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**RENOVATION OF MULTI-STORY OLD BUILDING IN
DOWNTOWN
(Capstone Project II Report)**

**Bachelor of Engineering
(Civil Engineering)**



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2018

DECLARATION

We hereby declare that this report entitled “*Renovation of Multi-Story Old Building in Downtown*” is the result of our own project work except for quotations and citations, which have been acknowledged in the report. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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Abstract

Deterioration of old buildings in urban areas due to outdated structural, geotechnical, and architectural designs calls for the need of renovation in order to reuse those buildings. Renovation is becoming an effective way to extend service life of aging buildings, which are emerging as a crucial factor effecting resident's wellbeing. Through performing renovation structure's service life is extended, the living condition of its residents is improved, sustainable design is implemented, as well as structural strength, architectural and geotechnical designs are improved. For this project, a Soviet Union 5-storey precast concrete panel building located in the downtown area of Astana is chosen for renovation. The renovation includes downwards and upwards extension of the structure, namely construction of 2-storey underground parking and additional 2 storeys on existing building. Due to increased gravity load from additional storeys, the existing structure's walls, slabs and connections between them are going to be strengthened and the old foundation is planned to be replaced to support the load of the new 7-storey structure and 2-storey underground parking.

The project scope includes architectural modification of the structure's exterior design presented as a 3D model in SketchUp software. The architectural plan of existing building is presented in AutoCAD drawings. As the entire project is performed based on American codes and the existing structure is constructed based on Soviet Union codes, the consistency of the existing structure's space use and occupancy is checked and verified to be in an agreement with American codes. The data on interior environment, ceiling and flooring of the existing structure is collected and described in architectural design section. The modifications, in addition to construction of the parking and 2 storeys, also include construction of elevators, the selection and design of which is also described in architectural design section.

The structural design includes analysis of existing building's structural elements, specifically their strength and ability to withstand additional loads from upward modification, and the procedure and results of strengthening details. In order to proceed in this, design loads, which include dead, live, snow and wind loads, were calculated. Then, 3D model was built in SAP2000 software, from which internal design loads for each wall, slab and connections were obtained. Additionally, the internal design loads of specific members for one vertical and one horizontal case were calculated manually. The values obtained from SAP2000 for these cases and manual calculations were compared and verified. In addition to this, lateral drift analysis and check under wind load was performed. Based on the data on existing building's structural members, namely reinforcement detailing, the design strength of each wall is calculated and presented in the form of M-N interaction curve. Design strengths of walls and joints are also calculated. From this analysis of design load and design strength, it is ensured that strengthening

is not required as the walls, slabs and joints are able to sustain additional loads from vertical extension. Structural detailing of all members are provided in AutoCAD drawings.

The geotechnical design was initiated by collecting data on soil profile of the region where the building is located. Based on the load from the structure, parking columns, slabs, micropiles and micropile connections are designed. Due to foundation replacement and excavation, dewatering is performed. Excavation process is designed in Plaxis to determine the settlement and ensure that it is within allowable limit. Sheet piles are also designed in Plaxis, as well as new foundation.

The project management section involves the scope of the project, risk assessment and mitigation plan, health and safety plan, waste management plan, cost and time estimations, and risk management. Additionally, detailed Work Breakdown Structure and scheduling are prepared.

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Chapter 1. Introduction

1.1. Problem statement and renovation purpose. Capstone II objectives

Aging and deterioration of old buildings are emerging as an important issue affecting resident's wellbeing. Due to outdated designs in old buildings, renovations are being performed in order to reuse the old buildings, especially in downtown areas, which are densely populated and require quick reconstruction. Through performing renovation structure's service life is extended, the living condition of its residents is improved.

The final goal of this capstone project is to analyze the structural and geotechnical strength of the structure and renovate in the aspects that require improvement. The renovation includes downwards and upwards extension of the structure, namely construction of underground parking and additional storeys on existing building. Due to increased gravity load from additional storeys, structural members (walls and slabs) and connections must be checked for strength. In addition, due to increased gravity loads from superstructure and parking, the existing foundation is replaced and a new foundation is designed. The architectural design of the structure is modified in accordance with the neighboring buildings in the area.

1.2. Background on chosen buildings

Initially, a complex of 3 residential 5-storey buildings located in Astana, on the cross-section of Birzhan-sal and Gete streets, were chosen for renovations (see **Figure 1.1**). However, for Capstone II it was decided to narrow the scope of the project by focusing on one residential building (see **Figure 1.2**), in order to perform more detailed engineering analysis on it. The location of the building is near the railway station and is surrounded by two main streets that lead from the station to the city (see **Figure 1.1**). Thus, they are the busiest streets and experience heavy traffic jams, as well as lack of parking lots, which lead to cars being parked on the roadsides, enhancing the existing traffic jam. Renovation of the project includes construction of 2-storey underground parking, which provides parking space and eliminates the cars on the roadsides, thus relieving intense traffic jam.



Figure 1.1. Cross-section of Birzhan-sal and Gete streets, Astana
(Source: Yandex map)

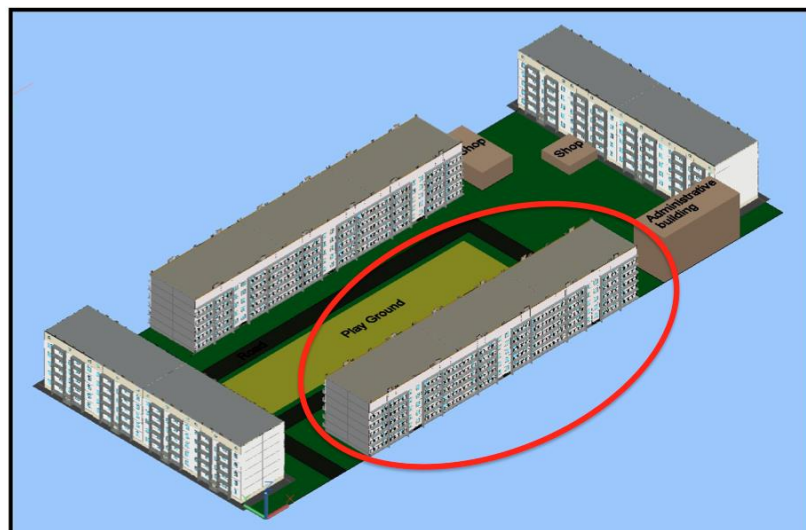


Figure 1.2. Narrowing the scope of the project: 1 residential building

The selected building is of Soviet Union design made of precast concrete panel walls, namely the reinforced concrete blocks fabricated in the plant and assembled on the site by welding the steel joints and filling in the gaps between the concrete blocks (see **Figure 1.3**). Renovation process will help to increase the number of available living spaces in the old building. It will allow improving building's structural, geotechnical and architectural designs of the building.



Figure 1.3. Precast panel buildings (Source: openleft.ru)

Architectural Design



Chapter 2. Architectural design

2.1. Existing building

2.1.1 General building height and areas

Existing 5-storey building was constructed in 1980s (see **Figure 2.1**) and provided 90 residential apartments for Astana city people (Karakova. T, Ryzhikova. E. 2014).

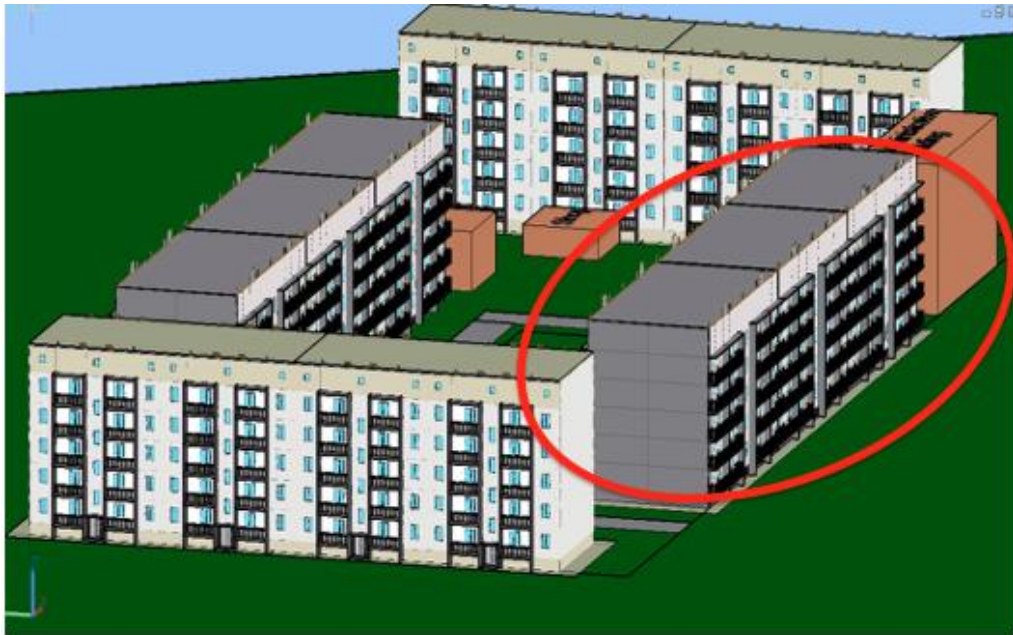


Figure 2.1. Existing building layout

Apartments of existing buildings are divided into 3 types: single, double room, triple room. Layout of the first floor is shown in the **Figure 2.2** below. Building consists of 3 sections, each floor has 2 subsections; each subsection has 3 apartments, thus in total in every floor 18 apartments are located.



Figure 2.2. Layout of first floor residential apartments

Existing building dimensions (for 1 section) are: 13.2 m in A - Г axis and 115.2 m in 1-13 axis, which is shown in **Figure 2.3**. Total area of it 507 m².

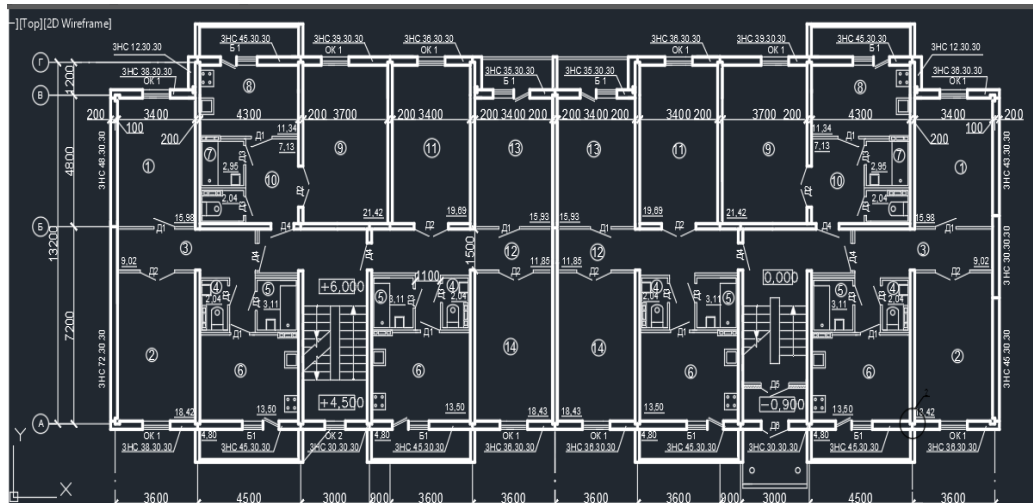


Figure 2.3. Layout of residential floor

Height of the room is 3m and total height of the building is 18.15m, which found from the drawing of cross section of the building in **Figure 2.4**. According to IBC, the building is considered as non-high-rise building, because total height of the building is less than 22.86m.

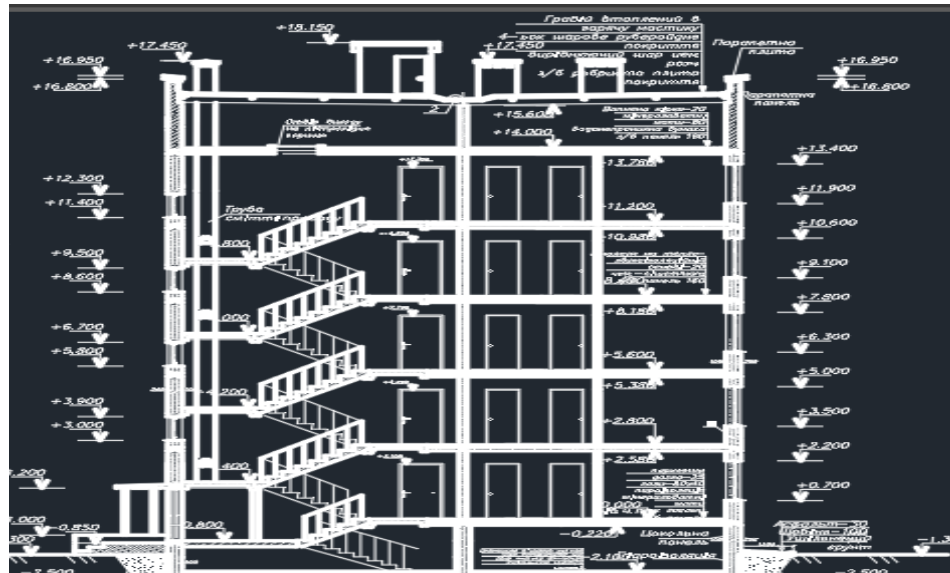


Figure 2.4. Section view of existing building

Moreover, existing building has cellar for communication devices and loft, which leads to the roof. Main architectural distinctive feature of the residential building is absence of elevators. Primary reason of constructing such kind of residential houses was to provide residents with apartments as soon and cheap as possible, thus simple construction was considered as a main target. For this reason, important properties such as fire protection system and sound transmission were done poorly.

2.1.2 Use and occupancy. Consistency with IRC

2.1.2.1 Total Occupancy Capacity

Gross living area of one building, which consists of 30 apartments, is 1046.76 m². One floor of five-story building contains three apartments: one single room, one two-room, and one three-room apartment. In the table below area of each room in all the type of apartments are given:

Table 2.1. Apartment space area allocation

Room types	Area, m²
Single room apartment	49.68
Common room	21.42
Hallway	7.13
Restroom	2.04
Bathroom	2.95
Kitchen	11.34
Loggia	4.8
double-room apartment	66.87
Bedroom	15.98
Common room	18.42
Hallway	9.02
Restroom	2.04
Bathroom	3.11
Kitchen	13.50
Loggia	4.80
Triple-room apartment	92.80
Bedroom	18.43
Bedroom 2	15.93
Common room	19.69
Hallway	11.85
Restroom	2.04
Bathroom	3.11
Kitchen	13.5
Loggia (bedroom)	3.45
Loggia (kitchen)	4.80

2.1.2.2 Requirements for living rooms according to IRC

According to the IRC there are several main criteria for living rooms in the apartment. Firstly, the minimum area of any habitable room must be greater than 11 m² (IRC R304.1). From above **Table 2.1** it can be seen that existing building satisfy with the given requirement, having minimum habitable room area 15.3 m². Moreover any horizontal dimensions of each habitable room must be greater than 2134 mm (IRC R304.3). From the **Figure 2.5** below it can be noticed that habitable room width and length is greater than 2134mm.

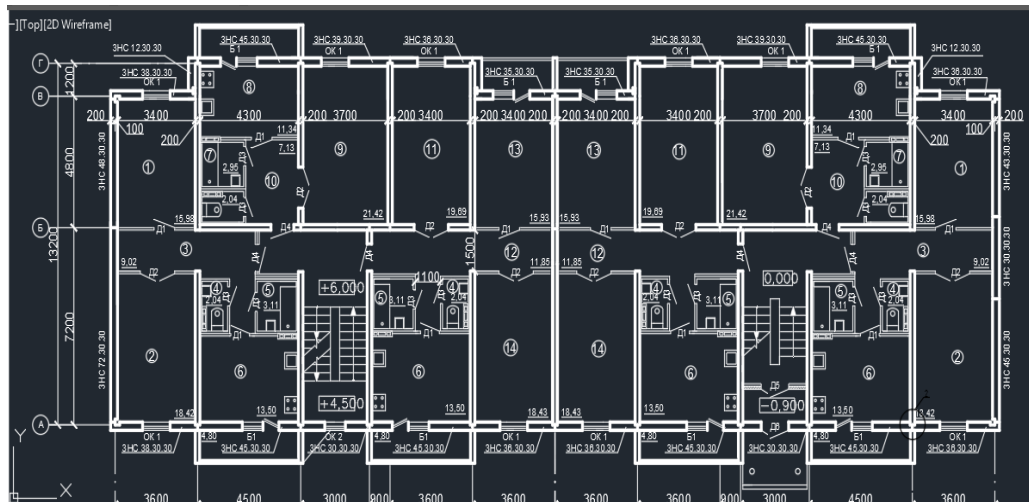


Figure 2.5. Layout of residential floor

Minimum ceiling height in the habitable space must be greater than 2134 mm (IRC R305.1). Bathroom and restroom must have minimum ceiling height of 2032 mm (IRC R305.1). As minimum height of the building in range of 2700 - 2900mm, it satisfies to the IRC R305.1 requirement.

2.1.2.3 Requirements for hallway and stairway according to IRC

According to IRC, the minimum width for both hallway and stairway is 0.91. In case of the existing building, width of the hallway and stairway are more than required minimum width, which are 1.5 and 1.2 respectively. Moreover, the maximum height difference of the stairway must be less than 3.6. In case of existing building, height difference between the start and end of the stairway is 1.5m.

To sum up, it can be concluded that architectural layout of the existing building is consistent with IRC. As it was mentioned before there is countless number of requirements for each apartment, however requirements listed above are the most important, as any change in mentioned requirements will lead to the change of the structure of the building.

2.1.3 Fire protection systems

As constructed object is considered as non-high-rise building, it does not require any specific code standard application. Existing building is classified to Type I - B construction building, which is non - combustible and residential building complex. But however, building does not meet the requirements of IBC code, by having poor fire protection system. According to IBC, automatic sprinkler system or alternative automatic fire – extinguishing system, portable fire extinguisher, standpipe system, fire detection system, emergency alarm system, smoke control system should be installed, which does not correspond in the case of existing building.

Building has 2-side staircase, width of the stairs between corresponding floors is 1400mm, the width of the ladder march is 1200mm. For fire safety purposes, 400 mm gap is provided between the marches for free passage of the fire hose.

2.1.4 Interior environment

2.1.4.1 Ventilation

According to International Mechanical Code Section 1203.4 all the buildings must be provided with natural ventilation system. Due to the fact that existing residential building already has natural ventilation system, in the current project this part of the construction set will not be included. As ventilation system is considered as mechanical engineering's responsibility, ventilation of the parking subcontractor who is specialized in ventilation systems will be involved to do it.

2.1.4.2 Lighting – Natural light

According to IBC Section 1205.2, the minimum glazed area must be higher than 8 % of floor total area of the room. In the existing layout all the rooms has higher than 8% glazed area, due to this fact changes in the lighting of the rooms are not necessary. In the **Table 2.2** below detailed information about the lightening are given.

Table 2.2. Percentage of glazing of each room

Room	Area of the room, m²	Area of the glazing, m²	% of glazing per room's area
<i>Single room apartment</i>			
Common room	21.42	1.8	8.4 %
Kitchen	11.34	2.24	19.8 %
<i>Two-room apartment</i>			
Bedroom	15.98	1.8	11.3 %
Common room	18.42	1.8	9.8 %
Kitchen	13.5	2.24	16.6 %
<i>Three-room apartment</i>			
Bedroom 1	18.42	1.8	9.8 %
Bedroom 2	15.93	2.24	14.1 %
Common room	19.69	1.8	9.1 %
Kitchen	13.5	2.24	16.6 %

Regarding to lightening of stairways in the dwelling, units must be illuminated with minimum of 1 foot-candle light (11 lux) (IBC 1205.4). For that purpose, in renovation process stairway lighting according to IBC will be provided.

2.1.5 Non-structural materials

Non-structural materials are also included in self-weight. Non-structural materials considered in this section are ceiling and wall finishing, as well as floor finishing.

2.1.5.1 Ceiling and walls

Exterior walls are composite of reinforcement concrete, polystyrene concrete and mineral wool plates. However, beside this, walls covered by finishing material – gypsum plaster, which covers area of 305.3 m² (area of walls in one floor in the building section). Ceiling covered by plasterboard with total area 170.8 m²

2.1.5.2 Flooring

Flooring has several layers as it can be seen from **Figure 2.6**. The layers on top of reinforced concrete include soundproofing gasket with width of 25mm, bitumen layer, cement screed with width of 45mm, polyethylene and laminate with the width of 10mm. The total area covered by these layers is 170.8 m² and this gives additional weight of 162.2 kN in one floor of building section.

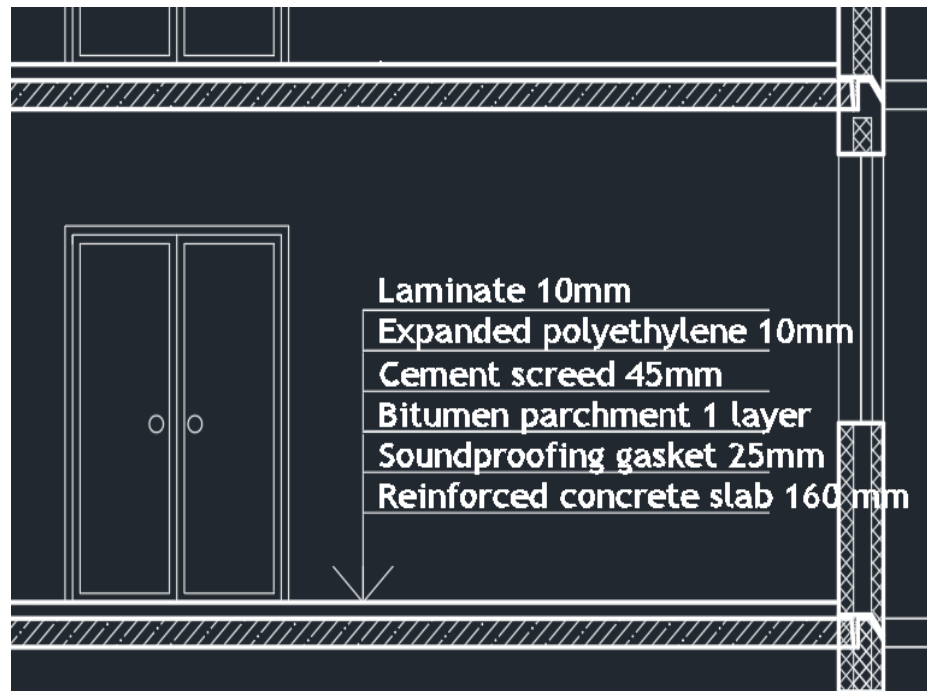


Figure 2.6. Shows materials in flooring coverage

2.2. Modifications

2.2.1 Additional two floors

From the owner's perspective, in order to make considerable profit by renting and selling the apartments, additional 2 floors will be constructed. The layout of the additional floors will be the same as the existing building in **Figure 2.6** that was mentioned above.

In order to select the appropriate material for additional 2 floors, 5 construction materials such as masonry, precast concrete, lightweight concrete, monolithic concrete, steel with sandwich panel are evaluated using the following criteria: load resistance, cost, time of construction, heat and noise isolation.

Masonry and steel with sandwich panel

Masonry is one of the widely used construction materials, because it has high fire and earthquake resistance. Moreover, its high noise and temperature insulation also play significant role in material selection. In order to calculate the total volume of the construction material, average unit brick with dimension 30 x 60.5 x 30 cm was used (Astana Satu. 2017). Knowing the

total perimeter and area, which is $115.2 \times 13.2 \text{ m}^2$, number of the units are found, thus multiplying the number of the brick to the average dimension total volume of the material is found. (See **Table 2.3**, **Table 2.4** below for detailed calculation). Steel concrete has high stiffness, low weigh and good thermal and noise isolation. Prefabricated and compacted steel with sandwich panel consist of the outer, inner rand isolation layers (Tata Steel. 2017).

Precast, lightweight, monolithic concrete

In comparison with other materials, precast concrete in construction process is faster and more efficient. Forms of precast concrete can be reused for several times, thus it is cheaper in comparison to onsite concrete casting and can be considered as practical construction material. Lightweight concrete reduces dead load, and improves thermal properties, fire resistance. Moreover, due to its light weigh it leads to saving in transporting, handling of materials (Ecoton. 2017). Monolithic concrete has similar characteristics as precast concrete, but the main difference between them is that precast concrete is prefabricated, whereas monolithic concrete is utilized in construction site (Estrealinvest.2017).

Selection of material

In the **Tables 2.3** and **Table 2.4** below detailed calculations are represented. Each material volume was calculated firstly, multiplying by unit weight, and then dividing by area, load was found. As load criteria is considered as one of the main characteristics of material, which shows amount of load transferred to the lower floors. Thus, steel with sandwich panel is considered as material, which has the lowest load. But, in selection of the material, cost also plays significant role. Thus, analyzing total cost, which is the sum of material cost and manpower cost, it is figured out that steel is the most expensive material. Conversely, precast concrete defined as the cheapest and efficient material. Moreover, during the construction process, equipment cost can also influence considerably. Multiplying the production rate to the total duration of the project equipment, cost can be obtained. Based on this analysis, precast concrete was chosen for additional 2 floors.

Table 2.3. Material selection analysis

Criteria/ material	Aver. density (kg/m ³)	Volume (m ³)	Weight (N)	Load (N/m ²)	Unit cost per m ³ (tg)	Total cost (tg)	Time of construction	Heat/noise isolation
<i>Masonry</i> [1]	1950	1728	33055776	7186	578	16 000 000	Slow, needs additional labour	Excellent
<i>Lightweight concrete</i> [2]	1640	1476	23746478	5162	711	16 801 520	Easy to use, require less time	Excellent
<i>Monolithic concrete</i> [3]	2400	988	23261472	5057	899	14 215 000	About 3 months	Poor noise isolation
<i>Precast concrete</i> [4]	2150	988	20838402	4530	708	11 200 000	Easy to use, require less time	Excellent
<i>Steel with sandwich panel</i> [5]	7800	6.47	495405	1220	440185	48 568000	Require less time	Excellent

Table 2.4. Manpower calculation

Material	Unit cost (tg of manpower per m ³)	Volume (m ³)	Cost of manpower (tg)	Total cost (tg)
Masonry	3200	1728	3369600	19369600
Precast concrete	1150	988	2124200	13324200
Lightweight concrete	1580	1476	2420640	19220640
Monolithic concrete	5638	988	2371200	16371200
Steel with sandwich panel	11285	6,70	75609,5	48075609,5

2.2.2 Elevators

In order to calculate minimum number of elevators, online toolbox developed by KONE Corporation was used. By inserting all the necessary input information (see **Figure 2.7**), following results were obtained:

Conventional Control Enter your planning information to get your required elevator solution

Metric | Imperial

Building information

Use of passenger elevator

Type

Usage

Zone information

Number of Stops

Travel for the zone (m)

Population in the zone

Applied parameters

Up peak handling capacity (% of population/5 minutes)

Acceleration rate (m/s²)

Interval (s)

Travel time (s)

System parameters

Acceleration

Speed (m/s)

Estimated

Actual

Elevator size (persons)

Estimated

Actual

Required number of elevators	
Required by interval <input type="button" value="i"/>	0.81
Required by handling capacity <input type="button" value="i"/>	0.26
Actually required <input type="button" value="i"/>	1

Figure 2.7. KONE Quick Traffic analysis

Due to shortage of the space inside of the building, it was decided to locate elevator in the hallway between two consequent floors (see **Figure 2.8**).

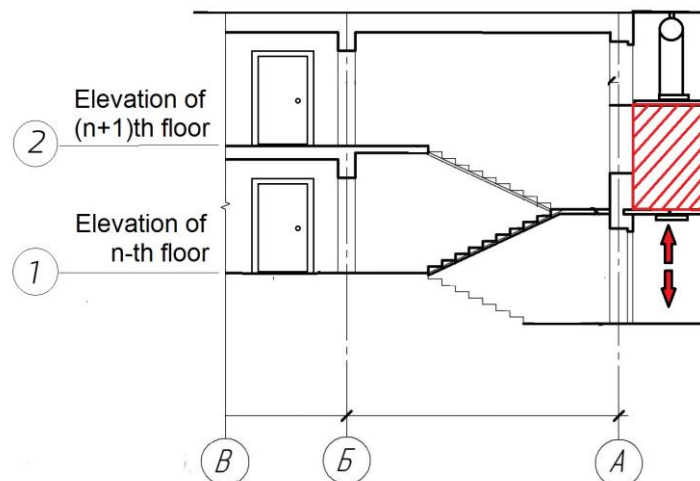


Figure 2.8. Location of elevator: Section view

After identifying the number of elevators, the dimensions of elevator and hoistway were evaluated. By calculating available space for elevator (2000 mm x 3100 mm), the type of elevator that suits for this location was found. The “Hyundai Elevators Co.” company that has an affiliate in Astana offers an elevator that passes all the requirements for the project. In the **Figures 2.9-10** location of the elevator and the design of it are presented.



Figure 2.9. Location of the elevator

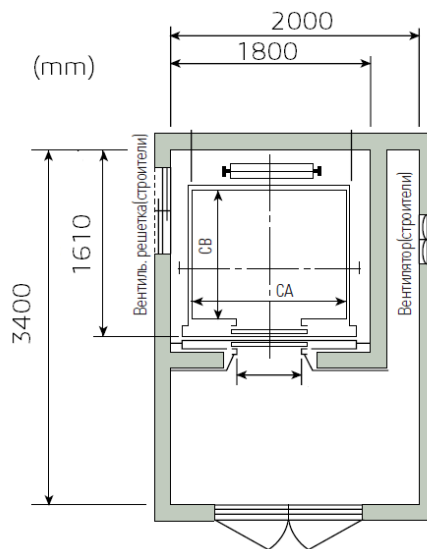


Figure 2.10. Design and dimension of the elevator

2.2.3 Exterior design

Exterior design of the building was prepared using Sketch up software program. Residential complex consist of 3 residential blocks and a hotel, but for better analysis this project focuses only on one residential building as it is indicated in **Figure 2.11** below. The main architectural direction of the buildings that was constructed in 1955-1991 pertains to Soviet Modernism, which refer to Khrushchev's and Brezhnev's ruling time and differs from contemporary architecture by simplicity and commonness. Major reason for construction of such panel houses was to provide apartments for city residents as soon as possible (*Soviet Modernism*. Garage.2017). As the chosen existing building is located in old downtown part of the city and is

surrounded by the same buildings that were constructed in the same period, exterior design does not differ much. According to Soviet Modernism fundamentals tranquil yellow was selected as a basis. In order to match for selected color set, elevator was chosen as bright blue.

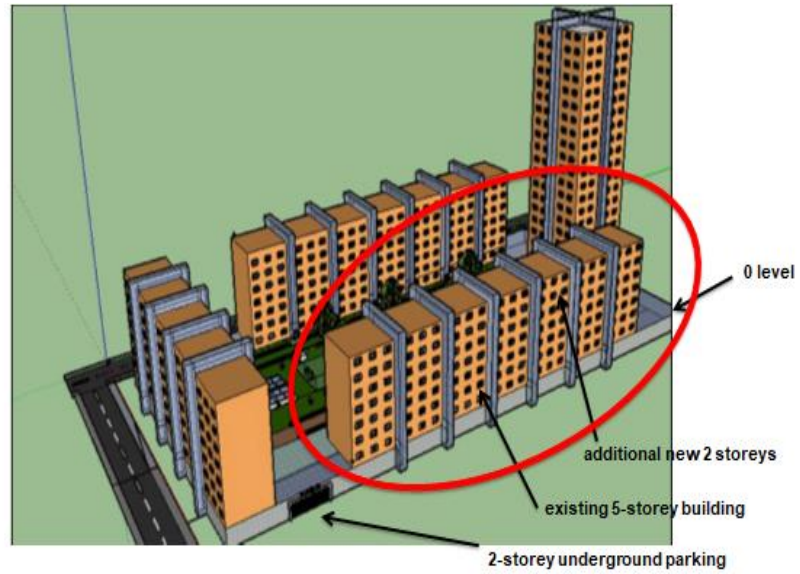
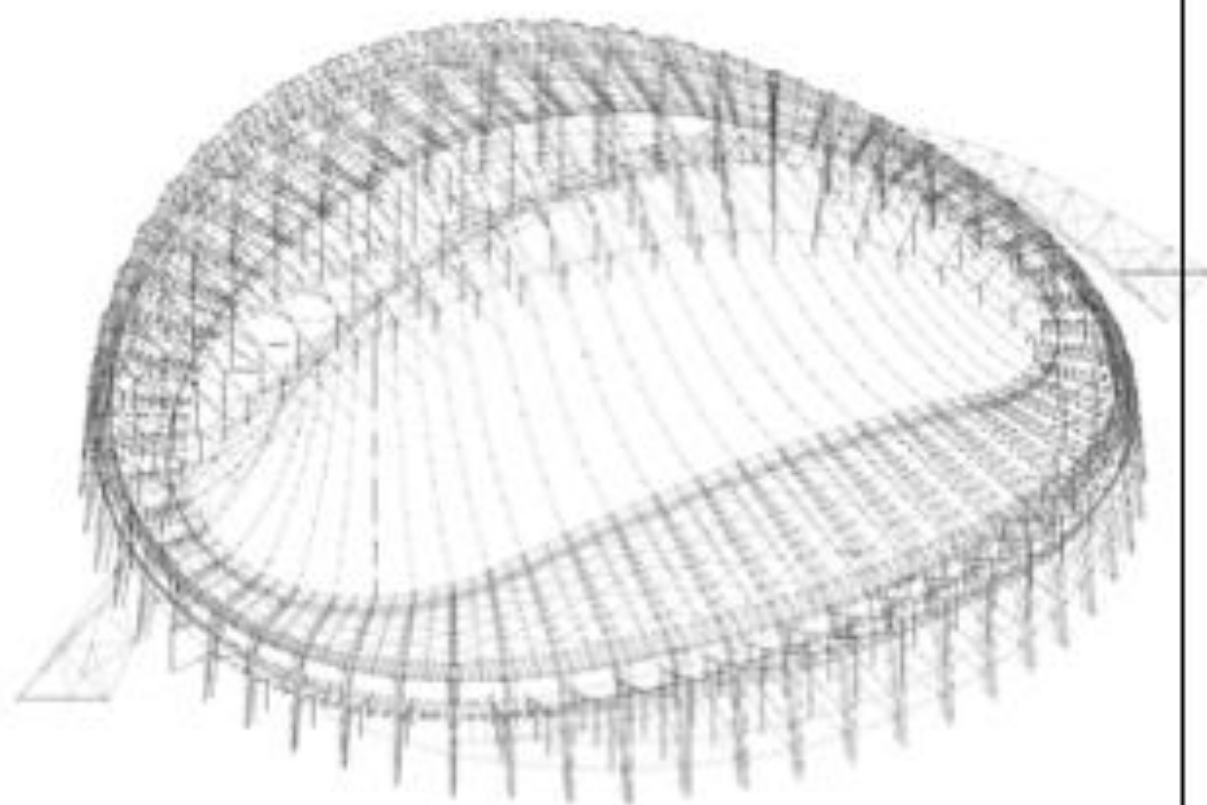


Figure 2.11. 3D view of chosen residential building with modifications

Structural Analysis



Chapter 3. Structural design

3.1. Existing building structural members

3.1.1 Walls

There are three categories of walls in the existing building: exterior load bearing walls, interior load bearing walls and partitions. The last mentioned category is not considered in this section since it is not a structural member. The external walls consist of several layers: polystyrene concrete, mineral wool plates and reinforced concrete. **Figure 3.1** shows a section view of the external wall.

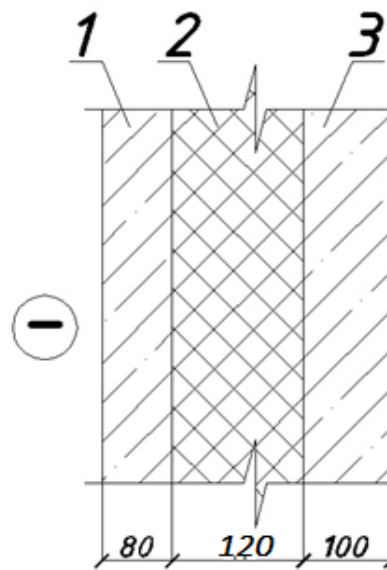


Figure 3.1. Section view of the wall with layout view

1 - polystyrene concrete;

2 - mineral wool plates;

3 - reinforced concrete;

The whole width is 300mm: 120mm of mineral wool plates, 80mm of polystyrene concrete, and 100 mm of reinforced concrete. Reinforcement allocation of the wall can be seen from the **Figure 3.2** and **Figure 3.3**.

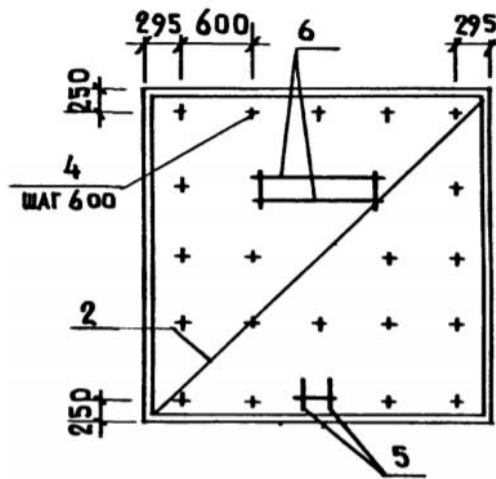


Figure 3.2. Rebar layout on the front section of view

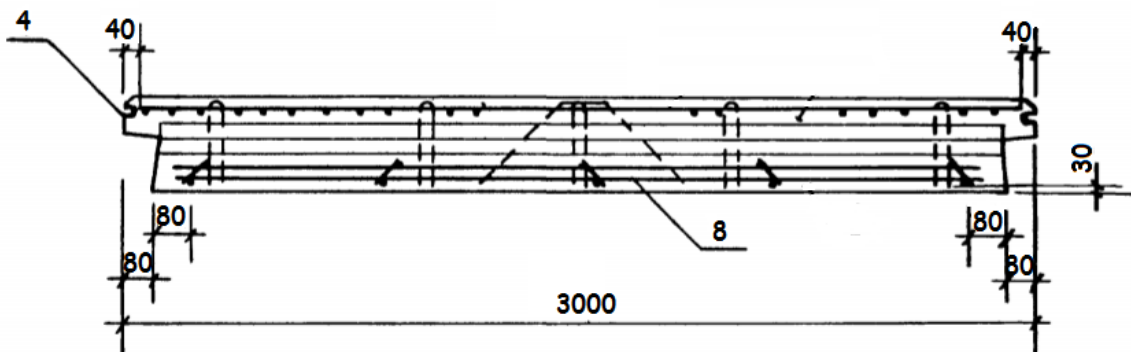


Figure 3.3. Top-side section view of rebar allocation

According to the figures, there is reinforcement in the concrete which has 2 layer nets of 8 mm diameter rebar with dimensions of the net 400*400mm and coverage of 80mm from both sides. There is rebar net in polystyrene concrete with dimension 100*200 mm and diameter of 4mm of the steel type A240. Type of concrete is B20 with average density 2500 kg/m³.

Interior wall has width of 200 mm of reinforced concrete. The type of concrete and steel is the same as in exterior wall. However, layout of reinforcement is different from exterior walls, which can be seen from the **Figure 3.4** below.

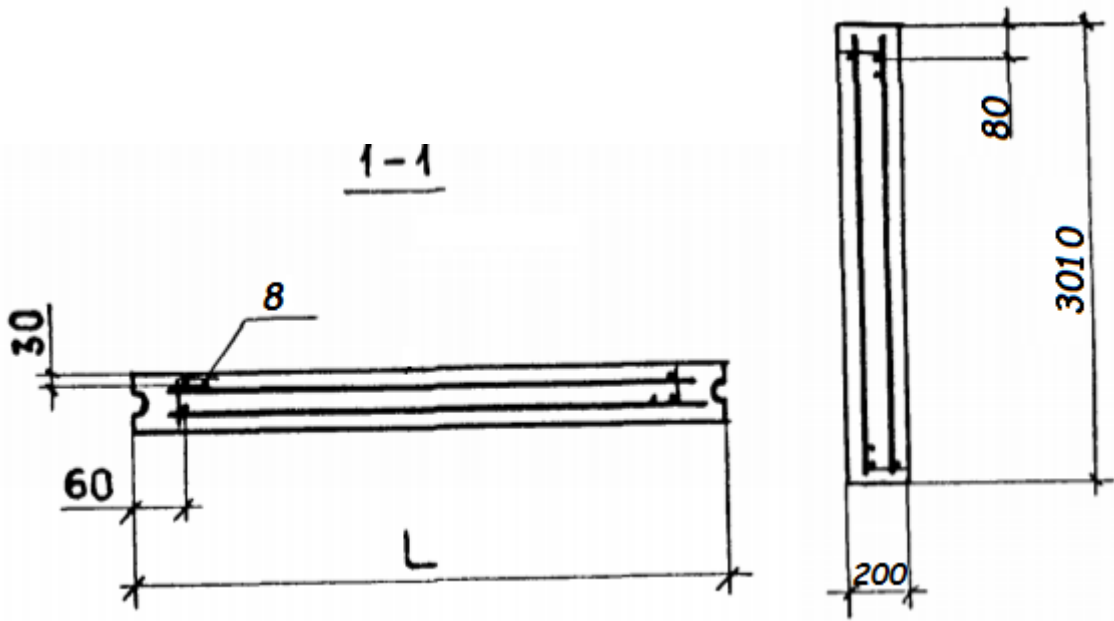


Figure 3.4. Section view of the rebar allocation in interior wall

3.1.2 Slabs

All the slabs in the existing building are pre-casted. There are seven different types of slabs presented in the building, they all described below in the **Table 3.1**. All of them are made of **M250** concrete with AIII steel reinforced. For reinforcement there is a mesh of $d = 16$ mm rebar, and connection of $d = 8$ mm.

Table 3.1. Slabs present in the existing building

#	Name	Size (length x width x height), mm	Quantity (in one block)
1	2PTS 48.30.16	4800 x 3000 x 160	24
2	2PTS 45.36.16	4500 x 3600 x 160	12
3	2PTS 45.24.16	4500 x 2400 x 160	84
4	2PTS 36.30.16	3600 x 3000 x 160	24
5	2PTS 36.24.16	3600 x 2400 x 160	120
6	LPF 30.28	3000 x 2800 x 160	30
7	LPF 30.14	3000 x 1400 x 160	24

3.1.3 Connections

In order to provide structural analysis, it is also necessary to take into account connections between walls themselves and between walls and slabs. The **Figure 3.6** below shows the connection between exterior walls and slab or roofing slab.

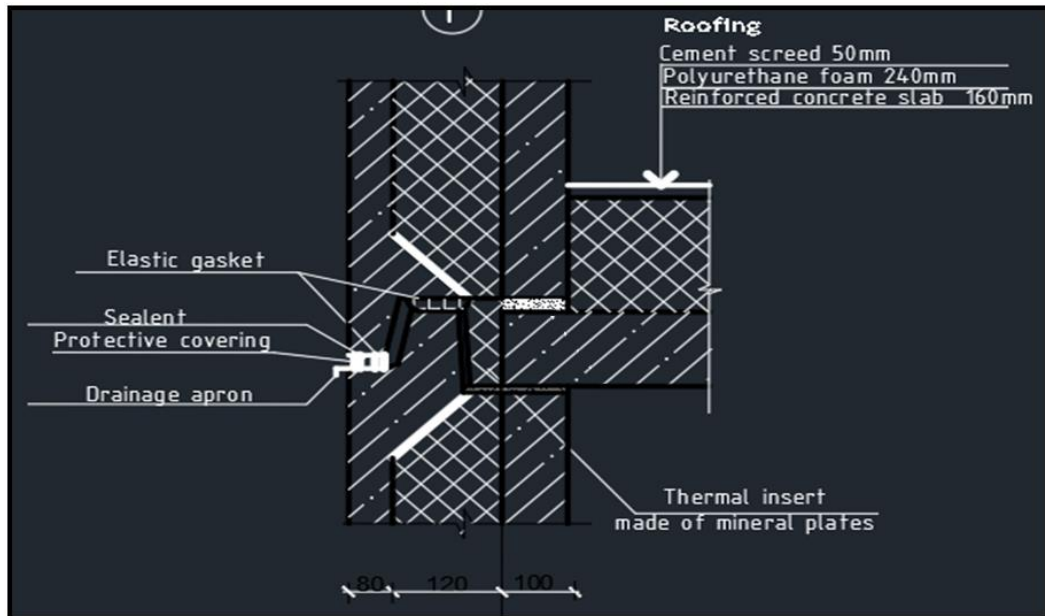


Figure 3.6. Connection of exterior walls and slab, side view

This figure represents connection of three elements: two vertical exterior walls and horizontal slab of the roof. The exterior slab is covered by thermo-layer of polyurethane foam, because it is roofing slab. Other slabs between floors are not covered by such layer, they have different layers, which can be seen from **Figure 2.6**. However, all slabs have identical width of reinforced concrete as 160mm.

The system has spike groove principle, and contiguity is filled by thermal material for thermal isolation. This thermal isolation contains elastic material to prevent cracking since different materials behave in various ways. Additionally, the connection is waterproof.

The connection between exterior walls and the interior wall can be seen from **Figure 3.7** below.

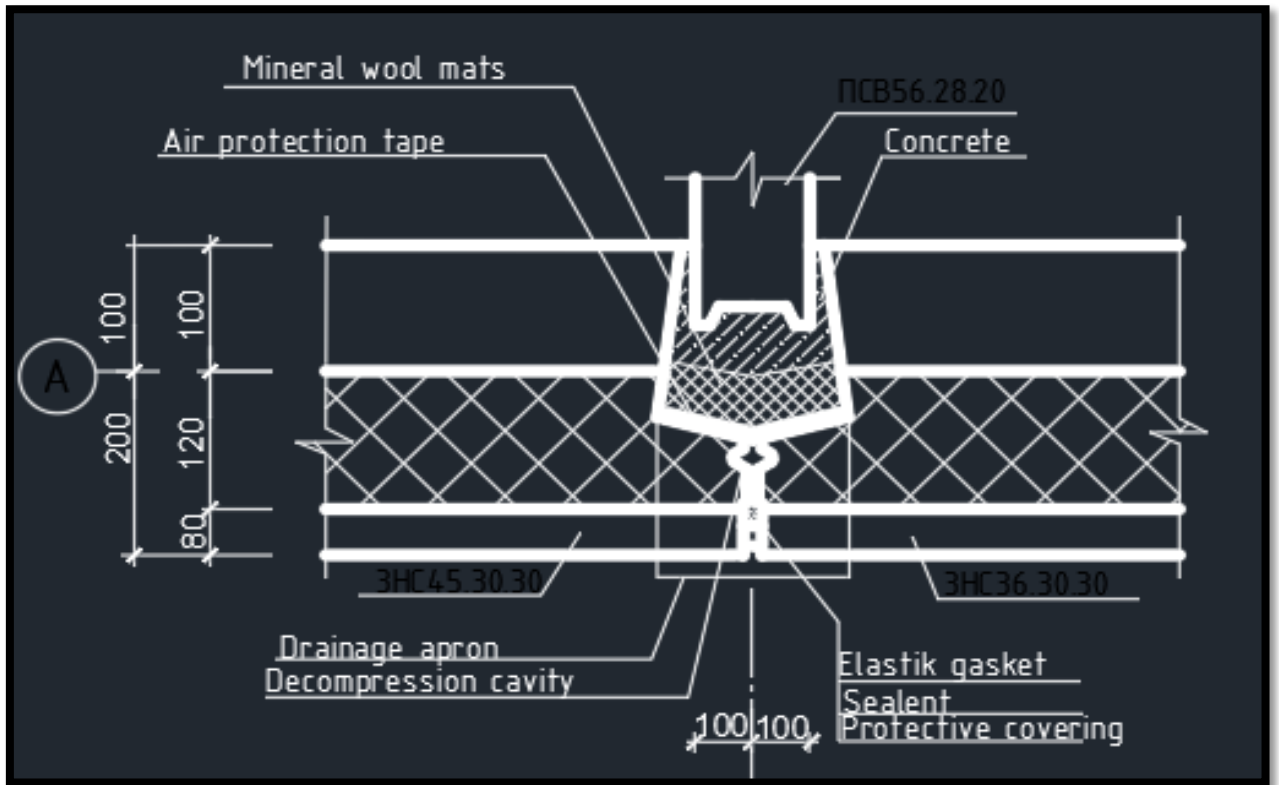


Figure 3.7. Connection of exterior walls and interior wall, top view

The top view shows the connection between interior wall (PCB56.28.20) and two exterior walls (3HC36.3030), the same materials are used for the connection. The contiguity between walls is filled by layer of mineral wool mats for thermo-isolation and by concrete. All elements are welded between each other before concreting.

3.2. Design load calculations

The structure is planned to undergo vertical extension of 2 storeys. Thus, due to increased gravity load, the walls, slabs, and connections of the initial storeys are checked to determine whether they are able to withstand additional load or certain strengthening amount is required. For that, procedure and calculations of design loads are presented in this section. The loads are calculated based on ASCE 7-10 Minimum Design Loads for buildings and other structures.

3.2.1 Dead load

According to Chapter 3 of ASCE 7-10, dead load consists of the weight of all materials, including, but not limited to, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, etc. **Table 3.2** presents dead load calculations.

Table 3.2. Dead load calculations

Member type	Member name*	Area (m ²)	Height or thickness (m)	Volume (m ³)	Unit weight (kg/m ^{2/3})	Quantity on each floor	Total weight on each floor (kN)
Walls Load-bearing (Exterior)	3HC 72.30.30	0,72	3	2,16	2500	1	53,0
	3HC 48.30.30	0,48	3	1,44	2500	1	35,3
	3HC 38.30.30	0,38	3	0,96	2500	2	47,1
	3HC 12.30.30	0,12	3	0,36	2500	1	8,8
	3HC 45.30.30	0,45	3	1,17	2500	2	57,4
	3HC 39.30.30	0,39	3	1,35	2500	1	33,1
	3HC 36.30.30	0,36	3	0,9	2500	2	44,1
	3HC 35.30.30	0,35	3	0,87	2500	1	21,3
	3HC 30.30.30	0,3	3	0,72	2500	1	17,7
Walls Load-bearing (Interior)	ПCB 71.30.20	1,42	3	4,26	2500	10	1029,3
	ПCB 30.30.20	0,6	3	1,8	2500	3	106,7
Slabs	2ПТС 36.24.16	8,64	0,16	1,38	2500	10	339,0
	2ПТС 45.36.16	16,2	0,16	2,59	2500	1	63,6
	2ПТС 45.24.16	10,8	0,16	1,73	2500	7	296,7
	2ПТС 48.30.16	14,4	0,16	2,3	2500	2	113,0
	2ПТС 36.30.16	10,8	0,16	1,73	2500	2	84,8
Stairway	ЛПФ 30.28	8,4	0,16	1,34	2500	1	33,0
	ЛПФ 30.12	4,2	0,16	0,67	2500	1	16,5
	ЛМФ 22.12.09	-	-	-	-	1	12,7
	ЛМФ 35.12.15	-	-	-	-	1	14,0
Roof	ПП	-	-	-	-	34	97,9

Table 3.2. Dead load calculations (Continuation)

Member type	Member name*	Area (m ²)	Height or thickness (m)	Volume (m ³)	Unit weight (kg/m ^{2/3})	Quantity on each floor	Total weight on each floor (kN)
Partition walls	Concrete partition	5,01	3	15,05	1200	1	177,1
Wall Finishings	Gypsum plaster	305,2	-	-	9	1	27,0
	Polystyrene concrete	4,42	3	13,1	1300	1	167,1
	Mineral wool plates	6,62	3	19,87	50	1	9,7
Slab finishings	Soundproofing gasket	170,8	0,025	-	0,3	1	0,5
	Cement-sand screed	170,8	0,045	7,69	1800	1	135,7
	Foamed polyethylene	170,8	0,01	1,71	180	1	3,0
	Laminate	170,8	0,01	1,71	953	1	16,0
	Polyurethane foam	170,8	0,1	17,1	60	1	10,1
Ceiling	Plasterboard	170,8	-	-	12	1	20,1
Windows	ОД 3С 15-12	-	-	-	-	4	1,0
	ОД 3С 15-9	-	-	-	-	1	0,2
Doors	ДВ1 ДГ 21-9	-	-	-	-	5	1,6
	ДВ1 ДГ 21-15	-	-	-	-	3	1,0
	ДВ6 ДГ 21-15	-	-	-	-	6	1,8
	ДВ2 ДГ 21-10	-	-	-	-	3	1,3
Structural load (Floor #1) (kN)							14562,0
Structural load (Floors #2, 3, 4, or 5) (kN)							14646,1
Non-Structural load (Floors #1, 2, 3, 4, or 5) (kN)							3439,6
Total Dead load (Floors #1) (kN)							18001,6
Total Dead load (Floors #2, 3, 4, or 5) (kN)							18085,7
Roof load (kN)							587,7
Total pressure on floor 1 (kPa)							12,0
Total pressure on floors 2-5 (kPa)							12,1
Total pressure on the roof (kPa)							0,4

*Member names (e.g. 3HC 72.30.30; ИСВ 71.30.20; 2ИТС 45.36.16) are given in Russian from the drawings.

The final pressure on first floor is **12,0 kPa**, whereas for the remaining floors – **12,1 kPa**. Roof pressure is **0,4 kPa**. These values are applied to the slabs of the 3D model in SAP2000.

3.2.2 Live load

According to Chapter 4 of ASCE 7-10, the live loads used in the design are the maximum loads expected by the intended use or occupancy, but will not be less than the minimum uniformly distributed unit loads required by Table 4-1, including any permissible reductions. **Table 3.3** and **Table 3.4** provide live load calculations, which are based on values obtained from Table 4-1 of ASCE 7-10. The reduction was performed on roof live load, however the live loads for rooms remained unreduced, due to the value of $K_{LL} \times A_r$ being less than $37,16 \text{ m}^2$ (see column 5 in **Table 3.3**). The uniform live loads were applied separately to the slab of each room of 3D model in SAP2000.

Table 3.3 Live load calculation results for floors 1-5

	Room	Area (m ²)	Uniform live load from ASCE 7-10 (kN/m ²)	$K_{LL} \times A_r$	Live load (kN)
Two-room apartment					
1	Bedroom	15,98	1,44	15,98	23,01
2	Common room	18,42	4,79	18,42	88,23
3	Hall (corridor)	9,02	4,79	9,02	43,21
4	Rest room	2,04	1,92	2,04	3,92
5	Bathroom	3,11	1,92	3,11	5,97
6	Kitchen	13,5	1,92	13,5	25,92
Three-room apartment					
14	Bedroom	18,43	1,44	18,43	26,54
11	Common room	19,69	4,79	19,69	94,32
6	Kitchen	13,5	1,92	13,5	25,92
13	Children room	15,93	1,92	15,93	30,59
5	Bathroom	3,11	1,92	3,11	5,97
12	Hall (corridor)	11,85	4,79	11,85	56,76
4	Rest room	2,04	1,92	2,04	3,92
One-room apartment					
9	Common room	21,42	4,79	21,42	102,60
10	Hall (corridor)	7,13	4,79	7,13	34,15
4	Rest room	2,04	1,92	2,04	3,92
7	Bathroom	2,95	1,92	2,95	5,66
8	Kitchen	11,34	1,92	11,34	21,77
Total live load on one floor (kN):					3614,2458

Table 3.4. Live load calculation results for the roof

Roof live load with reduction							
	Element name	Area (m ²)	Uniform live load from ASCE 7-10 (kN/m ²)	R1	R2	Reduced live load (kN/m ²)	Live Load (kN)
9	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
	III	25,3	0,96	0,92	1	0,88	22,39
Total roof live load (kN):							1343,17

3.2.3 Wind load

The procedure for calculating Main Wind-Force Resisting System (MWFRS) includes seven steps and is done in accordance with Chapters 26 and 27. In this section each step is considered separately.

Step 1. Risk category of the building is II.

Step 2. The basic wind speed is 40 m/s according to SNIP RK 2.1.01.07-85.

Step 3. Wind load parameters are given in **Table 3.5.**

Table 3.5. Wind load parameters

Wind direction. factor, Kd	Exposure category	Topograph. factor, Kzt	Gust factor, G	Building type	Enclosure class.	Internal pressure coef., GC(pi)
0,85	B	1	0,85	Rigid	Enclosed	0,18

Step 4. Velocity pressure exposure coefficient, K_z is calculated by **Equations 3.2.3.1-2:**

$$\text{For } 15 \text{ ft.} \leq z \leq z_g \quad K_z = 2,01 \times (z/z_g)^{2/\alpha} \quad (\text{Eq. 3.2.3.1})$$

$$\text{For } z < 15 \text{ ft.} \quad K_z = 2,01 \times (4,6/z_g)^{2/\alpha} \quad (\text{Eq. 3.2.3.2})$$

(where $z_g=365,76$ m. and $\alpha = 7$ for Exposure Category B)

Step 5. Velocity pressure, q_z is calculated by the **Equation 3.2.3.3:**

$$q_z = 0,613 \times K_z \times K_{zt} \times K_d \times V^2 \text{ (N/m}^2\text{)} \quad (\text{Eq. 3.2.3.3})$$

Step 6. External pressure coefficients, C_p , for leeward, windward, side walls and roof are given in **Table 3.6.**

Table 3.6. External pressure coefficients, C_p

Wall and roof pressure coefficients, C_p			
Surface	Wind direction	L/B	C_p
Windward wall		All	0,8
Leeward wall	⊥ to 115,2-m face	0-1	-0,5
	∥ to 115,2-m face	≥4	-0,2
Side wall	All	All	-0,7
Roof			-0,9

Step 7. Wind pressure, p , is calculated using **Equation 3.2.3.4:**

$$p = q \times G \times C_p - q_i \times (GC_{pi}) \text{ (N/m}^2\text{)} \quad (\text{Eq. 3.2.3.4})$$

The final calculation results for Steps 4-7 are given in **Tables 3.7-8.** The building is 13,2 meter by 115,2 meter. The wind is calculated for both directions: normal to short side (13,2-m) and normal to long side (115,2-m). **Figure 3.8** shows the sides of windward, leeward, side walls, at which pressures were applied in SAP2000.

