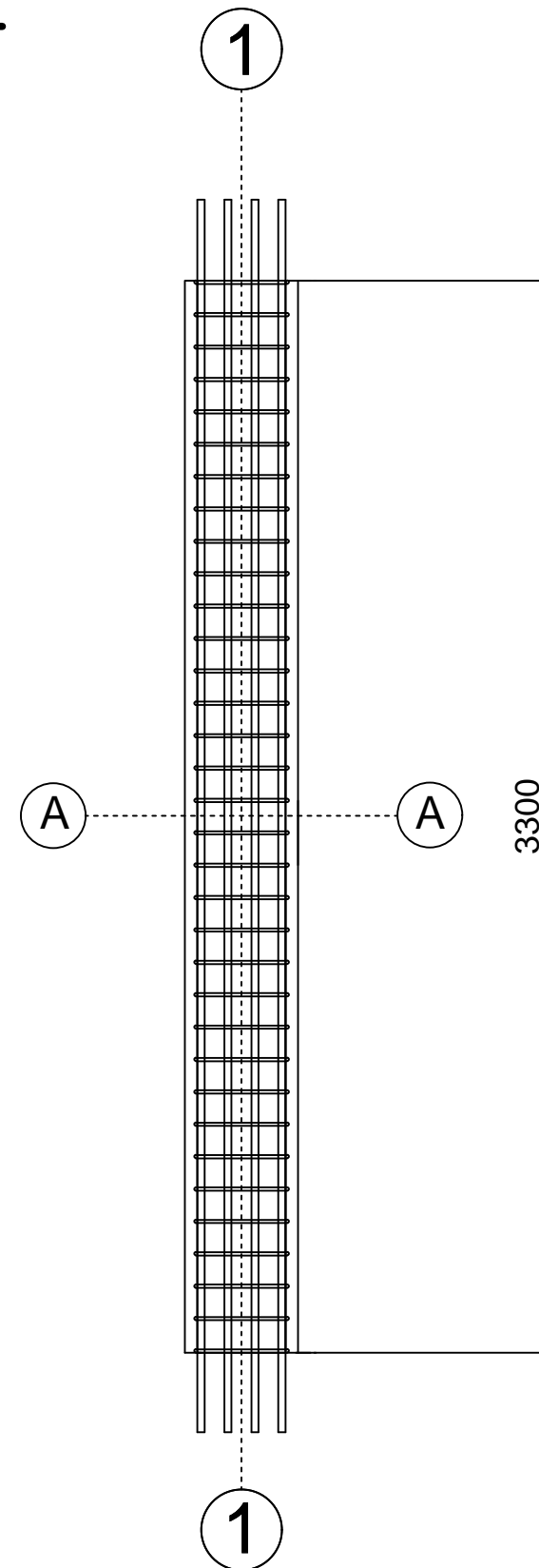
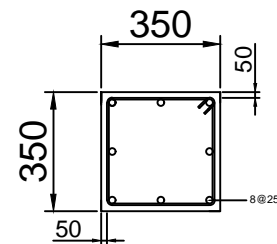
A - 101