Table 3.7. External pressures for MWFRS for wind normal to short side (13,2 m.)

For existing 7-storey building					
External Pressures for MWFRS for Wind Normal to 13,2-m Face					
Wall type Roof	Floor #	Elevation, z (m)	Velocity pressure exposure coef., K_z	Velocity pressure, q_z (N/m ²)	External pressure, p (N/m ²)
Windward wall	1	3	0,58	480	326
	2	6	0,62	518	352
	3	9	0,70	581	395
	4	12	0,76	631	429
	5	15	0,81	673	457

	6	18	0,85	709	482
	7	21	0,89	741	504
Leeward wall		All	All	765	-130
Side walls		All	All	765	-455
Roof		23,55	0,92	765	-585

Table 3.8. External pressures for MWFRS for wind normal to long side (115,2 m.)

For existing 7-storey building					
External Pressures for MWFRS for Wind Normal to 115,2-m Face					
Wall type	Floor #	Elevation, z (m)	Velocity pressure exposure coef., Kz	Velocity pressure, qz (N/m ²)	External pressure, p (N/m ²)
Windward wall	1	3	0,58	480	326
	2	6	0,62	518	352
	3	9	0,70	581	395
	4	12	0,76	631	429
	5	15	0,81	673	457
	6	18	0,85	709	482
	7	21	0,89	741	504
Leeward wall		All	All	765	-325
Side walls		All	All	765	-455
Roof		23,55	0,92	765	-585

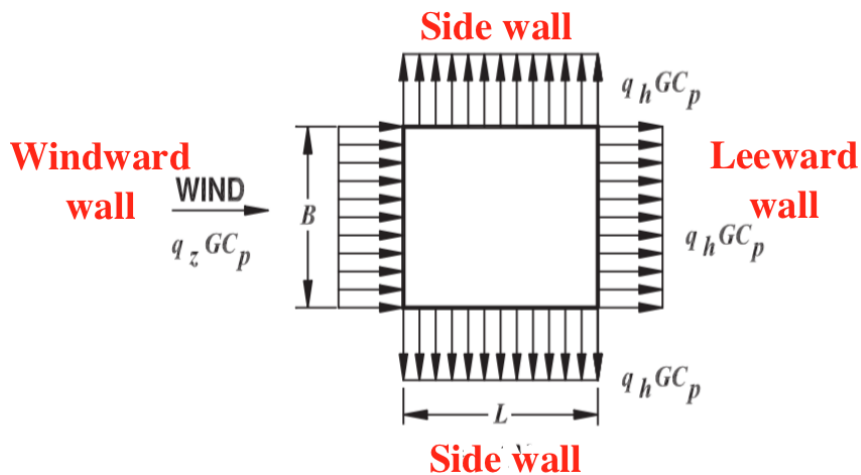


Figure 3.8. Plan view of the building with designation of windward, leeward and side walls
(Source: Figure 27.4-1, ASCE 7-10)

3.2.4 Snow load

Snow load was calculated based on Chapter 7 of ASCE 7-10. As the roof of the building is flat, **Equation 3.2.4** was used to calculate p_r :

$$p_f = 0,7 \times C_e \times C_t \times I_s \times p_g \quad (Eq.$$

3.2.4)

According to SNIP RK 2.1.01.07-85, snow ground pressure (P_g) is equal to $1,5 \text{ kN/m}^2$. As mentioned before, building's risk category is II. Terrain category is B and the roof is fully exposed. Thus, **Table 3.9** shows the calculation results for flat roof snow load. The final snow load value is **$1,35 \text{ kN/m}^2$** , which is applied to the roof of the 3D model in SAP2000.

Table 3.9. Snow load calculation results

Snow load					
$P_g \text{ (kN/m}^2\text{)}$	$C(e)$	$C(t)$	$I(s)$	$P_f \text{ (kN/m}^2\text{)}$	$P_m \text{ (kN/m}^2\text{)}$
1,5	0,9	1	1	1,35	0,96

3.2.5 Seismic load

According to SNIP RK 2-03-30-2006 “Construction in seismic areas. Seismic design code”, rules of seismic design are only applied to regions with seismic intensity of 7, 8, 9 and 10-points. The document considers four regions, namely Eastern-Kazakhstan region, Almaty region, Kyzylorda region, and Mangistau region, to which seismic design must be applied. However, Akmola region, in which Astana is located, has seismic intensity of point 5.8 and does not require seismic design. Thus, seismic load is not considered in this project.

3.3. Analytical model development in SAP2000

A 3-dimensional model of existing building was constructed in SAP2000 with the purpose of performing detailed structural analysis of the structure under lateral and gravity loads. The selected building was constructed by means of prefabricated panels, which are rigidly connected with welding. Therefore, load-bearing elements in the selected building are mainly precast panel walls.

3.3.1 Model geometry

The structure consists of 3 similar blocks connected in one building. Each block involves seven floors with height of 3m each. The width is 13,2m, and the length of each block is 38,4 m, the total length is 115,2m. In order to construct this model in SAP2000 the layout of one section depicted in the **Figure 3.10** was utilized. The construction of the model starts from definition of the element's coordinates in the coordinate system in x, y and z directions. The total amount of

exterior windows is 246, whereas the number of exterior doors is 6. The coordinates are defined considering the location of the windows and doors. **Figure 3.9** shows the coordinate definition process. Model's top view, side view in x-direction, side view in y-direction. The final appearance of the 3-D model is shown in the **Figure 3.15**.

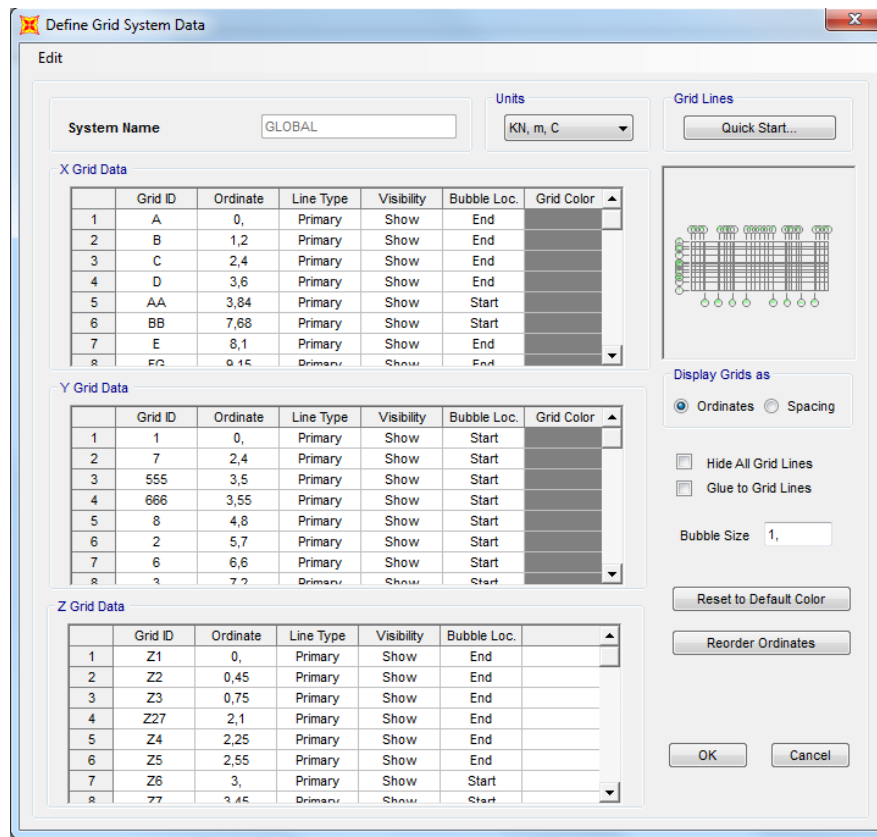


Figure 3.9. Grid System Data

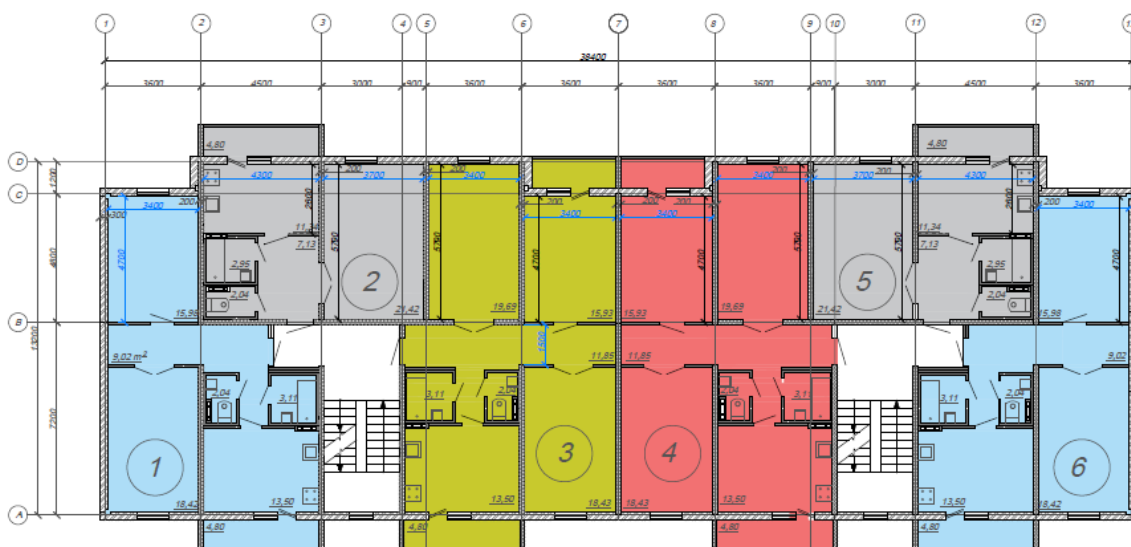


Figure 3.10. Layout of existing building

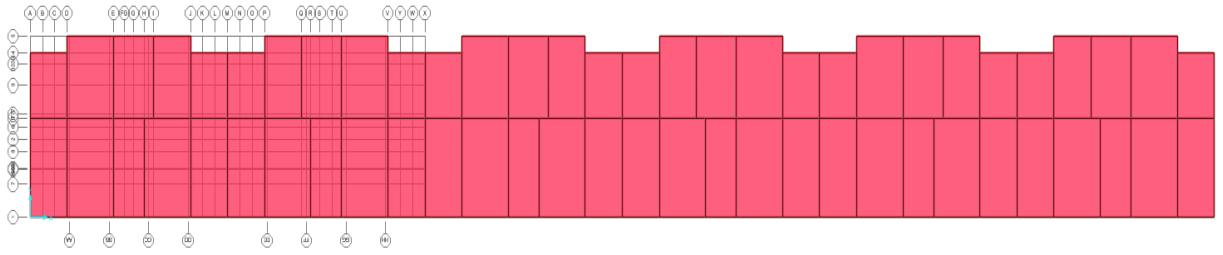


Figure 3.11. Model's top view

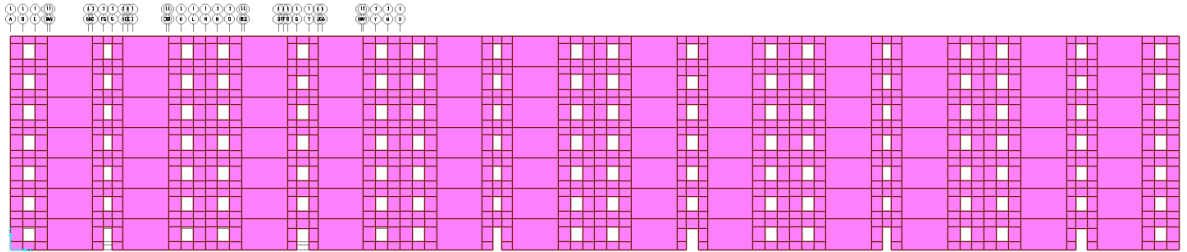


Figure 3.12. Model's side view in x-direction

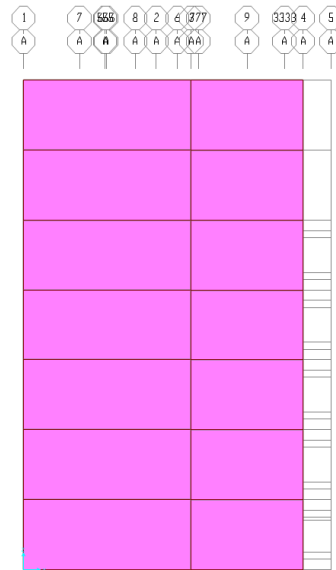


Figure 3.13. Model's side view in y-direction

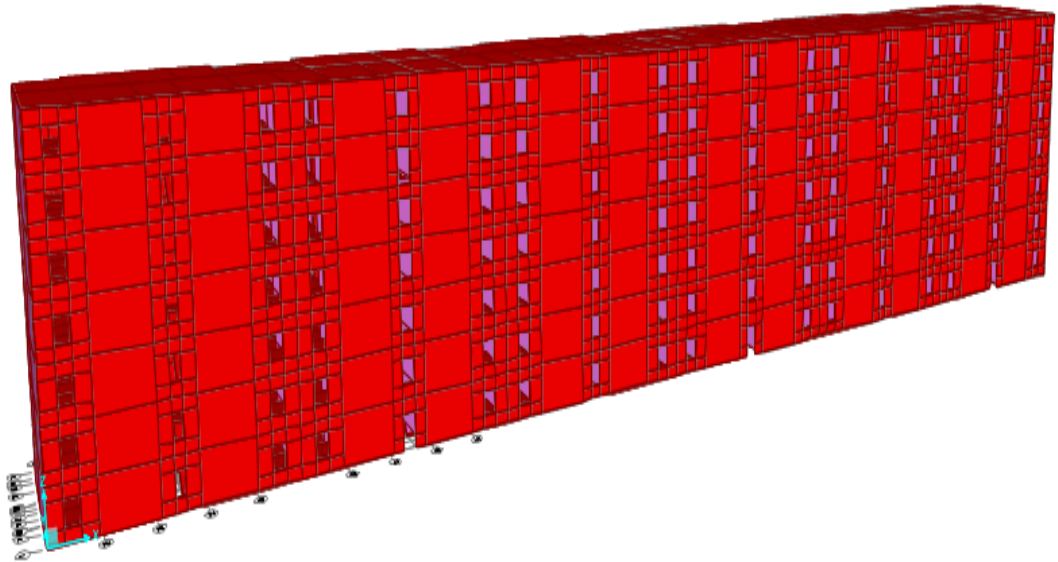


Figure 3.15. Finite element model

3.3.2 Elements and connections

The structure mainly consists of interior walls, exterior walls, floor slabs, ladder slabs and roof slabs. All these elements are constructed in SAP2000 by means of shell sections.

3.3.3 Materials and sections

According to the data on the constructed building, the type of concrete is the same for all elements. It worth mentioning, that selected structure is old Soviet building, which was built according to the old standards. Thus, brand of the concrete and steel grades are provided according to the Russian designations. During material definition process SAP2000 provides a choice between US, Russian, Indian standards, etc. The type of the concrete used for the model according to Russian Standard is B20 with elastic modulus equal to 27 MPa and specific compressive strength 15 MPa. The concrete strength reduction is insignificant over this period, thus it was neglected and full strength was inputted into SAP2000. The rebar for interior and exterior walls is A240, whereas for slabs it is A400. Properties of selected materials, namely Poisson ratio, Coefficient of Thermal Expansion, Shear Modulus and Yield Strength, are provided by the program, once the type of the concrete is chosen. The properties of the concrete B20 and steel grades A240 and A400 can be seen in the **Figure 3.16-3.19** respectively.

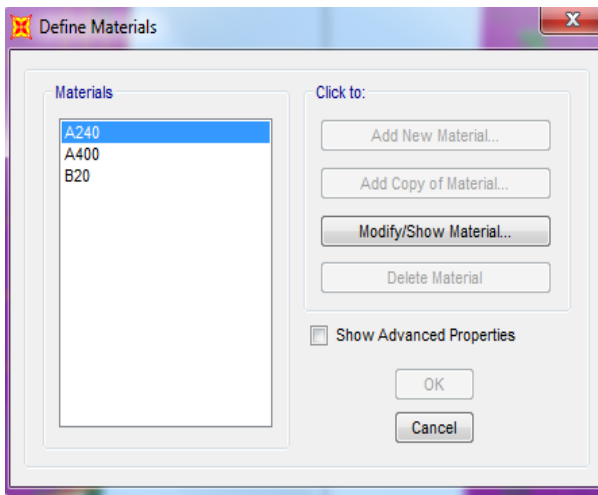


Figure 3.16. Materials Selection

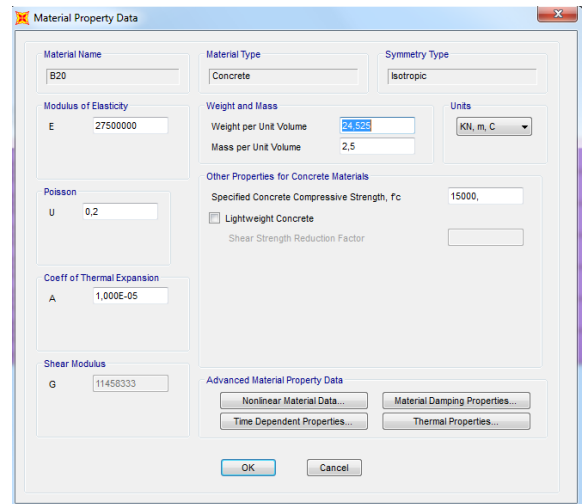


Figure 3.17. Properties of the B20 concrete

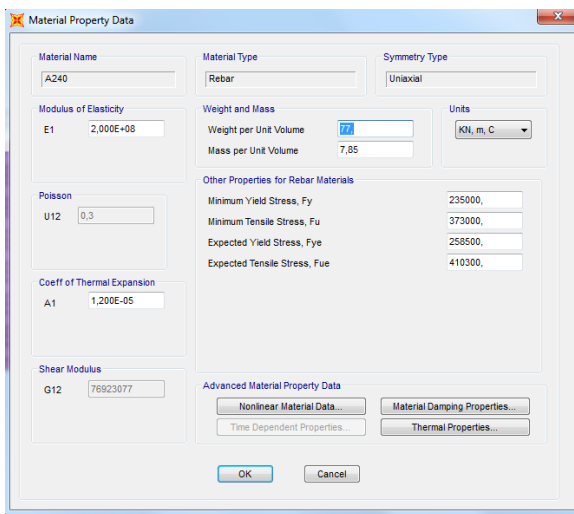


Figure 3.18. A240 steel rebar properties

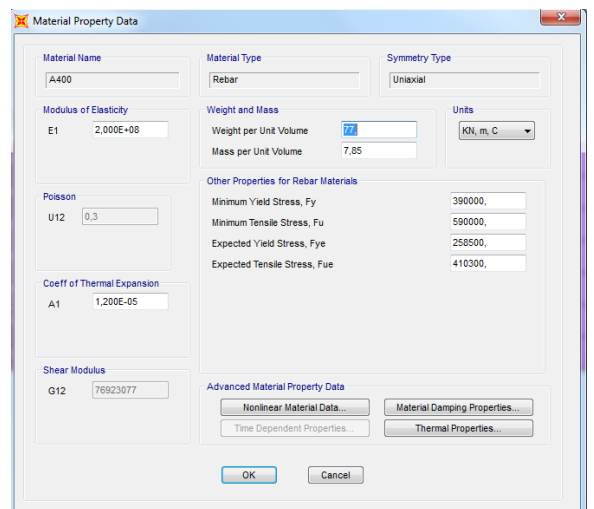


Figure 3.19. A400 steel rebar properties

The thickness of the interior walls is 200 mm, for exterior walls it is 100 mm and for slabs it is 160 mm. Roof and stair elements are of the same properties and dimensions as floor elements. In order to model main structural elements of the building such as slabs and walls, shell elements were utilized, because of its capability to support all loads. Since the thickness of the elements is mainly not greater than one tenth of the span, transverse shearing can be neglected, therefore thin-plate formulation is selected. As for material, previously defined materials are selected. The arrangement of the armature in the elements is taken as default value.

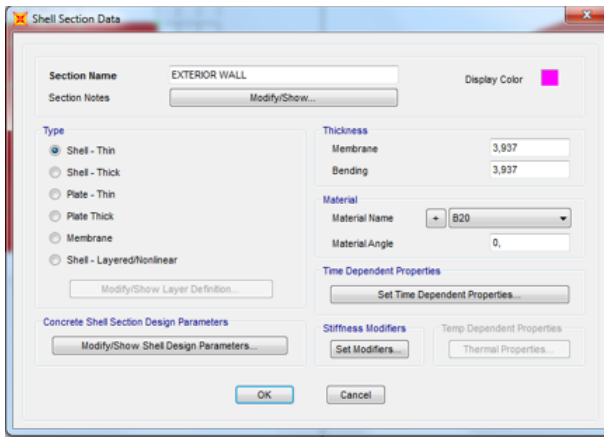


Figure 3.20. Shell section properties for exterior walls

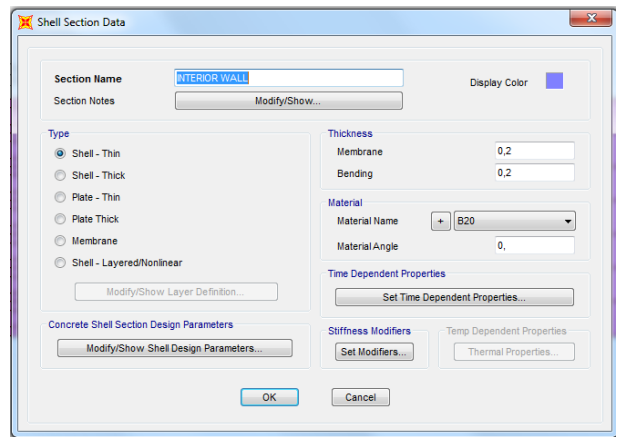


Figure 3.21. Shell Section Properties for interior walls

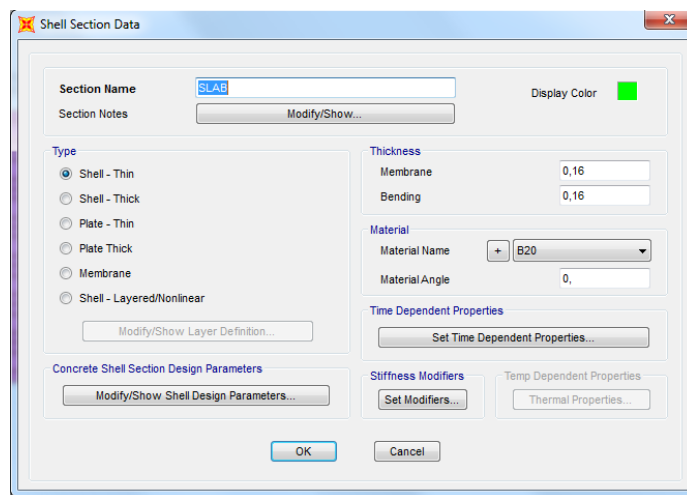


Figure 3.22. Shell section properties for the slabs

3.3.4 Load assignments

For model analysis, six load patterns were introduced; they can be seen in the **Figure 3.23** below. In this design, only Dead, Live and Wind load types were used, Seismic load was not considered due to the fact that the structure is built in seismically-stable region and the height of the structure is only 7 floors. All the loads are applied to the shell elements in the form of uniformly distributed pressure. The numerical values of the applied loads were calculated in the previous sections and applied to the designed model. DEAD 1-7, is the load pattern presenting the dead loads on the structure, for the first floor it is $12,1 \text{ kN/m}^2$, and for the remaining stories it is 12 kN/m^2 . LIVE load pattern represents live load applied to the floor. It worth mentioning, that the values of the live load vary depending on the room. So, for bedroom it is $1,44 \text{ kN/m}^2$, for hall and common room it is $4,79 \text{ kN/m}^2$, for kitchen, bathroom, restroom and children room it is $1,92 \text{ kN/m}^2$. Other load patterns are SNOW and ROOF. They are applied to the roof elements. For wind type loads, two load patterns were defined: WIND X DIRECTION and WIND Y DIRECTION. The first one is for load in x-direction, for the first case when the load is applied

normal to the 13,2 m side it is windward and leeward directions and the second one is for sidewalls values. In the case of wind load application normal to the 115, 2 m side, it is vice versa. WIND X DIRECTION is for sidewalls loads, whereas WIND Y DIRECTION is for loads applied to the windward and leeward walls.

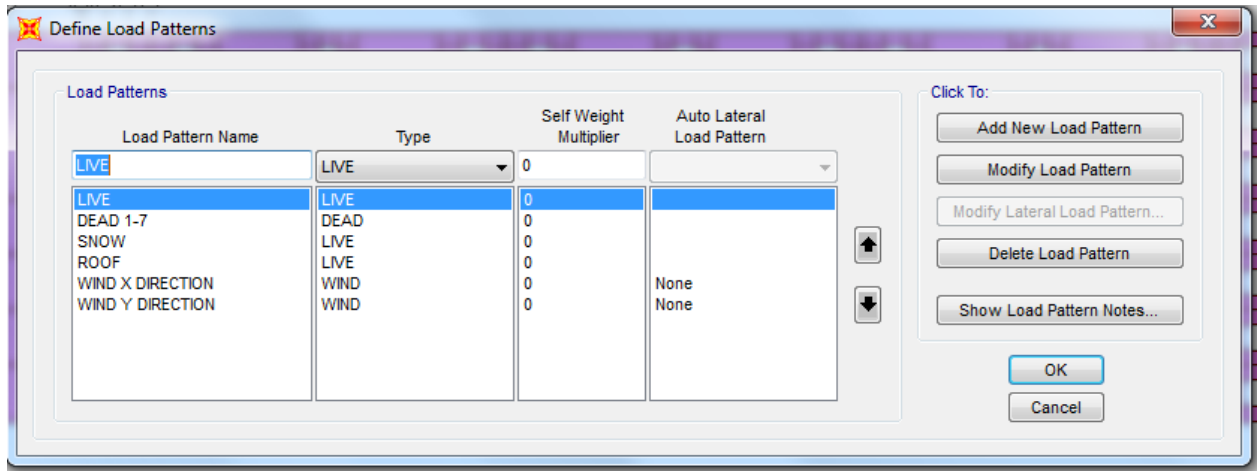


Figure 3.23. Load Patterns

3.3.5 Load combinations

Once the loads are defined and applied to the structural elements, the load combinations should be defined. Based on these patterns, following combinations were generated according to ASCE-7 Code:

- 1) 1.4 DL
- 2) 1.2DL+1.6LL,
- 3) 1.2DL+1.0LL+1.0WLX
- 4) 1.2DL+1.0LL-1.0WLX
- 5) 1.2DL+1.0LL+1.0WLY
- 6) 1.2DL+1.0LL-1.0WLY
- 7) 0,9DL+1.0WLX
- 8) 0,9DL-1.0WLX
- 9) 0,9DL+1.0WLY
- 10) 0,9DL-1.0WLY

The ENVELOPE combination represents the impact of all combinations mentioned above.

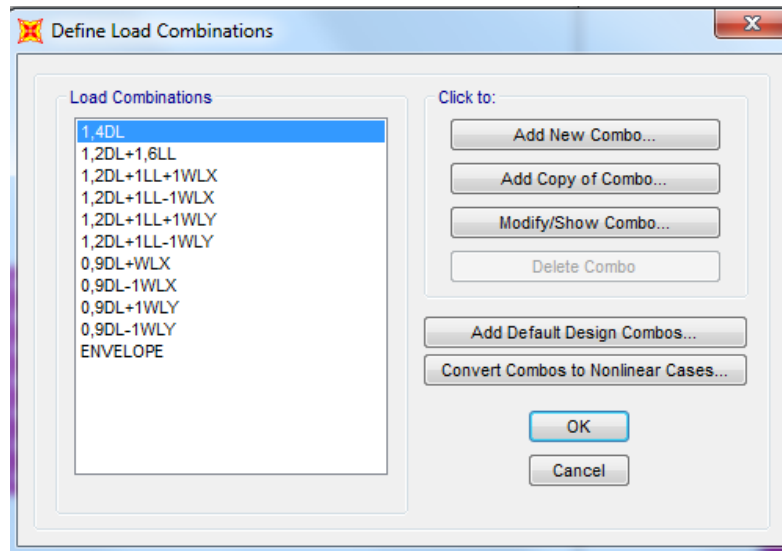


Figure 3.24. Load combinations

3.3.6 Boundary conditions

For this model, only fixed restraints at the foundation of the structure were used. For obtaining more accurate results, the area meshing of each floor elements as well as roof elements was done. This procedure allows transferring loads not only to the two edge points, but to the multiple numbers of joints assigned along the side exposed to loads which makes the analysis more realistic. Each slab element is divided into five rectangles; the view of the meshed stories can be seen in the figure below.

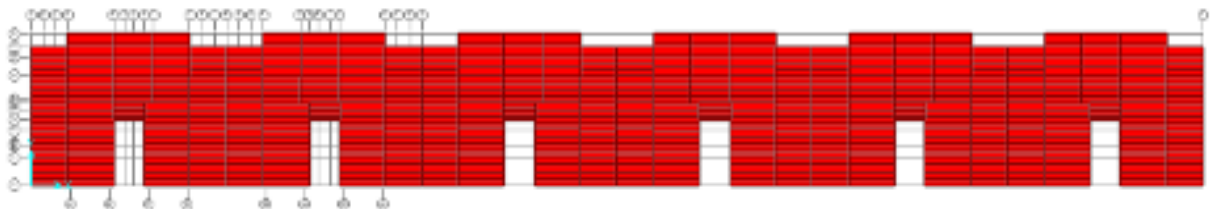


Figure 3.25. The meshed area of the typical floor

The figure below is the deformed shape of the structure under the action of all combinations.

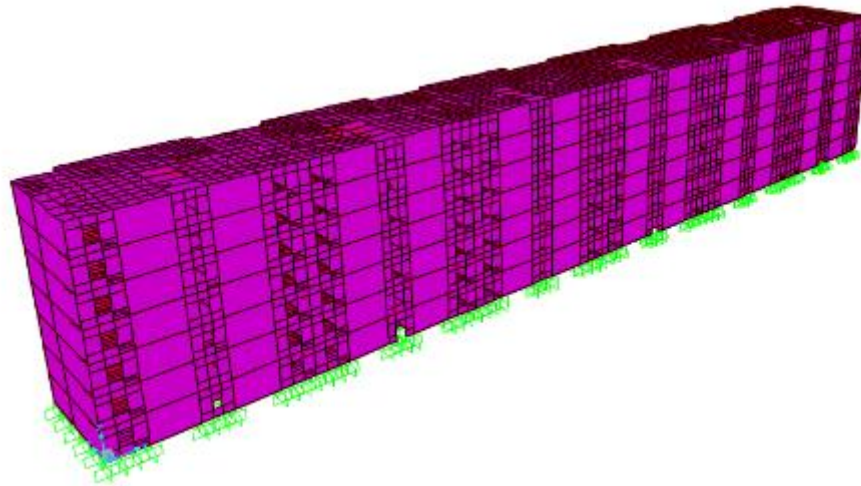


Figure 3.26. Deformed shape of the model

3.4. SAP2000 model check with hand calculations

In order to perform structural design part on the software (SAP2000), it is essential to ensure that values obtained from it are correct or at least in reasonable agreement with hand calculations. For that, vertical, horizontal load and lateral deflection results obtained from SAP2000 were compared with hand calculations.

3.4.1 Hand calculation verification for vertical load case

Live load is chosen as a vertical load case, for which hand calculation verification is done. For that, internal walls shown in the **Figure 3.27** are selected. The view of the wall from the side with area and joint numberings in SAP2000 are shown in the **Figure 3.28**.

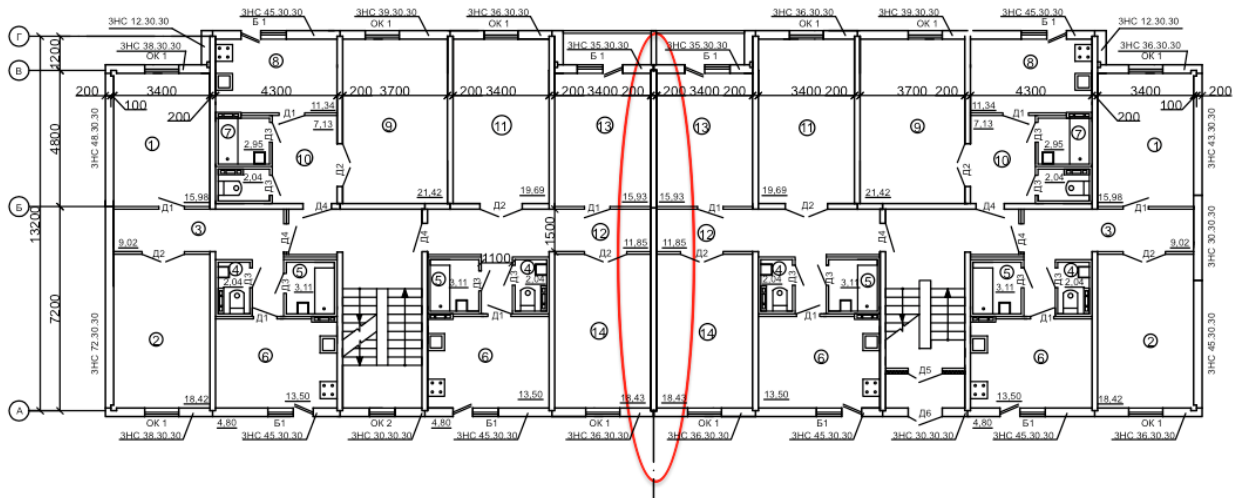


Figure 3.27. Internal walls for calculation verifications

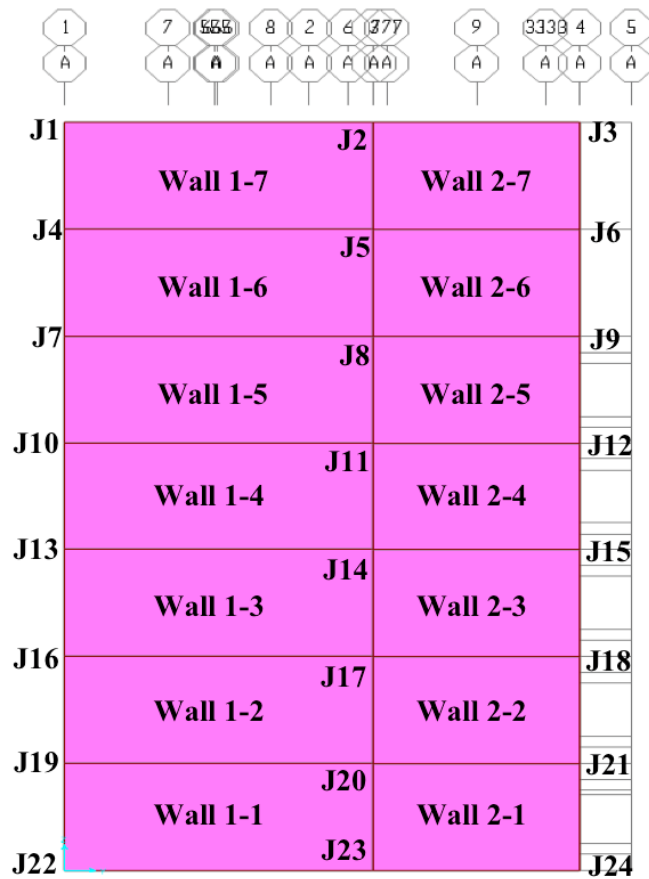


Figure 3.28. Selected wall and joint numberings

The hand calculations for total live load applied to a wall is calculated by multiplying the tributary area of the wall to the uniform pressure applied to the slab resting on those walls, and adding the live load on the floor above. The floors considered here include walls of 6 floors; walls of floor 7 are excluded, as they do not sustain floor live load due to being the walls of the top floor. SAP2000 provides point loads on the joints of the wall in x, y, and z-directions (F1, F2, F3, respectively). The vertical load case (z-direction) is represented by F3, while F2 represents y-direction, and F1 represents x-direction. Thus, the values of F3 for the wall joints are summed up to give the total vertical load on the wall. The output table with results from SAP2000 is presented in **Table 1** of Appendix. **Table 3.10** shows the final results of hand calculation and SAP2000. According to **Figures 3.29** and **3.30**, graphs of manual and software results coincide; thus, SAP2000 results for this vertical load case are verified.

Table 3.10. Live load hand calculation and SAP2000 results

Floor #	Wall name	Tributary area (m ²)		Live load (kN/m ²)		Total wall live load (kN)	Sap2000 results (kN)
		Room 1	Room 2	Room 1	Room 2		
6	Wall 1-6	5,1	19,38	4,79	1,44	52,34	53,6
5	Wall 1-5	5,1	19,38	4,79	1,44	104,67	103,6
4	Wall 1-4	5,1	19,38	4,79	1,44	157,01	157,5
3	Wall 1-3	5,1	19,38	4,79	1,44	209,34	209,4
2	Wall 1-2	5,1	19,38	4,79	1,44	261,68	261,4
1	Wall 1-1	5,1	19,38	4,79	1,44	314,02	314,4
6	Wall 2-6	16,32		1,92		31,33	30,9
5	Wall 2-5	16,32		1,92		62,67	62,7
4	Wall 2-4	16,32		1,92		94,00	93,9
3	Wall 2-3	16,32		1,92		125,34	125,0
2	Wall 2-2	16,32		1,92		156,67	156,5
1	Wall 2-1	16,32		1,92		188,01	187,2

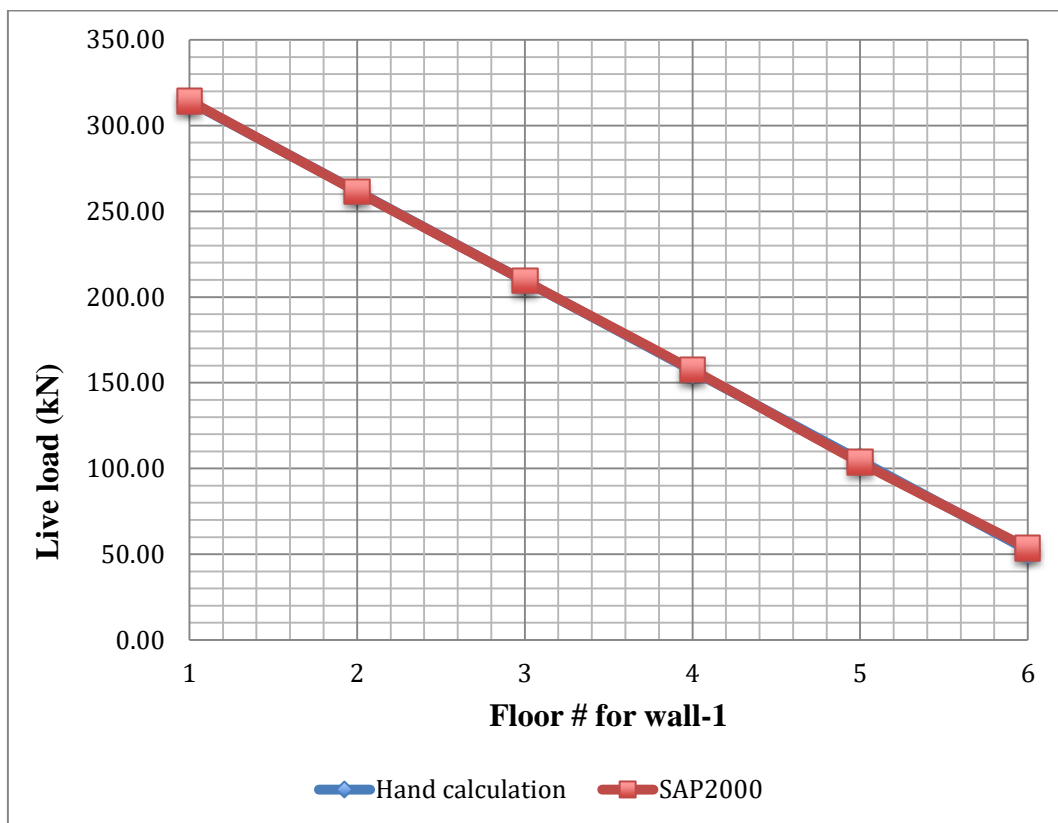


Figure 3.29. Hand calculation and SAP2000 results comparison for Wall-1

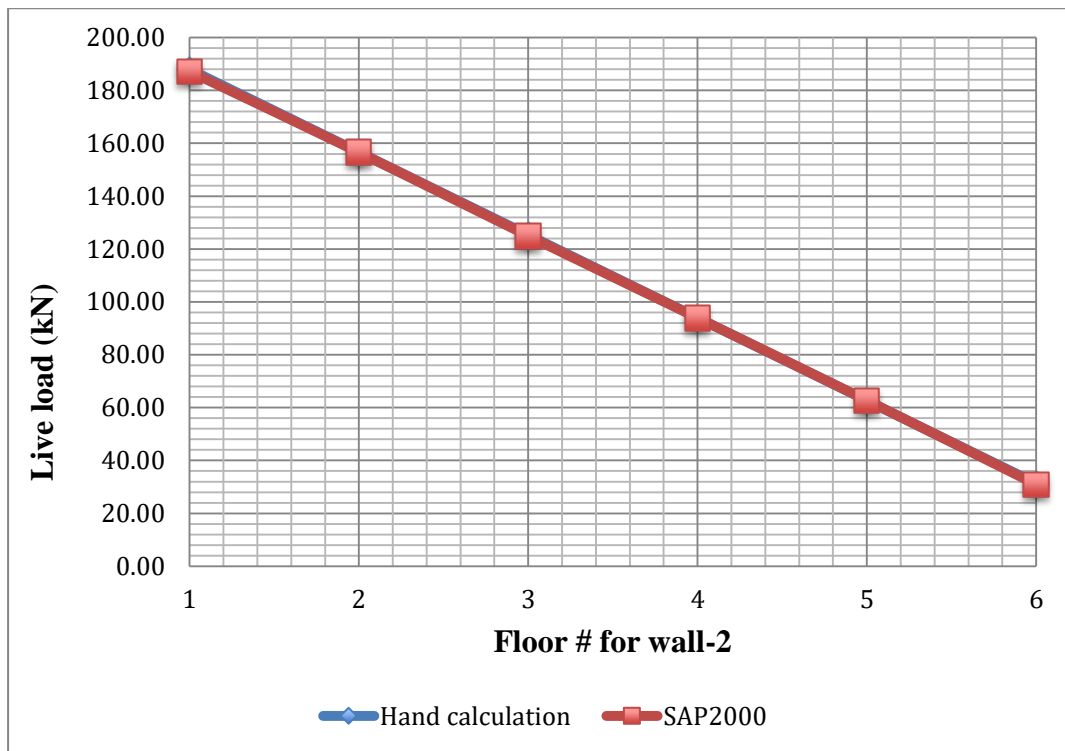


Figure 3.30. Hand calculation and SAP2000 results comparison for Wall-2

3.4.2 Hand calculation verification for horizontal load case

Wind load was chosen for horizontal load verification. SAP2000 provides forces for each joint/node in the wall resultant from wind load applied on the walls. In order to perform SAP2000 results verification with hand calculations, the total horizontal pressure (windward + leeward) for each floor calculated based on ASCE 7-10 was multiplied by story area (building length or width \times story height), and then summed up. Total force for the direction normal to 13,2m is 152,71 kN; whereas for 115,2m-side – 1804,46 kN. In order to obtain total wind force from SAP2000 results, wind load case in y-direction and x-direction were considered separately. For the wind load normal to 13,2m-side, wind load case in x-direction was taken: specifically, all the forces F1 were summed up. The resultant value is 150,59 kN (see **Table 3.11**). For the wind load normal to 115,2m-side, wind load case in y-direction was taken: specifically, all the forces F2 were summed up. The resultant value is 1793,45 kN (see **Table 3.12**). Overall, SAP2000 shows results, which agree with hand calculations within reasonable range. Thus, SAP2000 model is verified based on horizontal case also.

Table 3.11. Wind value comparison normal to short side

Normal to 13,2m-side					
Floor #	Floor area (m ²)	Windward pressure, p (N/m ²)	Leeward pressure, p (N/m ²)	Total pressure, p (N/m ²)	Total F (kN)
1	39,6	326	130	456	18,07
2	39,6	352	130	482	19,09
3	39,6	395	130	525	20,80
4	39,6	429	130	559	22,15
5	39,6	457	130	587	23,26
6	39,6	482	130	612	24,23
7	39,6	504	130	634	25,09
Total wind load from hand calculation (kN)				152,71	
Total wind load (F1) from SAP2000 (kN)				150,59	

Table 3.12. Wind value comparison normal to long side

Normal to 115,2m-side					
Floor #	Floor area (m ²)	Windward pressure, p (N/m ²)	Leeward pressure, p (N/m ²)	Total pressure, p (N/m ²)	Total F (kN)
1	345,6	326	325	651	225,12
2	345,6	352	325	677	234,01
3	345,6	395	325	720	248,96
4	345,6	429	325	754	260,67
5	345,6	457	325	782	270,43
6	345,6	482	325	807	278,89
7	345,6	504	325	829	286,39
Total wind load (kN)				1804,46	
Total wind load (F2) from SAP2000 (kN)				1793,45	

3.4.3 Lateral deflection analysis and check under wind load

Lateral drift due to wind load normal to 115,2m-side is calculated manually and compared to SAP2000 results. The detailed calculations for the drift are described next.

In order to perform drift calculations for a 7-storey building, the structure was treated as a cantilever beam fixed on one end (see **Figure 3.31**).

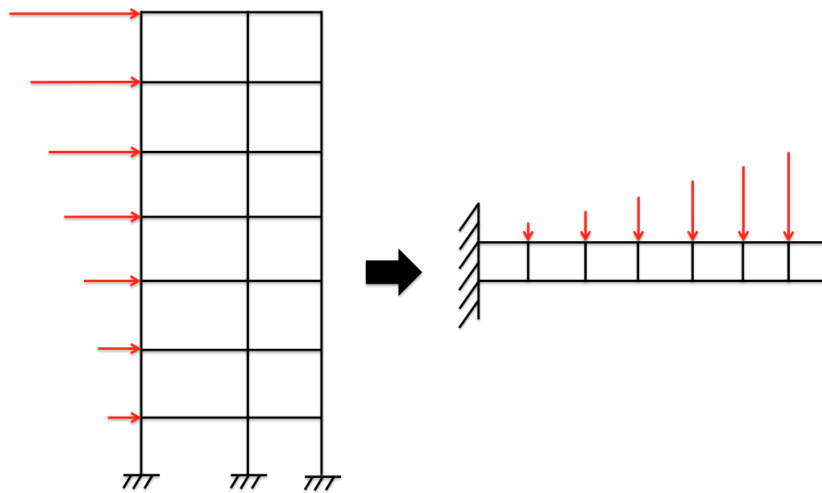


Figure 3.31. Simplification for drift calculation purpose: a structure treated as a cantilever beam

Initially, wind was applied as a pressure to each storey. For calculations, pressure was transferred into a point-load applied at the top of each storey (see **Table 3.13** and **Figure 3.32**) using equation:

$$F_i = \frac{p_i + p_{i+1}}{2} \times h \times l$$

where, p_i – pressure at storey i

p_{i+1} – pressure at storey $i+1$

h – height of the storey (=3m)

l – length of the structure (=115,2m)

Table 3.13. Calculation results for point-loads applied at each storey

Storey #	Storey height h (m)	Total pressure = Leeward + Windward p (N/m ²)	Point-load F (kN)
1	3	651	230
2	3	677	241
3	3	720	255
4	3	754	266
5	3	782	275
6	3	807	283
7	3	829	143

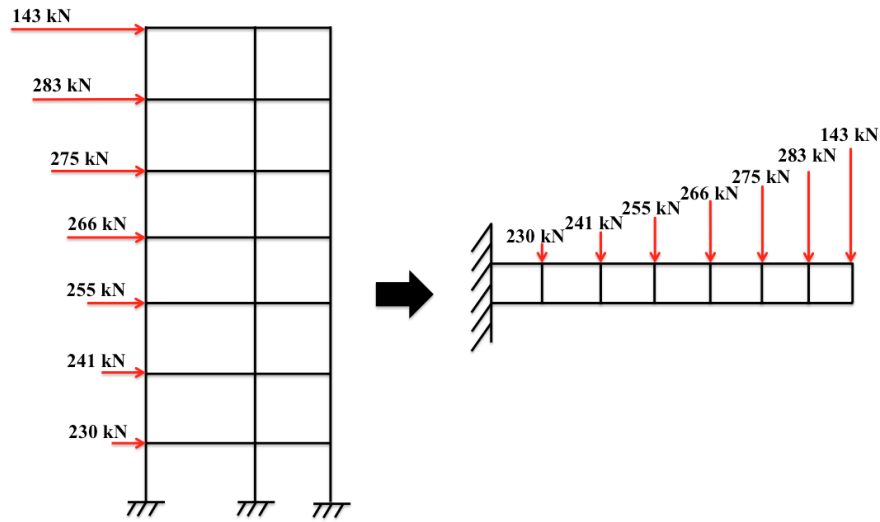


Figure 3.32. Point-loads applied at each storey

In order to find drift at each storey, virtual work method is used. The beam (structure) is considered separately at each case: 1-storey, 2-storey, 7-storey; and shear and moment equations for each case (location) are found. Also, virtual (unit) load is applied at each storey case; and, similarly, virtual shears and moments are found for each storey case. The total drift (flexural + shear) was found using the following equation:

$$\Delta_i = \underbrace{\frac{1}{EI} \times \int_a^b (\mathbf{m} \times \mathbf{M}) dx}_{\text{flexural drift}} + \underbrace{\frac{1}{GA} \times \int_a^b (\mathbf{v} \times \mathbf{V}) dx}_{\text{shear drift}}$$

where, E = modulus of elasticity = 27,5 MPa

$$I = \text{moment of inertia} = \frac{1}{12} b d^3 = \frac{1}{12} \times 0,2 \times 12^3 = 28,8 \text{ m}^4$$

$$G = \text{modulus of rigidity} = \frac{E}{2(1+\nu)} = \frac{27,5}{2(1+0,2)} = 11,458 \text{ MPa}$$

$$A = \text{cross-section area} = \frac{2}{3} \times 0,2 \times 12 = 1,6 \text{ m}^2$$

\mathbf{m} = virtual moment

\mathbf{M} = real moment

\mathbf{v} = virtual shear

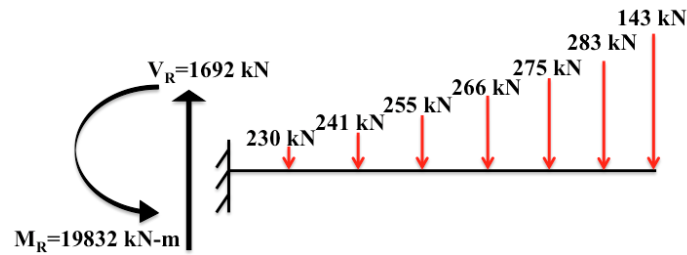
\mathbf{V} = real shear

\mathbf{a}, \mathbf{b} = start and finish locations for the specific case

\mathbf{i} = storey number

The following is the calculation process:

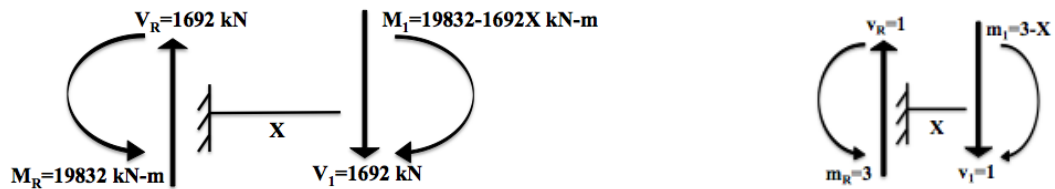
Reactions.



Real forces

Virtual forces

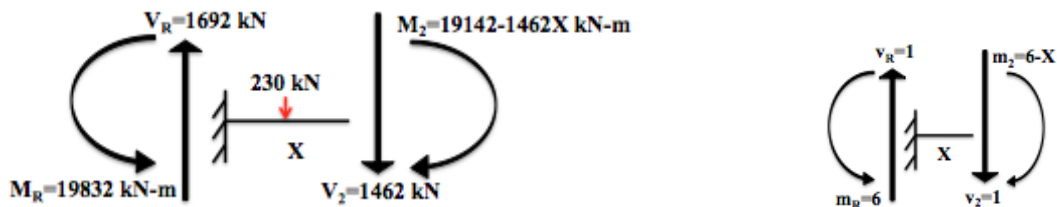
Case I. $x \in 0 \sim 3 \text{ m}$



$$\Delta_1 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^3 (3 - x) \times (19832 - 1692x) dx + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^3 1692 dx =$$

$$0,10307 \text{ mm} + 0,2769 \text{ mm} = \mathbf{0,37997 \text{ mm}}$$

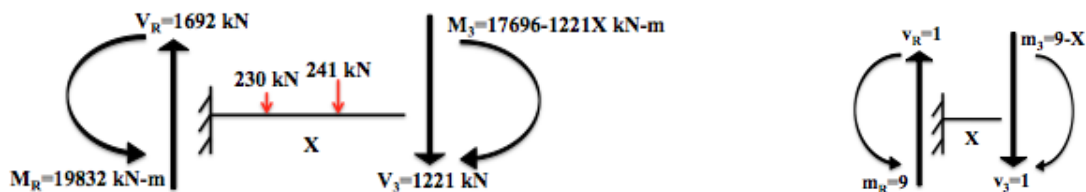
Case II. $x \in 0 \sim 6 \text{ m}$



$$\Delta_2 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^6 (6 - x) \times (19142 - 1462x) dx + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^6 1462 dx =$$

$$0,36859 \text{ mm} + 0,4785 \text{ mm} = \mathbf{0,84709 \text{ mm}}$$

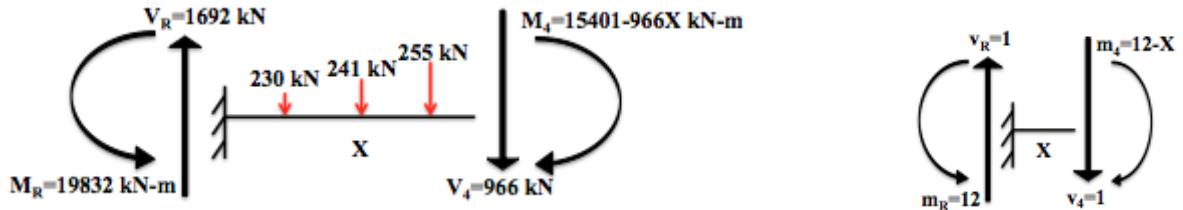
Case III. $x \in 0 \sim 9 \text{ m}$



$$\Delta_3 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^9 (9 - x) \times (17696 - 1221x) dx + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^9 1221 dx =$$

$$0,7176 \text{ mm} + 0,5994 \text{ mm} = \mathbf{1,317 \text{ mm}}$$

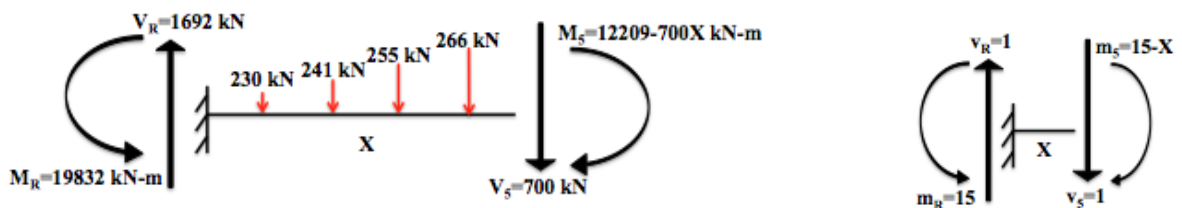
Case IV. $x \in 0 \sim 12 \text{ m}$



$$\Delta_4 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^{12} (12 - x) \times (15401 - 966X) dX + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^{12} 966 dx =$$

$$1,0488 \text{ mm} + 0,6323 \text{ mm} = \mathbf{1,6811 \text{ mm}}$$

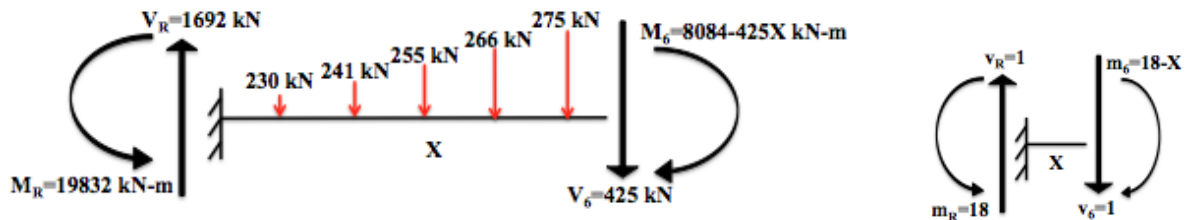
Case V. $x \in 0 \sim 15 \text{ m}$



$$\Delta_5 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^{15} (15 - x) \times (12209 - 700X) dX + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^{15} 700 dx =$$

$$1,237 \text{ mm} + 0,5727 \text{ mm} = \mathbf{1,8097 \text{ mm}}$$

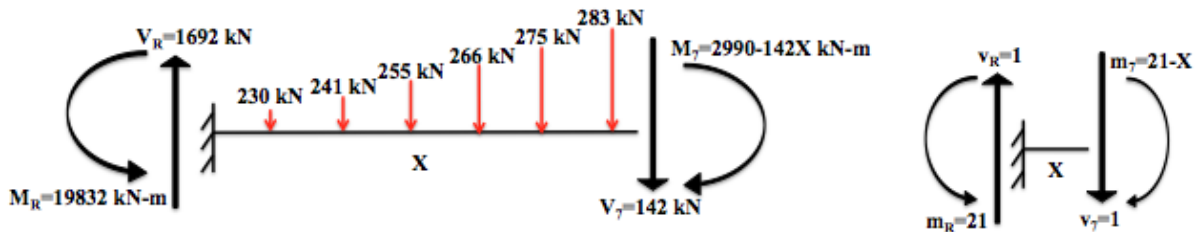
Case VI. $x \in 0 \sim 18 \text{ m}$



$$\Delta_6 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^{18} (18 - x) \times (8084 - 425X) dX + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^{18} 425 dx =$$

$$1,132 \text{ mm} + 0,4173 \text{ mm} = \mathbf{1,5493 \text{ mm}}$$

Case VII. $x \in 0 \sim 21 \text{ m}$



$$\Delta_7 = \frac{1}{27,5 \text{ MPa} \times 28,8 \text{ m}^4} \times \int_0^{21} (21 - x) \times (2990 - 142X) dX + \frac{1}{11,5 \text{ MPa} \times 1,6 \text{ m}^2} \times \int_0^{21} 142 dx =$$

$$0,5557 \text{ mm} + 0,1627 \text{ mm} = \mathbf{0,7184 \text{ mm}}$$

The **Table 3.14** and **Figure 3.33** show the result comparison for flexural, shear, total drifts, as well as drift obtained from SAP2000.

Table 3.14. Drift comparison

Floor #	Hand calculation			SAP2000 Drift (mm)
	Flexural Drift (mm)	Shear Drift (mm)	Total Drift (mm)	
7	0,5557	0,1627	0,7184	1,967
6	1,132	0,4173	1,5493	1,791
5	1,237	0,5727	1,8097	1,654
4	1,0488	0,6323	1,6811	1,423
3	0,7176	0,5994	1,317	1,275
2	0,36859	0,4785	0,84709	0,798
1	0,10307	0,2769	0,37997	0,347

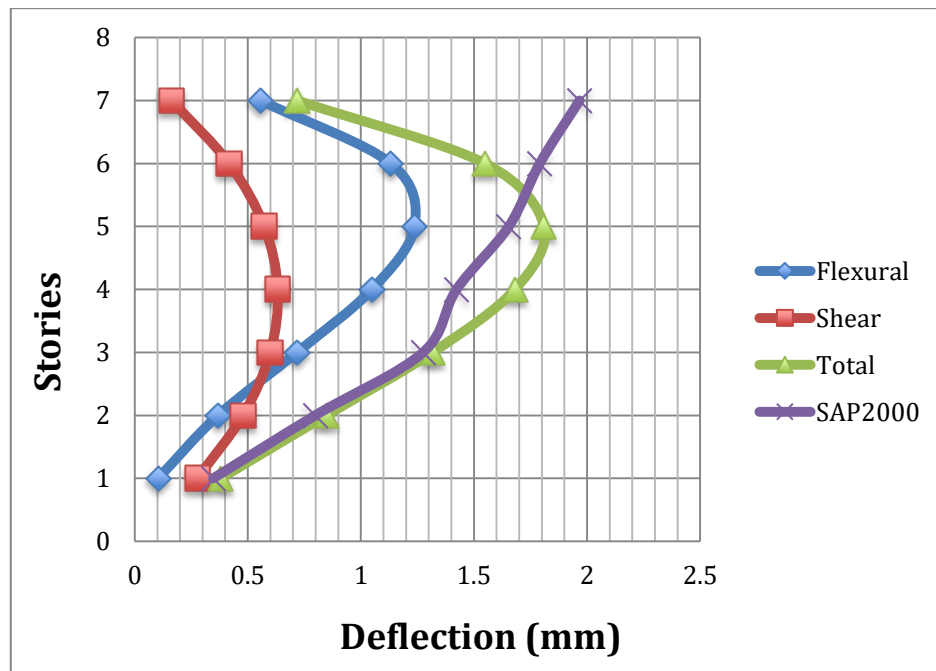


Figure 3.33. Flexural, shear, total and SAP2000 drift values comparison

Flexural drift has higher contribution to the total drift in upper floors, whereas shear drift – in lower floors. This behavior is predictable, because flexural drift dominates in high-rise buildings, whereas shear drift dominates in lower-rise buildings (see **Figures 3.34** and **3.35**).

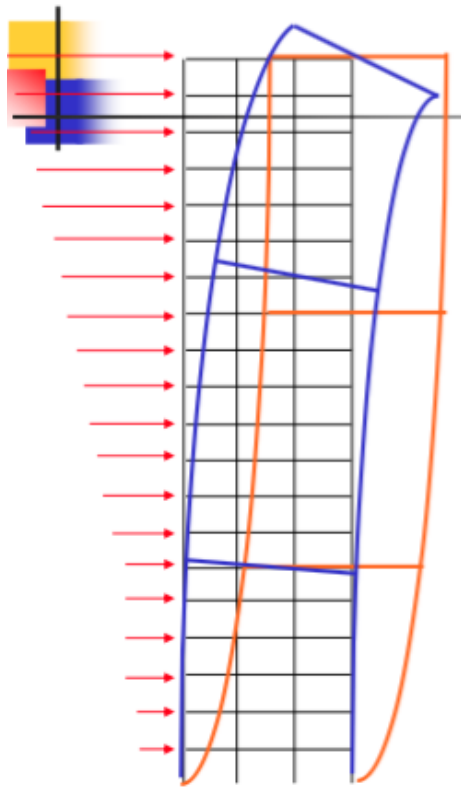


Figure 3.34. Flexural (blue) and shear (orange) drifts (**Source:** Zhang, 2018)

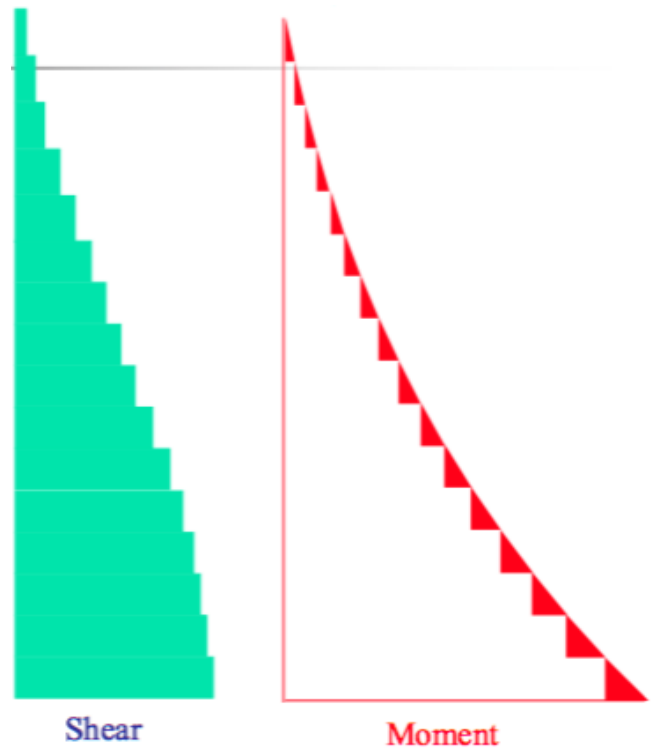


Figure 3.35. Shear and moment drifts for high-rise building (**Source:** Zhang, 2018)

Overall, the values are very small (in mm). This might be due to the reason that the structure is low-rise and experiences a small drift. SAP2000 also provides small values similar to hand calculations. Thus, SAP2000 model based on the drift may be considered as reliable.

3.4.4 Internal force (M , N , V) analysis under all possible loads

Once the model is constructed and the analysis is performed, software provides the output for all structural elements. SAP2000 interprets the internal forces as the forces and moments resulting from integration of stresses over the element thickness. Load combinations taken for the analysis of internal forces are those ten combinations mentioned in the section 3.4.5. For detailed analysis of the model in SAP2000, two cases were considered. In the first one the wind load was applied to the longest side, which is 115,2m. In the second case it was applied to the shortest side, which is 13,2m. Thus, the internal forces (M , N , and V) developed in each shell element of the structure under the action of load combinations mentioned earlier are provided in the appendix for both cases of wind load application (SAP2000, 2015).

Below are explanations of designations used in SAP2000 for expressing moment, axial and shear forces generated in shell elements. It worth mentioning, that these forces are measured per unit of in plane length. The direction of the forces is depicted in the **Figure 3.36**.

- F11- The area element internal F11 membrane direct force per length reported in the area element local coordinate system.
- F12- The area element internal F12 membrane shear force per length reported in the area element local coordinate system.
- F22- The area element internal F22 membrane direct force per length reported in the area element local coordinate system.
- M11-The area element internal M11 plate bending moment per length reported in the area element local coordinate system.
- M22-The area element internal M22 plate bending moment per length reported in the area element local coordinate system.
- M12-The area element internal M12 plate twisting moment per length reported in the area element local coordinate system.
- V13-The area element internal V13 plate transverse shear force per length reported in the area element local coordinate system.
- V23-The area element internal V23 plate transverse shear force per length reported in the area element local coordinate system.
- FMax- the maximum principal internal membrane force.
- FMin- The minimum principal internal membrane force.
- MMax- The maximum principal plate moment.
- MMin –The minimum principal plate moment.

Further, these results will be used for structural detailing of structural members, based on which strengthening amount will be figured out and final reinforcement detailing will be decided.

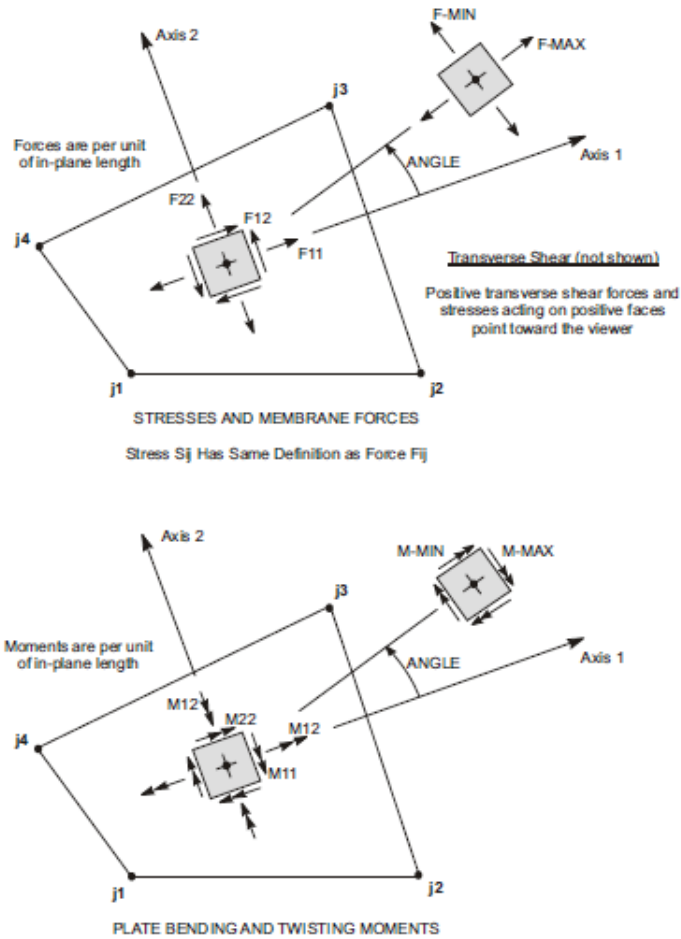


Figure 3.36. Shell Elements stresses, resultant internal forces and moments (Source: SAP2000 manual)

3.4.4.1. Shear walls

The structure, on which analysis is performed, consists of 10 types of shear walls. **Table 3.15** below describes dimensions and reinforcement detailings for their cross-sections.

Table 3.15. Dimensions and cross-section reinforcement detailings for shear walls

Wall type	Exterior/Interior	Wall length (mm)	Wall width (mm)	Rebar diameter (mm)	Spacing (mm)	Rebar # on 2 layers
1	Interior	5700	200	28	600	20
2	Interior	6000	200	28	650	20
3	Interior	4800	200	25	500	20
4	Interior	7200	200	32	800	20
5	Exterior	3000	100	16	300	20
6	Exterior	3600	100	16	400	20
7	Exterior	3900	100	16	400	20
8	Exterior	4500	100	18	450	20
9	Exterior	4800	100	18	500	20
10	Exterior	7200	100	22	750	20

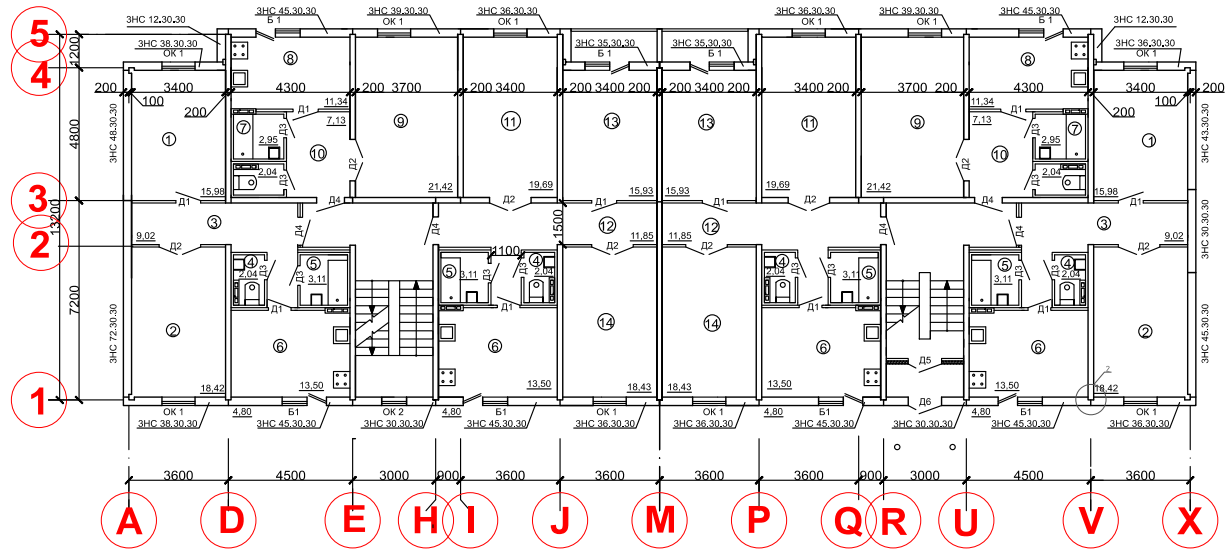


Figure 3.37. Wall locations on axis 1-5 and A-X (Plan view)

Figure 3.37 above and **Table 3.16** below show the location and wall type for each wall on one floor.

Table 3.16. Wall types and their locations on axis 1-5 and A-X

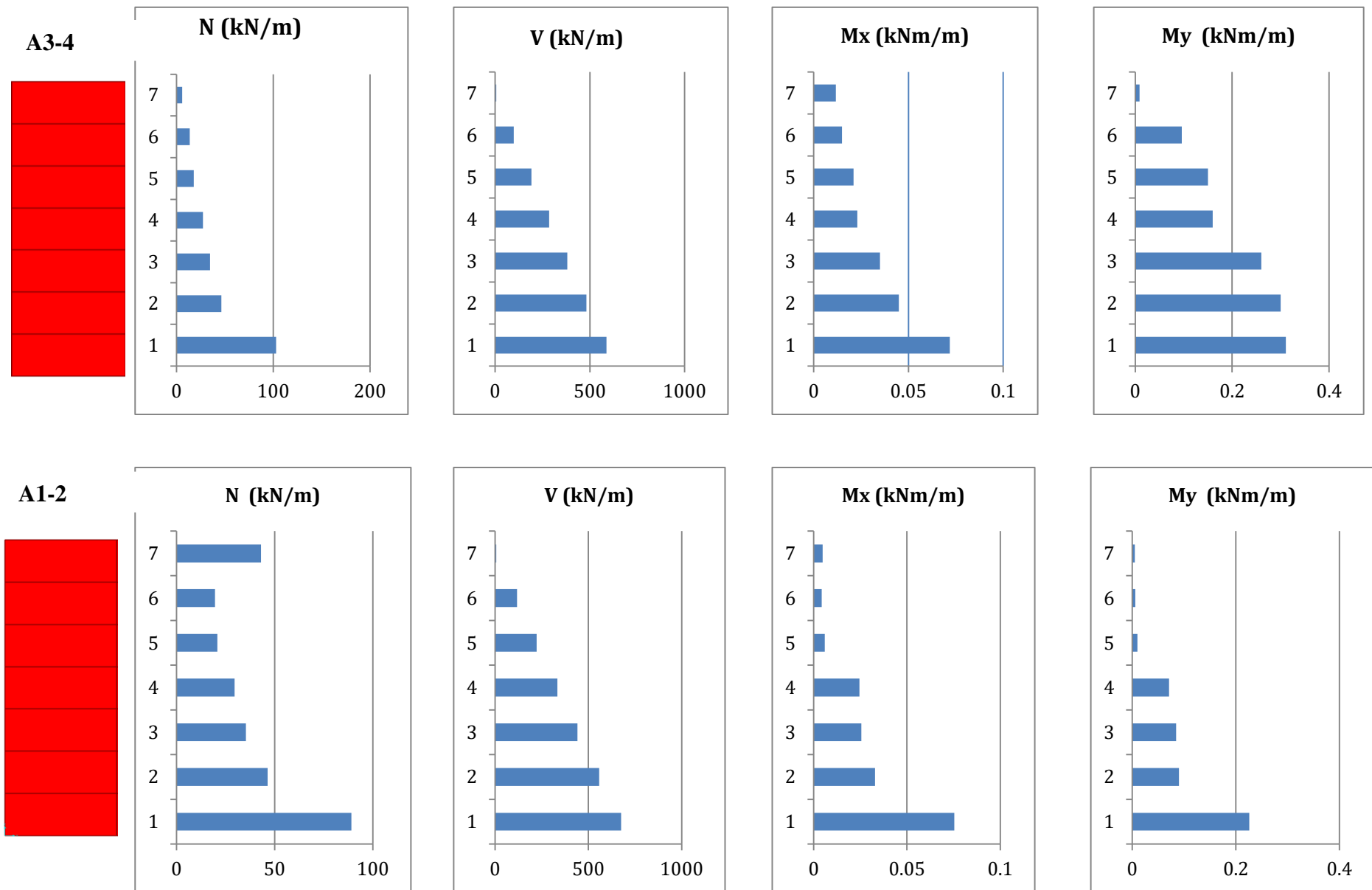
Wall type	Wall location on axis 1-5 and A-X (see Figure 3.37)
1	D 1-2; E 1-2; H 1-2; J 1-2; P 1-2; R 1-2; U 1-2; V 1-2
2	E 3-5; I 3-5; J 3-5; P 3-5; Q 3-5; U 3-5; V 3-5
3	M 3-4
4	M 1-3
5	1 E-H
6	1 R-U
7	5 E-H
8	1 D-E; 1 H-J; 1 P-R; 1 U-V; 5 D-E; 5 P-R; 5 U-V
9	A 3-4; X 3-4
10	A 1-3; X 1-3

Internal forces (M, N, V) for each specific wall were extracted from SAP2000 and their values are presented in **Table 3.17** and plotted in **Figures 3.38-43**. **Table 3.17** only contains values for walls located on A 3-4, A 1-3 and D 3-4; the values for the rest of the walls are in the **Table 3** (Appendix). Similarly, the graphs of these walls are presented in **Figures 3.38-39**; the graphs for the rest of walls are presented in **Figures 1** (Appendix).

Table 3.17. SAP2000 internal forces (M, N, V) values for walls located on A 3-4, A 1-3, D 3-4

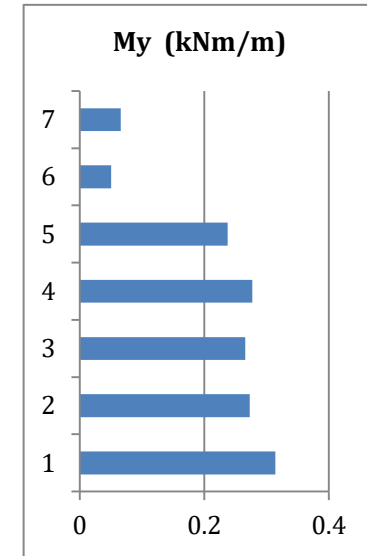
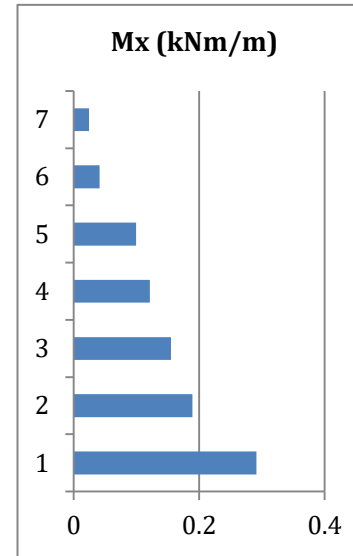
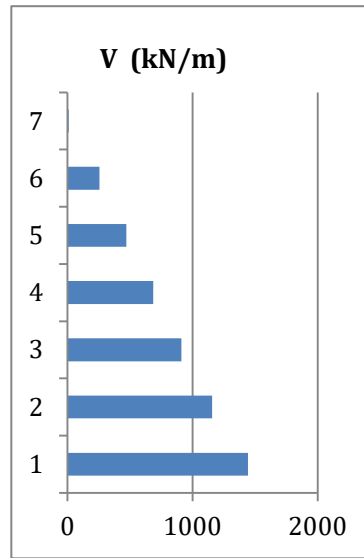
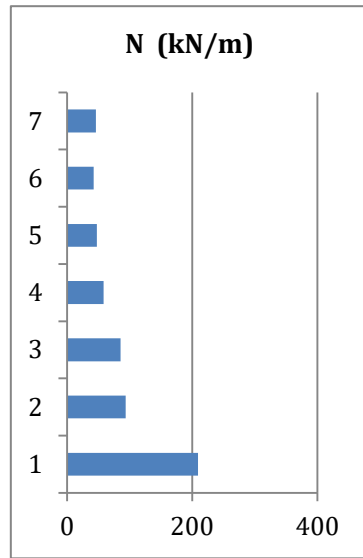
Wall location	Floor #	Wall name in SAP2000	N (F22) (kN/m)	V (F11) (kN/m)	M11 (kNm/m)	M22 (kNm/m)	M MAX (kNm/m)
A 3-4	1	271	-103,02	-588,16	0,033569	0,3109	0,4806
	2	272	-46,46	-482,12	-0,045	-0,3	0,6114
	3	273	-34,67	-381,52	-0,035	0,26	0,5778
	4	274	-27,26	-284,89	-0,023	0,16	0,5893
	5	275	-17,79	-192,3	-0,021	0,15	0,5965
	6	837	-13,744	-98,42	-0,015	-0,096	0,5011
	7	838	-5,95	-5,86	0,0117	0,009	0,9487
A 1-3	1	270	-89	-675,26	0,0754	0,2257	0,5612
	2	269	-46,4	-558,1	0,0329	-0,09	0,7153
	3	268	-35,31	-442,02	0,0256	-0,0845	0,6139
	4	267	-29,5	-333,78	0,0245	0,071	0,6988
	5	266	-20,78	-222,31	0,0059	-0,01	0,6123
	6	836	-19,6	-117,14	0,00436	0,006	0,5917
	7	835	-43,01	-5,7	0,00477	0,005	0,9714
D 3-4	1	631	-209,21	-1441,9	-0,2913	-0,31389	0,0325
	2	629	-93,43	-1156,6	0,189	0,27301	0,5736
	3	627	-85,64	-909,93	-0,155	-0,2655	0,2202
	4	625	-58,65	-685,33	0,1213	0,277	0,6079
	5	623	-47,64	-470,12	0,0993	-0,2373	0,3231
	6	1404	-42,45	-255,37	0,04138	0,05041	0,8057
	7	1402	-46,4	-9,46	0,02437	-0,06572	0,6353

The plots in **Figure 3.38-39** represent variation of axial force, shear force and moments in x and y directions with height for walls located on axis A 3-4, A 1-3, D 3-4 and D 1-2. It can be clearly seen that the values of axial force linearly increase as the height decreases, so the maximum value of the force is on the first floor, whereas the minimum value is on the seventh floor. The reason behind this is that the lower floors experience larger vertical loads due to the weight of the floors located above. Shear force also increases at lower height levels. This can be explained by portal frame method, the summation of shear forces at the highest level will be equal to the lateral load applied at this level. The summation of shear forces in each element at the lower floors will be equal to the summation of lateral forces applied to the floors above. Therefore the values of shear force at the lower heights will be larger. As for moments, then the situation is similar; the largest moment is at the first floor. It decreases with height; therefore at the 7th floor it has the lowest value. When the structure is exposed to the lateral loads, the distance between load applied and first floor is the largest, therefore it experiences the largest moment. The similar situation is with the vertical loads.

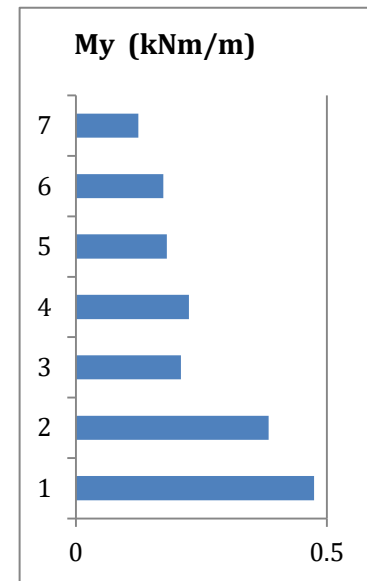
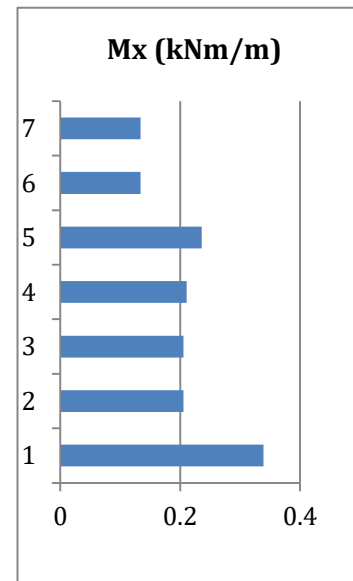
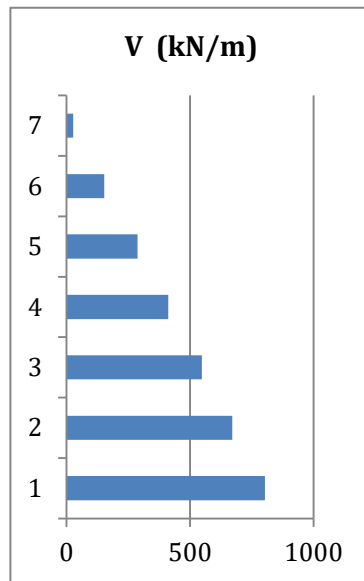
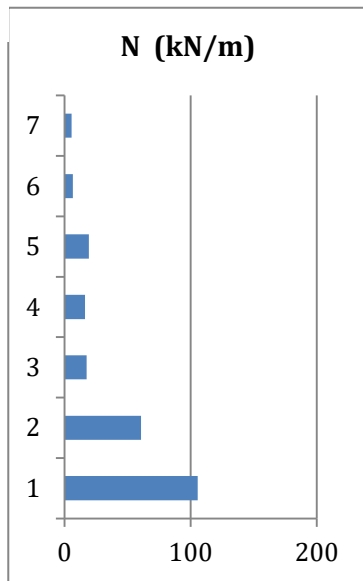
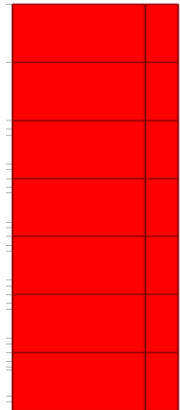


Figures 3.38. Internal forces diagrams (N, V, M) for Walls A3-4,A1-2

D 3-4



D 1-2



Figures 3.39. Internal forces diagrams (N, V, M) for Wall D3-4 and D 1-2.

Internal forces analysis is quite important in the element's design, because based on it, the material, reinforcement and dimensions of the structural elements are selected. In our case the analysis helps to determine whether the structure needs strengthening or not. From the visual representation of the results generated by SAP2000, it is clearly seen that the first floor of the structure is the critical since it is exposed to the highest axial and shear forces as well as bending moments in two directions. Further structural analysis in the following section will determine whether walls need strengthening or not.

3.4.4.2. Slabs

The structure consists of 7 types of slabs mentioned in **Table 3.1**. All the slabs of the second floor were given specific names in SAP2000 and extracted for further analysis. **Figure 3.40** below shows slab locations and names for one floor.

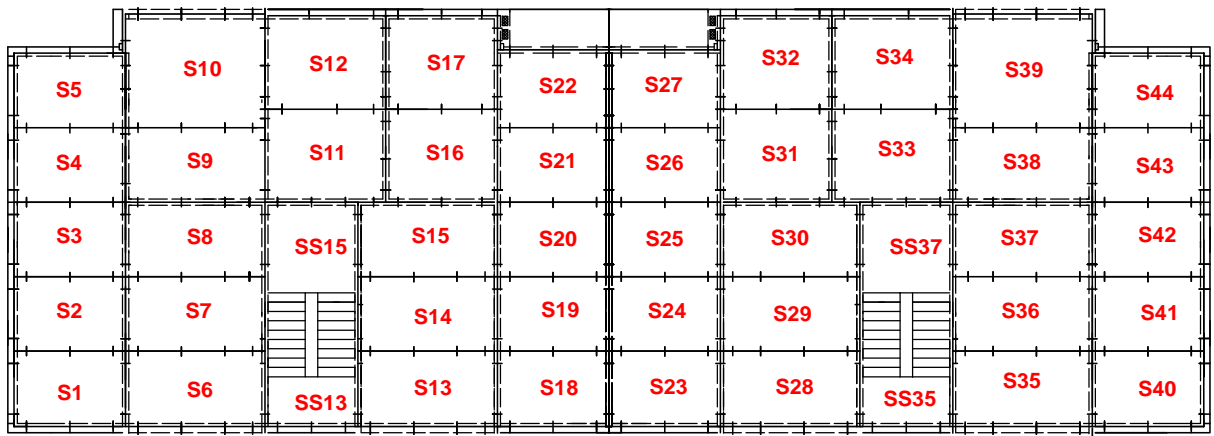


Figure 3.40. Slab names and locations for one floor

Table 3.18 shows the values obtained from SAP2000 for moment and shear. For slabs, these mentioned internal forces are considered as critical elements for design. However, the design is more controlled by the value of moment rather than shear, as bending and deflection govern the design and the values of shear are significantly smaller. Analysis in the following section will determine whether slabs have adequate bending and shear strength to sustain additional loads from vertical extension.

Table 3.18. SAP2000 values of moment and shear for slabs

Slab name in SAP2000	Shear V (kN)	Bending moment M (kNm)
s1	55,18	76,64
s2	55,06	75,04
s3	56,43	56,64
s4	58,01	74,40
s5	53,73	68,16

Table 3.18. SAP2000 values of moment and shear for slabs (Continuation)

Slab name in SAP200	Shear V (kN)	Bending moment M (kNm)
s6	65,02	55,20
s7	45,12	70,72
s8	64,02	70,24
s9	67,13	72,00
s10	71,98	89,60
s11	78,82	91,20
s12	81,35	94,00
s13	79,80	49,44
s14	79,40	70,72
s15	74,46	56,96
s16	93,95	96,48
s17	81,76	92,20
s18	37,20	75,00
s19	38,90	48,16
s20	43,70	37,28
s21	56,15	68,80
s22	61,26	68,80
s23	37,87	76,00
s24	30,72	64,64
s25	32,50	76,48
s26	61,02	56,96
s27	67,25	56,96
s28	29,45	76,48
s29	34,21	51,84
s30	33,89	37,92
s31	56,55	80,32
s32	41,29	65,92
s33	36,34	84,80
s34	41,40	80,64
s35	58,61	67,68
s36	59,39	72,48
s37	68,80	71,80
s38	64,28	99,20
s39	67,82	112,48
s40	55,76	93,48
s41	53,97	72,00
s42	70,51	91,40
s43	59,05	89,60
s44	59,26	93,96

3.4.4.3. Joints

In the precast concrete structure strength might be governed by joints' strength rather than by walls' and slabs'. For that, SAP2000 values for joints are extracted and compared to the design strength of the connection. The wall to slab connection (see **Figure 3.41**) is found to be mortar joint type. The interface between concrete surface of the wall and slab is filled with mortar, which causes compression effect and increases bearing capacity of the joint.

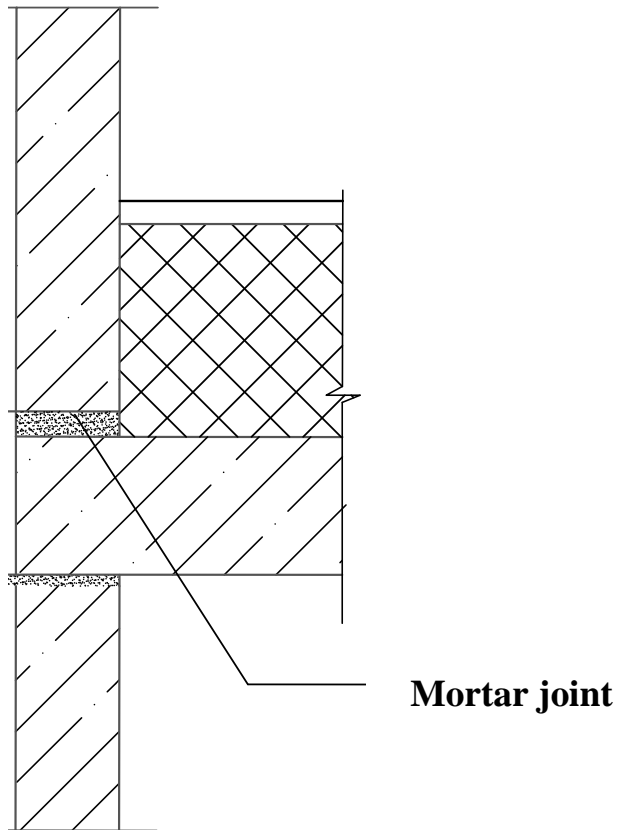


Figure 3.41. Wall to slab connection

The joints are given specific names in SAP2000 (see **Figure 3.42**) and the values of internal forces are extracted (see **Table 5**, Appendix)

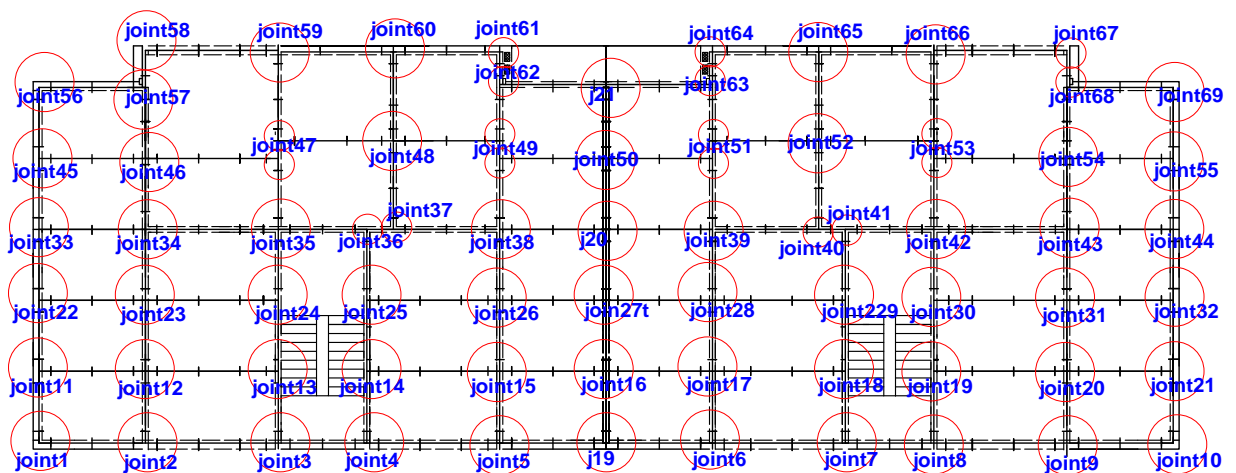


Figure 3.42. Joint names in SAP2000

The joint in **Figure 3.41** is treated as a pin connection and in the analysis section calculation of joint design strength is performed.

3.5. Analysis of existing structural members' strength

As the project deals with vertical extension of the structure by adding two additional storeys, it is required to ensure that existing structural members are able to sustain increased gravity loads; otherwise proper strengthening must be done. For that, analysis was performed on existing structural members' strength.

3.5.1. Shear walls

In order to ensure that existing shear walls are able to sustain increased gravity loads, M-N interaction curves for each specific wall, which represent existing strength of the walls, were plotted and compared to the internal forces (M and N) applied to each specific wall. M-N diagram was plotted using Excel spreadsheet, which computes interaction diagram according to ACI 318-2002. **Tables 6-8** in the Appendix represent the values for M-N for wall types 1-10 obtained from this spreadsheet and based on which the diagram was plotted. **Figure 3.43-44** below shows M-N interaction diagrams for wall types 1-10 (blue curves) and internal forces (M, N) in the walls that belong to that specific wall type (red dots). Each red dot in the figure represents a specific wall. Based on this analysis, all the walls in the structure are able to sustain increased loads, and no strengthening of walls is needed.

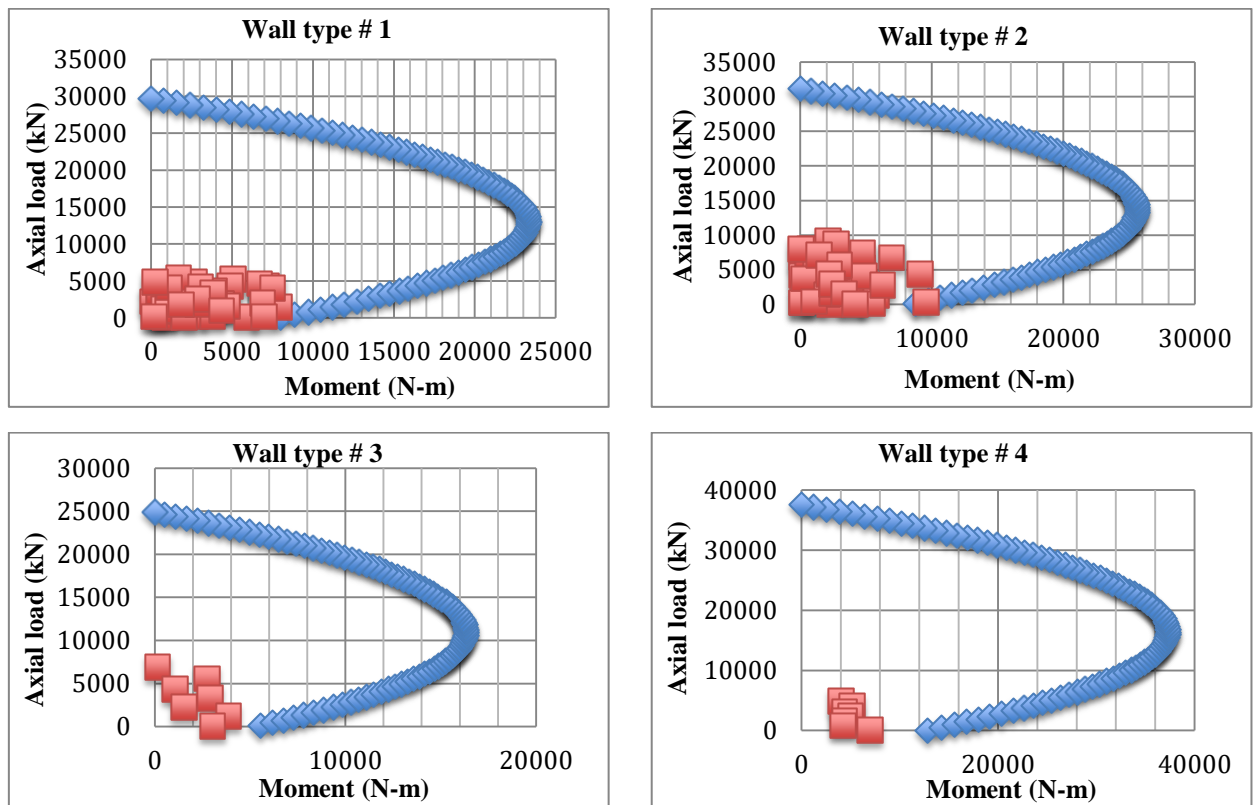


Figure 3.43. M-N interaction diagram for walls 1-4

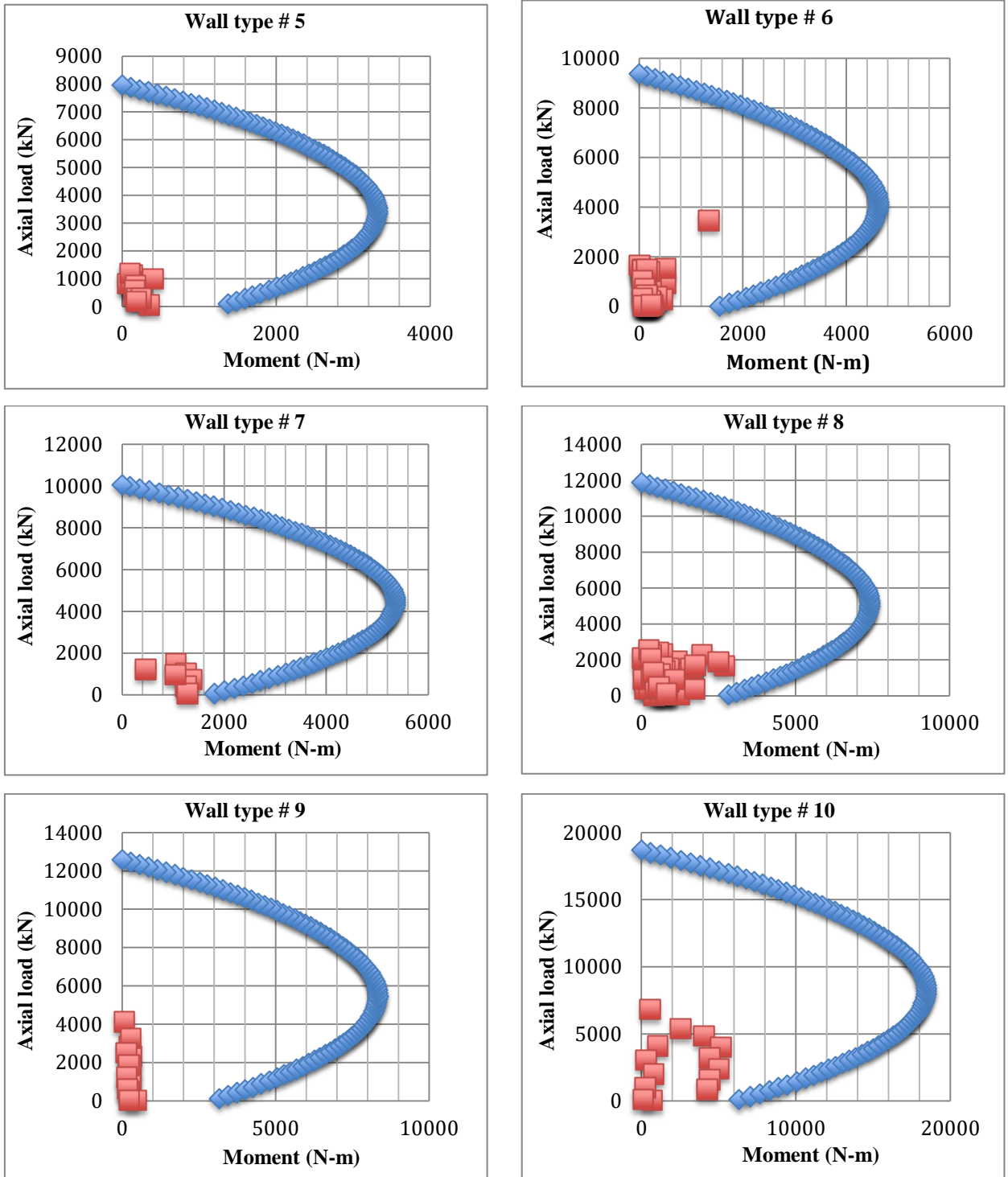
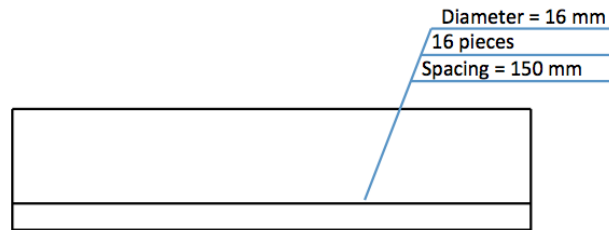


Figure 3.44. M-N interaction diagram for walls 5-10

3.5.2. Slabs

Next structural members, which design strength needs to be checked, are slabs. Slabs are one-way, supported by parallel walls, to which their loads are transferred. As bending occurs in one direction, the main tension reinforcing bars run in one direction, parallel to the span. In order to ensure that existing slab is able to sustain additional loads from vertical extension, bending and shear strengths are calculated. Calculations are done based on ACI Code 318-11.

Slabs S1-S9, S13-S15, S18-S30, S35-S37



1. Effective depth (d)

$$d = \text{height} - \text{cover} - \text{half diameter} = 160 - 25 - \frac{16}{2} = 127 \text{ mm}$$

2. Steel area (A_s)

$$A_s = \# \text{ of pieces} \times \pi \times \left(\frac{d}{2}\right)^2 = 16 \times \pi \times \left(\frac{16}{2}\right)^2 = 3217 \text{ mm}^2$$

3. Compression depth (a)

$$a = \frac{A_s \times f_y}{0,85 \times f_c \times b} = \frac{3217 \text{ mm}^2 \times 235 \text{ MPa}}{0,85 \times 27,5 \text{ MPa} \times 2400 \text{ mm}} = 13,5 \text{ mm}$$

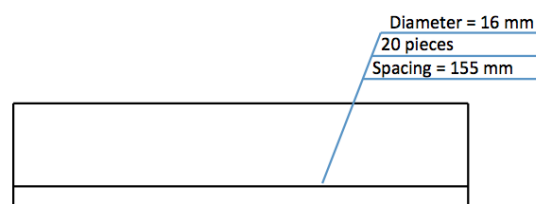
4. Factored moment (ϕM_n)

$$\begin{aligned} \phi M_n &= \phi \times A_s \times f_y \times \left(d - \frac{a}{2}\right) = 0,9 \times 3217 \text{ mm}^2 \times 235 \frac{\text{N}}{\text{mm}^2} \times \left(127 - \frac{13,5}{2}\right) \text{ mm} \\ &= \mathbf{81,8 \text{ kNm}} \end{aligned}$$

5. Shear strength (V_c)

$$V_c = 0,166 \times \sqrt{f_c} \times b \times d = 0,166 \times \sqrt{27,5} \times 2400 \times 127 = \mathbf{265,3 \text{ kN}}$$

Slabs S11-S12, S16-S17, S31-S34, S38, S40-S44



1. Effective depth (d)

$$d = \text{height} - \text{cover} - \text{half diameter} = 160 - 25 - \frac{16}{2} = 127 \text{ mm}$$

2. Steel area (A_s)

$$A_s = \# \text{ of pieces} \times \pi \times \left(\frac{d}{2}\right)^2 = 20 \times \pi \times \left(\frac{16}{2}\right)^2 = 4021 \text{ mm}^2$$

3. Compression depth (a)

$$a = \frac{A_s \times f_y}{0,85 \times f_c \times b} = \frac{4021 \text{ mm}^2 \times 235 \text{ MPa}}{0,85 \times 27,5 \text{ MPa} \times 3000 \text{ mm}} = 13,5 \text{ mm}$$

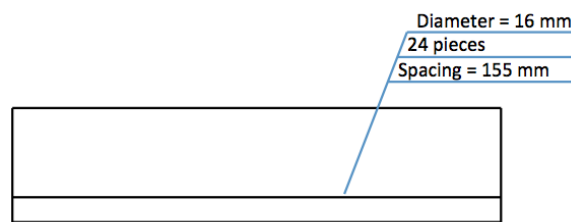
4. Factored moment (ϕM_n)

$$\begin{aligned} \phi M_n &= \phi \times A_s \times f_y \times \left(d - \frac{a}{2}\right) = 0,9 \times 4021 \text{ mm}^2 \times 235 \frac{\text{N}}{\text{mm}^2} \times \left(127 - \frac{13,5}{2}\right) \text{ mm} \\ &= \mathbf{102,3 \text{ kNm}} \end{aligned}$$

5. Shear strength (V_c)

$$V_c = 0,166 \times \sqrt{f_c} \times b \times d = 0,166 \times \sqrt{27,5} \times 3000 \times 127 = \mathbf{332 \text{ kN}}$$

Slabs S10-S39



1. Effective depth (d)

$$d = \text{height} - \text{cover} - \text{half diameter} = 160 - 25 - \frac{16}{2} = 127 \text{ mm}$$

2. Steel area (A_s)

$$A_s = \# \text{ of pieces} \times \pi \times \left(\frac{d}{2}\right)^2 = 24 \times \pi \times \left(\frac{16}{2}\right)^2 = 4825 \text{ mm}^2$$

3. Compression depth (a)

$$a = \frac{A_s \times f_y}{0,85 \times f_c \times b} = \frac{4825 \text{ mm}^2 \times 235 \text{ MPa}}{0,85 \times 27,5 \text{ MPa} \times 3600 \text{ mm}} = 13,5 \text{ mm}$$

4. Factored moment (ϕM_n)

$$\begin{aligned} \phi M_n &= \phi \times A_s \times f_y \times \left(d - \frac{a}{2}\right) = 0,9 \times 4825 \text{ mm}^2 \times 235 \frac{\text{N}}{\text{mm}^2} \times \left(127 - \frac{13,5}{2}\right) \text{ mm} \\ &= \mathbf{122,7 \text{ kNm}} \end{aligned}$$

5. Shear strength (V_c)

$$V_c = 0,166 \times \sqrt{f_c} \times b \times d = 0,166 \times \sqrt{27,5} \times 3600 \times 127 = \mathbf{398 \text{ kN}}$$

Based on the calculations and values obtained from SAP2000, the graphs on **Figure 3.44** were plotted. Blue dots in the graph represent demand values for shear and moment for each slab, whereas red dots represent shear and moment capacities of slabs. Based on the graphs, each slab has moment and shear capacity higher than demand, thus slabs are safe and do not require strengthening.

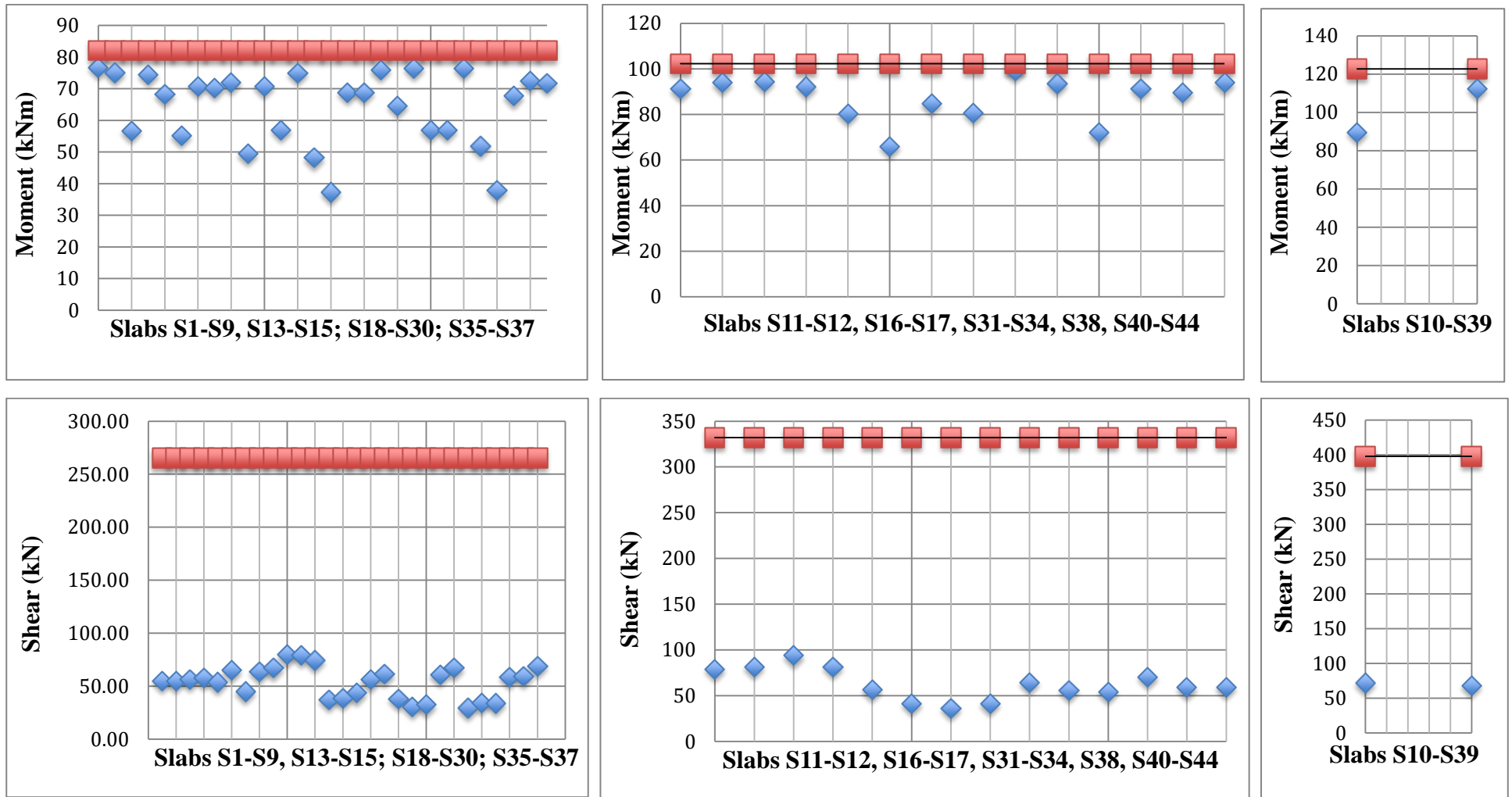


Figure 3.45. Comparison of design shear and moment strength (red) and ultimate shear and moment (blue) for slabs S1- S4

3.5.3. Joints

The connection in the **Figure 3.41** is a pin connection, which is able to transfer shear and axial loads, but no moment. For that, shear and axial strengths of the connection were calculated based on following procedure for joints filled with mortar.

The total shear capacity of the interface between mortar and concrete is defined by the following equation (Fib, 2008):

$$v_{Rdi} = cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) \leq 0.5vf_{cd}$$

where, c = coefficient according to Table 8-4 (Fib, 2008) = 0,5

$$f_{ctd} = \text{design tensile strength for the concrete grade} = \frac{f_{ctm}}{2} = 15\text{Mpa}$$

$$\mu = \text{coefficient of shear friction according to Table 8-4 (Fib, 2008)} = 0,9$$

$$\sigma_n = \text{stress per unit area of external normal force across joint interface} = 12 \text{ kPa}$$

$$\rho = \text{ratio of area of steel (if any) to area of joint interface} = 0 \text{ (no steel)}$$

$$v = \text{strength reduction factor} = 0,6 - f_{ck}/200 = 0,528$$

$$\text{Hence, } v_{Rdi} = 0,5 * 2,6 + 0,9 * 12/1000 = \mathbf{1,31 \text{ MPa}} \leq 5,28 \text{ MPa.}$$

$$V_{Rd} = 1,31 * 5,7 * 0,2 = 747 \text{ kN for wall with length 5,7m.}$$

The value of V_{Rdi} changes with each wall.

The total axial strength is equal to:

$$N_{Rd} = f^*_{cd} \times A_1 \leq 4f_{cd} \times A_1$$

$$f^*_{cd} = f_{cd} \times \sqrt{A_2/A_1}$$

where, A_1 =effective loading area = $5,7 \times 0,1$

$$A_2 = \text{distribution area} = 5,7 \times 0,2$$

Hence, $N_{Rd} = f^*_{cd} \times A_1 = f_{cd} \times \sqrt{A_2/A_1} \times A_1 = 20 \times \sqrt{2} \times 5,7 \times 0,1 = 16,1 \text{ MN} \leq 45,6 \text{ MN}$ (for wall with length 5,7m)

Shear and axial strengths were calculated for each connection. **Figures 3.46-3.47** show the final results. Blue dots represent demand values, whereas horizontal lines represent strengths. Based on the figures both axial and shear strengths satisfy the demand. Hence, no strengthening for connections is required.