Sheet 10



Column Design C15-1, C15-2, C15-3, C15-4
C15-5, C15-6, C15-7

Section A-A



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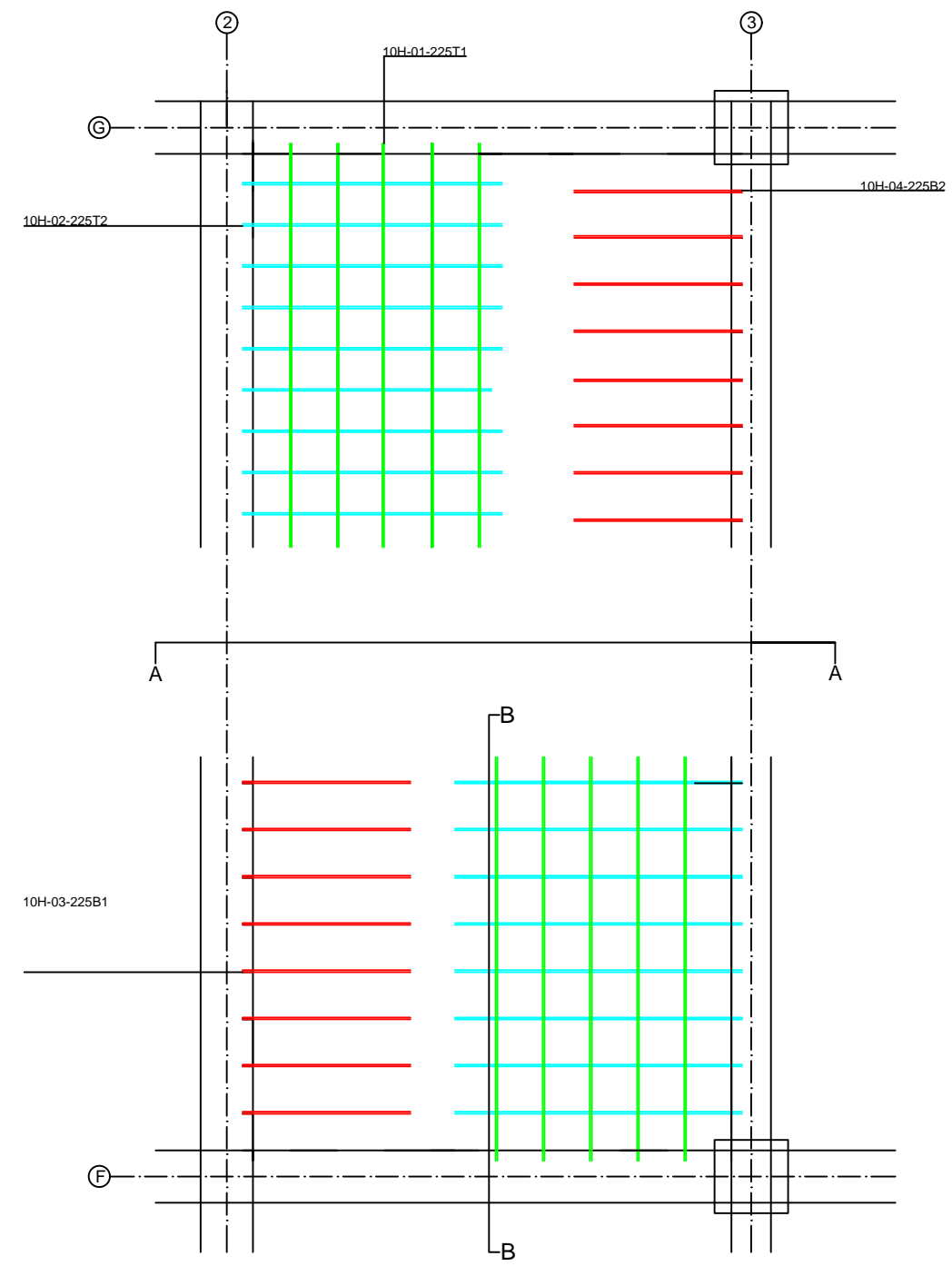
Column design 11

A - 101

Sheet 11

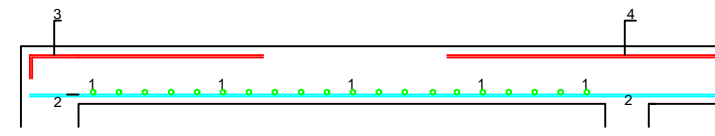
A1

Slab Reinforcement

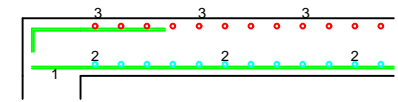


Plan of interior span of floor slab
 Cover to upper layers B1 and T1 = 25 mm

Section A-A



Section B-B



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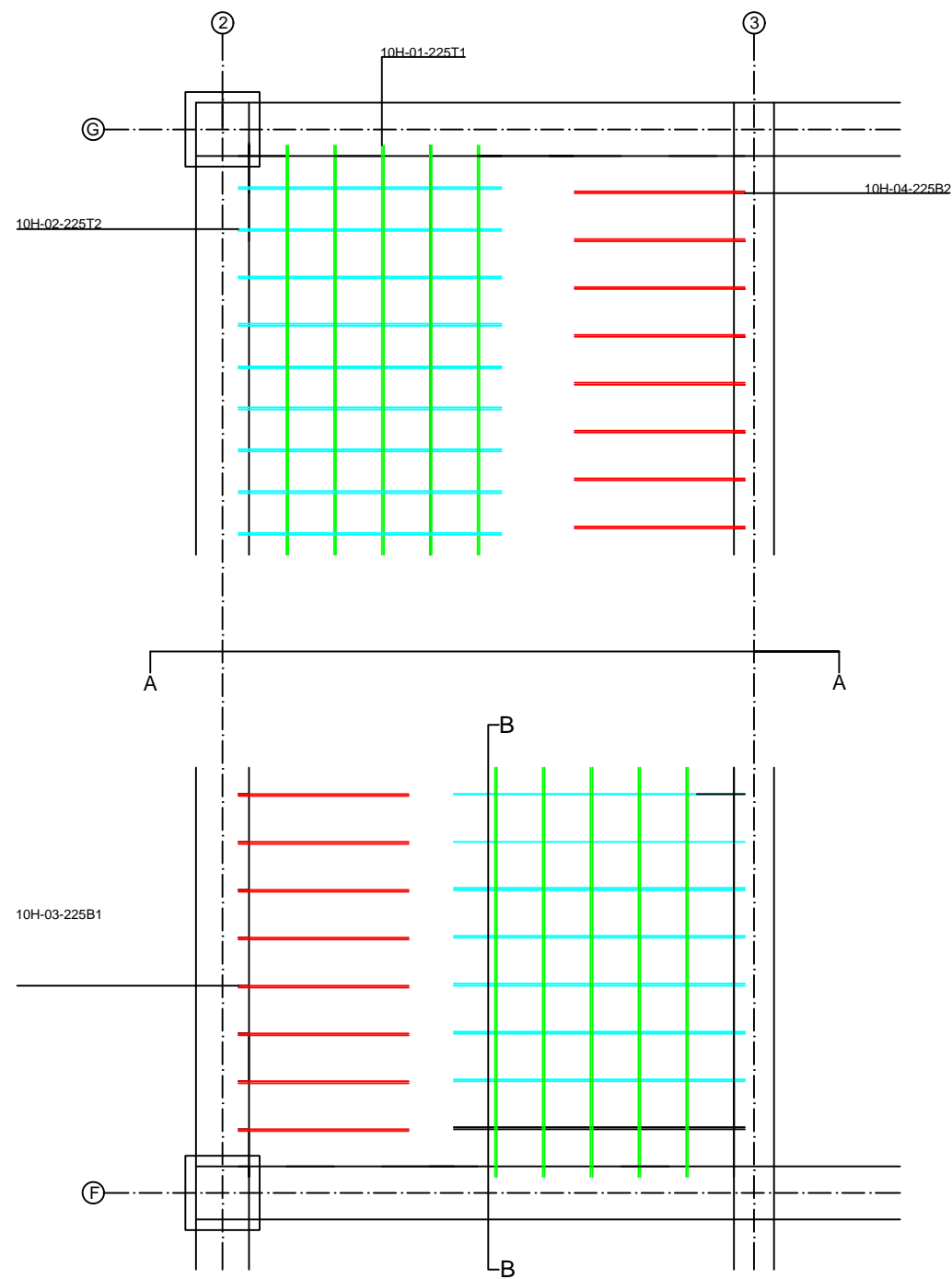
Sheet Title

Slab Reinforcement
 Interior span floor1

A - 101

A1

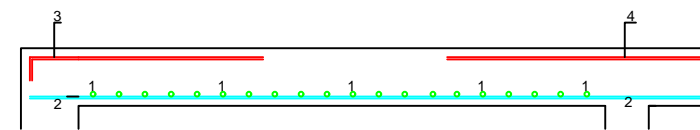
Slab Reinforcement



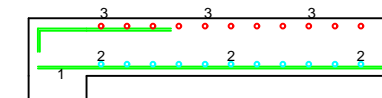
Plan of end span of floor slab

Cover to upper layers B1 and T1 = 25 mm

Section A-A



Section B-B



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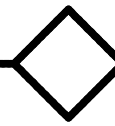
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Slab Reinforcement
End Span Floor 1