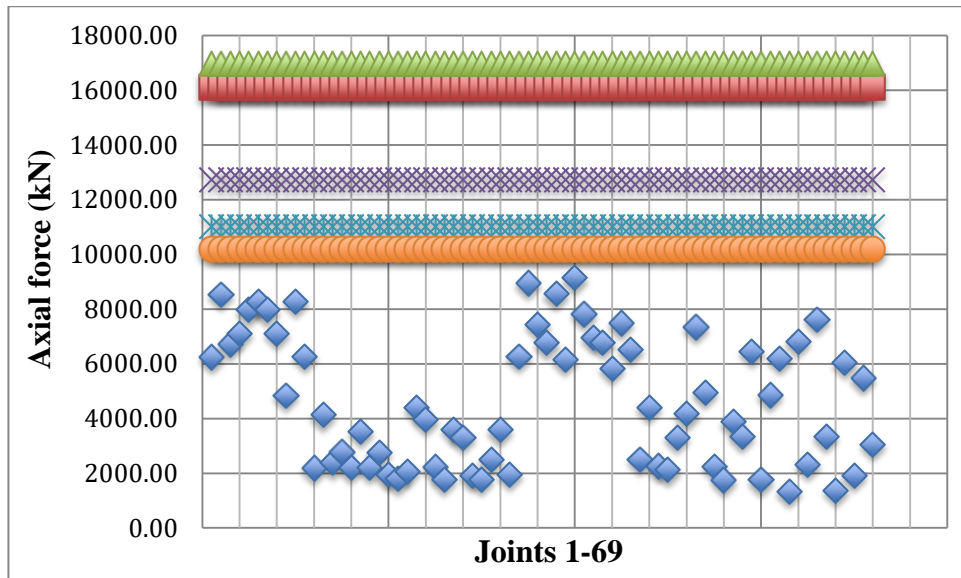


Figure 3.46. Comparison of joints' axial design strength and demand

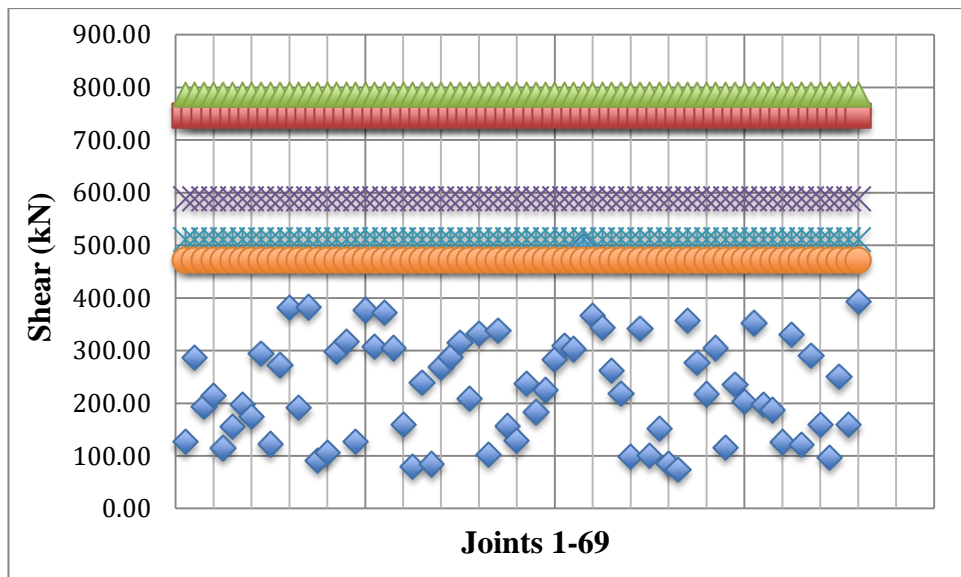


Figure 3.47. Comparison of joints' shear design strength and demand

3.5.3. Structural analysis conclusion

Based on the structural analysis of shear walls, slabs and joints, the existing structure is able to sustain additional loads from vertical extension. Thus, strengthening of structural members is not required. The cross-section detailings of walls (exterior and interior), slabs and joints are provided in the Drawings document.

3.6. Structural serviceability check

In order to ensure proper functioning of the structure, serviceability check must be performed. Crack check is one of the serviceability checks, which must be kept within specific limits based on building codes. It is dictated by a minimum number of rebars (n) for maximum allowable crack width. The following formula is used:

$$n = \left(\frac{0,076 \times 10^{-3} \times \beta_n \times f_s}{w_{max}} \right)^3 \times t_b \times A_e$$

where, $\beta_n = 1,2$

(distance from neutral axis to centroid) / distance from neutral axis to bottom bar)

$$f_s = 0,6 \times f_y = 0,6 \times 350 = 210 \text{ MPa} = 31 \text{ ksi}$$

$$t_b = \text{cover} = 25 \text{ mm} = 0,984 \text{ in}$$

$$w_{max} = \text{maximum width of crack} = 0,4 \text{ mm} = 0,015748 \text{ in}$$

$$A_e = 2 \times t_b \times y = 50 \times 3600 = 180000 \text{ mm}^2 = 280 \text{ in}^2$$

Hence, minimum number of rebars is equal to:

$$n = \left(\frac{0,076 \times 10^{-3} \times 1,2 \times 31}{0,015748} \right)^3 \times 0,984 \times 280 = 4 \text{ bars}$$

The minimum number of bars is 4, however the section has 16. Hence, crack check is performed. **Table 8** in the Appendix shows crack check for other cross-sections.

Another serviceability check is done for vertical deflection. For that service loads: $W_d = 12 \text{ kN/m}^2$ and $W_L = 4,79 \text{ kN/m}^2$ are used. Bending moments at the midspan is calculated by:

$$M_D = \frac{w_d \times l_n^2}{14} = \frac{12 \times 4,5^2}{14} = 17,4 \text{ kNm}$$

$$M_L = \frac{w_L \times l_n^2}{14} = \frac{4,79 \times 4,5^2}{14} = 6,9 \text{ kNm}$$

Assuming 30% of live load, sustained bending moment is $M_{sus} = M_D + 0,3M_L = 19,5 \text{ kNm}$.

Material properties for concrete are as follows:

$$f_R = \text{modulus of rupture} = 0,62 \times \lambda \times \sqrt{f_c} = 0,62 \times 1 \times \sqrt{27,5} = 3,25$$

$$E_c = 4700 \sqrt{f_c} = 4700 \sqrt{27,5} = 24647 \text{ MPa}$$

$$E_s = 200000$$

$$n = \text{modular ratio} = \frac{E_s}{E_c} = 8,1$$

$$I_g = \text{gross moment of inertia} = \frac{1}{12} \times b \times h^3 = \frac{1}{12} \times 2400 \times 160^3 = 8,192 \times 10^8 \text{ mm}^4$$

Thus, cracking moment:

$$M_{cr} = \frac{2f_r I_g}{h} = \frac{2 \times 3,25 \times 8,192 \times 10^8}{160} = 32,28 \text{ kNm}$$

The immediate deflection is calculated by:

$$\Delta_i = \frac{5KM_a l^2}{48E_c I_e}$$

where, K = deflection coefficient = $1,2 - 0,2 \times \left(\frac{M_0}{M_a}\right)$ for continuous span

$$M_0 = \frac{1}{8} \times w \times l^2$$

$$M_a = \frac{1}{14} \times w \times l^2$$

Hence, $K = 1,2 - 0,2 \times \left(\frac{19,44}{11,1}\right) = 0,85$

The immediate deflections are calculated by:

$$(\Delta_i)_D = \frac{5KM_D l^2}{48E_c I_e} = \frac{5 \times 0,85 \times 17,4 \times 10^6 \times 4500^2}{48 \times 24645 \times 8,192 \times 10^8} = 1,55 \text{ mm}$$

$$(\Delta_i)_{D+L} = \frac{5KM_{D+L} l^2}{48E_c I_e} = \frac{5 \times 0,85 \times 19,5 \times 10^6 \times 4500^2}{48 \times 24645 \times 8,192 \times 10^8} = 1,73 \text{ mm}$$

$$(\Delta_i)_L = (\Delta_i)_{D+L} - (\Delta_i)_D = 1,55 - 1,73 = 0,18 \text{ mm}$$

According to ACI, the maximum allowable immediate live load is $l/360 = 4500/360 = 12,5$ mm.

For long term deflection due to creep and shrinkage is:

$$\lambda_{cs} = \lambda_{\Delta} \times (\Delta_i)_{sus}$$

where $\lambda_{\Delta} = \frac{\xi}{1+50\rho'}$;

$\xi = \text{sustained load duration for 5 years and more} = 2.0$

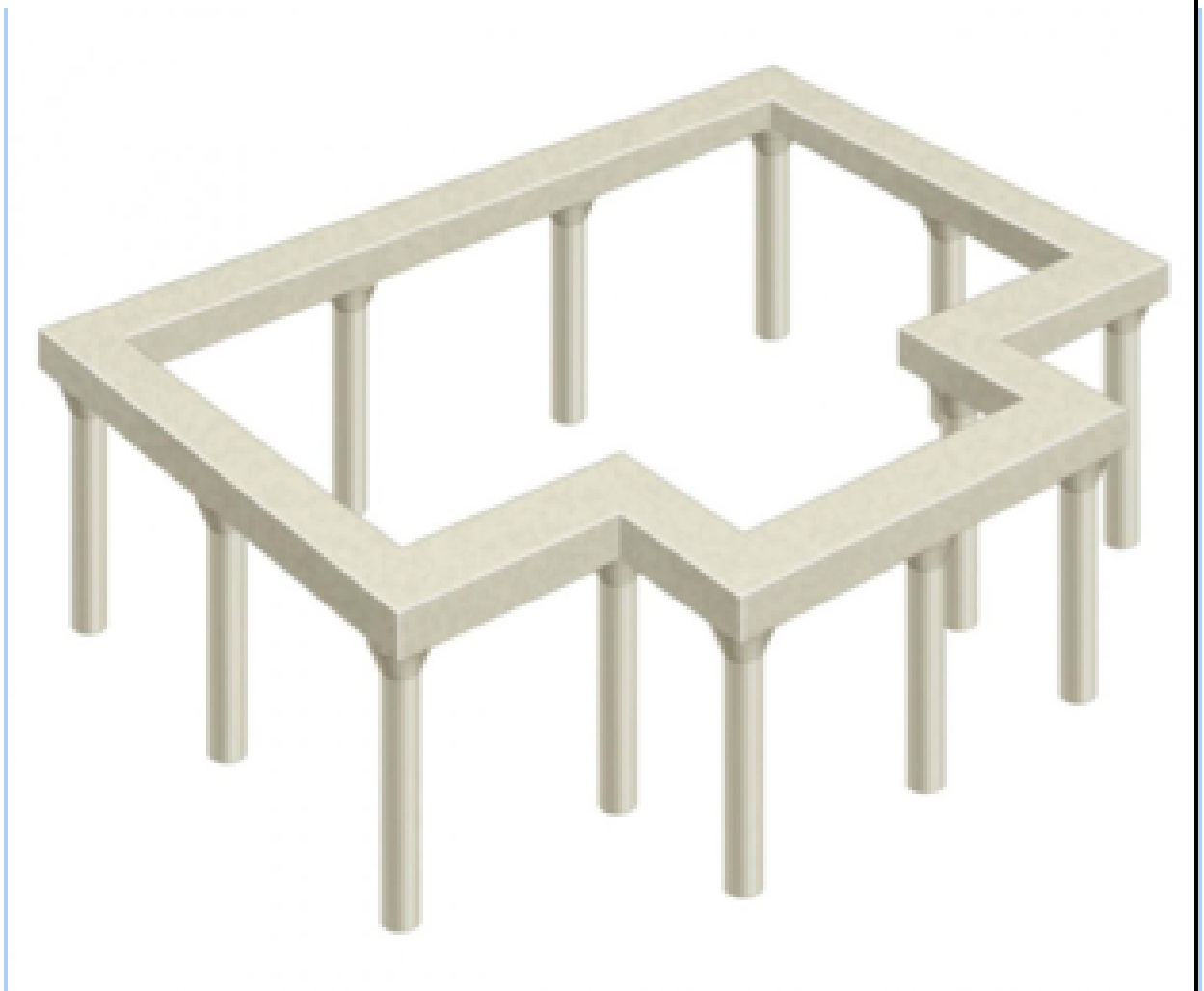
$$\rho' = \frac{A_s}{bd} = \frac{3217}{384000}$$

Hence,

$$\lambda_{cs} = \lambda_{\Delta} \times (\Delta_i)_{sus} = \frac{\xi}{1+50\rho'} \times (\Delta_i)_{sus} = \frac{2.0}{1+50 \times 0,000837} \times 1,73 = 3,5 \text{ mm}$$

The total deflection is the sum of immediate deflection and long-term deflection: $0,18$ mm + $3,5$ mm = $3,6$ mm, which is within reasonable range.

Geotechnical Analysis



Chapter 4. Geotechnical design

4.1. Existing building foundation

Existing foundation consist of grillage with the cross-section 800*800 mm² which distributes the load on footing and further on ground and foundation footings. The detailed description of footing is shown on the **Figure 4.1**. Also, the overall layout of foundation can be seen from the **Figure 4.2**.

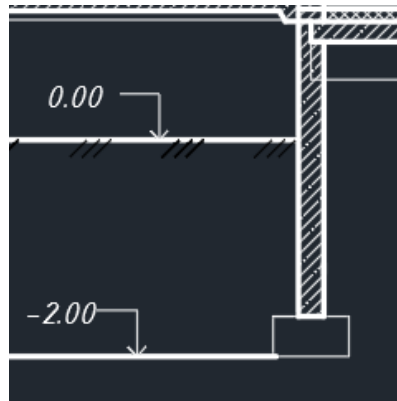


Figure 4.1. Dimensions of footing from side view

The distance between grillage and 0 ground level is 1.05 m, and distance between 0 ground level and bottom of foundation is 2 meters, which is in total gives 3.05 meters, which is enough for the equipment to get inside. For more details see procedure sections.

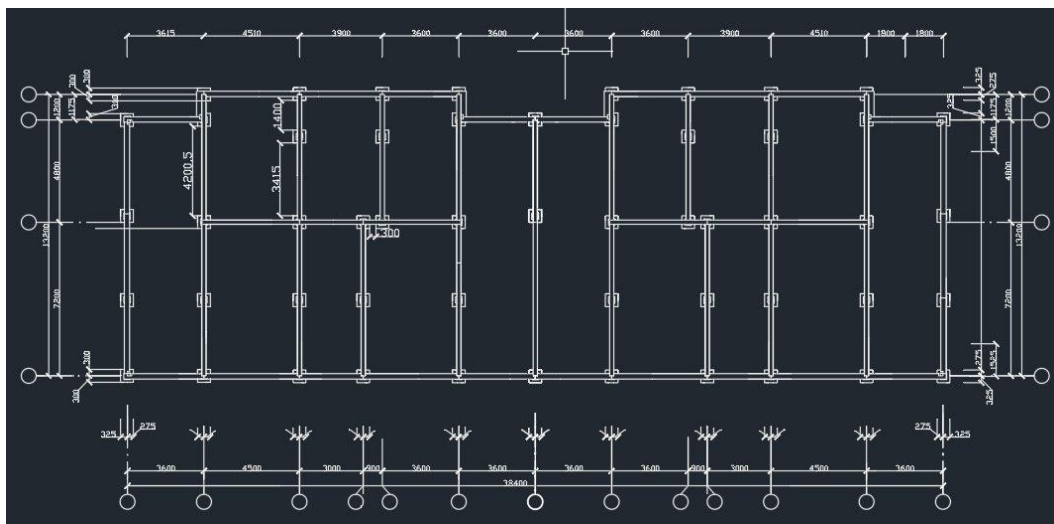


Figure 4.2. Layout of footing foundation of the building

4.1.1 Check for 7-storey

Before redesigning the existing foundation, it is necessary to check whether the existing building is able to sustain additional 2 floors and weight of slab. In case if it is able to sustain 7 storey structure, then the columns of the existing foundation will be used for new construction. The whole building stands on the foundation columns, which based on SAP2000 analysis are

able to sustain the increased gravity load from vertical extension. In order to prove it, it is necessary to provide hand calculations. From this calculations M-N interaction diagram was derived and different combinations of maximum vertical load and moment were checked for the safety (see **Figure 4.3**). Column with dimensions 400*400 mm and consist of 8 rebars with diameter of 16 mm.

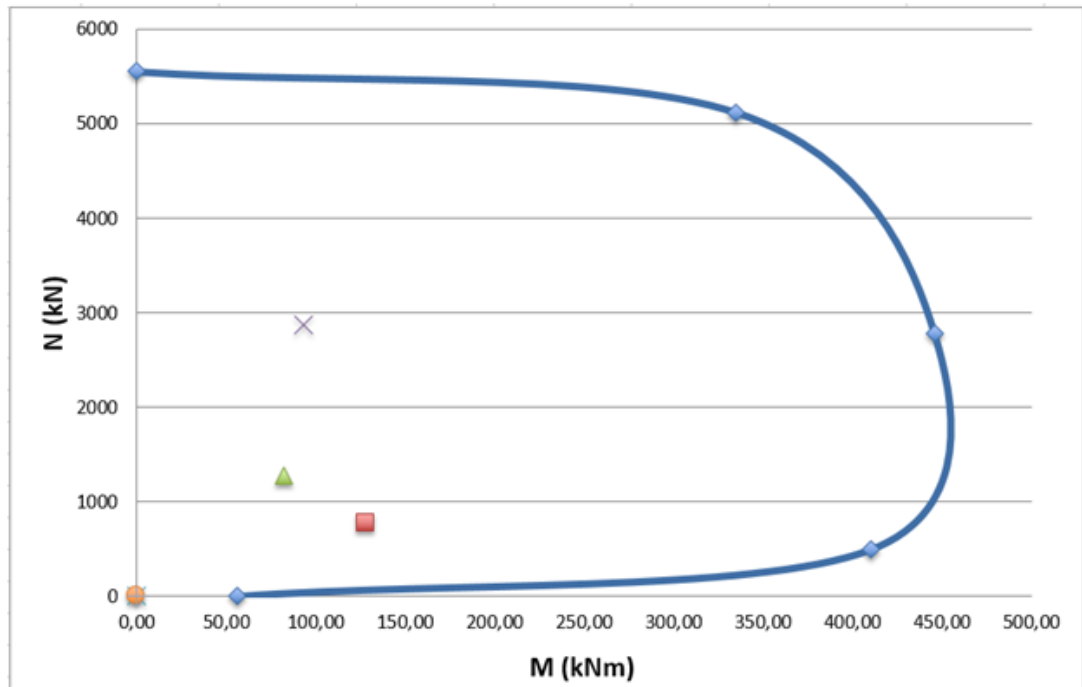


Figure 4.3. Final M-N diagram of column with maximum combinations of load and moment.

As it can be seen from the diagram above all combinations of maximum load and moment are in safe, meaning that elements are safe.

4.2 Parking design

4.2.1. Construction method

To evaluate possibility of construction of two-store underground parking, previous engineering experience in this area was analyzed. Seoul City Hall, building located in the capital of South Korea was taken as an example. As part of the renovation, whole foundation of the Seoul City Hall was replaced. By using micropiles and heavy hydraulic jack system, it was possible to replace foundation under the existing building. The excavation and construction of foundation is going to be discussed in the following chapters.

One of the main concerns about this project is an excavation process. To be able to reach basement of the building it will be necessary to excavate the large volume of soils which consequently will affect ground movement in the targeted area. In order to prevent such displacement it will be necessary to perform partly excavation.

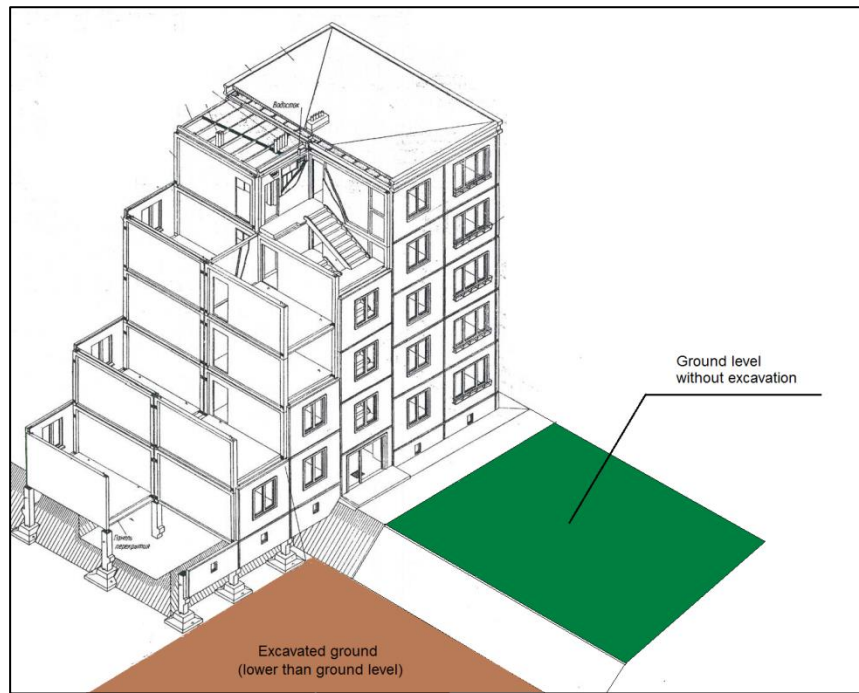


Figure 4.4. Demonstration of partial soil excavation.

To avoid any possible unnecessary settlement minimum excavation area per time must be used. Minimum side excavation must be enough for equipment to move inside and outside of the construction area.

4.2.2. Sequence of excavation & construction

To evaluate excavation & construction processes of the existing building hand calculation and finite element analyzing tool, Plaxis 2D, was used. Capability of Plaxis to calculate deformation and stability in soil, gives a chance to learn about soil behavior during the excavation works. In comparison with Plaxis 2D, Plaxis 3D's performance is more accurate, however availability and relative simplicity of 2D was decisive criteria for selecting it.

First of all, soil parameters and structural elements of the existing building were added. After that step-by-step excavation and construction processes were simulated. Results obtained from Plaxis 2D simulation are presented next.

4.2.2.1. Soil parameters

Parameters of soil were obtained from the report of “QaragandyGiiz”. Soil profile for targeted area is presented in the table below.

Table 4.1. Soil profile

Soil type	Depth range, m	Thickness, m	Density, g/cm ³	Friction angle	Cohesion, kPa	Elastic modulus, MPa	Permeability, m/day
Backfill	0-0.3	0.3	1.87	21	44	2.8	
Loam	0.3-2	1.7	1.97	22	18	6	0.60
Medium size sand	2-5.5	3.5	1.92	35	2	17	4.23
Coarse sand	5.5-6.3	0.8	2	38	1	21	9.20
Gravelly sand	6.3-10.3	4.0	2	38	1	21	12.94
Gravel soil	10.3-11.3	1.0	2.05	40	0	23	23.90
Loam	11.3-16.1	4.8	1.93	32	0	13.5	0.60

As it was shown, underground profile of area consists of six different layers. First, there is a 2 m deep loam layer right under the backfill. Following loam, there are two sand layers, medium size sand and coarse aggregate. Under which there are two layers of coarse sand and gravelly sand. In the bottom, starting from -11.3 m, there is also loam layer.

The water table of the targeted area is located -1 m from the ground level. However for the construction period it is going to be lowered to -6.7 m. **Table 4.1** below demonstrates soil profile of the targeted area.

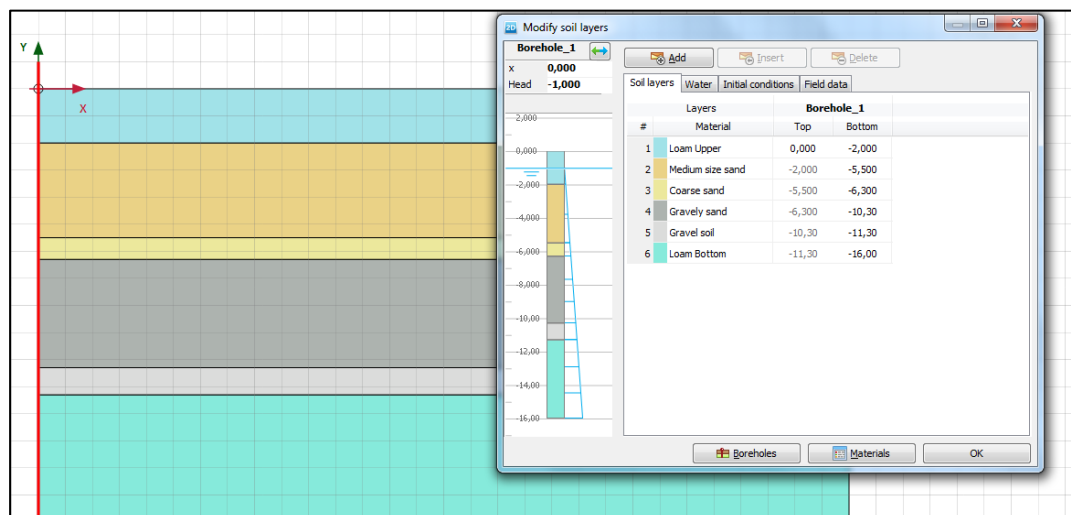


Figure 4.5. Soil profile evaluated in Plaxis 2D

Note: There is a 30 cm level of backfill on the surface, however for simplicity purpose it was replaced with layer of loam.

It was mentioned before that all soil parameters were obtained from Astana region soil profile file. However, only limited number of soil parameters was available for Astana region, for this reason all the unknown parameters were assumed as standard parameters in Plaxis 2D program. Full information about soil parameters is presented in **Appendix Table 9**.

4.2.2.2. Structural elements

In structure defining phase all the structures of the construction were prepared. Design of each structure was calculated by hand calculation. All structural elements used in simulation process are presented in the list below:

- Basement floor/slab – horizontal plates with defined parameters which were used to imitate slabs of the building. Parameters of slabs were taken from existing building and from slab calculations information that was not available was taken from standard Plaxis parameters. All parameters can be found in **Appendix Table 10**.
- Basement wall – vertical plates which plays a role of walls. All the parameters were taken from wall calculations and information that was not available was taken from standard Plaxis parameters. All parameters can be found in **Appendix Table 10**.
- Footing – Plates under columns of existing building. Size of footing is 2.6 x 2.6 m². Parameters of footing were obtained from existing building layout. All parameters can be found in the following chapters.
- Foundation columns – vertical nod-to-nod anchors were used as columns. Spacing in z-direction was used as 3 m, similar to existing building. All parameters can be found in **Appendix Table 11**.
- Micropiles – Embedded pile row was used to simulate micropiles in Plaxis. Parameters of micropiles were obtained from calculation. As a result it was evaluated that the length of each micropile will be 11 m. All parameters can be found in **Appendix Table 12**.
- Retaining wall – Sheet pile wall was used to conduct excavation process. For the parameters of sheet piles was used standard values from Plaxis and the height of wall was calculated manually in the previous parts. All parameters of wall can be found in **Appendix Table 13**.
- Strut – to be able to resist lateral earth pressure strut was installed. The parameters of strut was set as default as for Plaxis program. See **Appendix Table 14** for full parameters of strut.

Construction of all the structural elements in the Plaxis program resulted in the following picture (see **Figure 4.6** below):

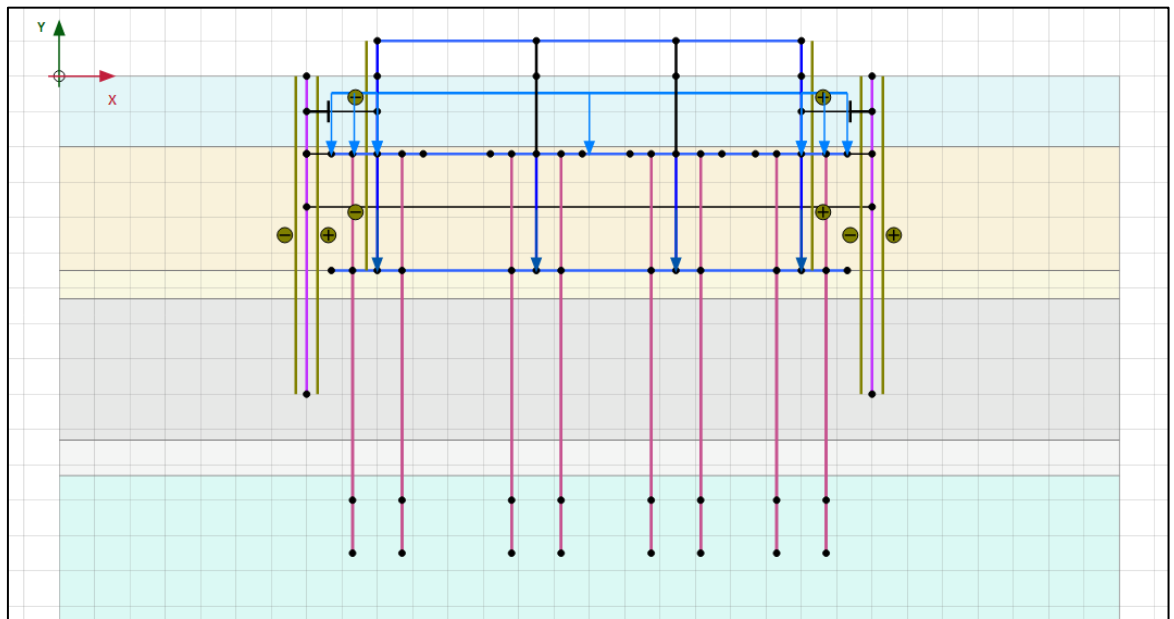


Figure 4.6. Model of existing underground structure

Step-by-by development of the following structure will be covered in the next part.

4.2.2.3. Construction stages

First stage for the project was to build a model of existing building in the Plaxis 2D. For the reason that Plaxis 2D gives a chance to evaluate only in two dimensions, it was decided to check side section of building with maximum loads on columns and footings. It was calculated from SAP model that the maximum load for one column is 1821 kN. To simulate load from the building it was decided to replace point load on column with linear load on footing. As a result 1821 kN was replaced with 152 kN/m on footing. Taking into consideration this information, following section was evaluated in Plaxis.

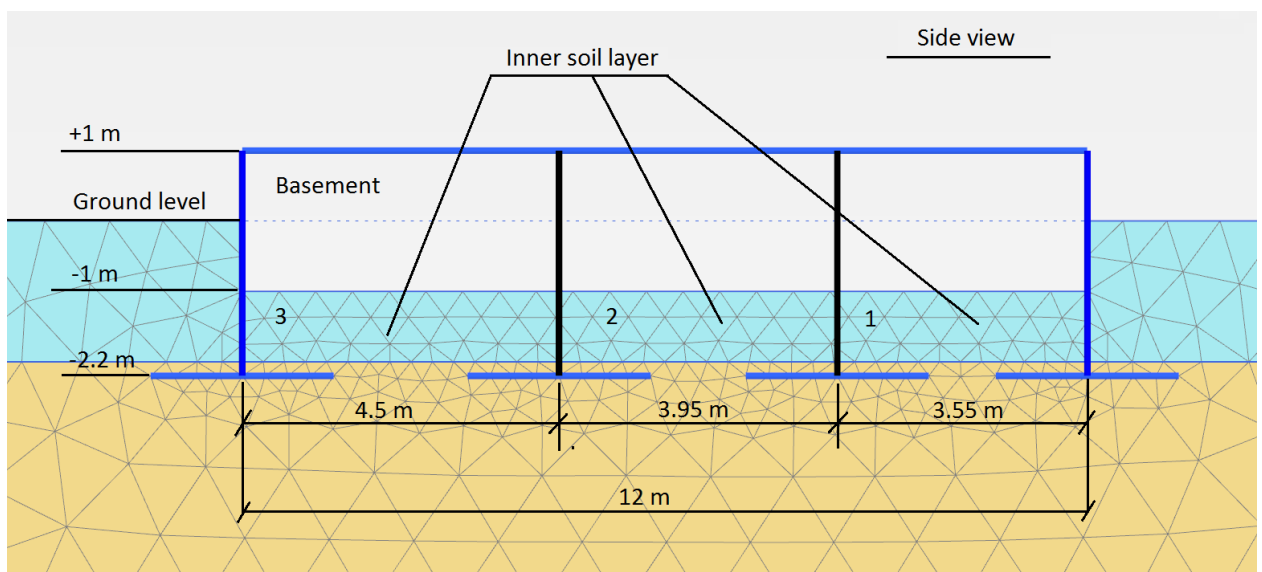


Figure 4.7. Side view of building's foundation

4.2.2.3.1. Design of existing building (Stage 1)

Existing building has a basement with height of 2 m. The bottom of basement is located at -1 m level from ground level and footings of building are located -2.2 m below the ground level. It means that the footing is located 20 cm below the loam soil layer. Total width of the building is 12 m and one column row contains four columns.

At this point several issues must be clarified. Actually, all four vertical lines are columns in real life. However for Plaxis simulation, it was assumed that two outside column are walls. This assumption was made due to the fact that columns in 2D Plaxis are counted as z-dimensionless object which means that in case if two outside lines are designed as column soil just outside of each line will collapse inside of the basement. For this situation two outside columns were changed to wall with similar properties as column. It can be noticed in picture above, two vertical blue lines are walls, and two black vertical lines are columns.

Second point to clarify is inner soil layer. First excavation stage is requiring removal inner soil layer by dividing its area to three sections (see 1-3 in **Figure 4.5-7** above). However, as it was mentioned before using columns in inner part creates an obstacle for removing inner soil part by part. For this situation inner soil layer was replaced with equivalent linear load on bottom surface. It was possible by knowing the unit weight and volume of the soil.

Unit weight of top loam layer is 18.7 kN/m^3 . The volume of inner soil layer that must be replaced with linear load:

$$\text{Volume} = \text{Length} \times \text{Width} \times \text{Height} = 12 \text{ m} \times 1 \text{ m} \times 1.2 \text{ m} = 14.4 \text{ m}^3$$

$$\text{Force by earth} = 18.7 \text{ kN/m}^3 \times 14.4 \text{ m}^3 = 269.28 \text{ kN}$$

$$\text{Linear load} = 269.28 \text{ kN} / 12 \text{ m} = 22.44 \text{ kN/m}$$

This is the effect of inner soil layer on bottom part, and now it can be replaced with linear load as in the picture below (**Figure 4.8**).

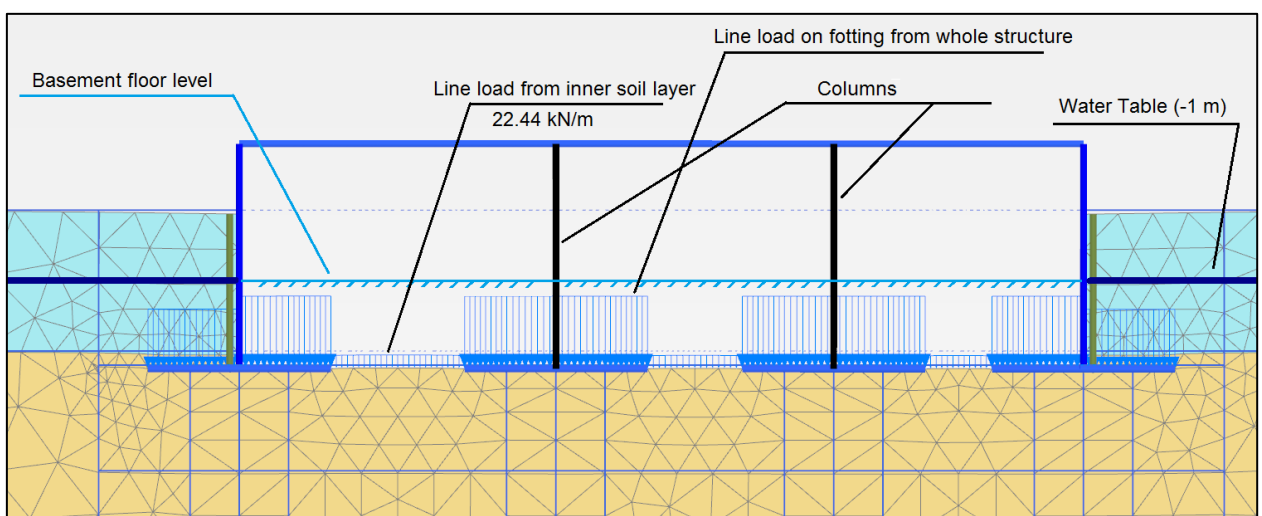


Figure 4.8. Graphic of replacing inner soil load with linear load

After designing the existing building structure, it was tested for consolidation for about 6000 days, after which displacement was reset to zero. Picture below, **Figure 4.8**, demonstrates building after long period settlement.

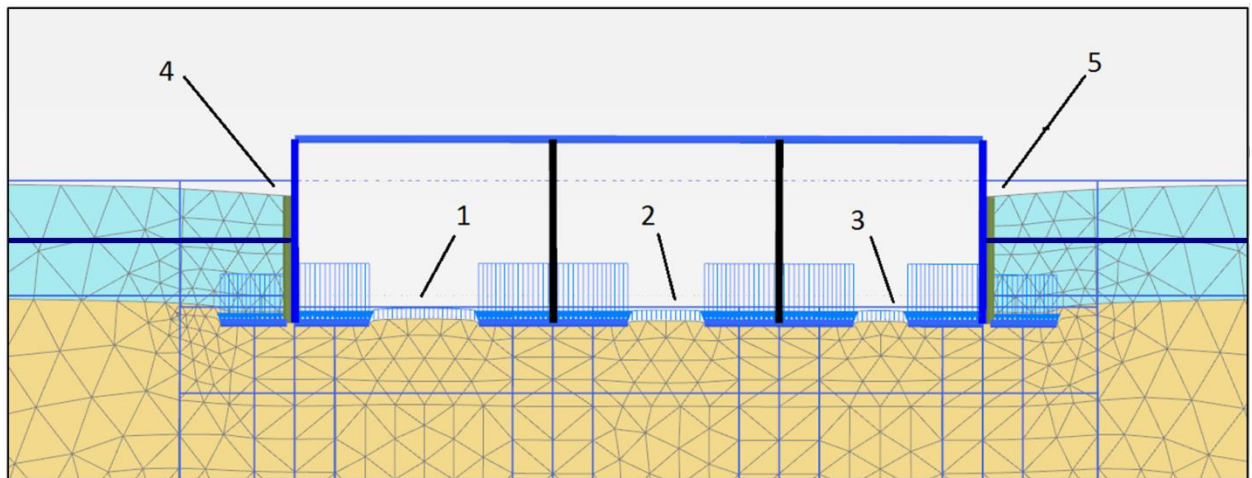


Figure 4.9. Result of building settlement in its lifetime period

From diagram it can be noticed that, soil from the side of the outer wall was settled, and soil in the center moved slightly up. In terms of deformation, maximum deformation at this stage was 5.8 mm.

4.2.2.3.2. Dewatering (Stage 2)

To be able to conduct underground works it is necessary to decrease water table at least to about -0.5-1.5 m below the excavation level for the reason that excessive ground water will cause difficulties during the construction (Cashman, 2002; Han, 2015).

According to geological survey, which was conducted in situ, it was discovered that the water table located 1.5-2.3 m below the ground level and during the spring period water level under the ground can increase to +0.3 m (Popov, 2015). Consequently, it can be assumed that maximum water level for the targeted area is -1.0 m from the ground level.

For the current project it is planned to excavate ground to the level of -5.2 m. For this purpose it is necessary to decrease water table at least to -7.7 m.

4.2.2.3.2.1. Permeability

The most important parameters of the soil for the dewatering are thickness and permeability of the soil. Permeability of the area with different soil types can be calculated through the formula:

$$K_{avg} = \frac{\sum_{i=1}^n K_i h_i}{h_{total}}$$

Calculating average permeability (from soil profile table):

$$K_{avg} = \frac{1.7 \times 0.6 + 3.5 \times 4.23 + 0.8 \times 9.2 + 4.0 \times 12.94 + 1.0 \times 23.9 + 4.8 \times 0.6}{16.1}$$

$$= 6.3183 \text{ m/day}$$

4.2.2.3.2.2. Dewatering methods

After the existing soil layer was evaluated, dewatering method must be identified. According to Vlasov et al. and Han there are three common methods of dewatering (Vlasov et al. 1987; Han 2015):

1. Dewatering by thin needle-filters – Elastic filter tubes 46-50 mm in diameter and about 8-9 m in length. Needle-filters are installed around excavation area by perimeter. One layer of needle-filters can effectively decrease water level to 4-5 m in case if permeability of soil is not less than 1 m/day.

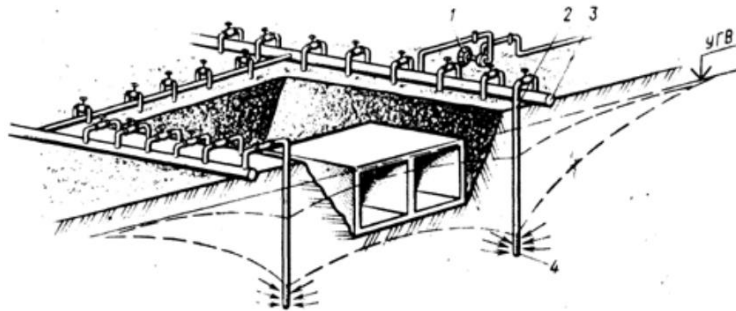


Figure 4.10. System of thin needle-filters around excavation

2. Dewatering by thin needle-filter based on ejection – Structure of filter is similar to previous one, however only difference is that this filter is based on ejection. By circulating water inside the tube, water around the filter is attached together to up going flow. This method is effective for the excavation of 10-12 m and when permeability of soil is around 0.5-1 m/day. Ejecting filter also placed by perimeter, however less frequently.

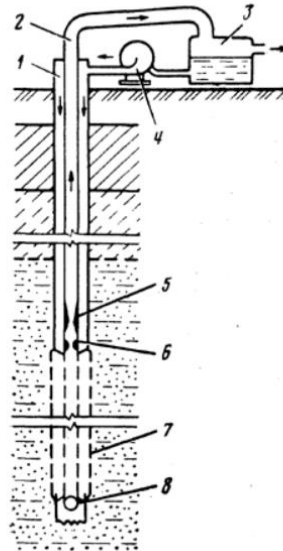


Figure 4.11 Needle-filter based on ejection:

1 – Inlet tube; 2 – Outlet tube; 3 – Reservoir; 4 – Pump

3. Dewatering by drilled wells with pump inside – most effective method of dewatering for large areas. For this method several wells are drilled in situ after which individual pumps are installed in each well. All the wells are then connected to common pipeline, which transfers excess water to the storage reservoirs.

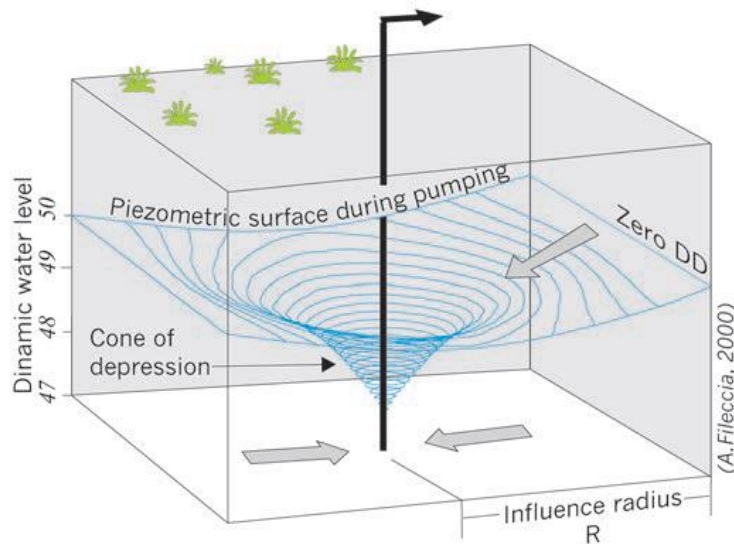


Figure 4.12. Illustration of deep well dewatering process

As it was mentioned before in soil profile table, targeted area has deposits of loam, which has permeability of 0.6 m/day. For such case it is not possible to use first method, due to the reason that it is constricted with criteria of permeability higher than 1 m/day.

Ejection needle-filters are acceptable for permeability criteria. However, ground area of current project is about 500 m² and it will be difficult to cover all the perimeter of site with

filters. Moreover, large dimensions of needle-filter can cause problem that center of the work site will not be drained properly.

For large areas like current project drilling dewatering wells will be the most suitable solution. Next step is to calculate number of wells and power of pumps.

4.2.2.3.2.3. Calculation

The main goal of calculations is to find radius of influence and discharge rate for the wells. Radius of influence is the horizontal distance, which is under influence of dewatering.

Formula for the radius of influence of single well is (Han, 2015):

$$R = C' \times h_{dd} \times \sqrt{k_{avg}}$$

Where:

R = radius of influence, m

C' = constant, 3000 for single wells, and 1500 or 2000 for single line wells

h_{dd} = depth of drawdown, m

k_{avg} = average permeability, m/s

Consequently, radius of influence for the well is:

$$R = C' \times h_{dd} \times \sqrt{k_{avg}} = 3000 \times (5.2 + 1.5) \times \sqrt{\frac{6.3183 \text{ m/day}}{86400 \text{ sec/day}}}$$

$$R = 197.54 \text{ m}$$

Next step is calculating discharge of well, and for this purpose rectangular area of the house must be converted to equal radius of excavation.

Areal dimensions of the existing building are 115.2 m x 13.2 m, and the area is 1520.64 m². To convert rectangular area to equivalent circle, following formula can be used:

$$r = \sqrt{\frac{\text{Area of rectangle}}{\pi}} = \sqrt{\frac{1520.64}{\pi}} = 22 \text{ m}$$

Next, using single well formula discharge rate can be calculated (Forchheimer, 1930):

$$Q = \frac{\pi k_{avg} (H^2 - h_{dd}^2)}{\ln R_f - \ln r}$$

Where:

H = distance from ground surface to the impermeable soil layer, in this case we have impermeable soil layer at -20 m.

As a result, discharge of the well

$$Q = \frac{\pi \frac{6.3183 \text{ m/day}}{86400 \text{ sec/day}} (20^2 - 6.7^2)}{\ln 197.54 - \ln 22} = 0.03566 \text{ m}^3/\text{s}$$

Checking for yield from a single well (Han, 2015):

$$Q_{MAX} = \frac{2\pi r h_{dd-well} \sqrt{k_{avg}}}{15}$$

Where: r = radius of well = 0.1 m;

Yield of a single well:

$$Q_{MAX} = \frac{2 \times \pi \times 0.1 \times 6.7 \sqrt{\left(\frac{6.3183 \frac{m}{day}}{86400 \text{ sec/day}}\right)}}{15} = 0.00276 \text{ m}^3/\text{s}$$

Hence, 13 wells are needed for the targeted area. After evaluation of necessary discharge rate, 6 pumps are installed inside of each well. All of them are going to be connected by pipe system and outflow water is going to be saved in water reservoirs.

It is planned to locate wells symmetrically along the long side of the building:

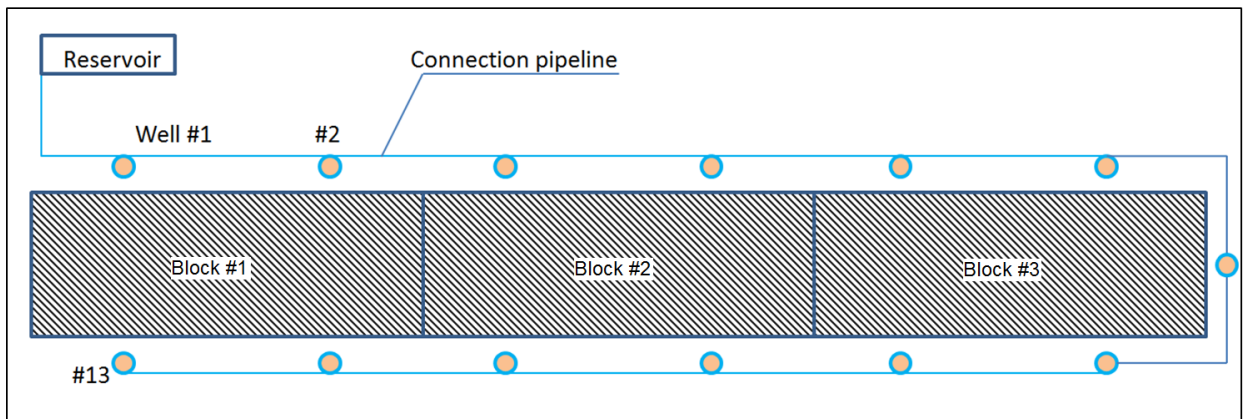


Figure 4.12. Location of dewatering wells

Dewatering can cause settlement to adjacent structural elements, and to reduce this effect recharging well is going to be installed (Han, 2015). But still settlement of the building due to the dewatering must be evaluated.

4.2.2.3.2.4. Settlement caused by dewatering

Dewatering process causes settlement of the soil which, consequently, results in building settlement. To evaluate soil movement dewatering process was simulated in Plaxis 2D. In **Figure X** above it can be noticed that in the beginning water table was located 1m below the ground level. In the next **Figure X** result of dewatering is demonstrated.

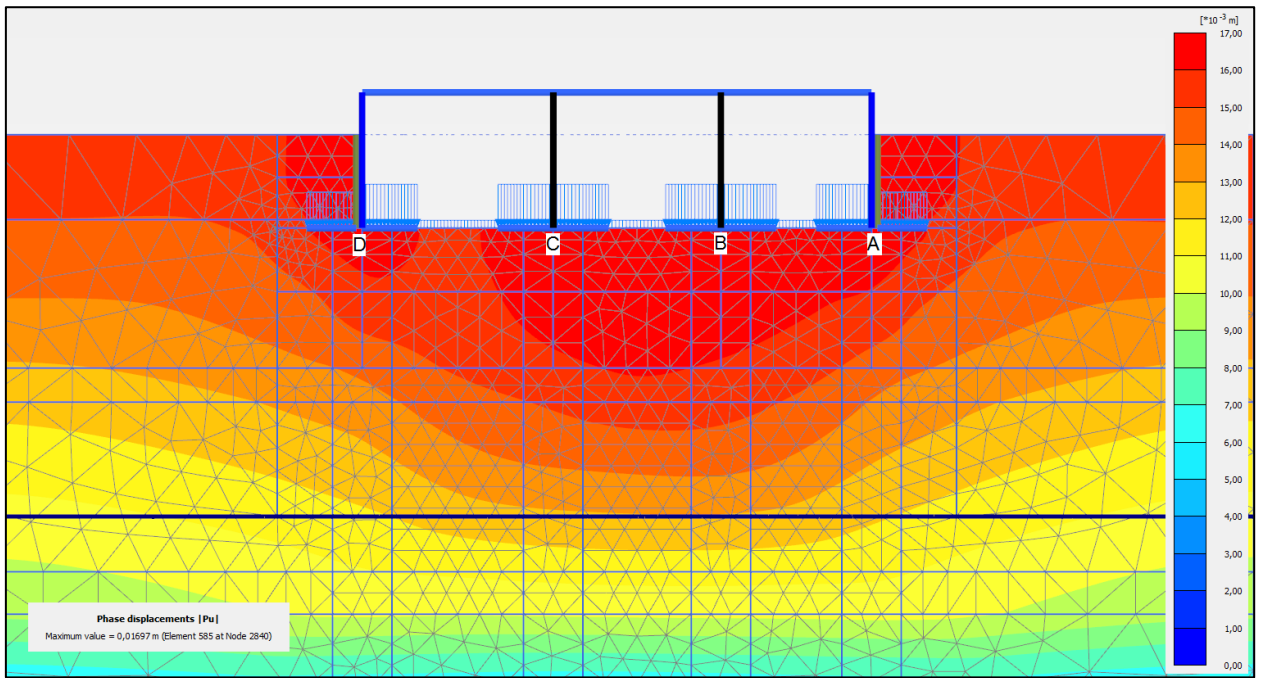


Figure 4.13. Settlement under the foundation caused by dewatering

As it can be noticed from the displacement graph, dewatering of targeted area causes significant settlement for the existing building. Checking displacement at points under the each footing following **Table 4.2** was obtained.

Table 4.2. Maximum settlement during dewatering of points A, B, C, D, (meters)

Point	Settlement (m)
A	0.01584
B	0.01648
C	0.01689
D	0.01534

Despite significance of the settlement, the fact that dewatering is going to be applied to the whole building will cause settlement of whole building together. First of all, it is important to check settlement of the points relative to each other, for the reason that if all points settled for 0.015 m, there is no stress effect on structural part of the building.

Table 4.3. Relative settlement during dewatering of points A, B, C, D, (meters)

Point	Displacement, relative to the minimum one (m)	Settlement (m)
A	0.01584	0.0005
B	0.01648	0.00114
C	0.01689	0.00155
D	0.01534 (minimum)	0

4.2.2.3.2.5. Checking for stability

After obtaining the settlement in this Plaxis, special spring model was created in SAP 2000 program. The idea was to install springs under each column to which it will be possible to give displacement values. As a result following model was created.

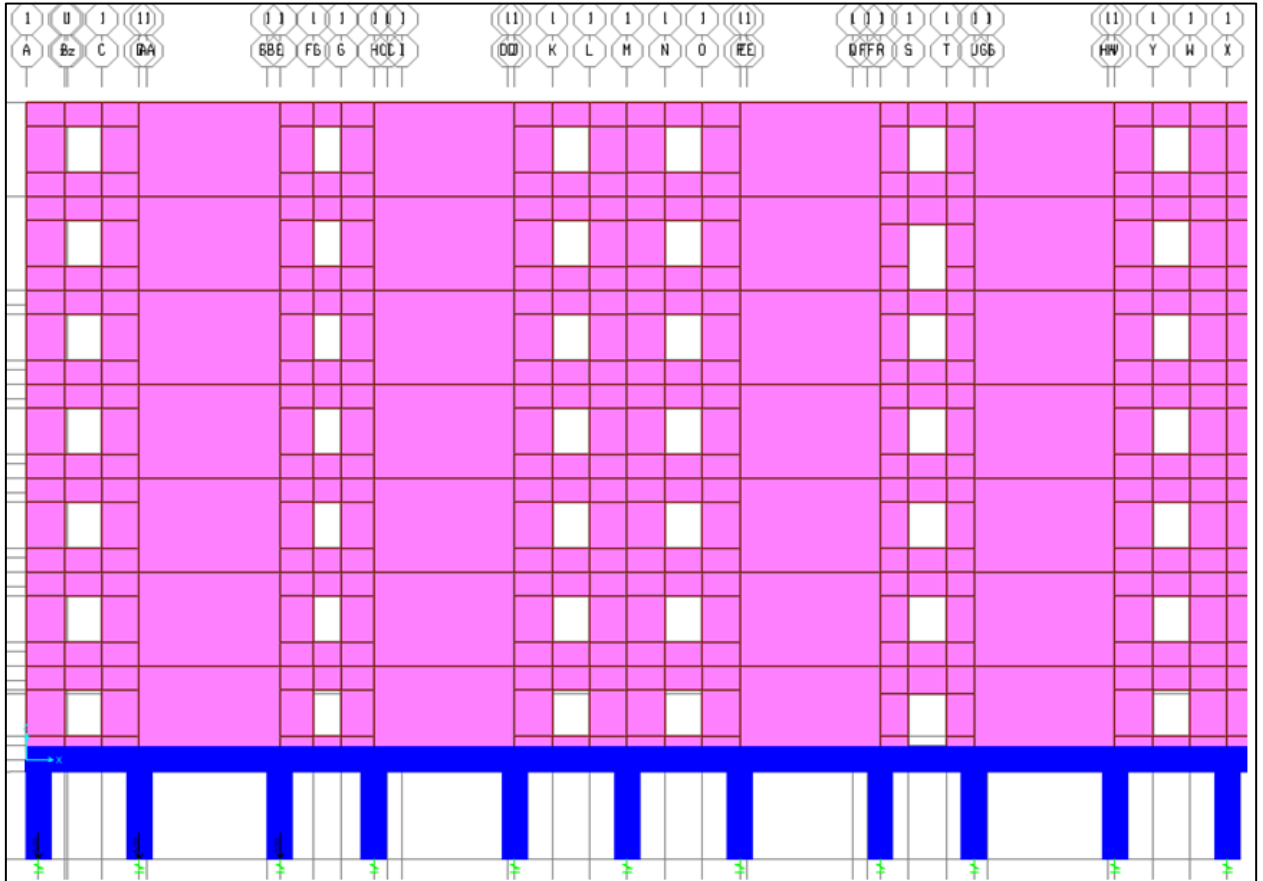


Figure 4.14. Location of springs

Green drawings under each column are springs and it can be noticed from the **Figure 4.14** above. The value of the spring constant was taken as $10,000,000 \text{ kN/m}^3$. Such high value of coefficient was given to give initial condition to the building.

To simulate settlement, obtained values of relative settlement was given to the column rows (see four rows in **Figure 4.15** below). This was due to the fact that whole building was influenced by settlement.