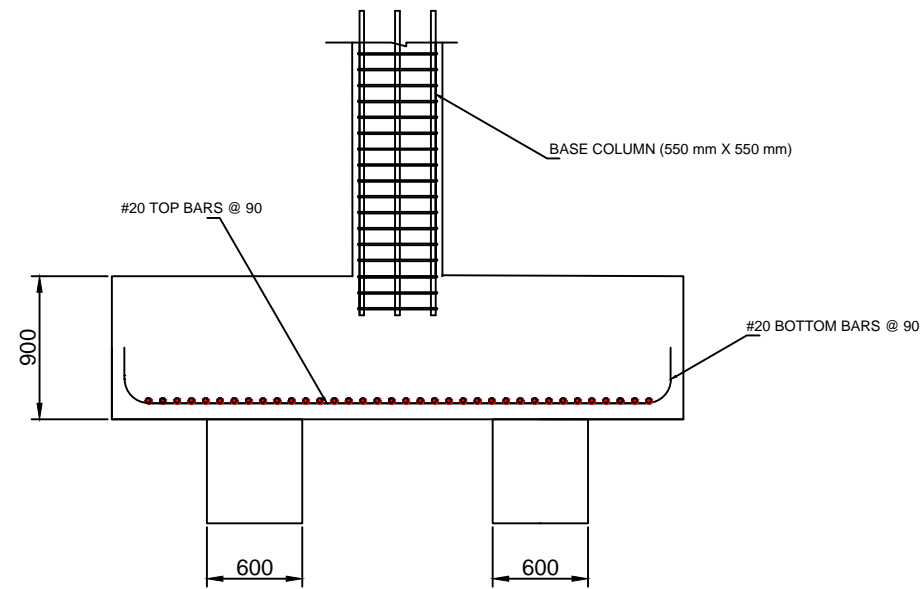
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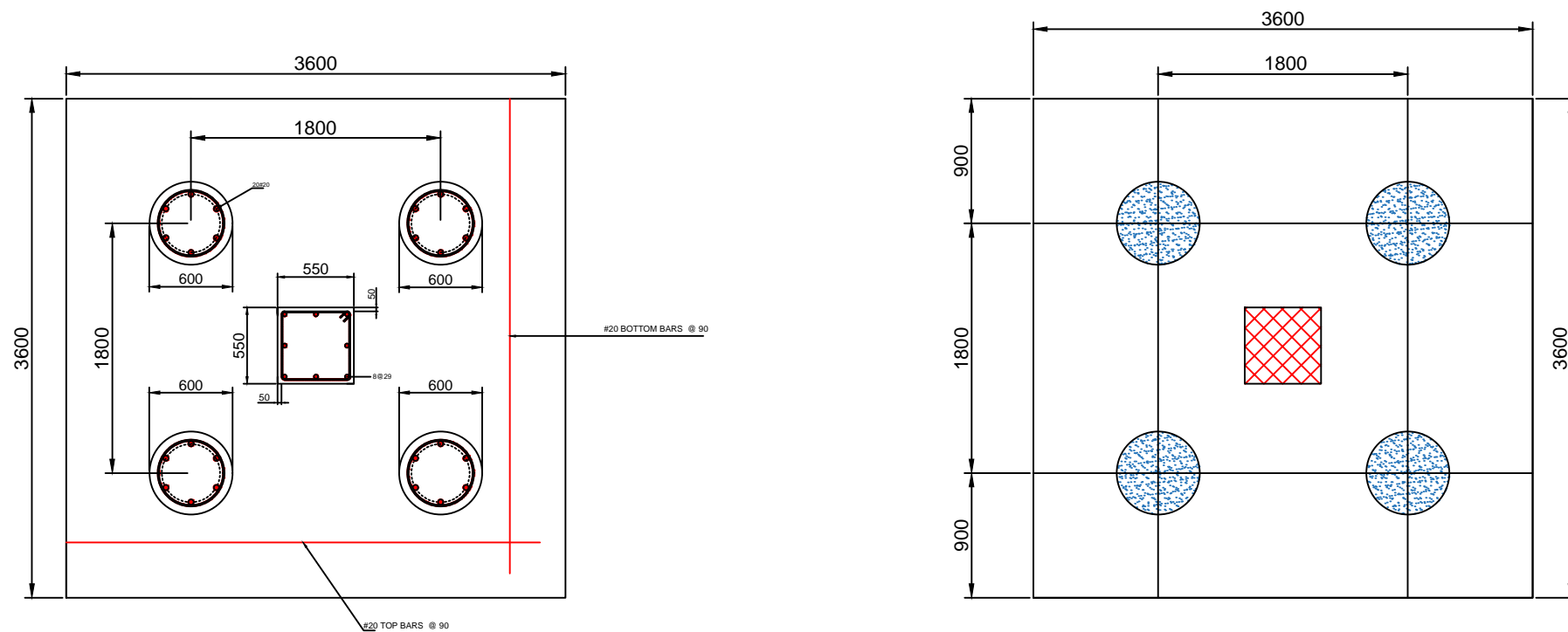


Pile Foundation

Side View



Top View



Energy Efficient Construction

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Pile Foundation

A - 101

Sheet 1