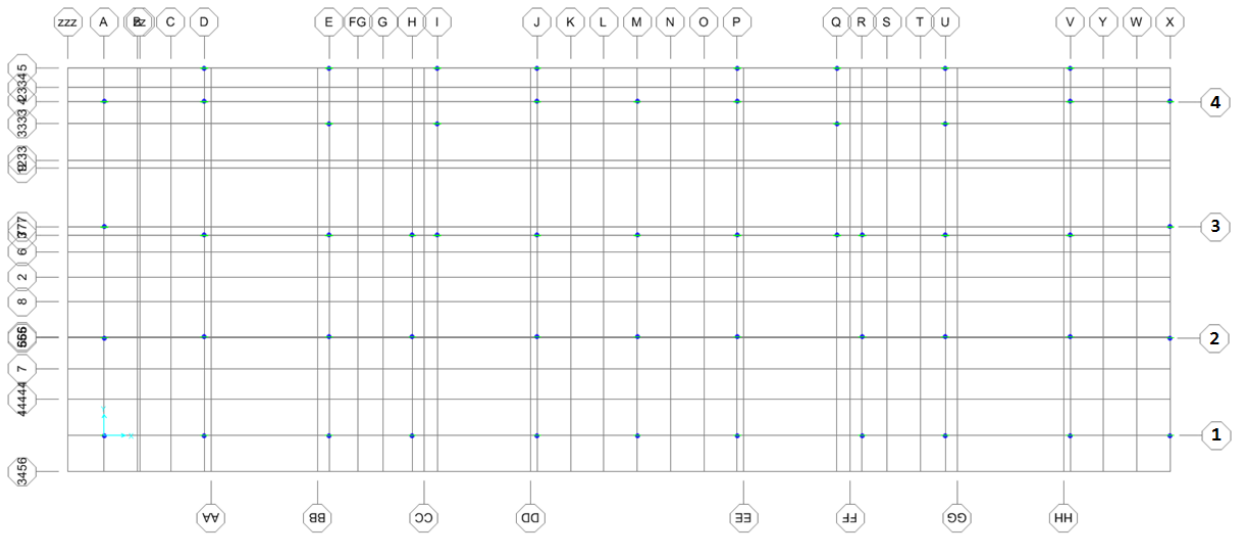


Figure 4.15. Location of springs

As a result of simulation no failure was discovered in any structural member of the building which is the most important point for the settlement.

All the stages that were analyzed in the Plaxis were checked for stability on SAP 2000.

4.2.2.3.3. Removal of inner soil layer (Stage 3-5)

Next step for excavation is to remove inner soil layer section by section. Starting from right to left, three section of inner soil layer was removed one by one. See **Figure 4.16** below.

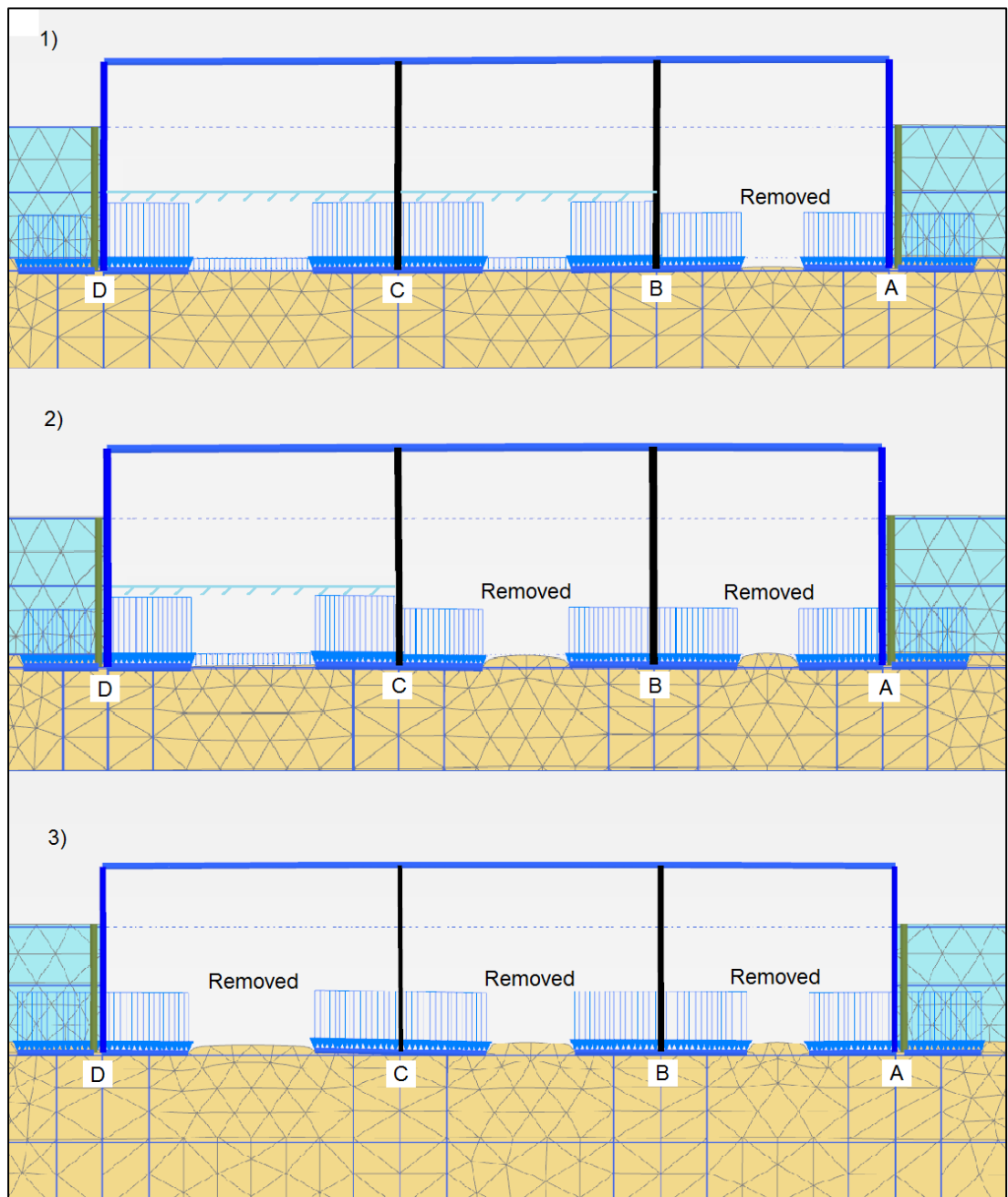


Figure 4.16. Soil deformation during three excavation phase (Scale: 20 times).

Note: deformation demonstrated in the diagrams above are not in scale, but 20 times larger in scale. It is done to show the deformation due to the fact that in true scale they are small to notice. To see true displacement, displacement map must be created as in following figures.

It can be noticed from graph above that removing each layer of inner soil layer (represented as linear load) causes upper movement of soil between footings. Moreover, upper movement from excavation causes settlement of the footings. To be able to predict settlement under each footing, points under them were checked for consolidation (points A, B, C, D). By obtaining consolidation at these points it is possible to check settlements under each column. At the **Figure 4.17-4.19** below it is possible to see deformation profile of soil during these three stages.

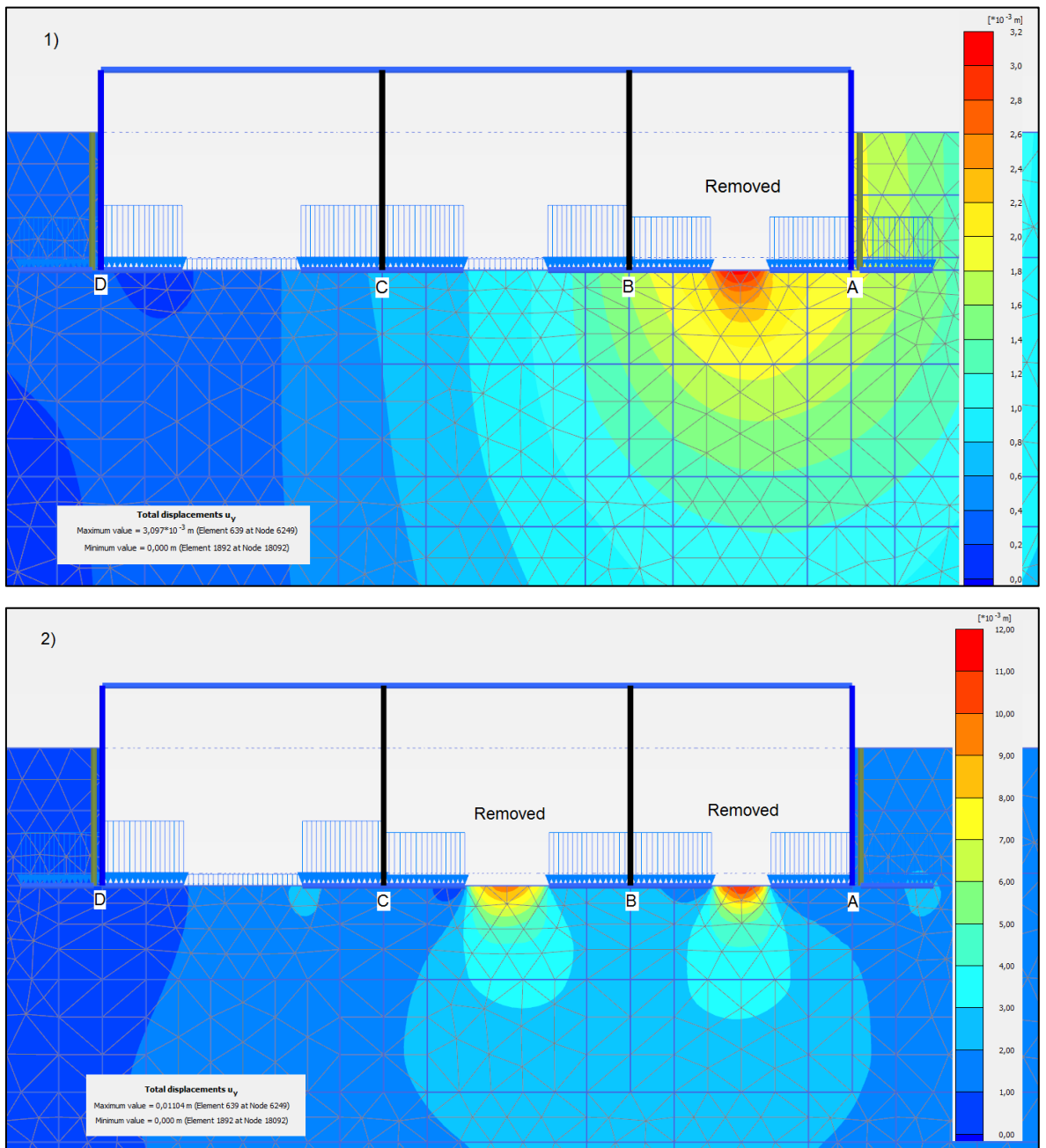


Figure 4.17 – 4.18. Removing inner soil layer step-by-step

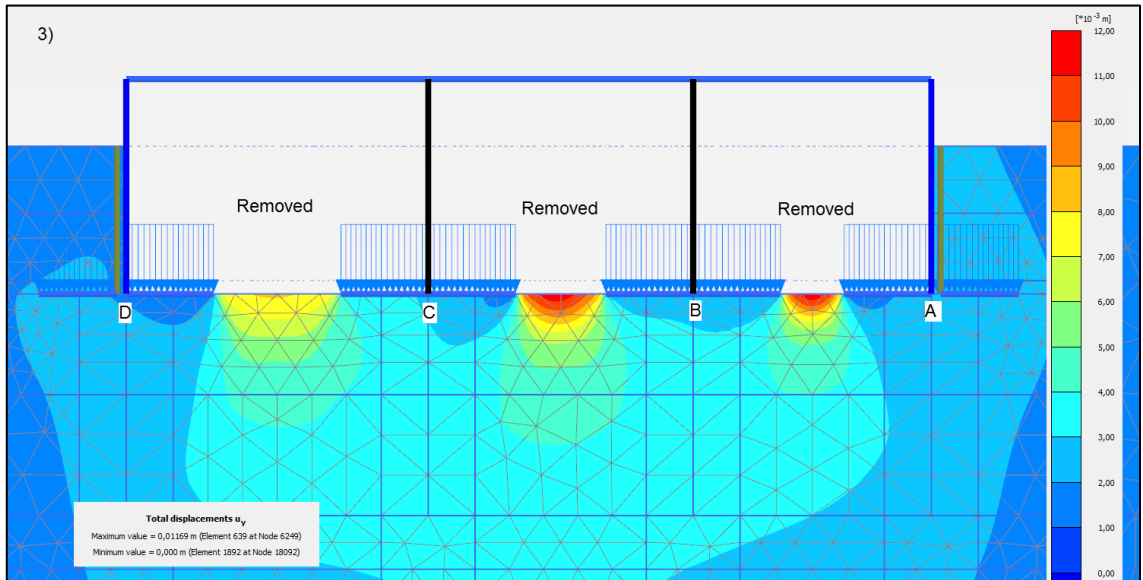


Figure 4.19. Removing inner soil layer step-by-step

Total displacement of each point is presented in the **Figure 4.20** below.

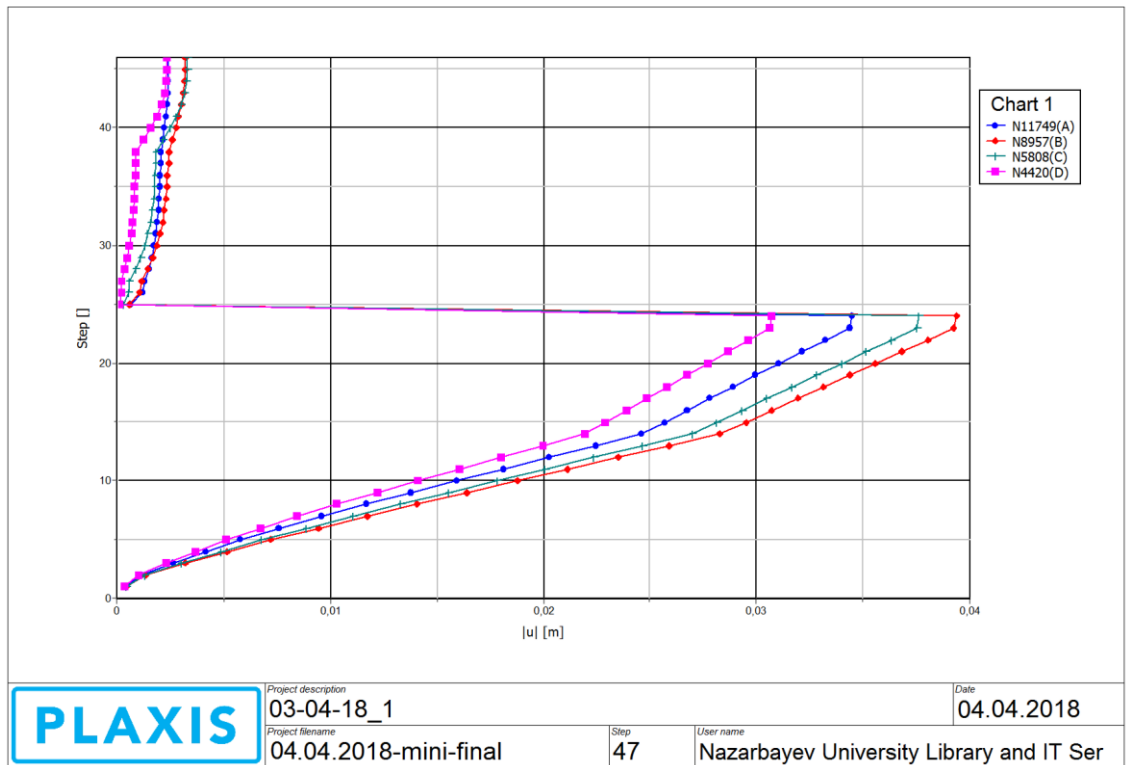


Figure 4.20. Displacement of soil under the footings

Step 0-27 corresponds to the first stage, construction of existing building. Drop of displacement after Step 27 corresponds to zeroing the displacement. It can be noticed from graph that during Step 27-47 displacement at four points are rising again, with its maximum value at Step 47. The maximum displacement for each stage can be found in below **Table 4.4**.

Table 4.4. Maximum displacement per stage of points A, B, C, D, (meters)

Point	Stage 2	Stage 3	Stage 4
A	0.00148	0.00206	0.00240

B	0.00145	0.00244	0.00322
C	0.00086	0.00186	0.00331
D	0.00036	0.00087	0.00233

After obtaining settlement for each excavation process of inner soil layer, it is possible to check stability of building. It can be done by applying settlement values for the SAP model of the house and evaluate it for failure.

Similar method as for dewatering was used, however instead of applying settlement for the whole structure (all the columns), only four row of columns were given displacement. This was made due to the fact that only part of the house is going to be excavated at once.

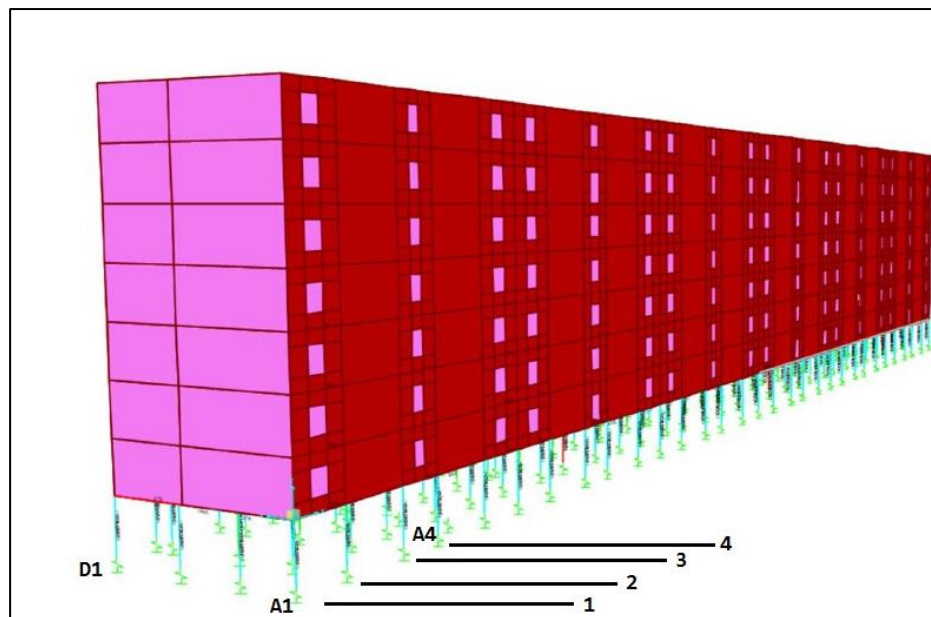


Figure 4.21. Spring design on SAP 2000

Obviously, it is incorrect to give the same value of settlement to the columns of A1 to A4, however due to the two-dimensional restriction in Plaxis 2D, it was decided to leave this way.

As a result of simulation of current step, it was discovered that structural elements can withstand such settlement.

4.2.2.3.4. Load from additional two floors (Stage 6)

For the reason that this project is going to be top-down construction, which means that foundation replacement and adding floor are going to be performed simultaneously. To be able to simulate additional floors, linear load on footing was increased, from 154 kN/m to 197 kN/m.

4.2.2.3.5. Sheet pile and strut installation (Stage 7-9)

4.2.2.3.5.1. Calculation of sheet pile and strut

To be able to install sheet pile and strut, the length and the location of strut must be calculated manually. First of all, soil profile was prepared for the calculation. It can be seen from **Figure 4.22** below the initial soil profile for the sheet pile calculation.

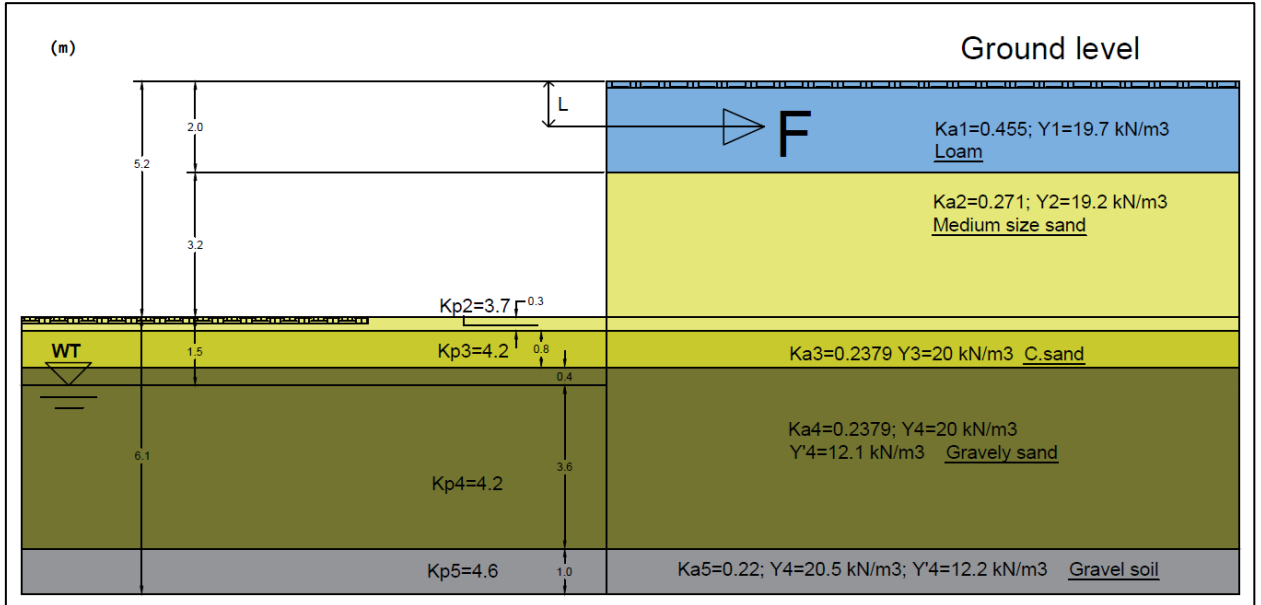


Figure 4.22. Soil profile

Above soil profile is considered as complicated for sheet pile calculation, for this reason several points were simplified. First of all, in order not to leave 0.3 m of passive side, it was decided to evaluate 5.5 m excavation instead of 5.2 m. Secondly, for Coarse sand and Gravelly sand all the values almost identical, for this reason they were mixed together in one layer. Finally, it was assumed that the length of sheet pile will be less than 5 m, and consequently will not affect last layer. As a result of above assumptions, following soil profile was evaluated:

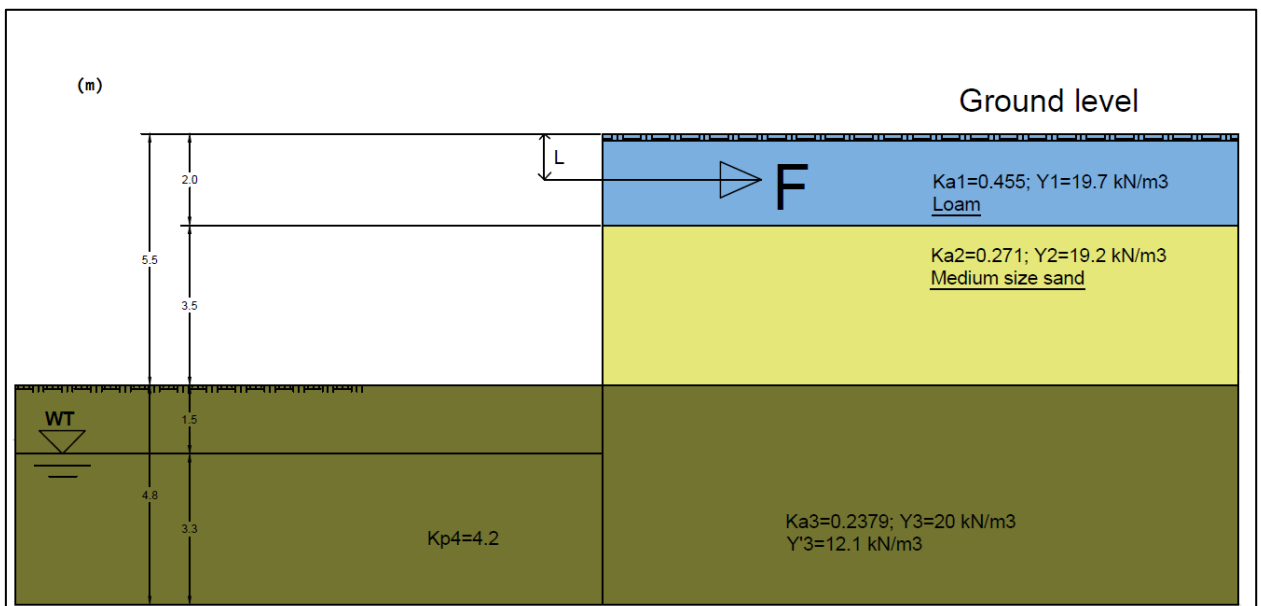


Figure 4.23. Simplified soil profile

The value of K_a and K_p , was evaluated from formulas:

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

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

All the parameters needed for sheet pile calculation is presented in **Table 4.5** below.

Table 4.5. Soil parameters

#	Soil type	y unsat (kN/m ³)	y sat (kN/m ³)	y' (kN/m ³)	c'	ϕ'	Ka	Kp
1	Loam	19.7	20.9000	11.1	18.00	22.00	0.4550	
2	Medium size sand	19.2	21.5000	11.0	0.00	35.00	0.2710	3.6902
3	Coarse sand	20	21.9000	12.1	0.00	38.00	0.2379	4.2037
4	Gravelly sand	20	22	12.2	0.00	38.00	0.2379	4.2037

As a scheme of the calculation following figure can be used

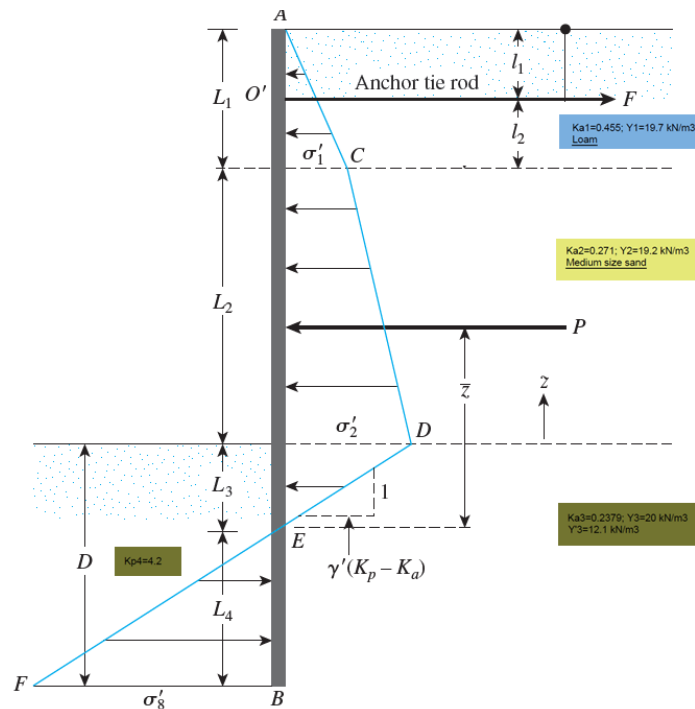


Figure 4.24. Stress profile of the soil

Where: $L_1=2\text{m}$; $L_2=3.5\text{m}$;

- 1) To find force by active side of the soil, P , all the forced was summed. The results of Force P calculation is presented in the **Table 4.6** below.

Table 4.6. Calculation of active earth pressure above dredge line

N	Size coef.	Force	Force	Distance from bottom (z)	F*z
1	0.5	$K_{a1} \cdot \gamma_1 \cdot L_1 \cdot L_1$	17.9255	4.17	74.69
2	1	$K_{a2} \cdot \gamma_1 \cdot L_1 \cdot L_2$	37.36952846	1.75	65.40
3	0.5	$K_{a2} \cdot \gamma_2 \cdot L_2 \cdot L_2$	31.86843036	1.166666667	37.18

		Total Force, P	87.1635	z	2.03
--	--	----------------	---------	---	------

2) For calculation of L3, active and passive stress of ground under the excavation line must be summed.

Active stress:

$$\sigma_a = P + \gamma_3 * K_{a3} * L_3$$

Passive stress:

$$\sigma_p = \gamma_3 * K_{p3} * L_3$$

Total stress:

$$\sigma' = \sigma_a - \sigma_p = P - \gamma_3 * (K_{p3} - K_{a3}) * L_3$$

At the distance L3, the net stress is 0:

$$\sigma' = 0;$$

$$L_3 = P / (\gamma_3 * (K_{p3} - K_{a3}))$$

As a result, **L3 = 0.4559 m.**

3) Now, to find total stress under the dredge line, σ'_8 , following formula can be used:

$$\sigma'_8 = \gamma_3 * (K_{p3} - K_{a3}) * L_4$$

By now, the value of L4 is not known, so the result is:

$$\sigma'_8 = 78.52 L_4$$

4) Now, applying force equilibrium to the system, it is possible to obtain following equation:

$$F = P - \frac{1}{2} * \sigma'_8 * L_4$$

$$F = P - \frac{1}{2} * 78.52 * L_4$$

$$F = 87.1635 - 39.63 * L_4$$

5) Moment equilibrium over point of F:

$$-P * (L_1 + L_2 + L_3 - z - L) + (\frac{1}{2} * \sigma'_8 * L_4) * L_4 = 0$$

Where L is the distance from top of the soil to the strut, and z=2.03 m result is:

$$-87.1635 (3.926 - L) + 39.63 * L_4^2 = 0$$

At this point, the depth of strut was assumed as L=1m. As a result it was find that **L4 = 1.375m**, and consequently, **F = 12.238 kN/m**

6) Final depth of the sheet pile:

$$D = L_3 + L_4$$

$$D \text{ (theoretical)} = 1.8309 \text{ m}$$

But,

$$D \text{ (actual)} = 1.5 \text{ to } 2 \text{ D (theoretical)}$$

$$\mathbf{D(actual) = 3.66 \text{ } \odot \text{ } 3.7 \text{ m}}$$

The location of strut is at -1m below the ground level and the force needed for strut is 12.238 kN/m.

4.2.2.3.5.2. Design of sheet pile and strut in Plaxis

By identifying length of the sheet pile and location of strut, it is possible to simulate model in the software. For this case 9.2 m long sheet pile was selected and strut was placed one meter below the ground level. Material properties of these two structural elements were taken from Appendix.

First of all, sheet piles are installed from both side of the building. The length of sheet piles is equal to 9.2 m. As a result of simulation following deformation mesh and displacement values for soil were obtained.

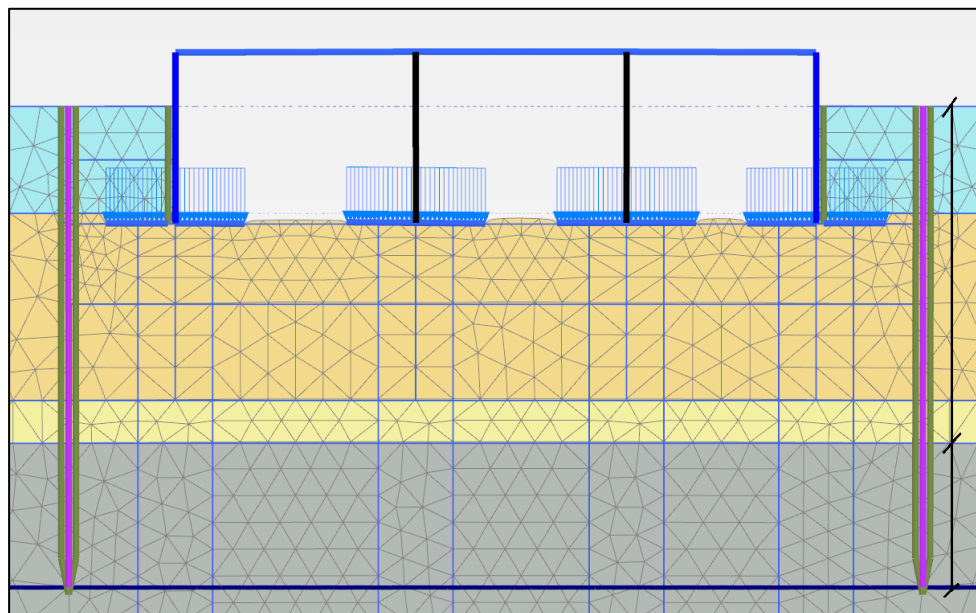


Figure 4.25. Deformation mesh after installation of sheet piles (Scale x20)

It was defined in calculation that strut must be installed -1 m below ground level. To be able to install them first of all small layer of soil must be excavated, then struts can be installed (See Figure X below).

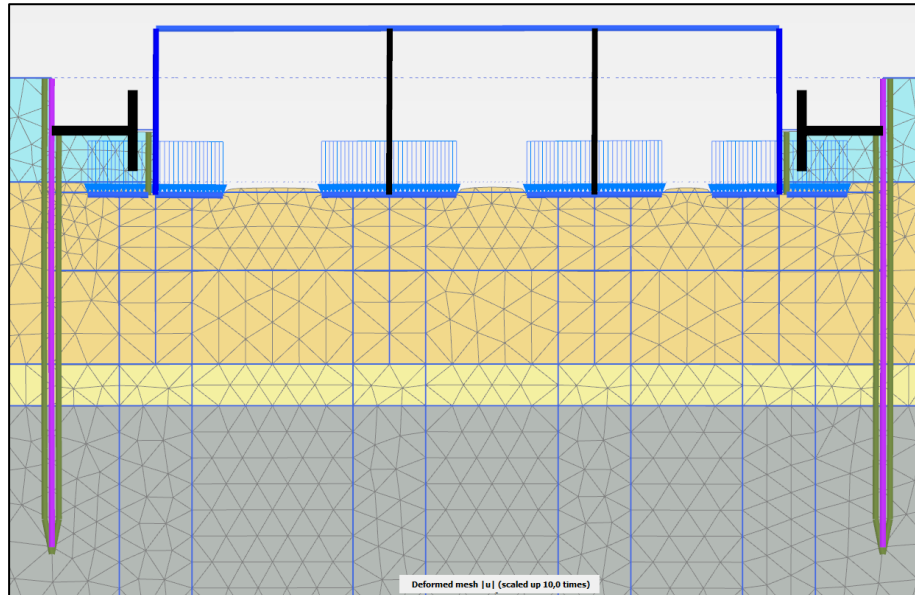


Figure 4.26. Removing soil layer and installation of struts (Scale x10)

In order to advance to micropiles installation process, it is necessary to excavate the footing layer (see **Figure 4.27** below).

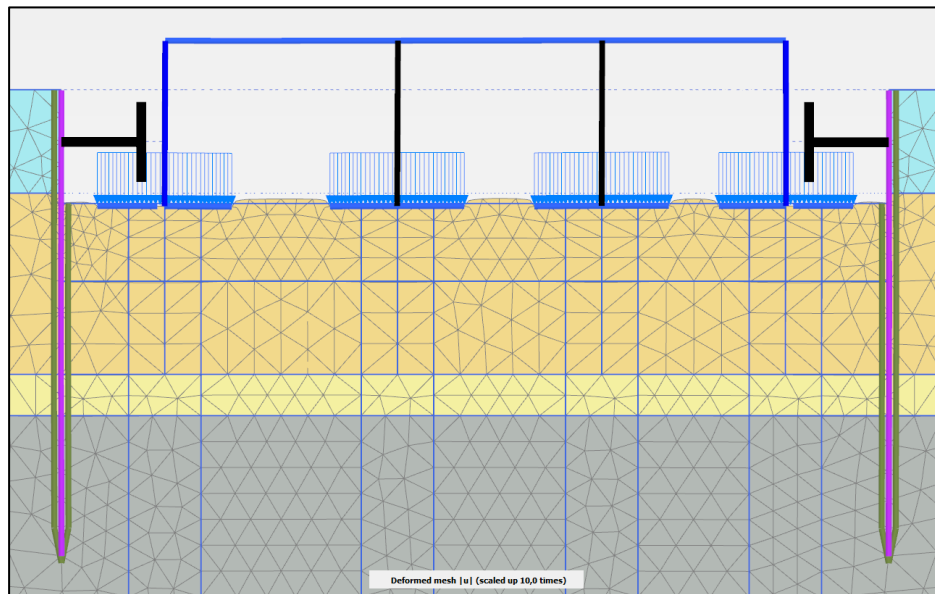


Figure 4.27. Installation of struts

4.2.2.3.6. Micropile installation (Stage 10)

4.2.2.3.6.1. Micropile design

In order to construct parking and relocate foundation below micropiles can be used. Micropiles is suitable for this case, as they can be installed in low headroom, as case consist of units that can be connected to each other, the grout inside can be poured by the pressure machine. The drilling machine and pump machine to not require high ceiling as it can be seen from the photos below **Figure 4.28**.



Figure 4.28. Installation of micropiles by drilling machine.

Micropiles will be installed through existing footing of foundation, there will be 4 micropiles through one footing. Every pile on the corner of footing. The procedure of micropile can be observed from the picture below **Figure 4.29.**

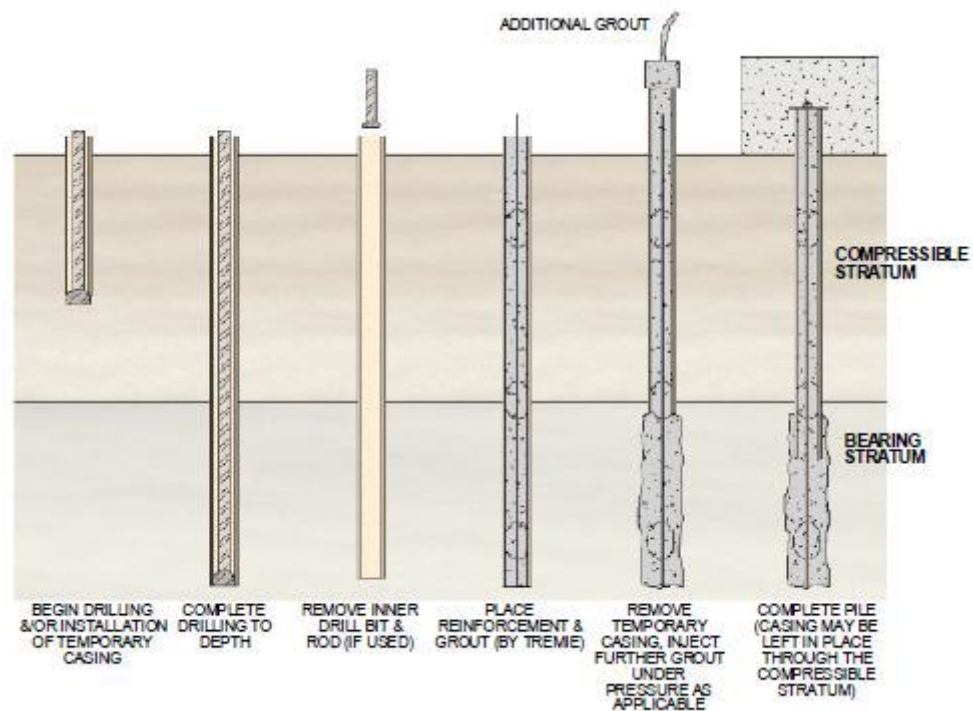


Figure 4.29. Procedure of micropile installation

Equipment drill the hole and simultaneously lower the temporary and permanent case in order to prevent hole failure. After drilling to the destined depth drill bit removes and the hole filled with reinforcement with grout (concrete), after that temporary case removed from bearing

stratum (below the parking level), and additional grout injected by the pressure in order to fill the void. Permanent case left in the space of further parking zone. The next step is to fix the connection of micropile with footing, this connection is shown below **Figure 4.30**.

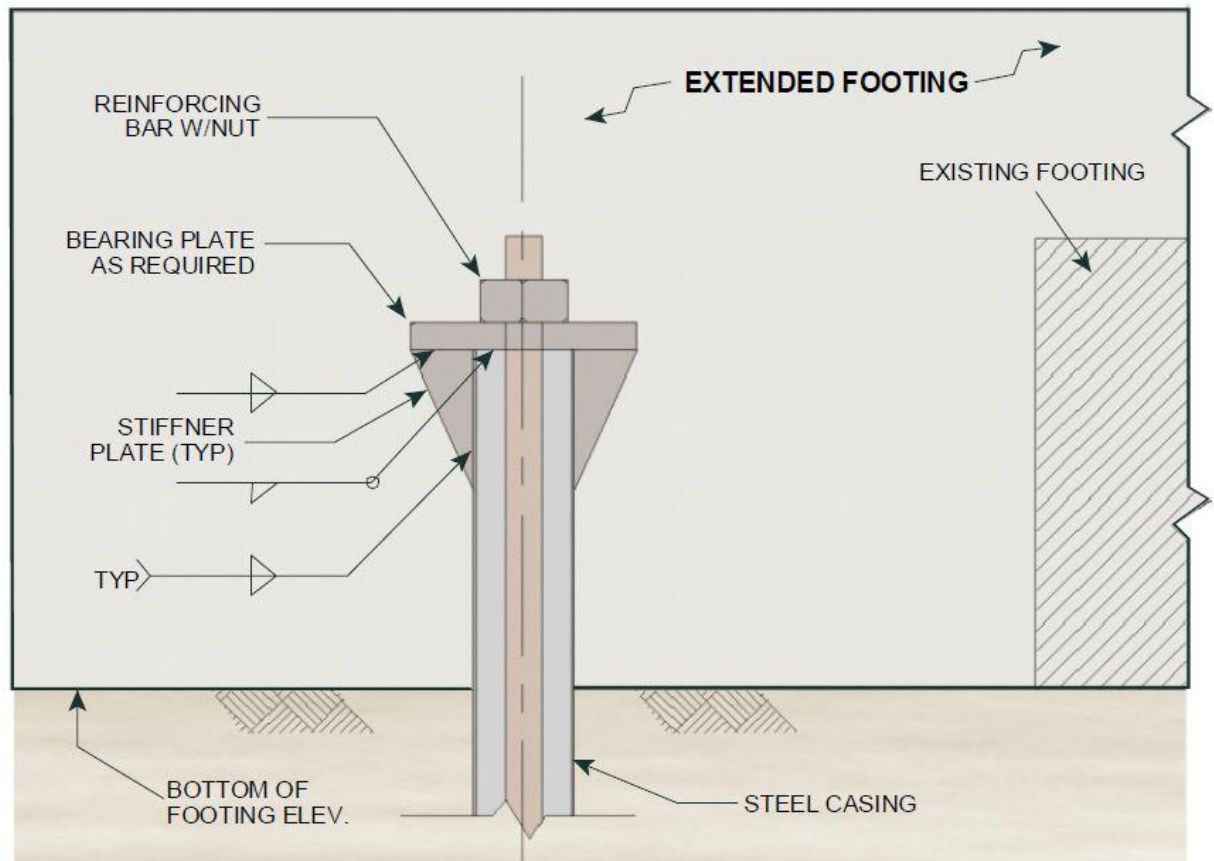


Figure 4.30. Pile to footing connection.

The foundation is extended as according to the manual piles, load on which extend the value of 100 kips =450 kN should be connected by extension. By calculations provided in the **appendix Table 15**, the load on each pile exceed 450 kN and it is 502,8 kN therefore in **appendix Table 18** calculations for the square bearing plate of 254*254 mm is designed with thickness 20 mm. And length of welding 1191 mm and with size of 6.35 mm, the calculation of it provided in the **appendix Table 19**. The detailed drawing is provided below in the **Figure 4.31**.

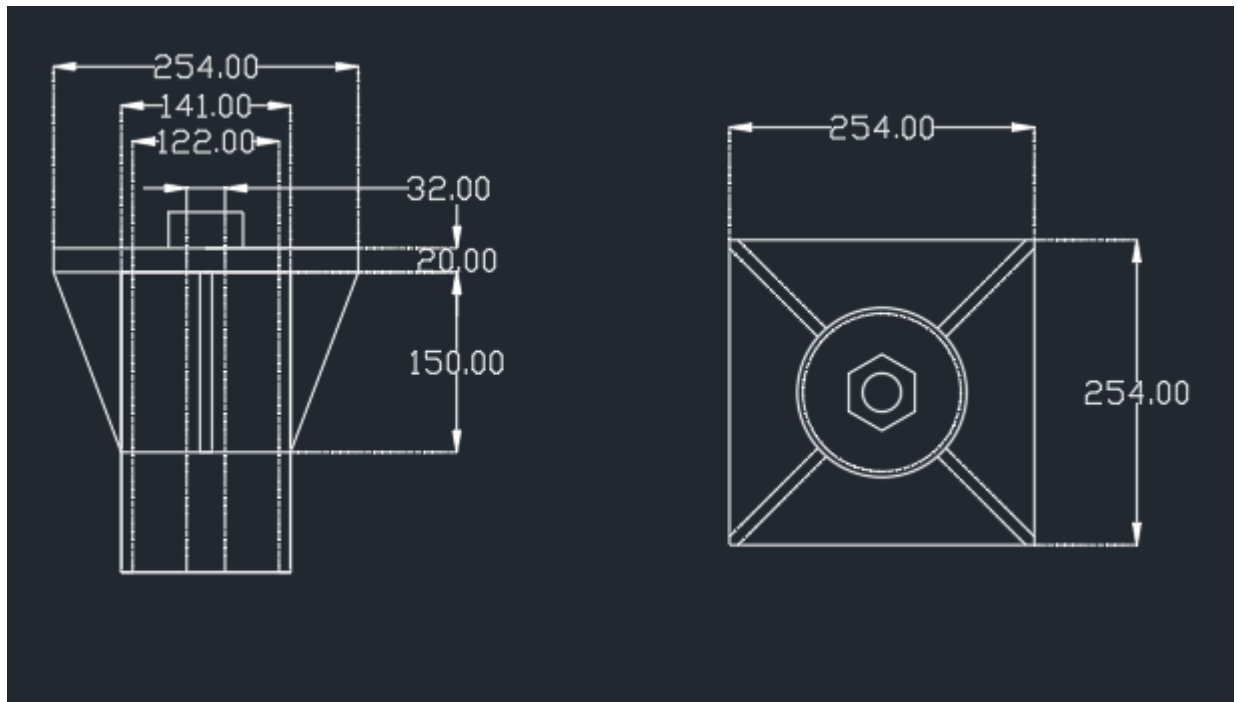


Figure 4.31. Detailed view of bearing plate of pile

The loads on the footing were also obtained from the SAP2000 and all combination of loads and moments were evaluated to find the maximum one, which is combination in 49 column. The detailed calculations provided in the **appendix Table 16**. Here is general information.

$$P_n = \frac{N}{4} + \frac{M_2}{1.1} - \frac{M_1}{1.1} = 502.83 \text{ kN}$$

Preliminary dimensions and general information:

Outside Diameter: OD = 141 mm

Wall thickness: t = 9.5mm

Inside diameter: ID = 122 mm

Area of casing: $A_c = 3922.645 \text{ mm}^2$

Reinforcing bar: d = 32mm

Area of bar: $A_b = 803.84 \text{ mm}^2$

$F_{y\text{-bar}} = 520 \text{ MPa}$

Grout Area: $A_{gr} = 10880.1 \text{ mm}^2$ & $A_{drill} = 27833.74 \text{ mm}^2$

$f'_{c\text{-grout}} = 34.5 \text{ MPa}$

$f_{y\text{-steel}} = \text{min } 241 \text{ MPa}$

Allowable compression load:

$$P_{c\text{-allow}} = \left(0.4 * f'_{c\text{-grout}} * A_{grout} + 0.47 * F_{y\text{-steel}} * (A_{bar} + A_{casing}) \right)$$

$$P_{c\text{-allow}} = 685.514 \text{ kN} > P_n = 502.83 \text{ kN}$$

Allowable tension load:

$$P_{t-allow} = (0.55 * F_{y-steel} * (A_{bar} + A_{casing}))$$

$$P_{t-allow} = 626.49 \text{ kN}$$

Allowable compression load on Uncased part:

$$P_{c-allow} = (0.4 * f'_{c-grout} * A_{drill} + 0.47 * F_{y-steel} * (A_{bar}))$$

$$P_{c-allow} = 580.56 \text{ kN} > P_n = 502.83 \text{ kN}$$

Allowable tension load:

$$P_{t-allow} = (0.55 * F_{y-steel} * (A_{bar}))$$

$$P_{t-allow} = 229.89 \text{ kN}$$

In order to calculate geotechnical allowable capacity, the **Table 4.7** is used, in this case Type B of Micropile and soil profile is used.

Table 4.7. Typical Grout-to-Ground Bond values for Micropile Design

Soil / Rock Description	Grout-to-Ground Bond Ultimate Strengths, kPa (psi)			
	Type A	Type B	Type C	Type D
Silt & Clay (some sand) (soft, medium plastic)	35-70 (5-10)	35-95 (5-14)	50-120 (5-17.5)	50-145 (5-21)
Silt & Clay (some sand) (stiff, dense to very dense)	50-120 (5-17.5)	70-190 (10-27.5)	95-190 (14-27.5)	95-190 (14-27.5)
Sand (some silt) (fine, loose-medium dense)	70-145 (10-21)	70-190 (10-27.5)	95-190 (14-27.5)	95- 240 (14-35)
Sand (some silt, gravel) (fine-coarse, med.-very dense)	95-215 (14-31)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Gravel (some sand) (medium-very dense)	95-265 (14-38.5)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	95-190 (14-27.5)	95-310 (14-45)	120-310 (17.5-45)	120-335 (17.5-48.5)
Soft Shales (fresh-moderate fracturing, little to no weathering)	205-550 (30-80)	N/A	N/A	N/A
Slates and Hard Shales (fresh- moderate fracturing, little to no weathering)	515-1,380 (75-200)	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	1,035-2,070 (150-300)	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	520-1,725 (75.5-250)	N/A	N/A	N/A
Granite and Basalt (fresh- moderate fracturing, little to no weathering)	1,380-4,200 (200-609)	N/A	N/A	N/A

Type A: Gravity grout only
 Type B: Pressure grouted through the casing during casing withdrawal
 Type C: Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting
 Type D: Primary grout placed under gravity head, then one or more phases of secondary "global" pressure grouting

And formula below is used:

$$L_b = \frac{P_{g-allow} * F.S.}{\alpha * \pi * D_b}$$

As the length of pile is oversized the length of one soil layer, the formula is rearranged and values obtained for each soil type. And applying safety factor, which is equal to 2, the length of uncased part- bearing part in the earth calculated to be 7.99 meters, which is rounded to 8 meters. The length of cased part is the part of future excavated parking zone, which equal to 3 meters. The detailed drawing of the pile design is shown below in the **Figure 4.32.**

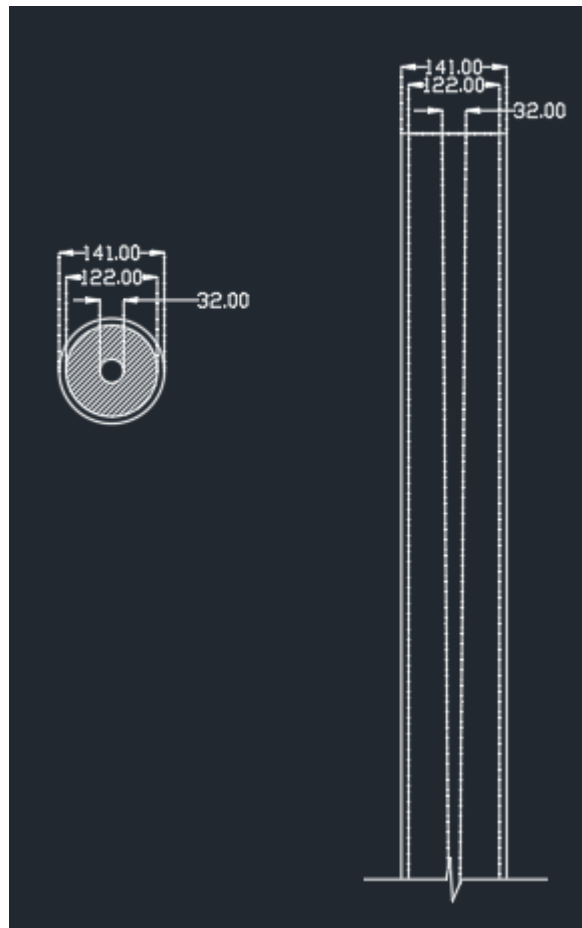


Figure 4.32. Detailed drawing of the pile design.

Elastic settlement of piles is calculated to be 1.17 mm for cased part and 1.96 mm for uncased part. The calculations of it provided in **appendix Table 17**.

4.2.2.3.6.2. Plaxis simulation of micropiles

Calculated length of micropiles is 11 m. For this section four pairs of micropiles are going to be installed. Their location and properties were discussed in previous parts. In Plaxis program deformation for installation stage is analyzed.

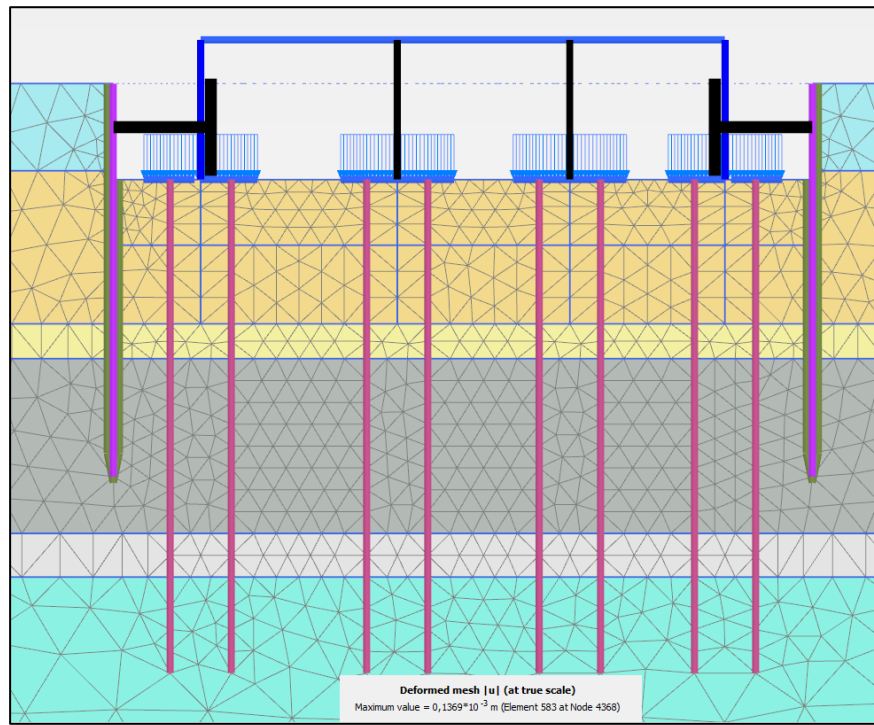


Figure 4.33. Installed micropiles

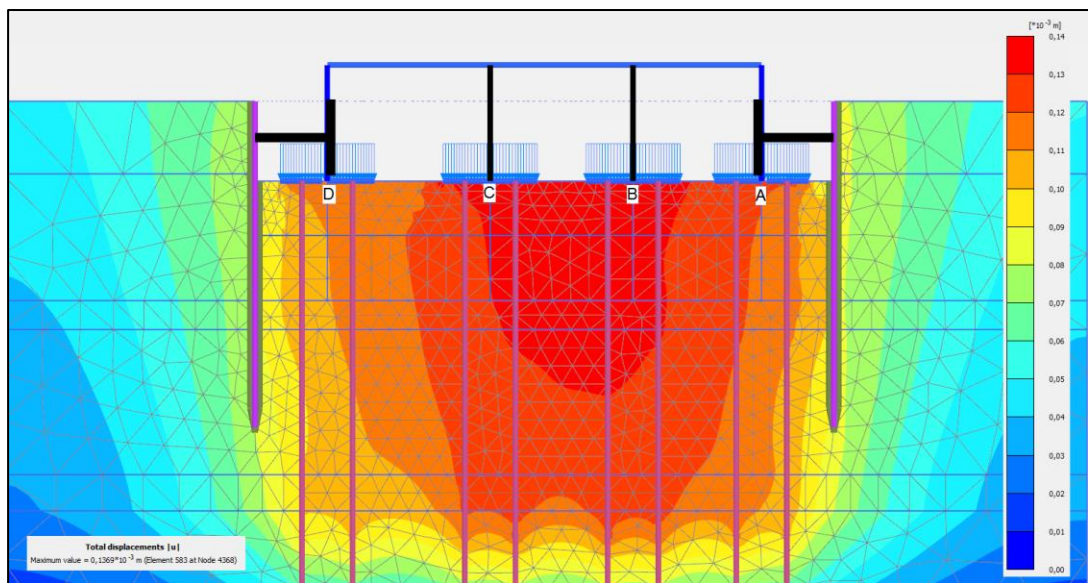


Figure 4.34. Deformation caused by installation of micropiles

Displacements at points A, B, C, D are demonstrated in the **Table 4.8** below.

Table 4.8. Displacements at points A, B, C, D

Point	Displacement (m)
A	0.0001690
B	0.0001313
C	0.0001277
D	0.0001121

Similar to the previous steps, settlement at this stage was checked on SAP 2000 program, and no failure was in any structural element of the building.

4.2.2.3.7. *Excavation of soil under the footing (Stage 11-12)*

After the successful installation of micropiles, it is possible to start excavation of soil under the footings. -2nd Floor is going to be excavated in two layers.

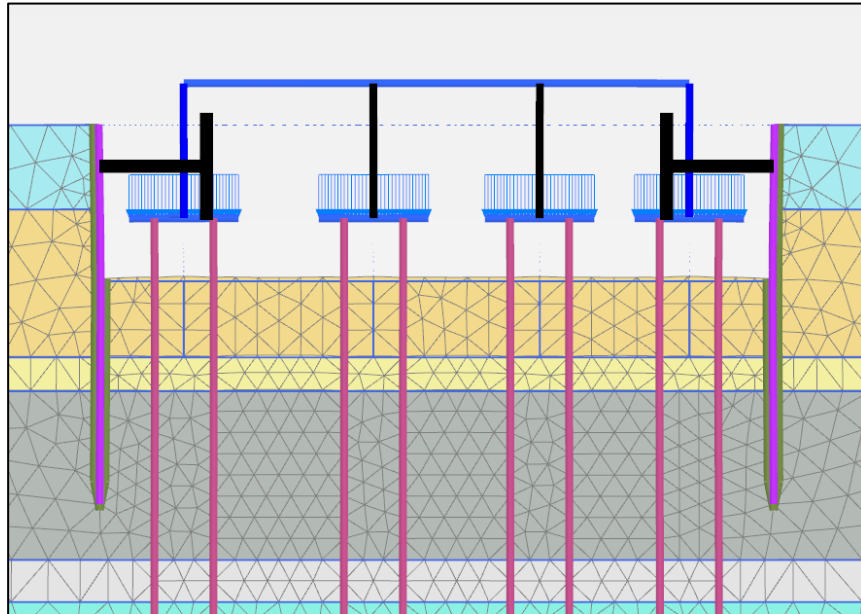


Figure 4.35. Excavation of soil under the footings in two steps. First step (Scale x20)

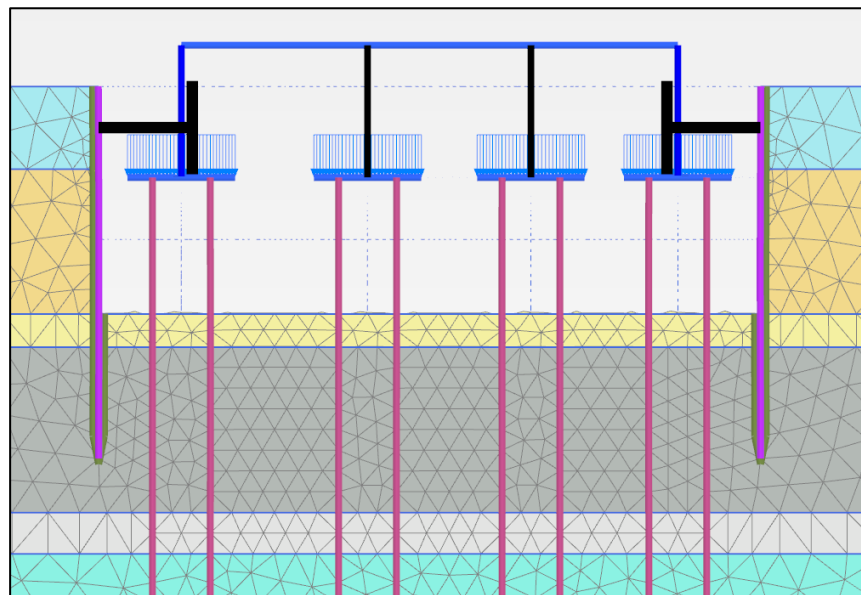


Figure 4.36. Excavation of soil under the footings in two steps. Second step (Scale x20)

4.2.2.3.8. *Installation of foundation, columns, walls and slab (Stage 13-15)*

After reaching final step of excavation inner structural elements of the building can be installed. All the calculations of structural elements were performed by hand and result was tested in Plaxis program.

Installation process is divided into three main parts:

1. Installation of common foundation for building
2. Installation columns, walls and slabs
3. Removing unnecessary structural elements.

4.2.2.3.8.1. Structural elements

Slab design

In this section slab design of parking will be considered. As -1 floor already has columns and footing, the design will be considered only between -1 and -2 floors, moreover because of those footings and installed micropiles, which restrain accessibility to construct common types of slab as one-way slab and two-way slab. Therefore, it was decided to design flat slab without drop. Despite that drop panel will not be designed, the ex-footing can act as drop panel, the detailed illustration provided in the **Figure 4.37**.



Figure 4.37. Cross section of slab.

From SAP2000 the loads on slab were obtained and used in design. The pressure on the slab was 12kPa, this load already includes live load and dead load of whole structure with necessary coefficients according to ACI code. The detailed calculations provided in the **appendix Table 20**.

General information:

$$\text{Steel: } f_y = 60000 \text{ psi}$$

$$\text{Concrete: } f'_c = 4000 \text{ psi}$$

$$\text{Density of concrete: } 150 \text{ lb/ft}^3$$

The self-weight of slab is 75 psf. So, in total load on the slab is calculated as 260psf. The dimension of slab is taken for maximum area on the column according to the layout of the structure, which is shown in **Figure 4.38**. Those dimension equal to $l_1 = 13.12 \text{ ft}$ and $l_2 = 9.84 \text{ ft}$.

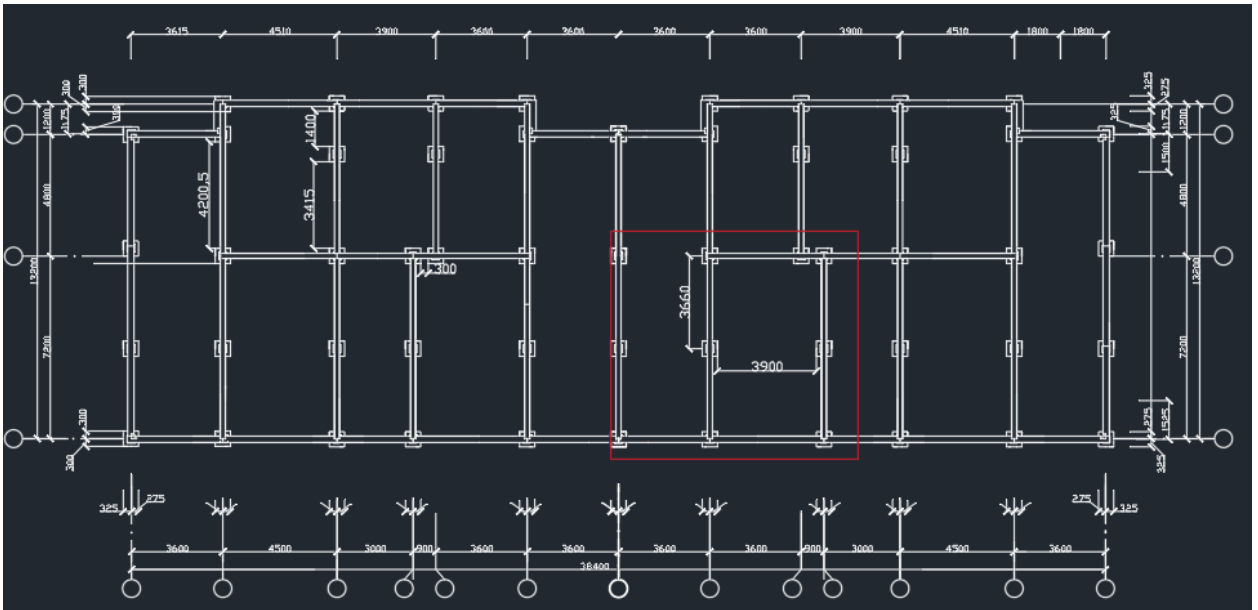


Figure 4.38. Slab design area with maximum span lengths.

Minimum Depth of the slab is taken as maximum length of slab divided by 30 according to the **Table 4.9**. Which gives the value of 5.9 inches, which is greater than 5 inches – gives satisfactory value for flat slab without drop panel according to ACI 318. By rounding thickness of the slab is equal to 6 inches.

Table 4.9. Thickness selection of the slab.

MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS						
Yield strength, f_y , psi*	Without drop panels [†]			With drop panels [†]		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams [‡]		Without edge beams	With edge beams [‡]	
40,000	$\frac{l_n}{33}$	$\frac{l_n}{36}$	$\frac{l_n}{36}$	$\frac{l_n}{36}$	$\frac{l_n}{40}$	$\frac{l_n}{40}$
60,000	$\frac{l_n}{30}$	$\frac{l_n}{33}$	$\frac{l_n}{33}$	$\frac{l_n}{33}$	$\frac{l_n}{36}$	$\frac{l_n}{36}$
75,000	$\frac{l_n}{28}$	$\frac{l_n}{31}$	$\frac{l_n}{31}$	$\frac{l_n}{31}$	$\frac{l_n}{34}$	$\frac{l_n}{34}$

Calculations of the moment.

- 1) Total static moment for short span:

$$M_0 = \frac{1}{8} * \frac{260}{1000} * 13.12 * 9.84^2 = 41.28 \text{ ft} - \text{kips}$$

Distribution:

$$\text{Negative Moment } -M: 41.28 * 0.65 = 26.83 \text{ ft} - \text{kips}$$

$$\text{Positive Moment } +M: 41.28 * 0.35 = 14.45 \text{ ft} - \text{kips}$$

Column Strip width:

$$2 * \frac{9.84}{4} = 4.92 \text{ ft}$$

$$\text{With } \frac{l_1}{l_2} = \frac{13.12}{9.84} = 1.33 \text{ and } \alpha=0 \text{ (no beams)}$$

Bending Moment for column strip:

$$-M: 26.83 * 0.75 = 20.12 \text{ ft} - \text{kips}$$

$$+M: 14.45 * 0.6 = 8.67 \text{ ft} - \text{kips}$$

2) For long span, static load:

$$M_0 = \frac{1}{8} * \frac{260}{1000} * 9.84 * 13.12 = 55.05 \text{ ft} - \text{kips}$$

Distribution:

$$-M: 41.28 * 0.65 = 26.83 \text{ ft} - \text{kips}$$

$$+M: 55.05 * 0.35 = 35.78 \text{ ft} - \text{kips}$$

Column Strip width:

$$2 * \frac{13.12}{4} = 6.56 \text{ ft}$$

$$\text{With } \frac{l_2}{l_1} = \frac{9.84}{13.12} = 0.75$$

Bending Moment for column strip:

$$-M: 35.78 * 0.75 = 26.83 \text{ ft} - \text{kips}$$

$$+M: 19.26 * 0.6 = 11.56 \text{ ft} - \text{kips}$$

Bending Moment for middle strip. Short span:

$$-M: 26.83 * 0.25 = 6.7 \text{ ft} - \text{kips}$$

$$+M: 14.45 * 0.4 = 5.78 \text{ ft} - \text{kips}$$

Bending Moment for middle strip. Long span:

$$-M: 35.78 * 0.25 = 8.95 \text{ ft} - \text{kips}$$

$$+M: 19.26 * 0.4 = 7.7 \text{ ft} - \text{kips}$$

Maximum moment (+ve and -ve)

$$\text{Short span: } 20.12 \text{ ft} - \text{kips}$$

$$\text{Long span: } 26.83 \text{ ft} - \text{kips}$$

ρ_{max} - maximum permitted reinforcement ratio

$$M_u = \phi \rho * f_y * b * d^2 * (1 - 0.59 * \rho \frac{f_y}{f'_c})$$

$$M_u = 0.9 * 0.0206 * 60000 * 9.84 * d^2 * (1 - 0.59 * 0.0206 * \frac{60000}{4000})$$

$$M_u = d^2 * 8950.45$$

$$d_1 = 1.49 \text{ inches}$$

$$d_2 = 1.73 \text{ inches}$$

Check for punching shear:

Assume footing as column with dimension 2600 mm = 4.26 ft

$$V_u = 350 * (13.12 * 9.84) - (4.26 + 0.5)^2 = 37255.12 \text{ lb}$$

$$1) V_{c1} = \left(2 + \frac{4}{\beta}\right) * \sqrt{f'_c * b_0 * d} = 296419.3 \text{ lb}$$

$$2) V_{c2} = \left(2 + \frac{\alpha_s * d}{b_0}\right) * \sqrt{f'_c * b_0 * d} = 218940.7 \text{ lb}$$

$$3) V_{c3} = 4 * \sqrt{f'_c * b_0 * d} = 346808 \text{ lb}$$

Min $V_c = V_{c2} > V_u$ Therefore it is safe.

Reinforcement:

Minimum steel area required according to Code is 0.0018 of gross area of concrete:

$$\text{In short direction : } \rho_{min} = \frac{0.0018 * 6 * 13.12}{6 * 9.84} = 0.0024$$

$$\text{In long direction: } \rho_{min} = \frac{0.0018 * 6 * 13.12}{6 * 13.12} = 0.0018$$

Calculation of steel area.

$$R = \frac{M_u}{\phi * b * d^2} = \rho * f_y * \left(1 - \frac{\rho * f_y}{f'_c * 1.7}\right)$$

Calculated ρ put in $A_s = \rho * b * d$

The whole calculations provided in Excel file,

For negative moment of short span, column strip : $A_s = 2.83 \text{ in}^2 / \text{ft}$

Select bar #8, at a spacing of 3.5 in, 6 pieces

For positive moment of short span, column strip : $A_s = 1.2 \text{ in}^2 / \text{ft}$

Select bar #6, at a spacing of 5 in, 4 pieces

For negative moment of short span, middle strip : $A_s = 0.92 \text{ in}^2/\text{ft}$

Select bar #5, at a spacing of 4 in, 4 pieces

For positive moment of short span, middle strip : $A_s = 0.53 \text{ in}^2/\text{ft}$

Select bar #3, at a spacing of 4 in, 4 pieces

For negative moment of long span, column strip : $A_s = 3.41 \text{ in}^2/\text{ft}$

Select bar #10, at a spacing of 4.5 in, 8 pieces

For positive moment of long span, column strip : $A_s = 1.59 \text{ in}^2/\text{ft}$

Select bar #6, at a spacing of 3 in, 4 pieces

For negative moment of long span, middle strip : $A_s = 1.14 \text{ in}^2/\text{ft}$

Select bar #5, at a spacing of 3.5 in, 6 pieces

For positive moment of long span, middle strip : $A_s = 1.04 \text{ in}^2/\text{ft}$

Select bar #5, at a spacing of 4.5 in, 4 pieces

Column Design

For Column design data also were obtained from SAP 2000. In order to find maximum load on the column all combinations were investigated. The final value of axial load and combinations of moments to the column is 2562 kN =576 kips investigated via Excel program, this load includes designed structural load and load of parking dead and live as well. The whole calculations were provided in the Excel file provided in the **appendix Table 20**. Here general information will be provided:

$$\text{Steel: } f_y = 60000 \text{ psi}$$

$$\text{Concrete: } f'_c = 4000 \text{ psi}$$

Reinforcement should be at the range of 1% to 8% according to the ACI 10.9.1.

Designed load: $P_u = 576 \text{ kips}$

$$\phi = 0.65$$

$$\alpha = 0.8$$

Firstly, let's check for slenderness:

$$k = 1$$

$$l_u = 3000\text{mm or } 118.11 \text{ inches}$$

$$r = 4.724$$

For long column:

$$\frac{k \cdot l_u}{r} > 34 - 12 \cdot \frac{M_1}{M_2}$$

$$25 < 28$$

Therefore, slenderness can be ignored.

By finding values of $K_n = 0.89$ and $M_n = 1.13 \cdot 10^{(-6)}$ for the Interaction diagram with $\gamma = 0.65$, show the percentage of reinforcement below 1%, which is not allowed by Code. Therefore, columns will be design by using formula below:

$$\phi P_u = \phi \cdot \alpha \cdot (0.85 \cdot f'_c \cdot (A_g - A_s) + f_y \cdot A_s)$$

From this area of column can be derived and therefore its dimensions by assuming area of steel to be 2%. One side of column derived as $a = 15.589 \text{ inches}$, but it is better to round up it to $a = 400\text{mm} = 15.748 \text{ inches}$, as higher standing column has dimensions of 400*400. The reason behind why existing column is more than calculated values of designed one, is because that column was made of concrete M300, which less compressive strength than currently using one, moreover steel in that column of A400C grading, which also provide less values of yielding strength.

By rearranging previous formula to $A_s = \frac{P_u - \phi \alpha (0.85 f'_c A_g)}{\phi \alpha (f_y - 0.85 f'_c)}$

Steel area can be derived: $A_s = 4.67 \text{ in}^2$, which is 8 pieces of #7 bars, bar selection were from **Table 4.10** and allocated according to the code (cover 1.5 inches, even number), column itself presented below.

Table 4 10 Selection of the reinforcement chart
Areas of groups of standard bars, in²

Bar No.		Number of Bars											
Inch-Pound	SI	1	2	3	4	5	6	7	8	9	10	11	12
4	13	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00	2.20	2.40
5	16	0.31	0.62	0.93	1.24	1.55	1.86	2.17	2.48	2.79	3.10	3.41	3.72
6	19	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52	3.96	4.40	4.84	5.28
7	22	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80	5.40	6.00	6.60	7.20
8	25	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32	7.11	7.90	8.69	9.48
9	29	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00
10	32	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16	11.43	12.70	13.97	15.24
11	36	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48	14.04	15.60	17.16	18.72
14	43	2.25	4.50	6.75	9.00	11.25	13.50	15.75	18.00	20.25	22.50	24.75	27.00
18	57	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00

Tie selected as #3, as according to the code for bar less than #8 tie bars #3 can be used with the spacing of 14 inches.

By checking the results, percentage of steel reinforcement obtained is equal to 1.935% and bar spacing is 4.68 inches, those values fit the range of the code of reinforcement 1%-8% and spacing less than 6 inches. The shear and lateral load are not taken into account in the structures with core walls and elevator shafts, in this case elevator shaft is presented.

Foundation design

In this case micropiles acts as foundation, but they will be partly removed namely cased part of micropiles, which locates in the parking zone will be removed. As the whole load will be transported to the new column. In order to provide load transfer from column to micropiles that left in the earth, new footing and its connection are designed. The calculations were done in the Excel file in appendix.

Pile Spacing=86.6 inches, Cap Edge Length = 139.5 inches.

General information:

$$\text{Steel: } f_y = 60000 \text{ psi}$$

$$\text{Concrete: } f'_c = 4000 \text{ psi}$$

$$L_{1-1} = 156.1 \text{ in}$$

$$L_{2-2} = 126.1 \text{ in}$$

$$M_{u \ 1-1} = 38 \text{ ft} - \text{kips} / \text{ft}$$

$$M_{u\ 2-2} = 67.8 \text{ ft} - \text{kips}/\text{ft}$$

$$\rho_{prov} = 0.042$$

Check the flexure capacity at column face

$$\rho = \frac{0.85f'_c \left(1 - \sqrt{1 - \frac{M_{u,\max}}{0.383bd^2f'_c}} \right)}{f_y} = 0.0032 < \rho_{prov}$$

$$\rho_{MAX} = \frac{0.85\beta_1f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_t} = 0.0206 < \rho_{prov}$$

$$\rho_{MIN} = \text{MIN} \left(0.0018 \frac{T}{d}, \frac{4}{3} \rho \right) = 0.0024 < \rho_{prov}$$

Check one way shear capacity at the face of column and pile according to ACI318, Section 22.5

$$L_{3-3} = 61.5 \text{ in}$$

$$V_{u\ 2-2} = 2 * \phi * P_n / L_{2-2} = 21.5 \frac{\text{kips}}{\text{ft}}$$

$$V_{u\ 3-3} = \phi * P_n / L_{3-3} = 22.1 \frac{\text{kips}}{\text{ft}}$$

$$\phi V_{c3} = \phi * \sqrt{f'_c} * b_0 * d = 22.8 \frac{\text{kips}}{\text{ft}} > V_u = 3.37 \text{ kips}$$

Where $\phi=0.75$

Check punching capacity of column:

$$\phi v_c(\text{psi}) = \phi(2 + y) \sqrt{f'_c} = 190 \text{ ksi}$$

$$v_u(\text{psi}) = \frac{P_{u,\text{col}}}{A_p} + \frac{0.5\gamma_v M_{u,\text{col}} b_1}{J} = 147 \text{ ksi, where}$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 0.4$$

$$J = \left(\frac{db_1^3}{6} \right) \left[1 + \left(\frac{d}{b_1} \right)^2 + 3 \left(\frac{b_2}{b_1} \right) \right] = 32 \text{ ft}^4$$

Therefore, $\phi v_c > v_u$

Check single pile punching capacity:

$$\phi v_c (\text{psi}) = \phi (2 + y) \sqrt{f'_c} = 95 \text{ ksi}$$

$$v_u (\text{psi}) = \frac{P_{u, \text{col}}}{A_P} + \frac{0.5 \gamma_v M_{u, \text{col}} b_1}{J} = 72 \text{ ksi, where}$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 0.4$$

$$J = \left(\frac{db_1^3}{6} \right) \left[1 + \left(\frac{d}{b_1} \right)^2 + 3 \left(\frac{b_2}{b_1} \right) \right] = 11 \text{ ft}^4$$

Therefore, $\phi v_c > v_u$

The detailed drawing of that connection is presented in the **Figure 4.39**.

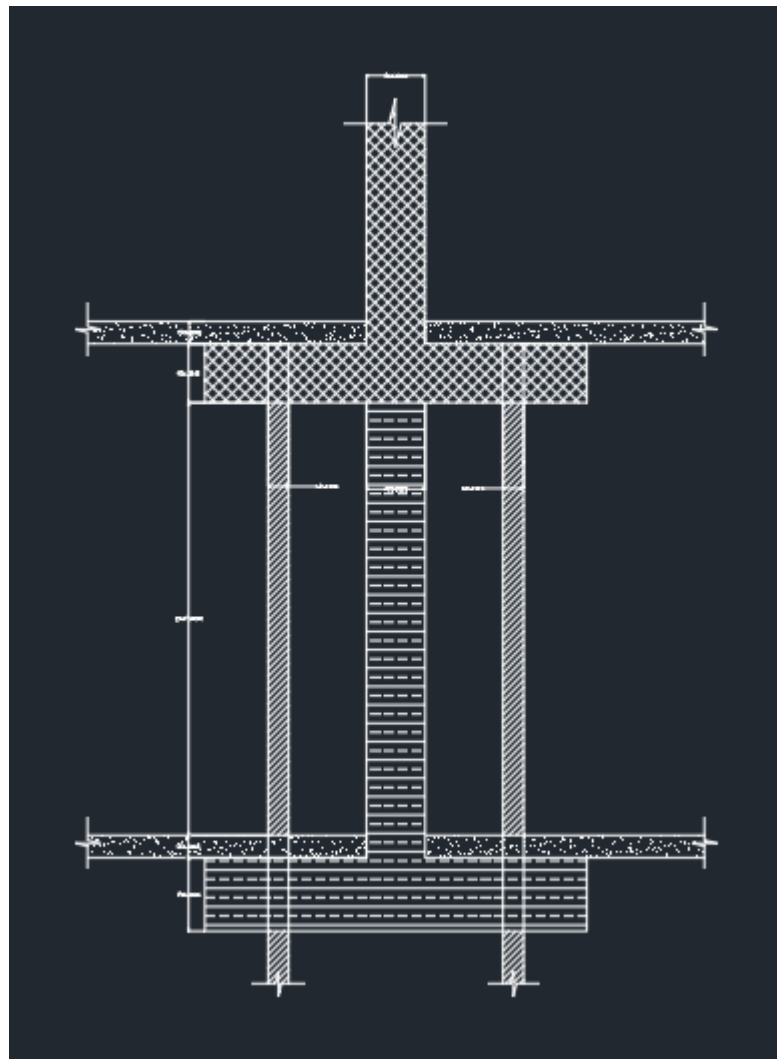


Figure 4.39. Representation of the connection of foundation.

4.2.2.3.8.2. Plaxis model

Three stages of construction were developed. All of them are presented in the **Figure 4.40** below.

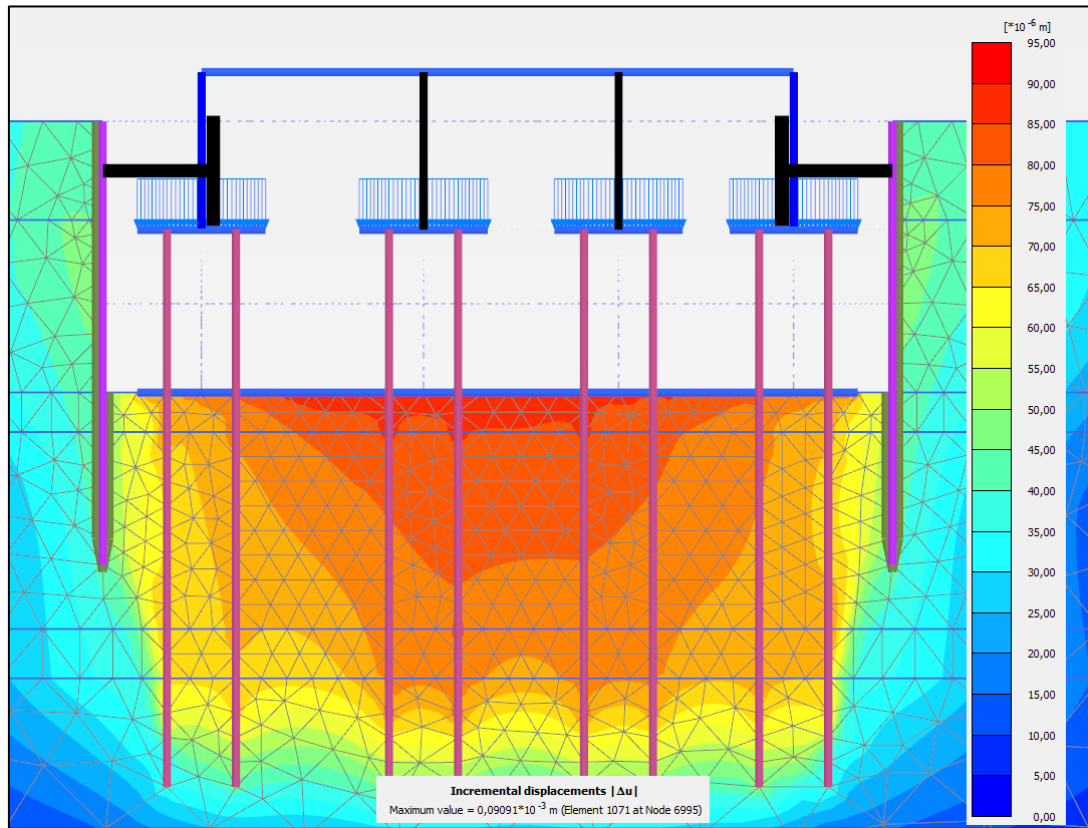


Figure 4.40. Installation of common foundation for building

For the stage of foundation installation soil layers consolidates under the weight of foundation. It can be noticed from displacement map that under the foundation weight soil goes through maximum displacement of 0.09 mm.

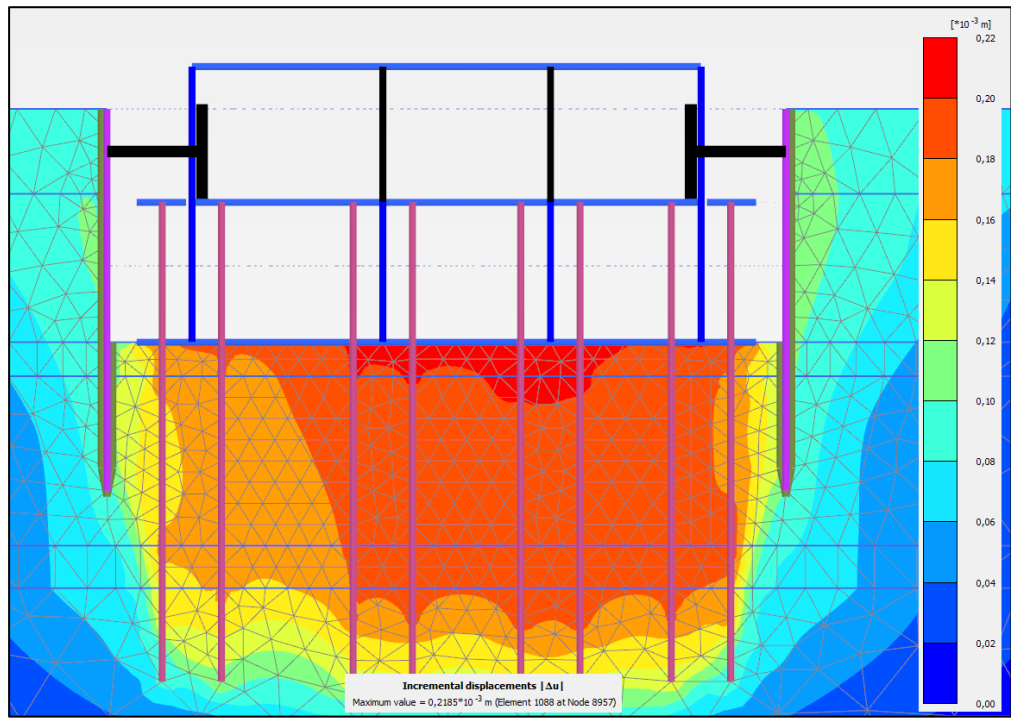


Figure 4.41. Installation columns, walls and slabs

Similar to previous step, installation of slabs, walls and columns increases load on soil and it goes through maximum deformation of 0.2185 mm.

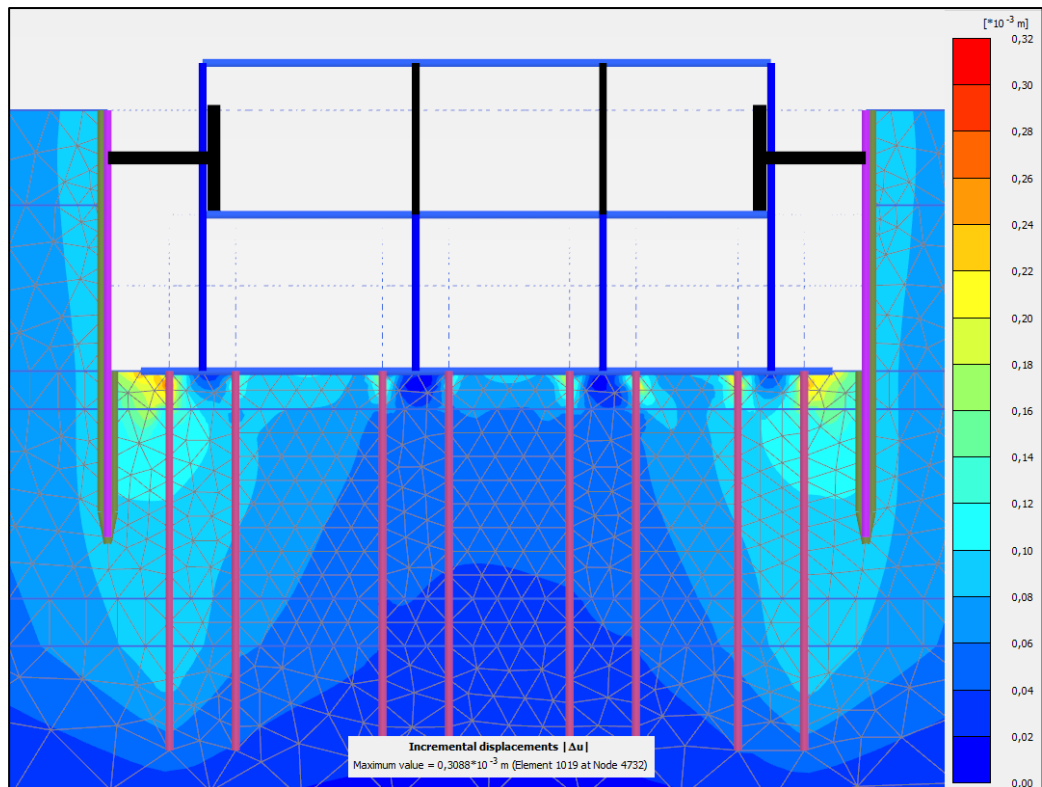


Figure 4.42. Removing unnecessary structural parts

For this stage as several structural elements were removed from building, structure is influenced by uplifting force from distressed soil. It can be seen from the map above that ending of foundation that are open is displaced to about 0.22-0.30 mm.

4.2.2.3.9. Backfilling (Stage 16-18)

Final step of the earth work is backfilling. Ground around the structure must be moved back and sheet piles must be taken away.

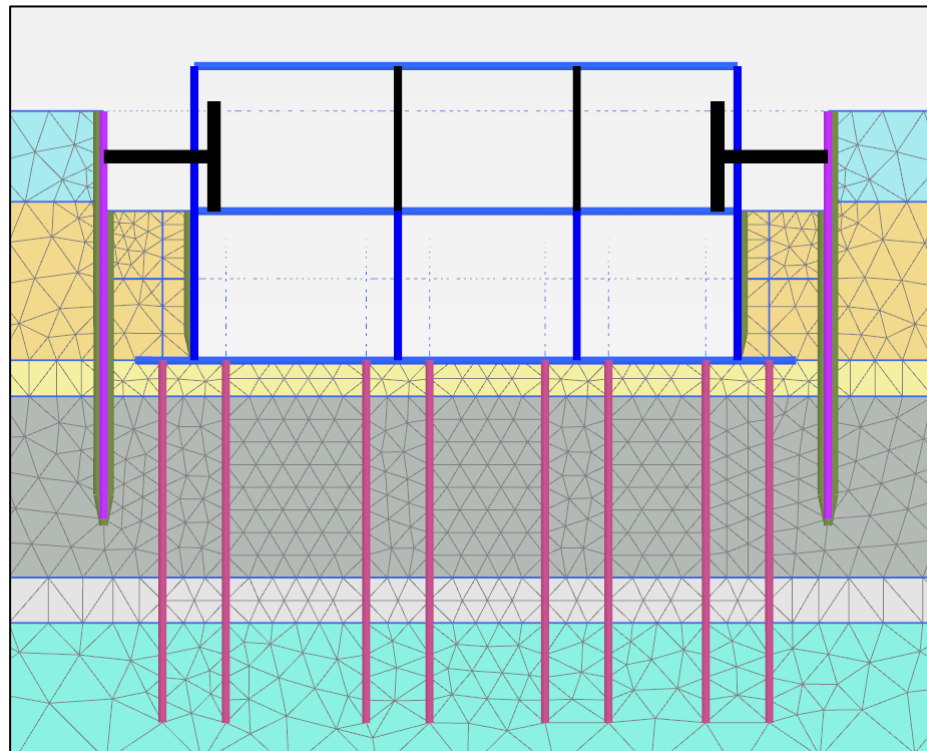


Figure 4.43. First layer of backfill (Scale x10)

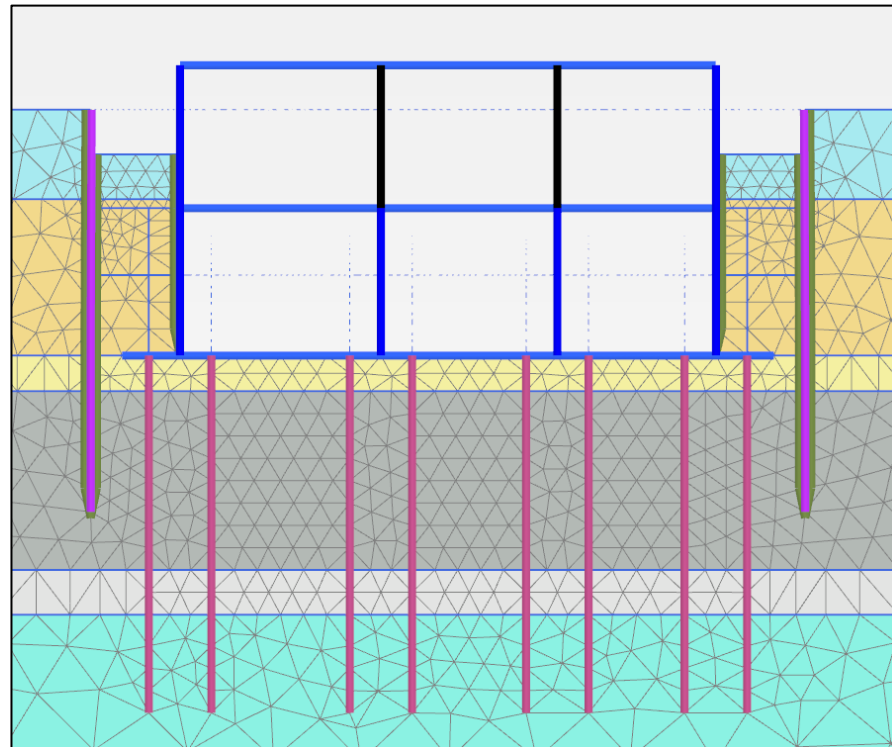


Figure 4.44. Second layer of backfill with strut removing process (Scale x10)

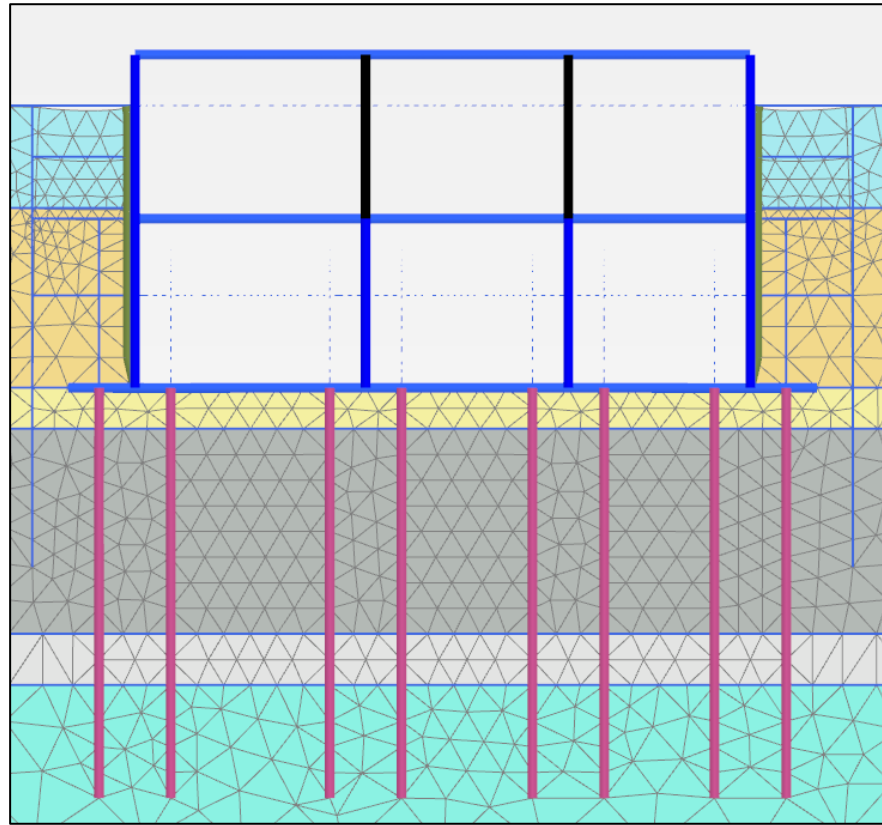


Figure 4.45. Final result of construction process (Scale x10)

The maximum total displacement over all the stages was equal to 15.9 mm. Point was located near point D. All the stages of settlement was then checked on SAP 2000 spring model which gave a chance to concluded that building can sustain such displacements. However, it must be clarified that deformation after initial construction , for the reason that initial condition was only for simulating long term settlement of existing building. Moreover, special analysis was performed for dewatering stage where whole building was under effect of settlement. In this case relative settlement was conducted to all the columns, however similar to the previous stages, there was no failure in structural elements.

Project Management



Chapter 5. Project management

Main consideration of project management in construction projects can be defined as accomplishment of precise concept by effective coordination and planning method, which includes proper management of time, resources, quality and cost. Thus, effective project management plays significant role in completion of project successfully and consists of following steps:

1. Defining the project
2. Scheduling
3. Health and Safety plan
4. Risk analysis
5. Waste management
6. Cost estimation

5.1. Defining scope

This part of the project includes gathering and analysis of information, setting clear goal, defining the scope of the project and related outcomes. It consists of the following stages:

1. Defining the project scope
2. Establishing project priorities
3. Creating WBS

5.1.1. Defining project scope

The project scope includes final result of the product and service that was provided. At that stage it is important to consider project purpose, deliverables and milestones.

The ultimate goal of the project is making considerable long-term profit by renovating the old building that was built in 1975, namely renting the apartments, which will be located in the constructed additional 2 floors. Moreover, as the location is densely populated and have little amount of parking slots near the residential complex, construction of underground 2 story parking will solve the problem and lead to beneficial long term profit for owner by renting them for residents.

Deliverables of the project are found to be architectural, structural, geotechnical design of the renovated building and cost analysis, project scheduling.

Milestones of the project contain major events that take place during the project lifecycle and can influence scheduling and completion of other activities. In this case of renovation project, preparation of site, finalizing foundation, completion of strengthening and technical inspection can be identified as milestones of the project.

Constrains of the project include location of the building which is densely populated and have dynamic movement of city residents because of nearby located train station. This limitation can significantly influence on the efficiency and duration of the excavation, filling and pile installation. Moreover, specific soil profile, namely high groundwater table, complicates geotechnical works.

Technical requirements of the project are used in order to ensure high quality of the works that will be performed in high safety level. It includes standards, codes, high engineering skills and estimations. List of the codes that are used for Renovation processes are:

- ASCE 7 -10 Minimum Design Loads and Associated Criteria for Buildings and Other Structures
- International Building Code (IBC)
- International Residential Code (IRC)

5.1.2. Establishing project priorities

In order to meet the goal of the project effectively important criteria should be identified. Considering all significant factors that can influence the project completion, 3 main criteria are developed: resource, time, quality (see **Table 5.1**). In renovation case, time is defined as the most significant meteward, because, as mentioned above, the ultimate goal is to provide residents with apartments as soon as possible and solve issue of parking. The second priority of the project is quality. For leading construction company it is important to save its outstanding reputation and to be able to compete with other companies in the construction market, emphasizing on safety of the workers and construction processes. Finally, the least substantial criteria is assigned to resource. Performing comprehensive cost estimation, labour and equipment analysis, it was found that any changes in cost, labour and equipment does not significantly effect on company’s budget.

Table 5.1. Project priority matrix

Criteria/priorities	Time	Quality	Resource
Least flexible			
Moderate flexible			
Most flexible			

5.1.3. Creating WBS

In order to consider total scope of the project without missing any deliverables, work breakdown structure of the project is developed. Dividing the workload into small elements, proper management and monitoring of the work progress can be performed. For renovation project WBS is shown in the **Figure 5.1**.

5.2. Scheduling

One of the factors that influence on the successful completion of the project is scheduling. It includes work durations and dependence, result is presented in **Figure 5.2** below.

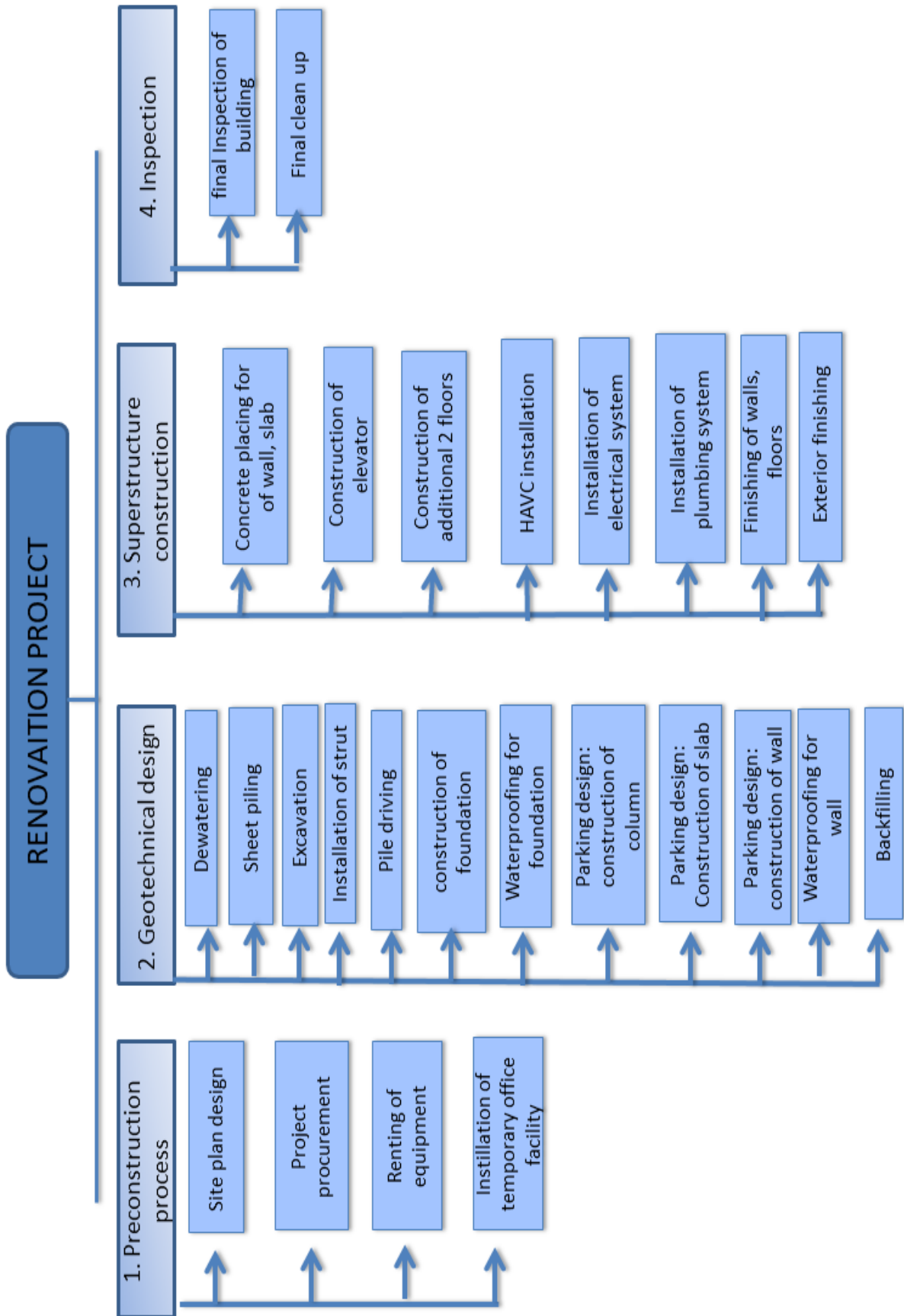


Figure 5.1. WBS of the project

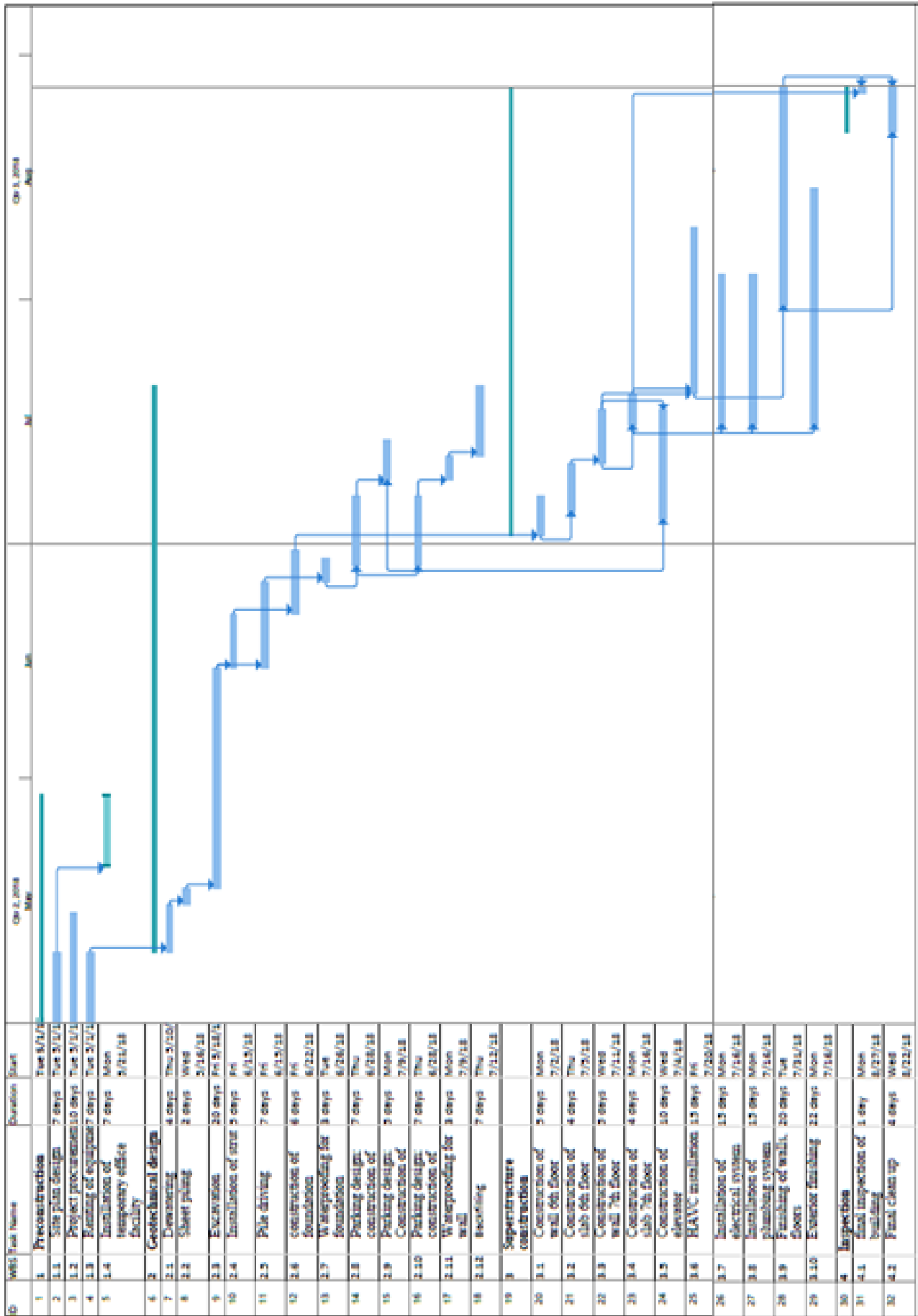


Figure 5.2. Project Scheduling

Table 5.2 below shows the detailed duration and dependance of each activity.

Table 5.2. Activity duration and dependence

	WBS	Task Name	Duration	Start	Finish	Predecessors	Predecessors
1	1	Preconstruction		Tue 5/1/18			
2	1.1	Site plan design	7 days	Tue 5/1/18	Wed 5/9/18		
3	1.2	Project procurement	10 days	Tue 5/1/18	Mon 5/14/18		
4	1.3	Renting of equipment	7 days	Tue 5/1/18	Wed 5/9/18	3SS	3SS
5	1.4	Instillation of temporary office facility	7 days	Mon 5/21/18	Tue 5/29/18	2	2
6	2	Geotechnical design					
7	2.1	Dewatering	4 days	Thu 5/10/18	Tue 5/15/18	4	4
8	2.2	Sheet piling	2 days	Wed 5/16/18	Thu 5/17/18	7	7
9	2.3	Excavation	20 days	Fri 5/18/18	Thu 6/14/18	8	8
10	2.4	Installation of strut	5 days	Fri 6/15/18	Thu 6/21/18	9	9
11	2.5	Pile driving	7 days	Fri 6/15/18	Mon 6/25/18	9	9
12	2.6	construction of foundation	6 days	Fri 6/22/18	Fri 6/29/18	10	10
13	2.7	Waterproofing	3 days	Tue 6/26/18	Thu 6/28/18	11	11
14	2.8	Parking design: construction of column	7 days	Thu 6/28/18	Fri 7/6/18	13SS+2 days	13SS+2 days
15	2.9	Parking design: Construction of slab	5 days	Mon 7/9/18	Fri 7/13/18	14,16SS+3 days	14,16SS+3 day
16	2.10	Parking design: construction of wall	7 days	Thu 6/28/18	Fri 7/6/18	14SS	14SS
17	2.11	Waterproofing for wall	3 days	Mon 7/9/18	Wed 7/11/18	16	16
18	2.12	Backfilling	7 days	Thu 7/12/18	Fri 7/20/18	17	17
19	3	Superstructure construction					
20	3.1	Construction of wall 6th floor	5 days	Mon 7/2/18	Fri 7/6/18	12	12
21	3.2	Construction of slab 6th floor	4 days	Thu 7/5/18	Tue 7/10/18	20SS+3 days	20SS+3 days
22	3.3	Construction of wall 7th floor	5 days	Wed 7/11/18	Tue 7/17/18	21	21
23	3.4	Construction of slab 7th floor	4 days	Mon 7/16/18	Thu 7/19/18	22SS+3 days	22SS+3 days
24	3.5	Construction of elevator	10 days	Wed 7/4/18	Tue 7/17/18	22FF,16SS	22FF,16SS
25	3.6	HAVC installation	15 days	Fri 7/20/18	Thu 8/9/18	22,23	22,23
26	3.7	Installation of electrical system	15 days	Mon 7/16/18	Fri 8/3/18	23SS	23SS
27	3.8	Installation of plumbing system	15 days	Mon 7/16/18	Fri 8/3/18	23SS	23SS
28	3.9	Finishing of walls, floors	20 days	Tue 7/31/18	Mon 8/27/18	25SS+7 days	25SS+7 days
29	3.10	Exterior finishing	22 days	Mon 7/16/18	Tue 8/14/18	23SS	23SS
30	4	Inspection					
31	4.1	final inspection	1 day	Mon 8/27/18	Mon 8/27/18	23FS+7 days,28FF	23FS+7 days,28

5.3 Health and Safety plan (HSP)

Construction is considered as a main type of industry that involves high level of hazards and accidents. Human resource is identified as a main value of the every sphere of industry, so construction area is not an exception. Many of the accidents that can lead to death of the workers are usually happen in the construction areas with poor health and safety management plan. Moreover, all construction related materials, tools, equipment used in construction activities include their own danger. Thus, importance and necessity of the health and safety plan is crucial (OSHA. *Safety considerations*. 2018). Implementation of the Health and Safety plan can significantly improve the quality of the construction, preventing the work related injuries and accidents. Furthermore, properly deliberated health and safety management can prevent increase of the cost of the project due to unexpected failures. And also, it directly can influence on the company's image and reputation, making company more competitive in industry market. In this part of the paper several very important parts of the Health and Safety Plan and specific considerations in Renovation project are presented.

Housekeeping

Keeping construction site clean and organized can significantly prevent construction related hazards. Housekeeping mainly consider provision of the safe access to the job site, immediate cleanup of soiled materials, placement of the debris and trash in proper containers, effective storage of the material. The appropriate area distribution for temporary building, storage and job site is indicated before in the architectural design part pf the project. Demolished building place will be used later for equipment and material storage. **Figure 5.3** below represents site organization plan and includes open, close material storage areas.

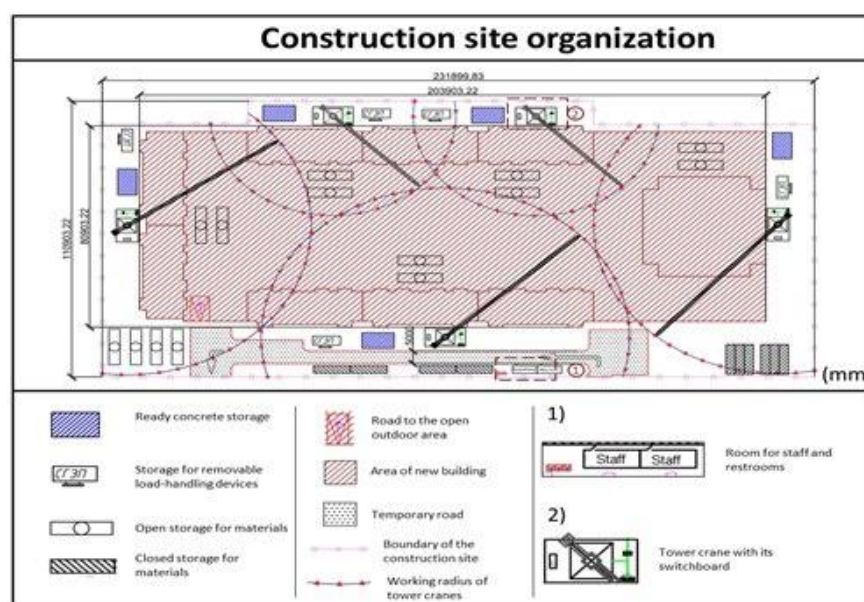


Figure 5.3. Construction site organization plan

Fire protection

Fire exposure is defined as one of the most popular and harmful accidents in all construction processes (OSHA 2018). Thus, fire protection is one of the must consider part in health and safety plan. It includes transportation and storage of the flammable, combustible liquids that can magnify the fire exposure. Protection of the ignition source, prevention of sparks caused by vehicle operations should be controlled carefully. According to OSHA, fire extinguisher must be provided within 50ft when more than 5gallons of the flammable liquid is located in the job site. Evacuation plan and fire mitigation should be introduced for every worker and practiced periodically.

Driving safety and traffic management

Vehicle operation is integral part of construction. As in a given case, in limited area of the construction, driving safety is one of the essential factors. Key responsibility lies on highly qualified and certified vehicle operators. Moreover, periodical maintenance and technical inspection of the vehicles play significant role on accident prevention. Strong recommendation in obeying traffic laws, such as speed limits, prohibition of the usage cell phone and personal safety rules could significantly contribute on the safety of the whole process in the construction site.

In addition to that, the selected building is located in the downtown, near the train station. Two main roads that surround the building from both sides make some difficulties during the transportation and delivering of material to the job site. Public safety measures consecution and monitoring the work schedule are important to minimize the delay in construction.

Industrial Hygiene and Occupational Health

In order to ensure safe environment for the workers, special construction guidelines, standards are developed. Those standards designed to manage safety issues, health measurements, risk assessment. Moreover, it includes instructions on the personal protective equipment and occupational health trainings (OSHA. *Division Consultation Program*. 2016).

As the renovation building includes construction of additional 2 floors, falling from the top is considered as main safety issue of the project. Moreover, given activity has high probability of occurrence and involves usage several equipment such as ladder, aerial lifts, cranes, scaffolds. Fall protection methods like guardrail systems, personal fall arrest systems, warning line systems, safety monitoring systems should be introduced in detail and used properly.

In addition to that, involvement of workers in reporting safety and health concerns add enormous effort in the controlling and monitoring the project processes and improve the relationship between employer and employee as well as improving the quality and of the work (Safety and Health program, OSHA 2016).

5.4. Risk management

Identifying and mitigation of influential risks is one the key factors that lead to competition of the project successfully. Otherwise, it results dangerous and costly consequences, affecting on the duration, cost and quality of the project. Thus, it is important to consider risk management which consists from the following sub parts: risk identification, risk assessment, response development. In order to cover all risks that could occur during the construction process each activity is investigated separately and detailed **Table 5.2** is developed.

Table 5.2. Risk assessment of the activities

Activity		risk	impact out of 5	probability out of 5	risk value out of 25	mitigation plan
	1.1. Project procurement	1.Unavailability of the materials 2. Later delivery of materials	4	3	12	1. control of deliverables 2. proper procurement management
	1.2 Site plan design	Unavailability of the technical data	4	3	12	Proper site investigation is needed.
1. PRECONSTRUCTION WORKS	1.3 Equipment renting	1.Break down of equipment 2.Equipment accidents	4	3	12	1.Regular technical checkup, pre rental inspection 2.Trained operators should have approved certificates. Usage of warnner labels appropriately. Developing of Rental agreement with dealerships
	1.4 Installation of temporary office facility	Physical injuries or workers	3	4	12	Health and safety regulations should be followed carefully
2. GEOTECHNICAL DESIGN	2.1 Dewatering	1. Possibility of polluting the surface of ground if waster has dangerous contaminants 2.Failure of pumps during the dewatering 3.Pipeline rupture 4. Can cause extra settlement	5	2	10	1.Improve drainage system, installation of groundwater cut of walls 2.Proper inspection of pumps 3.Monitoring procedures for early identification 4.Detailed soil profile and expedient dewatering design is needed.
	2.2 Installation of sheet pile	1. Contact with contaminated risings 2.Noise, vibration 3. Hazardous associated with	3	5	15	3. Crane operator should b trained well enough and workers should follow safety rules in the workplace where piles are being installed. 2. Monitoring the value

	falling of sheet piles during the lifting, slewing 4.Refuse or bend of sheet piles because of obstructions and stiff ground conditions 5.Influence on adjacent buildings				of noise and vibration and compare it with threshold values, investigation for reasons such as operational and obstructions. 3.Perposnal protective equipment should used. 4.Pre-drilling should be carries out, right methodology should be chosen. 5.Soil profile should be investigated and analyzed carefully
2.3 Excavation	1.Improper and time consumed excavation due to the limited area 2.Falling of workers, vehicles and materials to excavated area 3.Effects on the nearby structure	5	4	20	1.Selection of the effective equipment 2.Ensure the workers competent and experienced 3.Detailed study of soil profile and analysis of near located building.
2.4 Pile drilling	1.Drilling equipment can stuck the hole	5	2	10	1.Nozzle of the drilling equipment should be checked
2.6 Construction of foundation	1.Too much settlement 2.difficults with material delivery for underground	4	3	12	1.Detailed soil profile is needed 2.appopirate logistic management, provision of proper traffic routes for equipment movement
2.7 Waterproofing for foundation	1.Poor waterproofing 2.Damaging of waterproofing layer	4	3	12	1.Need high quality of installation and control of works 2.Protecting layer for waterproofing
2.8 Parking design: construction of column	1. Difficulty with concrete placement 2. Difficulty with concrete vibration	4	4	16	1. Concrete pumping equipment should be used 2. Self-consolidated concrete should be used.
2.9 Parking design: construction of slab	1. Difficulty with concrete placement 2. Difficulty with concrete vibration	4	3	12	1.Concrete pumping equipment should be used 2. Self-consolidated concrete should be used.
2.10 Parking design: construction of wall	1.Difficulty with concrete placement 2.Difficulty with concrete vibration	5	3	15	1.Concrete pumping equipment should be used 2. Self-consolidated concrete should be used.
2.11 Waterproofing for	1.Poor	4	3	12	1.Need high quality of

	wall	waterproofing 2.Damaging of waterproofing layer				installation is needed and control of works are required 2.Protecting layer for waterproofing
	2.11 Backfilling	Settlement of soil	3	5	15	Proper compaction of soil
3.SUPERSTRUCTURE CONSTRUCTION	3.1 Installation of framework	1.Falling of material 2.Falling down of workers from the height	5	3	15	1.Personal protective equipment should be used, safety rules during the construction process should be introduced in detail.
	3.2 Concrete placing for slab, wall	Sedimentation of mix design, poor curing process	4	3	12	Proper transportation method 2. Controlling curing process
	3.3 Construction of elevator	1. Loss of control machinery as a result of poor communication, which can lead to serious injuries. 2. Falling down and being cut by sharp edge of the elevator.	4	3	12	1.Regular competence and training of the workers. 1. Careful control and monitoring of communication system of machinery and elevator 2.Personal protective equipment, following safety regulations
	3.4 HAVC installation	1.Electrocution risk 2. Falling from great height 3.Asbestos exposure	5	3	15	1.Turning of power it if is not necessary, workers must be methodical in their safety practices; 2.Secure ladders and scaffolding should used; safety precautions should followed attentively 3.Using of respirators are strongly recommended
	3.5 Installation of electrical system	Cold related injuries, construction dust, electrocution, noise, stooped posture	3	4	12	Control of the temperature is needed and protective clothing equipment should be used; allowable limits and specific of dust should be identified and respirator should be used; double insulated tools are recommended to use; ear protections, such as ear plugs, ear muffs and noise attenuation helmets should be provided
	3.6 Installation of plumbing system	1.Hazardous from lifting and carrying 2. Overhead works that cause health problems	4	3	12	1.Maximum allowable height and acceptable weight for lifting should be identified 2. Physical exercise are suggested.

		for the workers				
	3.7 Finishing of walls, floors	1. Possibility of the breaking finishing equipment	2	2	4	1. Permanent control and on time maintenance of equipment.
	3.8 Exterior designing	Asbestos exposure	4	3	12	Enclose, and encapsulation, PPE and administrative control measurement should be used
4. INSPECTION	4.1 Final inspection of building	1. Redoing poorly performed construction activities which involve the above mentioned risks 2. Increase of the project cost and prolongation of the project duration 3. Delay in deliverables	4	3	12	Personal protective equipment should be used properly; safety regulations should be strictly followed. Control and monitor of the construction processes in time are crucial important.
	4.2 Final check up					

From the risk **Table 5.2** it can be seen that the most important risk with the highest risk values 20 was defined as excavation. Accordingly, mitigation plan such as personal safety of the workers and experience, detailed soil analysis, selection of the effective equipment was developed in order to prevent the risk. Moreover, other geotechnical works such as construction of the parking column, wall and backfilling were identified as the next level risk, having 15 and 16 risk value. In addition to that, superstructure contraction activates, having risk value in range of 12 to 15 were selected as an important risk too. Whereas finishing was recognized as risk which has the lowest value of 4.

5.5. Waste management

As an every construction project, given renovation project is carried out by excessive amount of waste. Waste is classified into several types: reusable, recycling and hazardous waste. Reusable waste from the existing building contains nonstructural elements such as door, window frame, ceiling etc. Recycling waste of the building include collecting, restoring and recovering of the certain materials that could be recycled in the future. Rubber, waste wood, plastic waste of the building are nowadays widely familiar as recycling potential elements in construction waste. By reducing disposal cost and carbon emission, recycling of construction waste significantly can reduce pollution of environment. In case of the given renovation project, in waste management subcontractors are playing important role. Waste that was is result of the demolition process and non-recycling, non-reused materials will be moved away by the two side agreement. Due to the

small construction area, waste could not be stored in the site for large amount, thus, they should be disposed periodically (Work Safe BC.2016).

In a given renovation process hazard waste management is considered as an important aspect, and it is necessary to conduct analysis for friable and non-friable containing materials. Personal protective equipment and safety consideration of the workers are crucially important. One of the widely faced hazardous material is asbestos. According to Environmental Protection Agency, it is required that construction workers should have passed Asbestos Awareness Training by WAC 296-62- 07722(6). Moreover, disposal of the toxic waste such as lead, glue, tar should be done carefully. Because of the high risk and significant harm, most of the toxic waste is not accepted by city dump, that’s why involvement of subcontractors is considerably important.

5.6. Cost estimation

Cost estimation is one of the essential parts of the project management. It is defined as important criteria in project prioritizing process. Widely used two types of estimation, namely top to down and bottom up methods are considered to estimate the project’s cost and duration. In **Table 5.3** below, advantages and disadvantages of two methods are shown.

Table 5.3. Comparison of cost estimation methods

Top down estimates	Bottom up estimates
<i>accuracy</i>	
minus 20% to plus 60%	minus 10% to plus 30%
<i>level of details</i>	
no need to consider all mechanism, more straightforward	avoid double checking, provide explicit guidance for each mechanism
<i>Scope of the project</i>	
unstable scope, more related to internal small projects	determined scope, usually practiced in large scale projects

From the **Table 5.3** it can be concluded that the best method of the estimation is found to be bottom up estimation. According to project’s priorities such as more accurate time prediction, qualitative construction that need detailed mechanism and considering the project’s scope top down estimation is less appropriate than bottom up estimation. Detailed cost estimation of the project is shown in table below (stroitelnyi.kz, rascenki. astana).

Table 5.4. Cost estimation of the project

ACTIVITY	MATERIAL			LABOUR COST	EQUIPMENT COST	SUB CONTRACTER COST	CUMULATIVE
	AMOUNT	UNIT COST	TOTAL COST				
PRECONSRUCTION WORKS:							
Building of temporary offices						87, 500	87, 500
GEOTECHNICAL WORKS:							
Drilling of borehole(m)	150 (m)	93.75	14, 062.5	Foreman \$11/hr Operator \$3.6/hr Pile driver \$8/hr Carpenter \$5/hr Crew hourly rate \$36.2 production rate 22m/h $\frac{150}{22} * 36.2 = \6.81	Drilling ring \$60/hr Equipment production rate 20m/hr $\frac{150}{20} * 60 = \$450$		14, 519
Deep well pumps	13 pieces	45	585	Labour cost: \$6, 700	\$27, 000		34, 285
Installation and operation of pipe	200 (m)	550	110,000	Labour: \$31.25/m Total:\$6, 250	Equipment cost: \$306/m Total: \$61, 250		177, 500
Excavation	46474 (m ³)	5	232, 370	Labor : \$46.87/m3 Total cost: \$217, 405	Equipment cost: \$53.2/m Total: \$2 472, 416		2 922,191
Pile driving	1696(m)	120	203, 520	Foreman \$11/hr Operator \$3.6/hr	Compressor \$6/h Pile hammer @19/hr		211, 155

				Pile driver \$8/hr Carpenter \$5/hr Crew hourly rate \$36.2 production rate 22m/h $\frac{1696}{22} * 36.2 = \$2,790$	equipment hourly rte = \$60/r $\frac{1696}{21} * 60 = \$4,845$		
Construction of column (m3)	916 (m3)	58	53,128	Labour \$62.5/m3 total: \$57,250	Truck mounted concrete pump \$37/hr Production rate 72m3/hr concrete truck mixer \$75/hr Production rate 3m3/hr Total cost: 2 646,324		2 577,402
Construction of foundation	245 (m ³)	58	14,210	Labour \$62.5/m3 total: \$15,312	Truck mounted concrete pump \$37/hr Production rate 72m3/hr concrete truck mixer \$75/hr Production rate 3m3/hr Total cost: 707,805		7 372,372
Construction of slab	811 (m ³)	58	47,038	Labour \$62.5/m3 total: \$50,687	Truck mounted concrete pump \$37/hr Production rate 72m3/hr concrete truck mixer \$75/hr Production rate 3m3/hr Total cost: 2 342,979		2 440,704
Construction of wall	462 (m ³)	48	22,176	Labour \$62.5/m3 total: \$28,875	Truck mounted concrete pump \$37/hr		1 281,765

					Production rate 72m3/hr concrete truck mixer \$75/hr Production rate 3m3/hr Total cost: 1 230, 714		
Waterproofing	5595 (m ³)	0,78	4364.1	Labour \$45.8/m3 total: \$253, 548	Total cost: 37, 800		2 595,712
Backfilling	5536 (m ³)	25	138, 400	Labor : \$46.87/m3 Total cost: \$259, 472	Equipment cost: \$53.2/m Total: \$294, 515		692,657
SUPERSTRUCTURE WORKS:							
Construction of slab	245 (m ³)	58	14, 210	Labour \$62.5/m3 total: \$15, 312	Truck mounted concrete pump \$37/hr Production rate 72m3/hr concrete truck mixer \$75/hr Production rate 3m3/hr Total cost: 707, 805		737,327
Construction of wall	821(m ³)	48	39, 408	Labour \$62.5/m3 total: \$51, 312	Truck mounted concrete pump \$37/hr Production rate 72m3/hr concrete truck mixer \$75/hr Production rate 3m3/hr Total cost: 237, 186		327, 906
Construction of elevator	8pieces	100000	800, 000	3000	1250		804,250
HAVC system	40 (m ³)	4	160			4166	

Electrical system installation	3254 (m ³)	9,37	30, 489			5 278	30, 489.98
Pumping system installation	37(m ³)	12165	450, 105			1244	450,105
Finishing wall, floors	37,5 (m ³)	12165	456, 187				456,187
INSPECTION WORKS:			0				
Final inspection of the building						150, 000	150,000
						SUBTOTAL:	23 363 936.81
						TAX: 10%	2 336 393.6
						CPI: 6,4%	1 495 291.92
						Engineering permits:1%	23 363.93
						Final clean up:0.5%	116 819.68
						TOTAL:	\$25 990 043.94

Constructed 2 story underground parking with 656 lots will be able to use by 91% of the residents. Moreover, selling each parking lot to \$6250, owner will get profit in \$4 100 000. Also, selling 1 m² of apartment to 1429, owner can get profit of \$12 423 726, which in total is \$16 523 726. Thus, it can be seen that, project revenue is approximately 63.5% of the project cost. But however, as mentioned before the main aim of the project is to provide city residents with residential apartments and solve the parking issue as soon as possible, so the cost is considered as the least important aspect of the project.

Chapter 6. Conclusion

Renovation of the old building is a good alternative to extend the service life of the structure. Nowadays, there are a lot of buildings whose structural, architectural, geotechnical and fire safety designs become out of date. Therefore, the suggested solution in the form of renovation may find a widespread application, especially in the downtown areas, where the long building process may be complicated by dense population and closely located buildings. Renovation process allows not only service life extension, but significant improvement of living conditions as well.

The main purpose of this capstone project is to renovate the old Soviet panel building in the downtown of Astana through the modernization of its structural, geotechnical and architectural designs. Renovation includes top extension in the form of additional two floors, bottom extension in the form of two-story parking and new fire safety design.

To implement this, structural analysis of the structure was performed with the purpose of determining the structural capacity of the existing structure under the gravity and lateral load combinations. From the point of geotechnical analysis, for parking construction under already existing building, foundation replacement procedure was carried out. Finally, concerning architectural design, the structure is modified so that it is capable of meeting the modern code requirements. It includes noticeable modifications in the appearance of the whole structure and addition of elevators.

The structural analysis of the structure was performed through the following procedures: load calculations, modeling of the structure in SAP2000 software for detailed analysis, drift check, validation of Sap 2000 results by means of hand calculations for the selected walls, structural analysis of walls, slabs and joints and serviceability check. The results obtained during the structural analysis showed that the walls are strong enough to sustain the gravity loads from the additional loads, as a result they don't require strengthening. The same situation is with slabs, from P-M Interaction Curve, it is seen that they are strong and strengthening is unnecessary. Connections between structural elements are mortar joints; therefore they were designed as pin connections. The analysis revealed that despite the fact that they are pin connections, they are also strong and do not need strengthening as well. Geotechnical analysis involves parking design, development of foundation replacement procedure and consequently designs of micro piles, footings and load bearing columns for parking, soil behavior analysis during the excavation and underground parking construction. For soil behavior analysis 2D Plaxis Model was developed. The results show that at the different settlement values the structural elements

did not fail. In terms of architectural design, the Soviet Code and the American code were studied deeply. The conclusion made is that the building is consistent with the American code and it can be applied for the further analysis of the building. The future appearance of the structure was modeled by means of SketchUp Software. The elevator's location plan was also developed. In the project management part, the scope of the project is identified, risk assessment and mitigation are carried out; health and safety plan as well as waste management plan is established. In addition to these, cost and time estimations, as well as risk management are thoroughly considered. Due to complexity of the project detailed work breakdown structure with scheduling is prepared.

Appendix

Table 1. SAP2000 results for live load

TABLE: Element Joint Forces - Areas							
Area	Area Elem	Joint	Output Case	Case Type	F1	F2	F3
Text	Text	Text	Text	Text	KN	KN	KN
wall1-1	wall1-1	J22	LIVE	LinStatic	0,048	39,65	164,1
wall1-1	wall1-1	J23	LIVE	LinStatic	0,117	4,716	150,3
wall1-1	wall1-1	J20	LIVE	LinStatic	-0,12	-23,986	-162,563
wall1-1	wall1-1	J19	LIVE	LinStatic	-0,045	-20,38	-150,889
wall1-2	wall1-2	J19	LIVE	LinStatic	-0,029	37,01	142,335
wall1-2	wall1-2	J20	LIVE	LinStatic	-0,1	11,722	119,1
wall1-2	wall1-2	J17	LIVE	LinStatic	0,097	-21,228	-137,1
wall1-2	wall1-2	J16	LIVE	LinStatic	0,033	-27,504	-123,493
wall1-3	wall1-3	J16	LIVE	LinStatic	-0,077	41,624	119,026
wall1-3	wall1-3	J17	LIVE	LinStatic	-0,065	24,726	90,345
wall1-3	wall1-3	J14	LIVE	LinStatic	0,05	-31,299	-115,118
wall1-3	wall1-3	J13	LIVE	LinStatic	0,092	-35,051	-94,013
wall1-4	wall1-4	J13	LIVE	LinStatic	0,147	49,974	96,443
wall1-4	wall1-4	J14	LIVE	LinStatic	0,223	24,502	61,05
wall1-4	wall1-4	J11	LIVE	LinStatic	-0,243	-31,096	-91,445
wall1-4	wall1-4	J10	LIVE	LinStatic	-0,128	-43,379	-63,258
wall1-5	wall1-5	J10	LIVE	LinStatic	0,011	45,951	68,432
wall1-5	wall1-5	J11	LIVE	LinStatic	0,00714	35,765	35,14
wall1-5	wall1-5	J8	LIVE	LinStatic	-0,011	-39,46	-63,457
wall1-5	wall1-5	J7	LIVE	LinStatic	-0,007323	-42,256	-33,567
wall1-6	wall1-6	J7	LIVE	LinStatic	-0,07	47,822	39,452
wall1-6	wall1-6	J8	LIVE	LinStatic	-0,194	26,489	14,175
wall1-6	wall1-6	J5	LIVE	LinStatic	0,178	-28,934	-42,776
wall1-6	wall1-6	J4	LIVE	LinStatic	0,086	-45,377	-11,821
wall2-1	wall2-1	J23	LIVE	LinStatic	-0,012	10,85	101,993
wall2-1	wall2-1	J24	LIVE	LinStatic	-0,146	-10,347	85,2
wall2-1	wall2-1	J21	LIVE	LinStatic	0,156	-8,326	-91,444
wall2-1	wall2-1	J20	LIVE	LinStatic	0,002121	7,823	-97,12
wall2-2	wall2-2	J20	LIVE	LinStatic	-0,23	9,265	87,29
wall2-2	wall2-2	J21	LIVE	LinStatic	-0,385	-0,702	69,25
wall2-2	wall2-2	J18	LIVE	LinStatic	0,367	-8,772	-80,664
wall2-2	wall2-2	J17	LIVE	LinStatic	0,248	0,208	-72,35
wall2-3	wall2-3	J17	LIVE	LinStatic	-0,151	13,869	71,968
wall2-3	wall2-3	J18	LIVE	LinStatic	-0,306	1,916	52,985
wall2-3	wall2-3	J15	LIVE	LinStatic	0,316	-11,129	-67,887
wall2-3	wall2-3	J14	LIVE	LinStatic	0,141	-4,656	-58,175
wall2-4	wall2-4	J14	LIVE	LinStatic	-0,042	13,87	59,97
wall2-4	wall2-4	J15	LIVE	LinStatic	-0,29	9,483	33,978
wall2-4	wall2-4	J12	LIVE	LinStatic	0,304	-14,8	-58,437

wall2-4	wall2-4	J11	LIVE	LinStatic	0,028	-8,552	-36,338
wall2-5	wall2-5	J11	LIVE	LinStatic	-0,118	16,604	41,3
wall2-5	wall2-5	J12	LIVE	LinStatic	-0,311	7,856	21,442
wall2-5	wall2-5	J9	LIVE	LinStatic	0,288	-12,106	-40,805
wall2-5	wall2-5	J8	LIVE	LinStatic	0,141	-12,354	-22,01
wall2-6	wall2-6	J8	LIVE	LinStatic	-0,307	14,775	23,02
wall2-6	wall2-6	J9	LIVE	LinStatic	-0,395	11,141	7,892
wall2-6	wall2-6	J6	LIVE	LinStatic	0,427	-15,303	-23,652
wall2-6	wall2-6	J5	LIVE	LinStatic	0,275	-10,613	-8,783

Table 2. SAP2000 results for wind load (values of U2 are taken for wind in y-direction)

TABLE: Joint Displacements			
Joint	OutputCase	CaseType	U2
Text	Text	Text	m
j1	WIND Y DIRECTION	LinStatic	0,001967
J2	WIND Y DIRECTION	LinStatic	0,001967
J3	WIND Y DIRECTION	LinStatic	0,001967
J4	WIND Y DIRECTION	LinStatic	0,001791
J5	WIND Y DIRECTION	LinStatic	0,001791
J6	WIND Y DIRECTION	LinStatic	0,001791
J7	WIND Y DIRECTION	LinStatic	0,001654
J8	WIND Y DIRECTION	LinStatic	0,001654
J9	WIND Y DIRECTION	LinStatic	0,001654
J10	WIND Y DIRECTION	LinStatic	0,001423
J11	WIND Y DIRECTION	LinStatic	0,001423
J12	WIND Y DIRECTION	LinStatic	0,001423
J13	WIND Y DIRECTION	LinStatic	0,001275
J14	WIND Y DIRECTION	LinStatic	0,001275
J15	WIND Y DIRECTION	LinStatic	0,001275
J16	WIND Y DIRECTION	LinStatic	0,000798
J17	WIND Y DIRECTION	LinStatic	0,000798
J18	WIND Y DIRECTION	LinStatic	0,000798
J19	WIND Y DIRECTION	LinStatic	0,000347
J20	WIND Y DIRECTION	LinStatic	0,000347
J21	WIND Y DIRECTION	LinStatic	0,000347
J22	WIND Y DIRECTION	LinStatic	0
J23	WIND Y DIRECTION	LinStatic	0
J24	WIND Y DIRECTION	LinStatic	0

Table 3. SAP2000 internal forces (M, N, V) under all load combinations for all shear walls

Wall location	Floor #	Wall name in SAP200	N (F22) (kN/m)	V (F11) (kN/m)	M11 (kNm/m)	M22 (kNm/m)	M MAX (kNm/m)
A 3-4	1	271	-103,0	-588,2	0,0336	0,2184	0,4806
	2	272	-46,5	-482,1	-0,0256	-0,0235	0,6114
	3	273	-34,7	-381,5	-0,0174	0,0048	0,5778
	4	274	-27,3	-284,9	-0,0230	0,0025	0,5893
	5	275	-17,8	-192,3	-0,0297	0,0150	0,5965
	6	837	-13,7	-98,4	-0,0718	-0,0960	0,5011
	7	838	-6,0	-5,9	0,0117	0,3109	0,9487
A 1-3	1	270	-89,0	-675,3	0,0754	0,2957	0,5612
	2	269	-46,4	-558,1	0,0329	-0,0178	0,7153
	3	268	-35,3	-442,0	0,0256	-0,0345	0,6139
	4	267	-29,5	-333,8	0,0245	0,0710	0,6988
	5	266	-20,8	-222,3	0,0019	-0,0709	0,6123
	6	836	-19,6	-117,1	0,0436	0,0050	0,5917
	7	835	-43,0	-5,7	0,0477	0,1959	0,9714
D 3-4	1	631	-209,2	-1442,0	-0,0913	-0,3890	0,0325
	2	629	-93,4	-1156,7	0,0890	0,3007	0,5736
	3	627	-85,6	-909,9	-0,0155	-0,1655	0,2202
	4	625	-58,7	-685,3	0,1213	0,2770	0,6079
	5	623	-47,6	-470,1	0,0993	-0,2373	0,3231
	6	1404	-42,5	-255,4	0,4138	0,5041	0,8057
	7	1402	-46,4	-9,5	0,2437	-0,6572	0,6353
D 1-2	1	630	-60,8	-803,0	-0,0724	-0,0350	0,2136
	2	628	5,9	-670,2	-0,2054	-0,1920	0,0549
	3	626	17,6	-547,7	-0,0380	0,2092	0,4534
	4	624	-16,3	-411,8	0,2110	-0,2255	0,5944
	5	1198	19,4	-287,1	0,2357	0,3814	0,6983
	6	1403	-6,8	-152,5	-0,3390	-0,4743	-0,1935
	7	1401	-105,7	-26,8	-0,0587	0,1244	0,4774
E 3-5	1	641	-186,7	-1214,5	0,1861	0,0622	0,3881
	2	639	-122,5	-1035,8	0,2127	-0,2514	0,2538
	3	637	-99,2	-840,3	0,2202	0,3320	0,4836
	4	635	-89,0	-644,4	0,0526	-0,1435	0,1578
	5	633	-35,4	-439,8	-0,0389	-0,0204	0,2410
	6	1414	-43,0	-36,5	-0,1346	-0,4178	-0,0175
	7	1412	-43,0	-36,5	-0,1346	-0,4178	-0,0175
E 1-2	1	640	-89,3	-840,5	-0,6470	-0,5109	0,4715
	2	638	29,9	-589,1	-0,5124	-0,7813	0,4760
	3	636	32,9	-558,7	-0,8838	4,4095	7,3885
	4	634	98,7	-259,1	-2,0910	-3,7106	0,3179
	5	632	49,4	-239,3	0,7663	-1,2756	1,4327
	6	1413	-10,9	-176,8	1,1672	0,2297	1,8323

	7	1411	27,4	-10,7	0,3194	0,4601	1,4448
H 1-2	1	646	6,4	-912,4	0,1873	0,0024	0,8877
	2	645	-132,9	-752,7	0,2051	0,2161	0,8568
	3	644	-137,3	-324,1	-0,1280	0,2559	0,7629
	4	643	-64,1	-402,9	-0,5296	1,8352	2,5892
	5	642	-137,3	-324,1	-0,1280	0,2559	0,7629
	6	1419	-182,9	-384,5	-0,9957	-0,6666	-0,0157
	7	1418	11,5	-11,3	-0,5383	-0,0655	1,0339
I 3-5	1	651	-175,4	-1325,3	-0,2380	-0,1936	0,1239
	2	650	-108,9	-1112,9	-0,3413	0,1816	0,3323
	3	649	-86,0	-893,6	-0,3509	-0,3074	0,1875
	4	648	-70,8	-673,1	-0,2031	0,2044	0,2935
	5	647	-40,1	-459,3	-0,2367	-0,1919	0,2716
	6	1424	-24,7	-242,9	-0,1308	-0,0331	0,1986
	7	1423	-40,5	-28,4	0,0689	0,4438	0,6380
J 3-5	1	661	-200,7	-1402,2	0,0355	0,2265	0,4483
	2	659	-93,6	-1132,5	-0,1203	-0,5320	0,0229
	3	657	-81,2	-886,2	-0,0803	0,4883	0,7814
	4	655	-58,2	-689,8	-0,3637	-0,4702	-0,0137
	5	653	-45,0	-443,3	-0,0821	0,1079	0,3143
	6	1434	-35,0	-216,5	-0,0775	0,0673	0,5717
	7	1432	-48,8	5,4	-0,2867	0,0849	0,3752
J 1-2	1	660	-12,0	-747,9	0,0180	-0,2505	0,1129
	2	658	-33,1	-626,5	0,1984	0,1266	0,3520
	3	656	19,0	-522,3	0,2243	0,0987	0,4276
	4	654	30,5	-385,6	0,0919	0,0137	0,2207
	5	652	-32,6	-267,7	-0,0276	0,0854	0,1720
	6	1433	8,2	-150,2	0,0068	0,0732	0,1266
	7	1431	-94,9	-25,3	-0,0163	-0,0142	0,0738
P 3-5	1	681	-217,0	-1536,2	0,0006	-0,1511	0,3573
	2	679	-100,4	-1234,5	0,2089	0,4260	0,7900
	3	677	-78,6	-932,0	-0,0728	-0,3787	0,3387
	4	675	-60,6	-720,6	0,1524	0,3418	1,5157
	5	673	-47,2	-486,6	0,2240	0,0136	0,7274
	6	1454	-35,2	-218,7	0,1523	-0,0320	0,9648
	7	1452	-54,4	12,3	0,2323	-0,2981	0,9070
P 1-2	1	680	-68,4	-764,7	-0,0817	-0,0793	0,1640
	2	678	27,3	-638,5	-0,1941	-0,1709	-0,0829
	3	676	-17,8	-529,5	0,0474	0,1681	0,3932
	4	674	-7,4	-395,1	0,2252	-0,1416	0,5914
	5	672	55,7	-282,5	0,2428	0,5401	0,8236
	6	1453	-33,0	-157,0	-0,1797	-0,4627	0,1764
	7	1451	-98,1	-27,1	-0,0716	0,3095	0,6685
Q 3-5	1	686	-84,8	-1150,3	0,0194	-0,0427	0,0919

	2	685	3,8	-990,5	0,0206	-0,1426	0,1999
	3	684	11,8	-817,6	0,0784	0,2188	0,4280
	4	683	2,3	-625,6	-0,0283	-0,0753	0,0486
	5	682	11,2	-425,3	-0,0493	-0,0966	0,3732
	6	1459	-8,2	-228,7	0,0413	0,1463	0,3394
	7	1458	-110,5	-58,0	-0,0116	-0,2032	0,1765
R 1-2	1	691	-117,0	-956,7	0,1031	-0,3673	0,2955
	2	690	54,6	-749,9	0,2402	-0,4913	1,2494
	3	689	0,2	-642,4	-0,8639	4,2552	6,8769
	4	688	245,5	-216,4	-2,1779	-3,9073	0,1369
	5	687	153,4	-257,8	0,4064	-1,1634	1,3881
	6	1464	-89,9	-185,1	0,6823	1,5653	1,8530
	7	1463	73,3	-8,5	0,0126	-0,3299	0,3549
U 3-5	1	701	-207,3	-1330,9	-0,2345	-0,4960	-0,0116
	2	699	-151,8	-1124,8	-0,1787	0,3463	1,1601
	3	697	-79,9	-901,3	-0,1506	-0,1804	0,4675
	4	695	-71,2	-687,1	-0,0655	-0,1598	0,8112
	5	693	-39,9	-464,7	0,0473	0,5473	1,0434
	6	1474	-12,3	-249,2	-0,1932	-1,0257	0,4001
	7	1472	-54,5	-49,8	0,3613	1,5200	1,5918
U 1-2	1	700	-47,9	-812,0	-0,0416	0,4601	1,1761
	2	698	-72,3	-714,8	-0,0715	0,0590	1,9613
	3	696	-174,5	-703,3	-2,9810	-4,9362	-1,3168
	4	694	-52,0	-415,3	-1,5344	4,0189	6,0619
	5	692	-48,5	-339,6	-1,0658	0,1975	1,2064
	6	1473	-140,3	-328,2	-1,5221	-2,3345	-0,5082
	7	1471	-36,6	-23,4	-0,5627	0,6882	1,2222
V 3-5	1	711	-221,0	-1454,2	0,0192	0,2437	0,4544
	2	709	-116,5	-1182,9	-0,1187	-0,2394	0,2418
	3	707	-100,4	-948,0	-0,0438	0,1287	0,4992
	4	705	-69,3	-721,6	-0,1093	-0,0669	0,3662
	5	703	-49,8	-490,0	-0,0238	-0,0040	0,4000
	6	1484	-36,7	-258,4	0,0991	0,1075	0,5558
	7	1482	-23,4	-5,5	-0,1210	0,0117	0,6977
V 1-2	1	710	-33,8	-850,8	-0,0444	-0,3031	0,0402
	2	708	-4,8	-722,5	0,3149	0,2215	0,5348
	3	706	18,6	-597,7	0,3421	0,0464	0,6715
	4	704	9,0	-450,5	0,0551	0,1896	0,5004
	5	702	-3,7	-308,3	0,0815	-0,0070	0,3260
	6	1483	-0,9	-166,4	0,1157	0,2601	0,7642
	7	1481	-113,2	-27,7	-0,1796	-0,1468	0,0291
X 3-4	1	1166	-149,0	-860,8	-0,0067	-0,0316	0,0137
	2	1167	-75,6	-680,7	0,0021	0,0135	0,0581
	3	1168	-58,5	-521,6	0,0030	-0,0016	0,0311

	4	1169	-46,3	-387,5	0,0055	0,0030	0,0439
	5	1170	-31,5	-257,8	-0,0013	-0,0032	0,0325
	6	1730	-22,9	-129,7	-0,0123	-0,0107	0,0380
	7	1731	-34,0	5,3	-0,0036	0,0229	0,0469
X 1-3	1	1165	-140,4	-946,6	-0,0056	-0,0174	0,0078
	2	1164	-84,4	-747,3	-0,0187	0,0051	0,0352
	3	1163	-66,2	-569,0	-0,0352	-0,0100	0,0147
	4	1162	-52,9	-425,1	-0,0258	0,0209	0,0441
	5	1161	-32,1	-277,9	-0,0369	-0,0393	-0,0110
	6	1729	-26,9	-137,0	-0,0117	0,0028	0,0362
	7	1728	-60,3	12,8	0,0038	0,0022	0,0185
1 A-D	1	2;9	-463,3	-3090,0	-0,1301	-0,0357	0,1456
	2	58;62	-294,7	-3355,4	3,0256	-0,4489	5,9673
	3	110;114	-246,7	-2739,1	3,3455	0,4786	7,7816
	4	162;166	-186,0	-2077,9	3,2611	-2,4290	6,6851
	5	214;218	-109,3	-1425,0	3,4888	1,8695	10,3428
	6	1;40	44,7	-768,0	4,2643	-4,7686	6,8813
	7	256;279	255,6	-100,6	4,1159	5,4717	14,0424
1 D-E	1	13	-77,2	-480,8	-0,0475	-0,1368	0,1099
	2	66	-40,1	-423,5	-0,0475	0,0908	0,2540
	3	118	-28,4	-339,2	-0,0398	0,0043	0,1967
	4	170	-26,1	-274,8	0,0688	-0,0676	0,3468
	5	222	-7,1	-186,6	0,0928	0,2778	0,4421
	6	44	8,4	-85,2	-0,2138	-0,5186	0,0280
	7	328	57,9	0,2	0,0821	0,8022	0,9223
1 E-H	1	20;16	-270,6	-3027,4	0,7491	-0,8999	2,5687
	2	73;69	-376,9	-2753,5	2,5540	-1,4281	4,3887
	3	125;121	-183,7	-2189,0	2,7139	2,1814	8,0496
	4	177;173	-136,2	-1911,4	2,2148	-5,2169	5,4243
	5	229;225	-13,8	-1114,8	2,1575	4,1806	11,8158
	6	95;47	168,8	-454,7	3,5139	-6,3404	6,4821
	7	335;331	333,1	-124,1	3,4175	6,0006	13,2894
1 H-J	1	22	-83,9	-513,2	-0,0110	0,0404	0,1512
	2	75	-65,3	-429,6	-0,1691	-0,4868	0,0424
	3	127	-50,1	-332,9	0,1137	1,0888	1,1063
	4	179	-35,4	-290,1	-0,4051	-1,8006	-0,3608
	5	231	-18,7	-163,2	0,4514	2,3087	2,3194
	6	97	2,6	-80,4	-0,5705	-2,5277	-0,5639
	7	337	27,6	-14,4	0,4956	2,7015	2,7035
1 J-M	1	23;27	-402,1	-3607,4	-0,0980	-1,2926	1,7892
	2	76;80	-283,3	-3126,9	0,2861	-1,1926	3,2703
	3	128;132	-232,0	-2590,3	0,5435	0,7543	5,5115
	4	180;184	-223,5	-1936,1	0,5329	-4,0263	3,4661
	5	232;236	-107,9	-1338,4	0,7597	3,3050	9,0175

	6	98;146	12,8	-774,4	0,2215	-7,0899	3,3168
	7	338;342	-76,1	-118,9	1,2724	6,1905	12,4157
1 M-P	1	31;35	-380,6	-3612,2	-0,1001	-1,3655	1,7032
	2	84;88	-196,8	-3153,0	0,0608	-0,7126	3,3910
	3	136;140	-99,9	-2590,2	-0,0526	-0,7591	3,9352
	4	188;192	-92,1	-1922,4	-0,3837	-1,5394	3,9629
	5	240;244	-18,4	-1336,4	0,1380	-0,5269	5,0705
	6	150;198	74,1	-757,6	-0,1627	-3,1891	3,9459
	7	346;350	-52,4	-118,3	1,0504	2,4422	8,0494
1 P-R	1	39	-81,1	-495,8	-0,0467	-0,0968	0,0841
	2	92	-53,4	-442,9	-0,0942	-0,0452	0,2262
	3	144	-37,5	-330,5	-0,0393	-0,0870	0,3957
	4	196	-25,5	-290,9	0,2362	0,6168	0,7366
	5	248	-5,9	-198,0	-0,1234	-0,9741	0,0170
	6	202	4,8	-75,9	0,1908	1,0690	1,3775
	7	354	16,5	-18,3	-0,1801	-0,8213	0,1868
1 R-U	1	1200;1206	-295,7	-3014,7	1,7091	-0,3054	3,7056
	2	1255;1259	-396,7	-2825,6	2,0397	-1,6914	3,3661
	3	1248;1252	-256,4	-2170,8	2,2371	0,6349	5,8049
	4	1241;1245	-193,5	-1969,7	0,9494	-1,2049	5,4564
	5	1208;1212	-92,4	-1166,9	1,4295	-2,5950	6,3709
	6	1753;1758	-95,0	-1293,1	0,1403	1,8477	8,2374
	7	1782;1786	-61,4	-1419,1	0,5539	-2,3980	6,1311
1 U-V	1	48	-89,0	-538,5	-0,0051	0,0278	0,1240
	2	101	-62,8	-462,3	-0,2079	-0,3471	-0,0077
	3	153	-48,3	-369,6	-0,0877	0,5962	0,5987
	4	205	-39,7	-304,7	-0,2534	-0,9098	-0,1704
	5	237	-12,7	-232,8	-0,5349	0,8560	0,8664
	6	203	-9,8	-79,8	-0,3991	-1,3664	-0,3833
	7	355	18,6	-10,7	0,2862	1,5532	1,5714
1 V-X	1	49;54	-412,0	-4533,6	-0,1646	-1,3934	1,8312
	2	102;106	-257,3	-3908,9	0,4251	-1,4736	3,7436
	3	154;158	-222,8	-3167,2	1,1349	1,3240	6,1651
	4	206;210	-190,3	-2373,9	0,3291	-4,4085	3,3395
	5	258;262	-103,7	-1609,7	1,3991	3,7412	9,3591
	6	204;256	-21,0	-994,8	0,4635	-6,4604	4,4529
	7	356;360	-423,9	-117,7	1,8702	7,4044	14,0429
3 D-E	1	616	-239,9	-1567,1	0,0364	0,7333	1,4090
	2	610	-127,9	-1282,3	-0,3116	-0,9429	1,2810
	3	604	-104,6	-1026,3	0,2432	1,8037	2,4434
	4	598	-74,0	-782,5	-0,4977	-2,0675	1,0251
	5	592	-47,4	-541,6	0,6070	3,0232	4,0390
	6	1390	-6,7	-281,3	-0,6925	-3,3423	1,5148
	7	1384	36,9	-23,2	0,6075	3,4587	5,6383

3 E-I	1	617	-277,5	-1392,4	0,0565	-0,2811	0,7744
	2	611	-208,8	-1251,1	-0,0228	-0,8731	1,7165
	3	605	-131,9	-1024,3	0,6455	2,7555	2,7726
	4	599	-103,3	-784,9	-0,7673	-4,1100	0,0231
	5	593	-57,1	-535,0	1,2657	6,5135	6,5138
	6	1391	-5,9	-273,2	-1,7874	-8,7240	-1,7854
	7	1385	84,3	-60,9	2,2257	11,0164	11,0187
3 I-J	1	618	-229,9	-1532,3	-0,0478	0,2836	1,2270
	2	612	-116,5	-1289,0	-0,3298	-0,4691	2,3175
	3	606	-104,5	-1025,6	0,1960	1,0996	1,9765
	4	600	-87,2	-809,8	-0,0716	-0,9424	2,8149
	5	594	-39,4	-512,4	0,2836	1,6348	3,8196
	6	1392	-15,6	-254,4	-0,4925	-2,3743	2,5504
	7	1386	8,2	-19,4	0,7944	3,2586	6,2036
3 P-Q	1	619	-238,3	-1593,6	-0,0327	0,0797	1,4868
	2	613	-141,7	-1341,7	-0,1574	-0,6179	2,2860
	3	607	-109,8	-1033,8	0,2520	2,0645	3,5213
	4	601	-85,4	-811,9	-0,6344	-3,0589	0,2324
	5	595	-59,3	-560,6	1,0479	4,7880	5,2854
	6	1393	1,2	-245,3	-1,2886	-5,8339	-0,4196
	7	1387	48,8	-1,3	1,2709	6,9675	7,5362
3 Q-U	1	620	-277,7	-1398,9	-0,2366	-1,2347	1,0533
	2	614	-222,1	-1290,8	0,1450	0,2553	3,2153
	3	608	-142,2	-1052,9	0,4587	2,1178	2,9144
	4	609	-113,5	-1062,8	0,0560	0,5214	1,4089
	5	596	-61,9	-526,1	1,3274	6,6081	6,6209
	6	1394	0,4	-272,4	-1,9221	-9,7441	-1,5479
	7	1388	99,4	-58,7	2,6880	13,0818	13,1090
3 U-V	1	621	-247,6	-1557,4	-0,0595	-0,2474	1,2875
	2	615	-146,4	-1331,9	0,0494	0,7099	2,6149
	3	609	-113,5	-1062,8	0,0560	0,5214	1,4089
	4	603	-83,2	-809,7	-0,1602	-1,0598	1,9619
	5	597	-51,8	-530,5	0,4332	2,5273	4,1201
	6	1395	-7,7	-262,9	-0,9585	-4,3008	1,5120
	7	1389	22,0	-33,8	1,0430	5,3539	7,8811
4 A-D	1	287;291	-234,1	-5589,0	0,4135	3,0420	16,8840
	2	295;299	-49,9	-3063,0	2,0668	-0,0109	4,9131
	3	303;307	-4,3	-658,4	2,9652	-2,3122	4,9601
	4	311;315	-2457,1	-28771	9,5329	22,8943	111,6110
	5	319;323	-27,3	-2325,8	1,5002	0,1336	5,0808
	6	847;851	-46,4	-1977,3	2,4961	-1,1258	6,2946
	7	855;859	-21,4	-1234,5	2,6862	0,9016	7,0264
4 J-M	1	377;381	-365,8	-3261,0	-0,0840	-1,0107	2,1377
	2	413;417	-274,8	-2837,9	0,2303	-0,5890	3,0430

	3	449;453	-213,7	-2300,6	0,4235	-0,1539	3,7422
	4	485;489	-5245,0	-65923	25,7983	41,8063	229,4089
	5	521;525	-86,4	-1150,1	-0,1677	1,1289	4,3730
	6	896;900	31,6	-597,3	-0,0325	-2,8594	2,3331
	7	1849;926	387,1	-134,7	1,0066	1,7121	5,4552
4 M-P	1	385;389	-406,5	-3457,2	-0,0787	-1,0574	2,1197
	2	421;425	-275,2	-3016,9	0,2405	-0,3164	3,2769
	3	457;461	-185,8	-2371,3	-0,5015	-0,7576	2,4112
	4	493;497	-127,0	-1812,0	-0,8217	-0,9809	2,4442
	5	529;533	-62,9	-1224,5	0,6276	-0,5251	3,7084
	6	904;908	19,3	-654,7	0,0860	-1,2542	3,6011
	7	1852;934	411,7	-148,6	1,0257	0,0231	4,2077
4 V-X	1	547;551	-284,5	-3747,4	-0,7042	-0,9252	1,7175
	2	563;567	-195,0	-3546,5	0,1895	-0,9502	3,0077
	5	562;566	-63,4	-1496,7	0,8287	1,7248	5,7548
	3	571;575	-123,1	-2770,0	0,0612	0,5579	3,9549
	4	579;583	-93,0	-2107,7	-0,7827	-2,6776	1,8240
	6	1321;1325	-6,7	-808,1	-0,4927	-3,9043	2,1972
	7	1855;1333	-13,0	-187,8	1,0013	2,6313	6,5365
5 D-E	1	367	-66,6	-509,9	0,0960	0,2142	0,4372
	2	403	-20,7	-419,4	0,1031	0,0749	0,5567
	3	439	-21,7	-340,0	0,0342	-0,0959	0,3559
	4	475	-15,4	-253,3	0,0909	0,2946	0,7100
	5	511	-11,4	-172,5	-0,0720	-0,4234	0,2005
	6	887	-5,8	-90,1	0,1179	0,6554	1,0065
	7	913	4,1	-19,2	-0,1683	-0,9514	-0,1341
5 E-H	1	374;370	-309,2	-2920,3	0,1199	-6,7855	11,9798
	2	410;406	-383,9	-2575,4	4,5237	4,2583	27,0790
	3	446;442	-268,6	-2124,4	5,7236	0,9754	31,9800
	4	482;478	-187,9	-1630,8	5,9218	1,4287	34,9813
	5	518;514	-105,9	-1135,4	6,0361	-0,6833	32,0464
	6	894;890	15,5	-627,4	5,5569	4,8680	32,9002
	7	1848;916	248,4	-137,0	1,5985	3,6814	26,8300
5 H-J	1	972	-56,3	-416,6	-0,2489	-0,7155	0,7786
	2	971	-26,1	-371,2	-0,2270	0,5030	1,4671
	3	970	-23,5	-313,2	-0,4586	0,0916	1,4173
	4	968	-16,3	-243,0	-0,5123	0,0373	1,5788
	5	967	-10,1	-165,3	-0,5408	-0,1155	1,3388
	6	1541	-4,6	-92,0	-0,3632	0,4772	1,5774
	7	1540	10,2	-33,2	-0,0341	0,1113	1,1110
5 P-R	1	978	-67,2	-508,2	-0,2462	-0,7264	0,7005
	2	970	-23,5	-313,2	-0,4586	0,0916	1,4173
	3	976	-24,1	-355,7	-0,4210	0,1882	1,5205
	4	975	-16,8	-267,7	-0,5092	-0,1251	1,3337

	5	973	-14,4	-177,5	-0,4849	-0,0015	1,5909
	6	1546	-0,8	-86,2	-0,5388	-0,2491	1,2805
	7	1545	-13,1	-19,4	-0,4204	0,0672	1,7082
5 R-U	1	1266;1270	-378,6	-3251,2	0,5162	-4,8724	12,0131
	2	1273;1277	-415,8	-2872,0	3,5075	3,4004	25,2050
	3	1281-85	-323,1	-2337,6	4,5107	0,5654	30,2420
	4	1289-93	-239,6	-1791,1	4,6420	1,3348	32,8781
	5	1297-301	-146,8	-1213,4	4,6679	-2,3833	29,6979
	6	1792-96	-79,3	-596,5	4,4577	1,5359	33,3428
	7	1858-803	117,8	4,5	4,4184	-7,4386	27,7963
5 U-V	1	402	-71,1	-567,0	0,1137	0,2877	0,5583
	2	438	-18,3	-458,0	0,0940	0,0451	0,6725
	3	474	-22,3	-374,3	0,0298	-0,1176	0,3887
	4	510	-17,2	-287,9	0,1104	0,3641	0,8677
	5	546	-11,4	-199,2	-0,0868	-0,5567	0,2410
	6	912	-10,3	-112,7	0,1714	0,8560	1,2696
	7	938	7,0	-24,9	-0,2619	-1,2673	-0,1806

Table 5. SAP2000 internal forces (shear and axial) under all load combinations for joints

Joint name in SAP2000	Shear V (kN)	Axial N (kN)
joint1	126,93	6259,35
joint2	287,00	8553,23
joint3	194,01	6718,15
joint4	214,43	7122,05
joint5	115,05	7980,00
j19	155,40	8278,16
joint6	197,60	7975,13
joint7	174,35	7119,15
joint8	294,07	4854,20
joint9	122,55	8292,30
joint10	272,45	6263,30
joint11	381,50	2190,10
joint12	191,10	4138,15
joint13	382,80	2376,20
joint14	91,29	2775,55
joint15	106,76	2213,80
joint16	299,07	3527,90
joint17	317,60	2201,10
joint18	128,00	2771,75
joint19	377,27	1942,40
joint20	307,80	1794,05
joint21	372,20	2080,25
joint22	305,00	4399,00

joint23	159,10	3973,15
joint24	80,40	2225,80
joint25	239,70	1783,65
joint26	84,84	3604,25
joint27	268,87	3318,40
joint28	288,13	1928,65
joint29	315,00	1788,25
joint30	209,76	2501,15
joint31	332,40	3580,85
joint32	103,14	1965,20
joint33	337,60	6272,15
joint34	156,40	8953,10
joint35	128,60	7445,17
joint36	237,00	6771,10
joint37	183,55	8559,32
joint38	225,46	6158,65
j20	282,72	9145,25
joint39	310,33	7826,65
joint40	303,20	6950,65
joint41	499,00	6776,45
joint42	367,00	5836,65
joint43	344,57	7499,70
joint44	261,76	6538,73
joint45	219,40	2500,00
joint46	99,00	4419,00
joint47	342,20	2265,25
joint48	100,95	2129,05
joint49	151,47	3315,50
joint50	86,00	4175,00
joint51	73,88	7347,00
joint52	357,93	4958,75
joint53	277,00	2256,95
joint54	217,60	1754,20
joint55	305,00	3880,00
joint56	116,22	3344,60
joint57	236,00	6457,84
joint58	204,19	1764,25
joint59	352,47	4865,30
joint60	197,87	6194,10
joint61	187,24	1323,05
joint62	126,22	6830,04
j21	330,80	2319,90
joint 63	121,56	7628,83
joint 64	290,55	3340,45
joint 65	160,46	1371,20
joint 66	97,00	6043,27

joint 67	250,13	1926,45
joint 68	159,20	5485,36
joint 69	393,00	3058,45

Table 6. Values for Axial load (P) and moment (M) for each wall types 1-4

Wall type # 1		Wall type # 2		Wall type # 3		Wall type # 4	
P (kN)	M (N-m)	P (kN)	M (N-m)	P (kN)	M (N-m)	P (kN)	M (N-m)
24,3	7977,1	149,5	8741,4	126,7	5540,3	37,2	12824,8
415,4	8870,0	555,3	9717,6	452,0	6165,4	529,5	14246,7
785,9	9700,5	940,5	10627,4	760,8	6747,8	996,1	15569,9
1141,2	10481,0	1310,7	11483,7	1057,4	7296,0	1443,8	16813,4
1485,3	11219,5	1669,7	12294,8	1344,9	7815,2	1877,5	17990,3
1820,7	11921,5	2020,0	13066,7	1625,5	8309,4	2303,1	19114,8
2221,1	12701,9	2437,0	13922,9	1952,3	8847,8	2814,8	20373,0
2611,1	13445,3	2841,9	14736,4	2276,2	9367,7	3306,8	21558,9
2983,2	14137,4	3226,8	15490,3	2589,9	9860,1	3765,0	22632,7
3322,4	14745,5	3581,1	16159,6	2875,2	10291,6	4193,4	23603,5
3655,9	15327,1	3929,6	16800,1	3153,9	10701,0	4614,6	24532,0
3984,6	15883,7	4273,4	17413,3	3428,8	11093,0	5029,7	25420,4
4352,2	16456,0	4656,8	18042,8	3732,2	11492,8	5498,0	26339,2
4722,4	17011,0	5042,1	18652,4	4040,0	11882,1	5965,8	27225,6
5085,2	17539,5	5419,8	19233,1	4341,7	12252,9	6424,0	28069,8
5441,3	18042,6	5791,1	19786,2	4638,3	12606,2	6874,0	28873,5
5791,8	18521,5	6156,6	20312,7	4930,4	12942,5	7316,8	29638,2
6137,2	18976,8	6517,1	20813,5	5218,4	13262,3	7753,2	30365,4
6505,8	19417,8	6900,9	21298,2	5524,2	13571,7	8219,9	31070,1
6881,6	19839,9	7291,8	21762,1	5836,3	13867,9	8695,0	31744,5
7251,2	20239,4	7676,5	22201,2	6143,4	14148,3	9162,0	32382,7
7615,1	20616,8	8055,4	22616,1	6446,0	14413,2	9621,9	32985,5
7971,2	20968,4	8425,8	23001,5	6744,6	14663,0	10067,8	33538,8
8310,3	21279,6	8780,0	23344,7	7027,8	14882,6	10496,3	34035,8
8664,0	21565,8	9148,8	23660,6	7322,7	15084,4	10942,7	34493,0
9029,1	21828,3	9528,9	23950,6	7626,1	15269,8	11404,3	34912,1
9389,5	22072,2	9904,5	24220,0	7925,9	15442,1	11860,0	35301,4
9745,8	22297,6	10275,9	24469,0	8222,4	15601,3	12310,5	35661,3
10098,3	22504,6	10643,5	24697,6	8515,8	15747,5	12756,1	35991,7
10447,2	22693,2	11007,6	24905,9	8806,5	15880,7	13197,2	36292,8
10804,2	22853,2	11379,3	25083,4	9104,7	15993,2	13646,6	36550,3
11174,3	22980,0	11764,5	25224,6	9412,0	16083,8	14114,6	36752,3
11540,2	23089,9	12145,6	25347,1	9716,1	16162,4	14577,4	36927,3
11902,3	23182,8	12522,8	25450,7	10017,1	16228,9	15035,3	37075,1
12260,9	23258,6	12896,5	25535,3	10315,3	16283,3	15488,7	37195,8
12616,1	23317,4	13266,9	25601,0	10611,0	16325,6	15937,9	37289,3
12973,0	23347,5	13638,0	25636,1	10910,4	16346,6	16384,5	37340,3
13335,6	23325,7	14015,8	25615,0	11211,8	16334,0	16843,2	37304,4
13695,0	23288,8	14390,3	25577,0	11510,7	16310,4	17297,7	37244,3

14051,3	23236,5	14761,8	25521,8	11807,1	16275,9	17748,4	37159,8
14404,8	23168,8	15130,4	25449,4	12101,3	16230,3	18195,4	37050,7
14755,5	23085,5	15496,3	25359,6	12393,3	16173,5	18639,0	36916,7
15106,5	22980,4	15861,8	25247,5	12687,5	16097,9	19079,4	36757,6
15473,4	22820,1	16243,9	25074,6	12992,3	15988,6	19543,5	36499,9
15837,3	22645,6	16622,9	24885,9	13294,6	15869,0	20003,8	36219,6
16198,3	22456,7	16999,0	24681,0	13594,7	15739,1	20460,4	35916,4
16556,5	22253,3	17372,4	24459,8	13892,6	15598,7	20913,5	35590,1
16912,2	22035,1	17743,3	24222,1	14188,5	15447,7	21363,4	35240,2
17265,5	21802,0	18111,7	23967,7	14484,7	15280,9	21810,2	34866,7
17635,2	21501,8	18495,8	23643,9	14791,7	15073,9	22273,9	34399,1
18000,9	21187,5	18876,6	23302,1	15095,7	14857,0	22736,1	33894,9
18357,2	20857,6	19248,1	22942,8	15392,1	14628,9	23186,9	33365,9
18711,2	20514,1	19617,3	22568,2	15686,7	14390,8	23634,7	32815,2
19063,1	20156,6	19984,3	22177,9	15979,5	14142,7	24079,8	32242,3
19412,9	19785,0	20349,2	21771,9	16270,8	13884,4	24522,2	31647,1
19760,7	19399,2	20712,2	21349,9	16560,4	13615,8	24962,1	31029,2
20106,6	18999,0	21073,3	20911,7	16848,6	13336,9	25399,6	30388,4
20450,7	18584,2	21432,6	20457,3	17135,3	13047,5	25834,8	29724,4
20793,1	18154,7	21790,1	19986,4	17420,8	12747,5	26267,9	29037,0
21133,9	17710,4	22146,1	19498,9	17704,9	12436,9	26698,8	28326,0
21473,1	17251,1	22500,4	18994,8	17987,7	12115,6	27127,8	27591,2
21810,8	16776,7	22853,3	18473,7	18269,4	11783,4	27554,9	26832,5
22147,1	16287,2	23204,7	17935,8	18550,0	11440,3	27980,1	26049,5
22482,0	15782,4	23554,8	17380,7	18829,5	11086,3	28403,6	25242,3
22814,5	15262,5	23902,6	16808,8	19106,9	10721,5	28824,4	24410,9
23140,4	14728,8	24243,6	16221,5	19379,3	10346,7	29236,4	23557,7
23465,3	14179,5	24583,6	15616,7	19650,8	9960,8	29647,1	22679,6
23789,1	13614,5	24922,6	14994,4	19921,6	9563,6	30056,6	21776,5
24112,0	13033,8	25260,7	14354,6	20191,6	9155,1	30464,8	20848,3
24434,0	12437,1	25597,8	13697,1	20460,9	8735,3	30871,8	19894,8
24755,1	11824,6	25934,0	13021,8	20729,5	8304,1	31277,7	18916,0
25075,3	11196,1	26269,4	12328,8	20997,4	7861,5	31682,6	17911,8
25394,8	10551,5	26604,0	11617,9	21264,7	7407,5	32086,4	16881,9
25713,4	9890,9	26937,8	10889,1	21531,3	6942,0	32489,2	15826,5
26031,3	9214,1	27270,8	10142,3	21797,4	6464,9	32891,0	14745,2
26348,4	8521,0	27603,1	9377,5	22062,9	5976,3	33291,9	13638,2
26664,9	7811,7	27934,7	8594,6	22327,8	5476,0	33691,9	12505,2
26980,6	7086,1	28265,6	7793,5	22592,2	4964,2	34091,0	11346,2
27294,9	6345,0	28595,2	6974,9	22854,9	4441,6	34489,2	10161,4
27604,0	5591,6	28919,4	6142,7	23114,0	3909,7	34879,8	8958,3
27912,5	4821,7	29243,1	5292,1	23372,7	3366,0	35269,8	7728,8
28220,6	4035,2	29566,4	4423,0	23631,0	2810,4	35659,1	6472,8
28528,3	3232,1	29889,2	3535,5	23889,0	2243,1	36048,0	5190,4
28835,5	2412,3	30211,5	2629,5	24146,7	1663,9	36436,2	3881,4
29142,2	1575,8	30533,4	1704,9	24404,0	1072,7	36823,9	2545,7

29431,7	770,8	30837,2	815,1	24646,7	503,9	37189,7	1260,4
29748,9	0,0	31155,4	0,0	24894,1	0,0	37598,3	0,0

Table 7. Values for Axial load (P) and moment (M) for each wall types 5-8

Wall type # 5		Wall type # 6		Wall type # 7		Wall type # 8	
P (kN)	M (N-m)	P (kN)	M (N-m)	P (kN)	M (N-m)	P (kN)	M (N-m)
90,3	1371,7	5,8	1557,0	69,4	1800,6	57,0	2822,5
188,7	1484,7	130,4	1735,8	201,3	2006,0	207,8	3088,7
283,3	1591,0	248,2	1902,0	326,4	2197,5	351,9	3338,0
375,0	1691,7	361,1	2058,1	446,7	2377,9	490,8	3573,1
477,2	1797,4	470,4	2205,8	563,3	2548,7	625,9	3796,2
586,1	1905,6	576,8	2346,1	677,1	2711,4	785,5	4041,8
690,9	2007,5	697,1	2495,4	806,1	2884,8	944,7	4280,4
792,3	2103,8	820,6	2643,5	937,0	3055,4	1098,2	4505,6
883,5	2187,1	940,1	2783,7	1063,9	3217,0	1237,3	4702,4
972,3	2265,8	1053,2	2912,7	1182,9	3364,0	1371,2	4886,2
1069,2	2345,4	1158,5	3028,5	1295,7	3498,6	1503,0	5061,9
1171,0	2424,6	1262,3	3139,3	1407,0	3627,5	1650,2	5242,5
1270,4	2499,9	1375,6	3251,5	1528,4	3758,0	1800,3	5418,8
1367,8	2571,5	1492,3	3361,8	1652,6	3885,7	1947,1	5586,6
1463,3	2639,6	1606,8	3466,8	1774,5	4007,4	2091,0	5746,3
1557,3	2704,3	1719,1	3566,8	1894,4	4123,4	2232,3	5898,1
1657,4	2766,9	1829,7	3662,0	2012,5	4233,8	2371,4	6042,4
1760,8	2827,0	1938,6	3752,5	2129,0	4338,9	2520,3	6182,2
1862,2	2883,9	2054,1	3840,0	2252,1	4440,4	2672,8	6316,2
1961,8	2937,6	2172,5	3923,9	2378,0	4537,6	2822,5	6443,0
2059,9	2988,1	2288,9	4003,3	2502,0	4629,6	2969,7	6562,7
2155,2	3034,7	2403,6	4078,3	2624,2	4716,6	3114,7	6675,6
2252,4	3074,7	2516,7	4149,0	2744,8	4798,6	3251,6	6774,3
2352,1	3111,5	2625,6	4212,8	2860,6	4872,2	3394,7	6864,2
2450,5	3145,6	2737,5	4269,7	2980,0	4938,6	3542,3	6946,6
2547,5	3177,2	2852,4	4322,0	3102,4	4999,7	3687,9	7023,1
2643,4	3206,1	2965,8	4370,5	3223,4	5056,4	3831,7	7093,9
2738,1	3232,6	3078,0	4415,4	3343,1	5108,8	3973,8	7158,9
2837,5	3253,8	3189,0	4456,6	3461,7	5157,0	4114,3	7218,1
2938,7	3271,1	3298,8	4494,2	3579,1	5200,8	4259,3	7267,4
3038,7	3286,1	3412,6	4525,3	3700,1	5237,6	4409,1	7306,5
3137,5	3298,8	3529,0	4550,8	3824,1	5267,7	4557,1	7340,3
3235,2	3309,1	3644,1	4572,8	3946,7	5293,7	4703,4	7368,9
3331,8	3317,1	3758,0	4591,5	4068,2	5315,8	4848,2	7392,2
3432,6	3318,7	3870,8	4606,7	4188,6	5333,9	4991,5	7410,1
3532,7	3315,2	3982,6	4618,5	4308,0	5348,0	5137,5	7417,8
3630,5	3309,1	4097,4	4623,2	4429,9	5354,3	5284,0	7409,2
3727,4	3300,8	4212,0	4619,7	4551,9	5350,9	5429,1	7396,0
3823,4	3290,5	4325,0	4612,7	4672,5	5343,7	5572,8	7378,0
3918,6	3278,1	4437,1	4602,7	4792,1	5332,8	5715,2	7355,3
4017,6	3258,4	4548,3	4589,6	4910,9	5318,2	5856,5	7327,8

4117,7	3234,5	4658,6	4573,4	5028,8	5299,9	5999,5	7290,8
4216,9	3208,7	4771,4	4549,6	5148,7	5273,8	6147,8	7237,9
4315,1	3181,0	4886,7	4518,3	5271,6	5238,6	6294,9	7180,8
4412,6	3151,3	5001,1	4484,2	5393,5	5200,0	6440,6	7119,3
4509,3	3119,7	5114,6	4447,2	5514,6	5157,9	6585,2	7053,4
4609,4	3080,0	5227,2	4407,3	5634,8	5112,4	6728,6	6982,9
4710,4	3036,0	5339,0	4364,4	5754,2	5063,3	6872,7	6904,1
4810,5	2990,4	5452,9	4313,8	5875,1	5006,6	7022,3	6806,5
4907,3	2942,5	5569,1	4254,9	5998,8	4939,5	7170,3	6704,8
5003,4	2892,8	5684,3	4193,3	6121,6	4869,1	7314,0	6598,2
5098,9	2841,2	5796,3	4128,5	6241,2	4794,9	7456,7	6487,5
5193,6	2787,7	5907,5	4060,9	6360,1	4717,4	7598,4	6372,6
5287,8	2732,4	6018,2	3990,5	6478,3	4636,6	7739,3	6253,4
5381,4	2675,1	6128,2	3917,3	6595,8	4552,4	7879,2	6129,9
5474,5	2615,8	6237,5	3841,2	6712,8	4464,8	8018,3	6002,0
5567,0	2554,6	6346,3	3762,2	6829,2	4373,7	8156,7	5869,6
5659,0	2491,3	6454,6	3680,3	6945,0	4279,1	8294,2	5732,8
5750,6	2426,1	6562,3	3595,4	7060,3	4181,0	8431,1	5591,4
5841,6	2358,7	6669,5	3507,6	7175,1	4079,4	8567,3	5445,4
5932,3	2289,4	6776,2	3416,7	7289,4	3974,2	8702,8	5294,9
6022,5	2218,0	6882,5	3322,8	7403,3	3865,4	8837,7	5139,6
6111,6	2144,5	6988,3	3225,9	7516,7	3752,9	8972,0	4979,7
6199,1	2069,2	7093,7	3125,8	7629,7	3636,9	9105,1	4815,2
6286,2	1991,8	7198,1	3022,9	7741,7	3517,3	9235,4	4646,5
6373,0	1912,2	7300,7	2917,0	7851,9	3394,3	9365,2	4473,0
6459,5	1830,5	7403,0	2808,1	7961,8	3267,6	9494,7	4294,7
6545,7	1746,6	7505,0	2696,0	8071,4	3137,2	9623,7	4111,5
6631,7	1660,5	7606,7	2580,7	8180,6	3003,0	9752,3	3923,5
6717,4	1572,3	7708,1	2462,3	8289,6	2865,1	9880,5	3730,5
6802,8	1481,9	7809,2	2340,7	8398,3	2723,4	10008,4	3532,6
6888,1	1389,2	7910,1	2215,8	8506,8	2578,0	10135,9	3329,7
6973,0	1294,4	8010,7	2087,8	8615,0	2428,7	10263,0	3121,9
7057,8	1197,3	8111,1	1956,5	8722,9	2275,7	10389,8	2909,1
7142,3	1098,0	8211,2	1822,0	8830,7	2118,8	10516,3	2691,3
7226,7	996,5	8311,2	1684,3	8938,2	1958,0	10642,5	2468,5
7309,6	893,3	8410,9	1543,3	9045,4	1793,5	10768,4	2240,6
7391,9	788,1	8510,4	1399,1	9152,5	1625,1	10893,2	2008,2
7474,0	680,6	8608,8	1252,0	9258,7	1453,3	11016,1	1771,9
7555,9	570,8	8706,2	1102,2	9363,7	1278,1	11138,9	1530,5
7637,8	458,8	8803,5	949,0	9468,5	1099,1	11261,4	1284,0
7719,5	344,4	8900,6	792,5	9573,2	916,2	11383,7	1032,3
7801,0	227,8	8997,6	632,7	9677,8	729,4	11505,8	775,4
7878,0	115,5	9094,5	469,6	9782,2	538,6	11627,8	513,4
7970,3	0,0	9191,2	303,1	9886,5	343,9	11742,9	261,2
		9282,4	142,9	9984,9	156,6	11879,9	0,0
		9376,9	0,0	10080,1	0,0		

Table 8. Values for Axial load (P) and moment (M) for each wall types 9-10

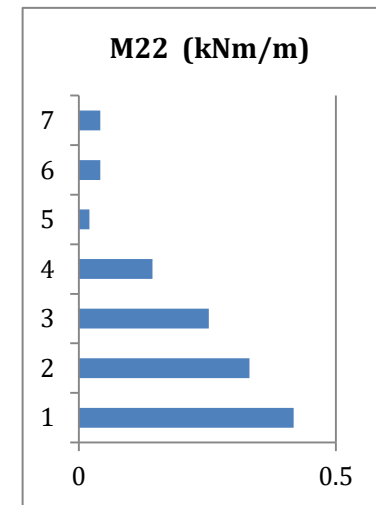
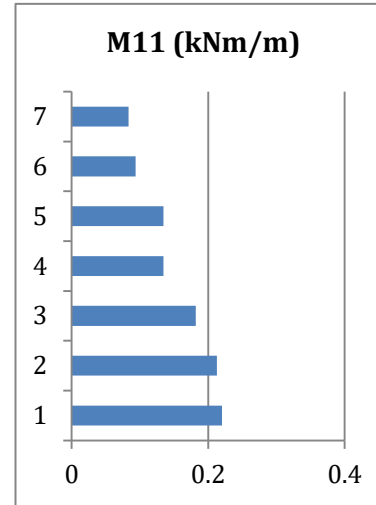
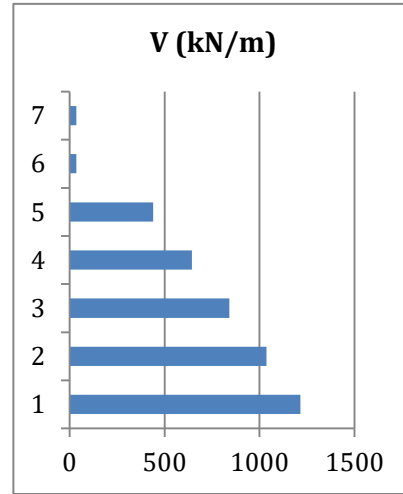
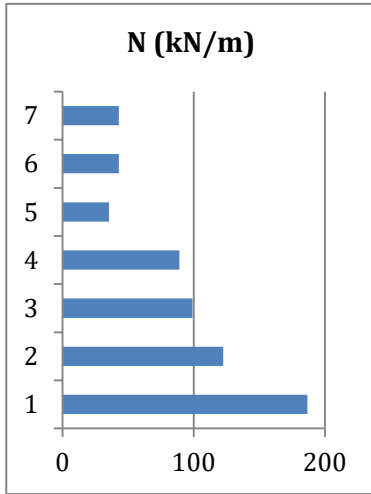
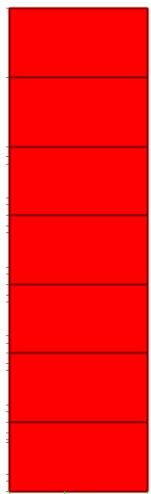
Wall type # 9		Wall type # 10	
P (kN)	M (N-m)	P (kN)	M (N-m)
127,6	3164,8	97,6	6322,4
285,8	3462,9	340,3	7023,7
437,2	3742,6	570,7	7677,4
583,6	4006,8	792,2	8292,8
726,0	4257,9	1007,1	8875,9
894,3	4534,2	1218,2	9433,5
1060,9	4801,1	1470,0	10053,5
1221,8	5053,2	1712,4	10638,6
1366,9	5272,2	1938,9	11169,9
1508,3	5479,6	2151,3	11651,8
1647,6	5678,0	2360,3	12112,9
1802,9	5881,5	2566,4	12554,3
1960,5	6079,8	2797,7	13009,3
2114,7	6268,5	3028,7	13448,2
2266,1	6448,2	3255,2	13866,4
2415,0	6619,2	3477,8	14264,7
2561,7	6781,7	3697,0	14643,8
2718,2	6939,0	3913,1	15004,4
2878,2	7089,8	4143,7	15353,6
3035,4	7232,4	4378,1	15687,7
3190,2	7367,2	4608,8	16003,9
3342,7	7494,2	4836,1	16302,6
3486,6	7605,1	5056,7	16577,2
3637,1	7706,9	5269,2	16824,4
3792,3	7800,2	5490,1	17052,0
3945,5	7886,9	5718,2	17260,8
4096,8	7967,1	5943,5	17454,8
4246,5	8040,7	6166,3	17634,1
4394,6	8107,7	6386,8	17798,7
4546,8	8164,0	6605,3	17948,7
4704,2	8208,8	6827,6	18077,3
4859,8	8247,6	7058,7	18178,9
5013,6	8280,4	7287,3	18267,0
5166,0	8307,2	7513,7	18341,5
5316,8	8327,9	7737,9	18402,4
5469,8	8337,5	7960,1	18449,6
5623,9	8329,0	8181,1	18476,0
5776,5	8315,2	8407,9	18460,6
5927,9	8296,0	8632,6	18432,9
6077,9	8271,3	8855,5	18393,0
6226,8	8241,1	9076,7	18340,6
6376,8	8201,2	9296,3	18275,8
6532,8	8143,2	9514,4	18198,2

6687,4	8080,2	9743,6	18073,4
6840,7	8012,1	9971,1	17937,0
6992,9	7938,9	10196,8	17789,1
7143,9	7860,6	10420,9	17629,5
7295,0	7774,1	10643,5	17458,0
7452,3	7665,9	10864,6	17274,5
7607,8	7553,0	11093,7	17045,9
7759,1	7434,5	11322,0	16799,1
7909,3	7311,2	11545,0	16539,8
8058,6	7183,0	11766,6	16269,5
8207,1	7049,8	11986,9	15988,0
8354,6	6911,7	12205,9	15695,2
8501,3	6768,5	12423,7	15390,8
8647,2	6620,1	12640,4	15074,8
8792,4	6466,6	12856,1	14747,2
8936,8	6307,9	13070,7	14407,7
9080,6	6143,8	13284,3	14056,2
9223,7	5974,5	13496,9	13692,8
9366,2	5799,9	13708,7	13317,2
9508,0	5619,8	13919,6	12929,5
9648,8	5434,5	14129,7	12529,5
9786,7	5244,3	14338,5	12117,3
9924,1	5048,5	14543,2	11694,0
10061,1	4847,3	14747,2	11258,2
10197,7	4640,5	14950,7	10809,8
10333,9	4428,0	15153,5	10348,8
10469,7	4210,0	15355,9	9875,0
10605,1	3986,3	15557,6	9388,6
10740,2	3757,0	15758,9	8889,3
10874,9	3521,9	15959,7	8377,1
11009,3	3281,2	16160,0	7852,1
11143,4	3034,7	16359,8	7314,1
11277,1	2782,4	16559,3	6763,1
11410,6	2524,5	16758,2	6199,2
11543,1	2261,2	16956,8	5622,1
11673,7	1993,4	17154,9	5032,1
11804,0	1719,8	17349,5	4432,7
11934,1	1440,4	17543,7	3820,0
12063,9	1155,0	17737,7	3194,0
12193,6	863,8	17931,4	2554,8
12323,2	566,6	18124,8	1902,2
12445,4	280,7	18318,0	1236,3
12583,2	0,0	18500,2	595,5
		18693,3	0,0

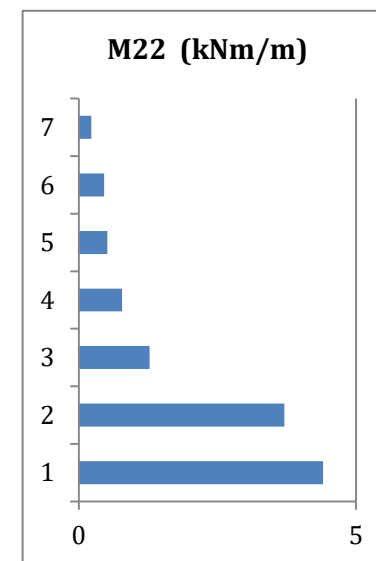
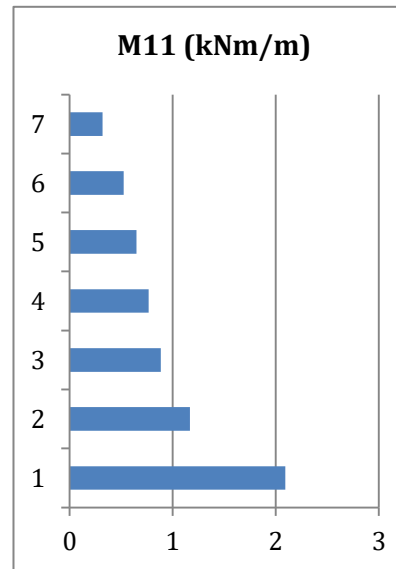
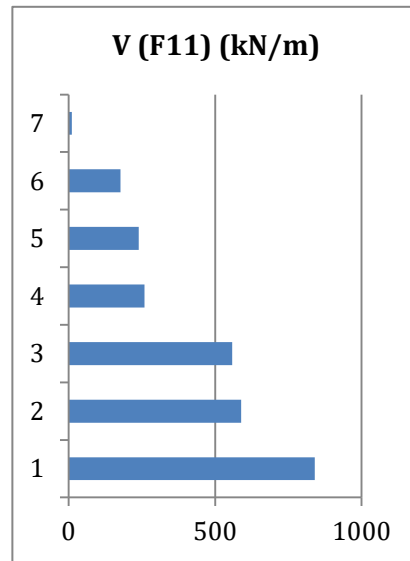
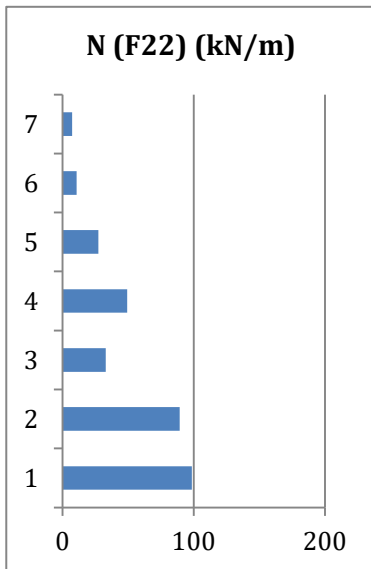
Table 8. Crack check for slab cross-section

slab y (in)	βn	fs (ksi)	w_max (in)	tb (in)	Ae (in²)	n	n actual
94,488	1,2	31	0,016	0,984	371,906	3	16
118,110	1,2	31	0,016	0,984	464,882	4	20
141,732	1,2	31	0,016	0,984	557,859	4	24

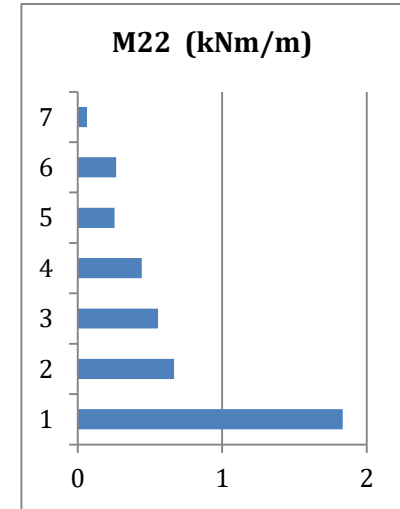
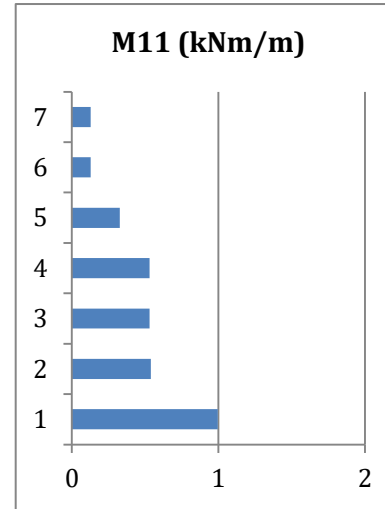
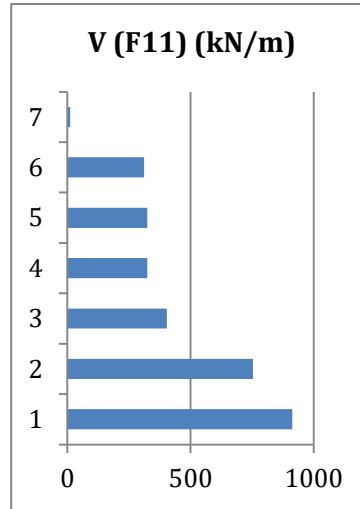
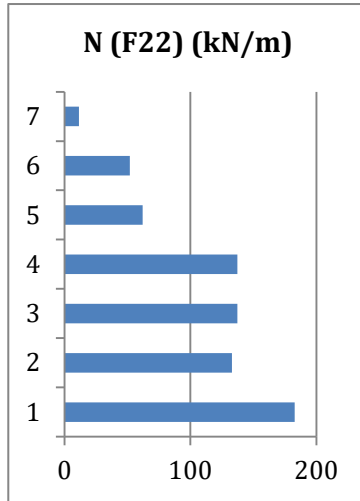
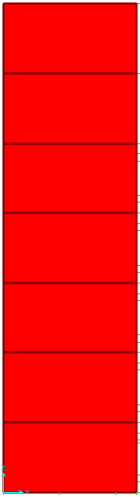
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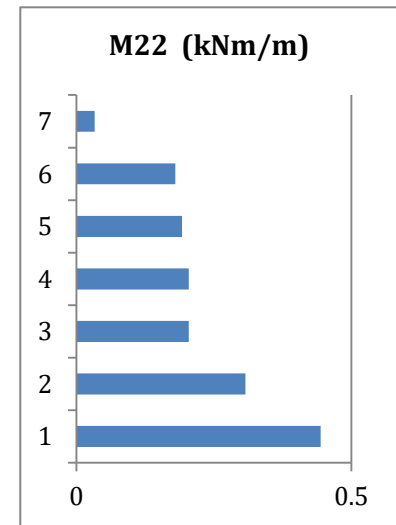
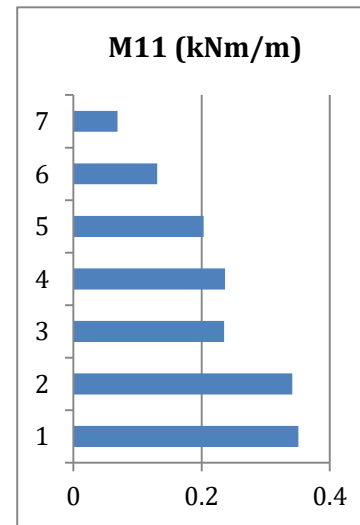
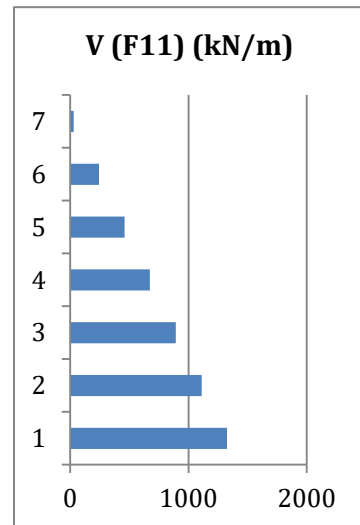
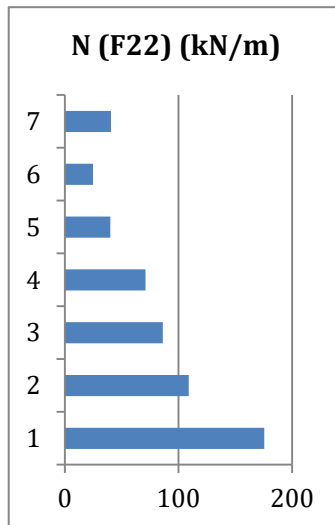
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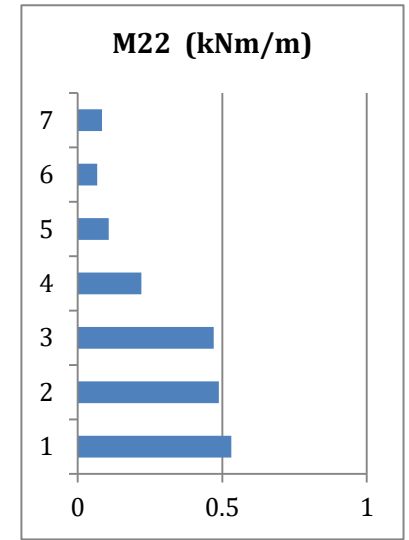
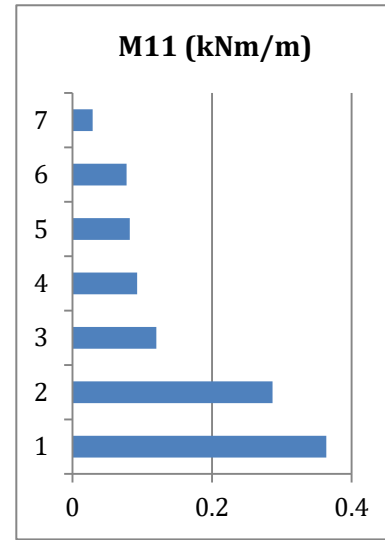
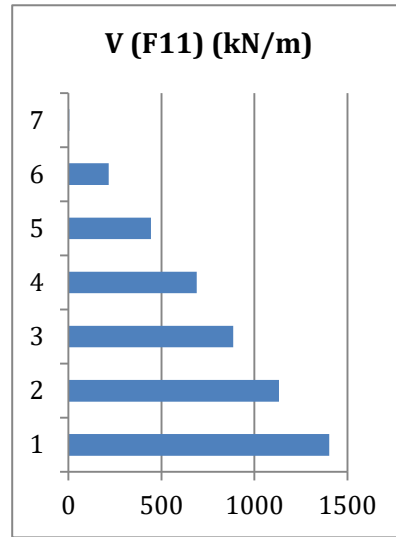
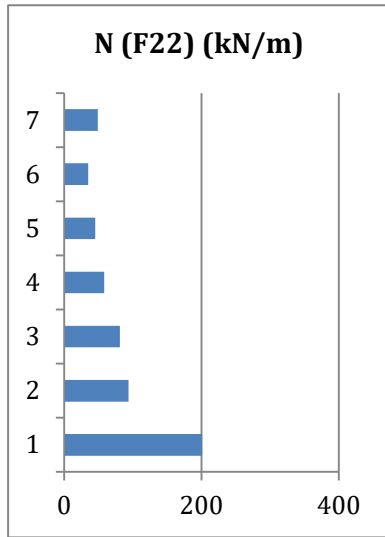
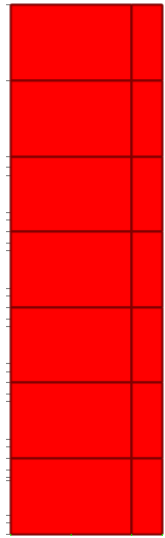
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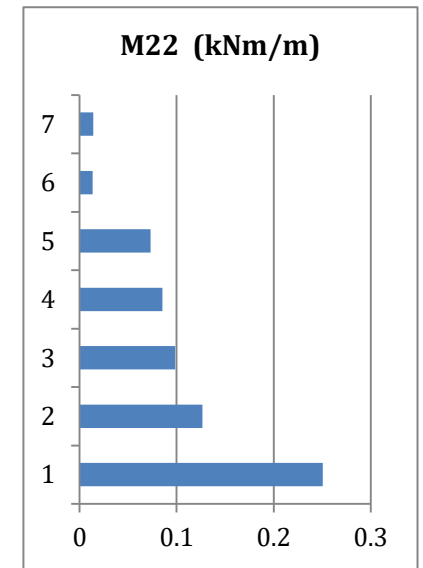
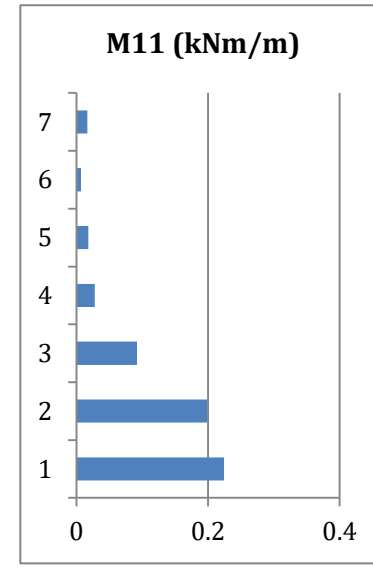
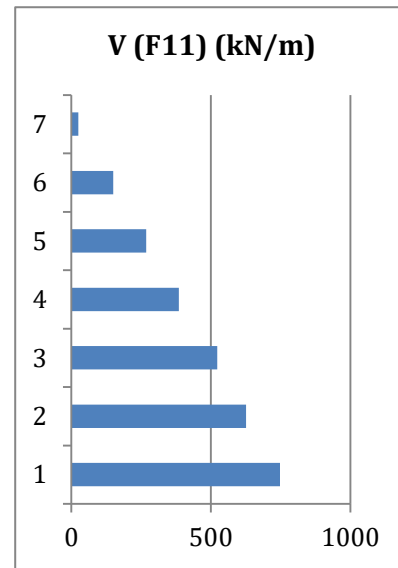
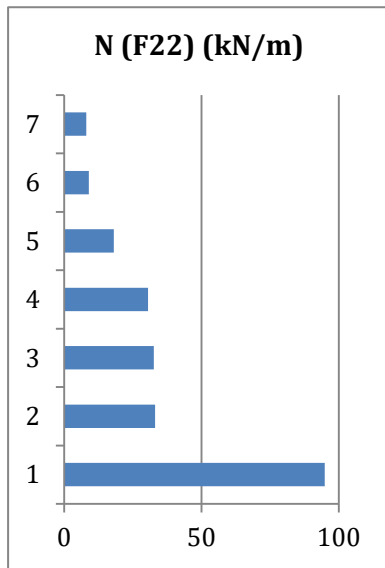
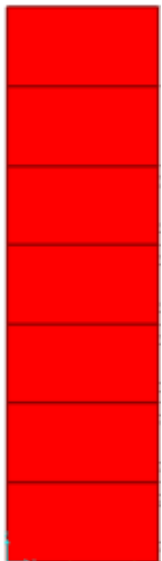
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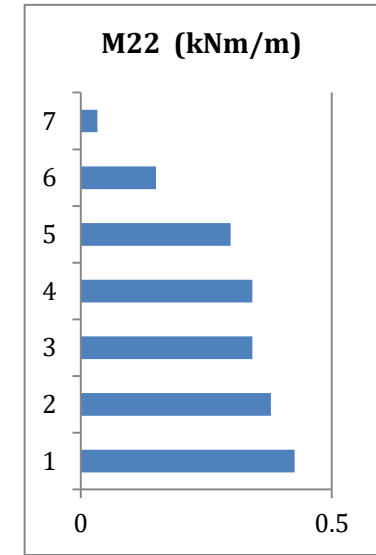
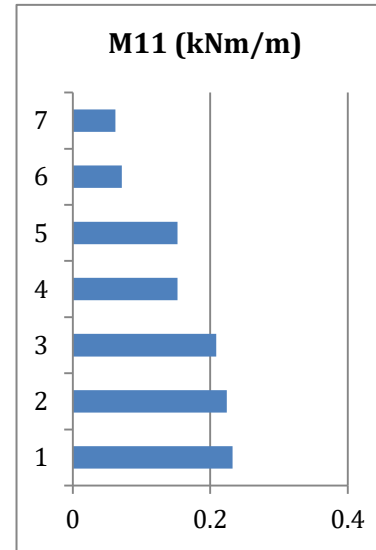
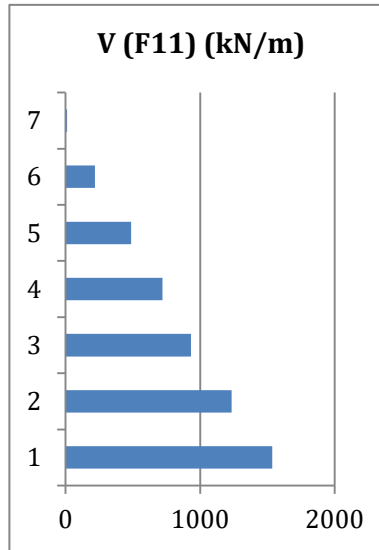
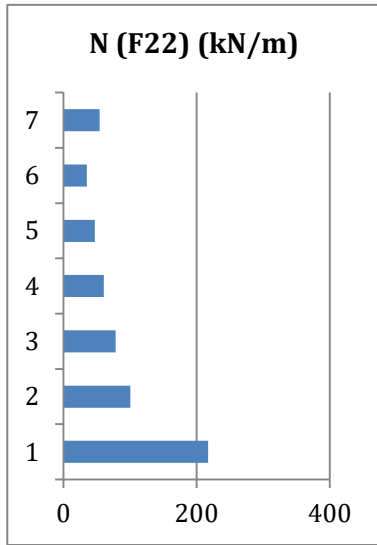
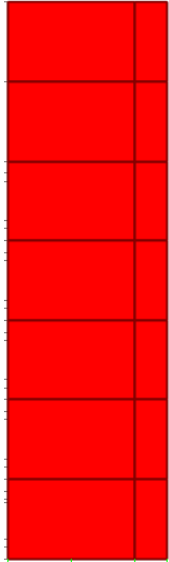
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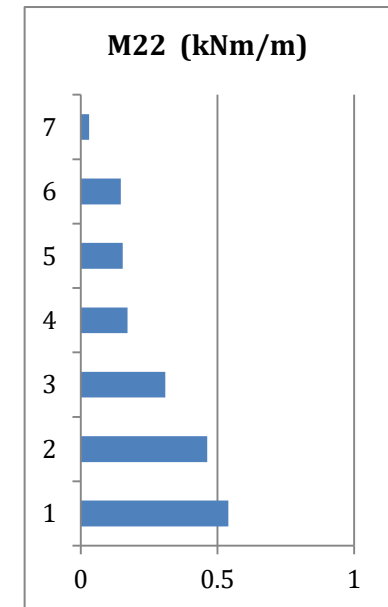
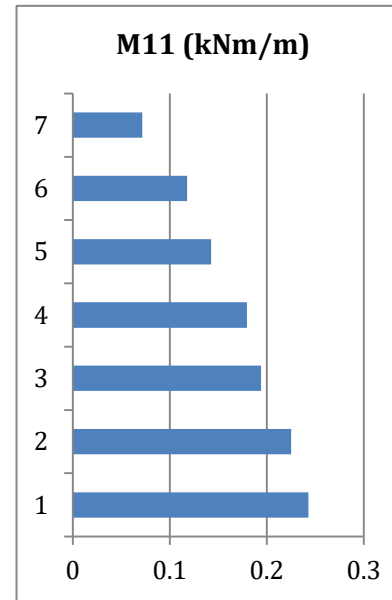
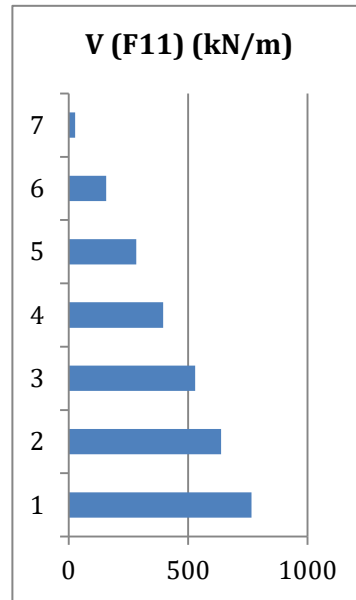
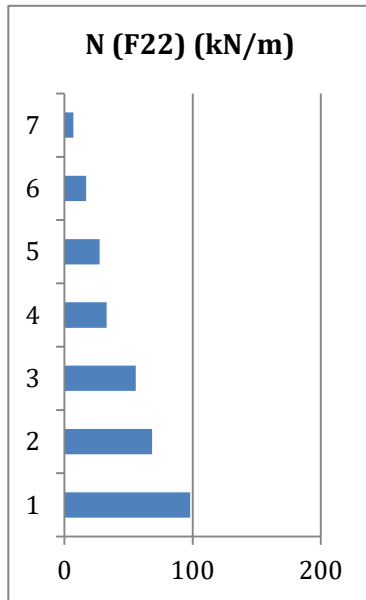
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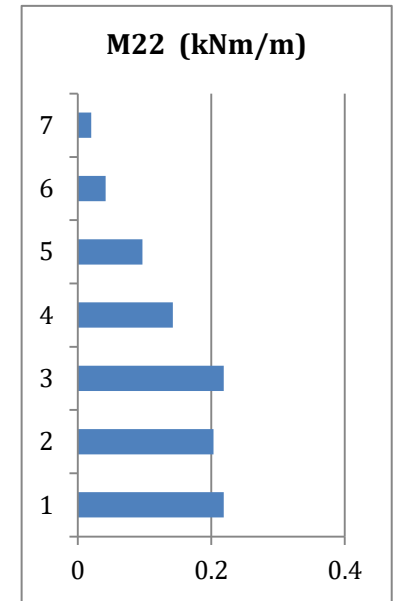
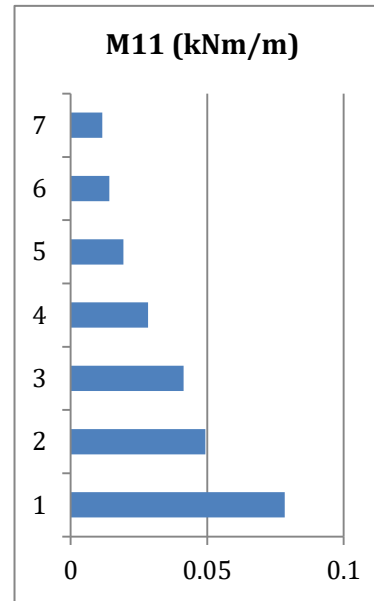
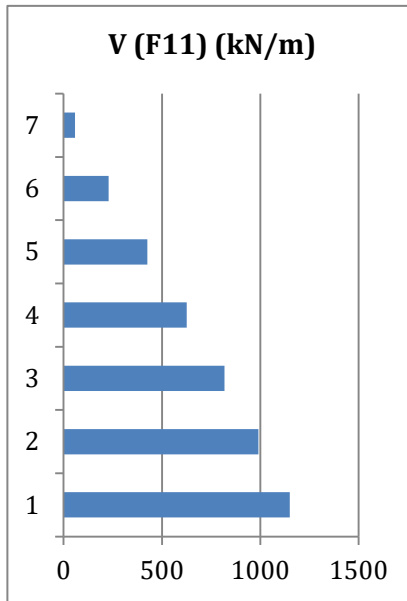
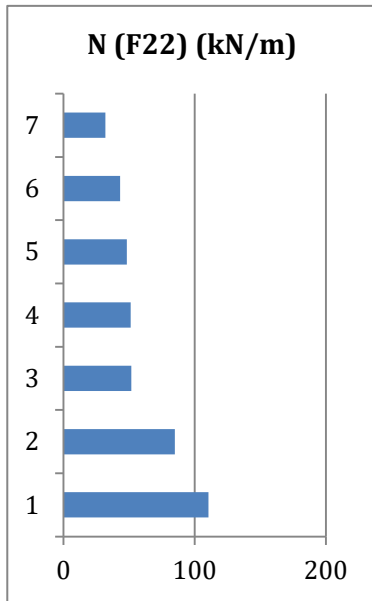
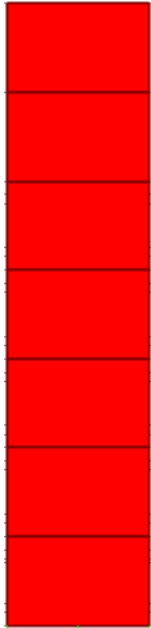
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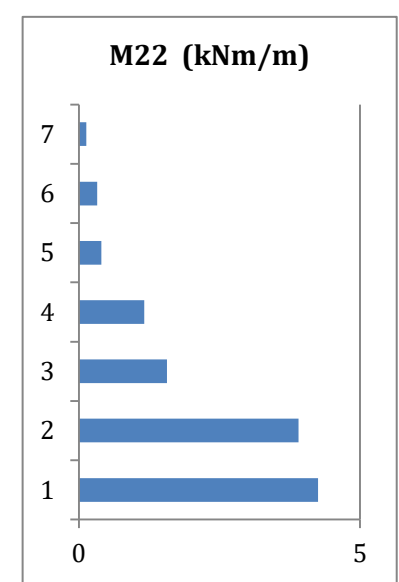
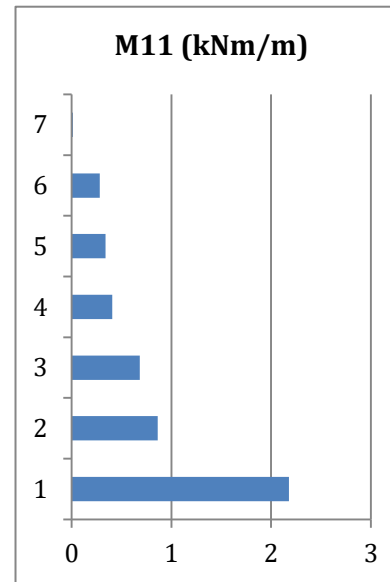
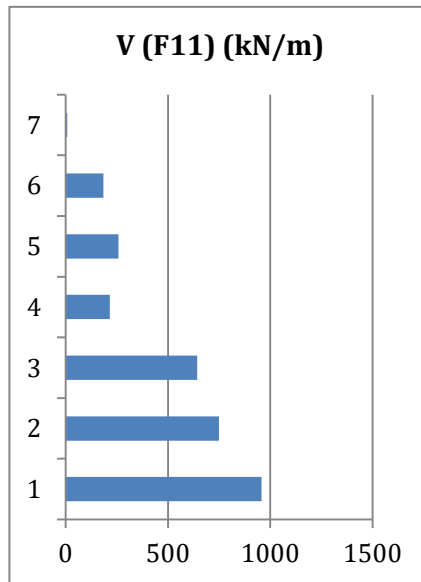
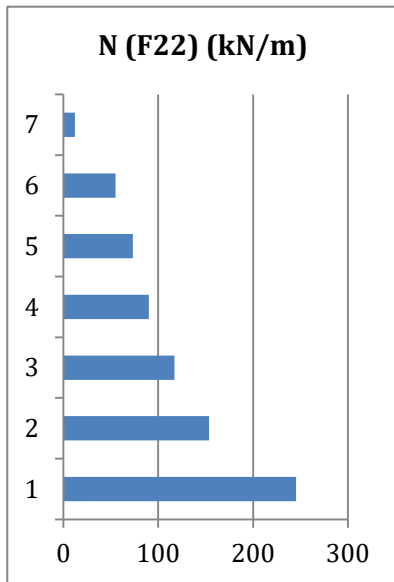
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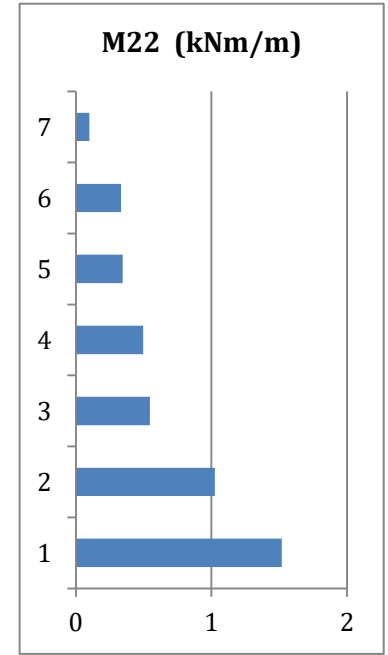
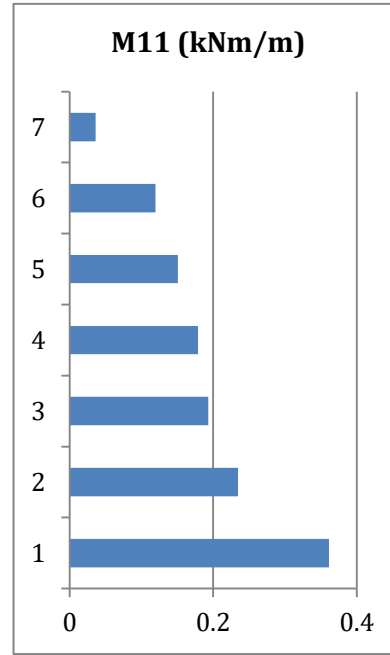
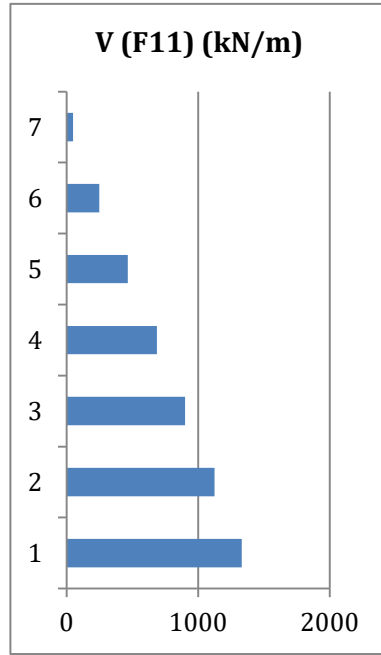
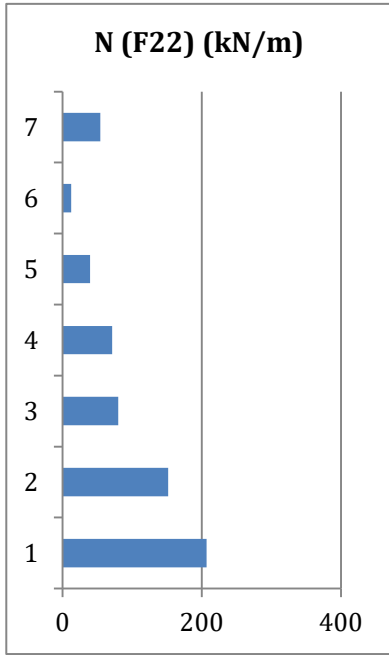
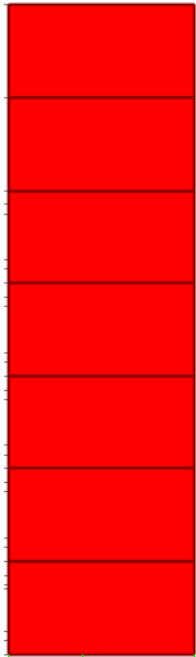
Q 3-5



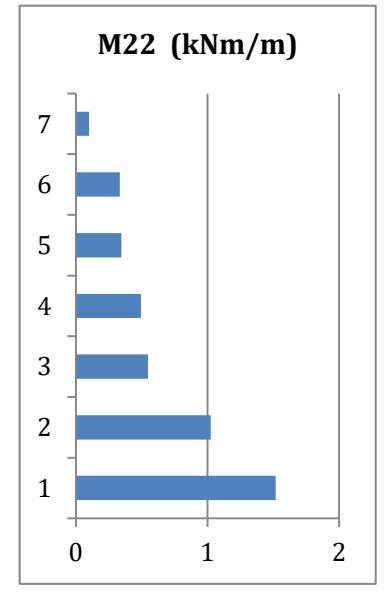
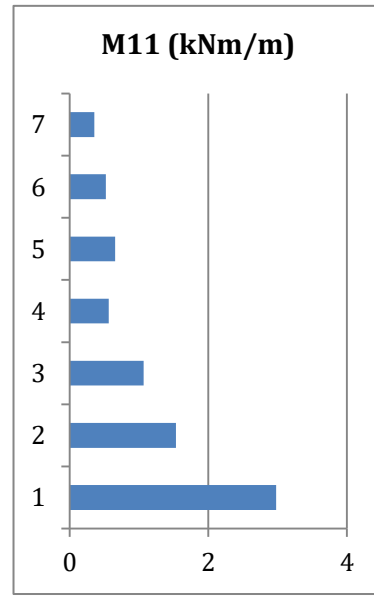
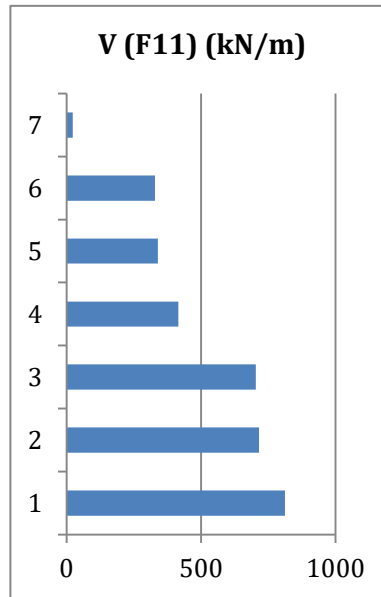
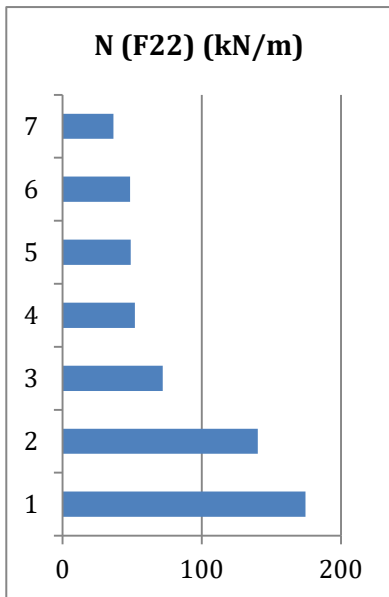
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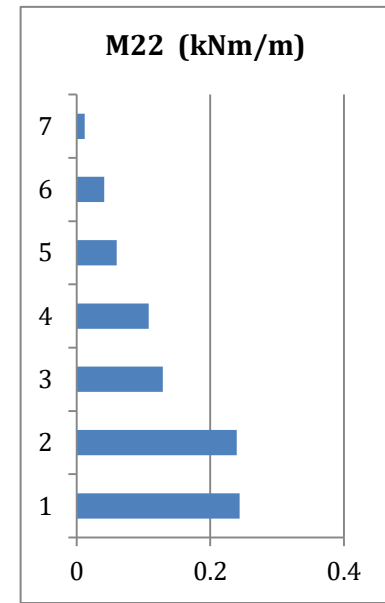
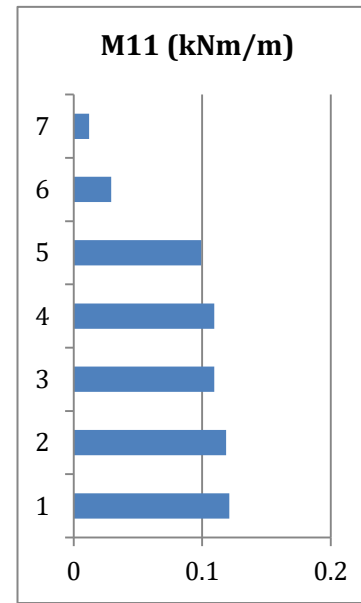
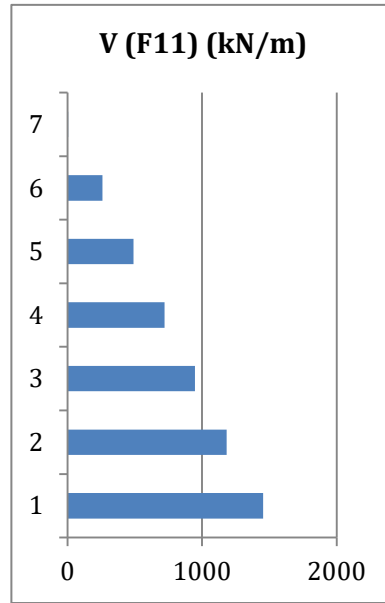
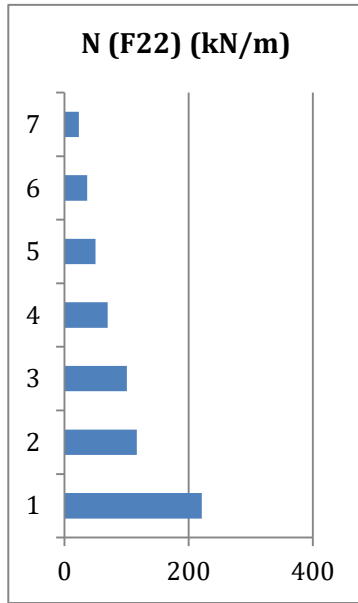
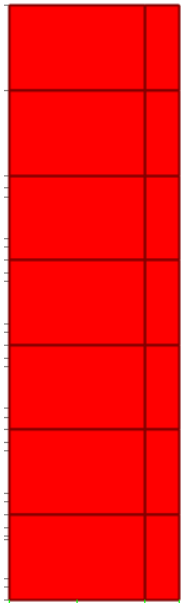
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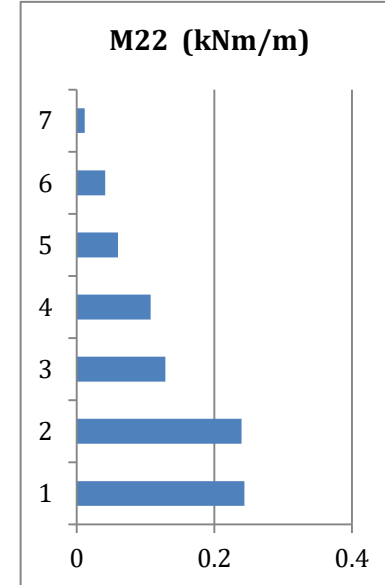
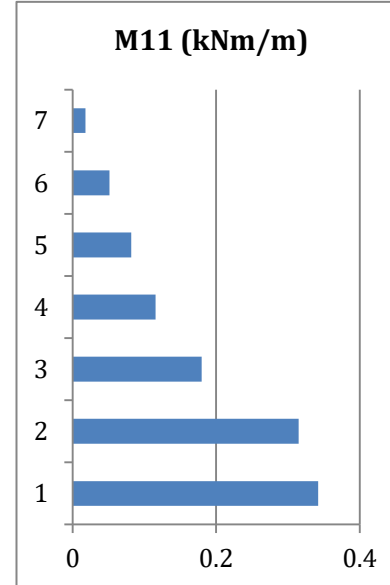
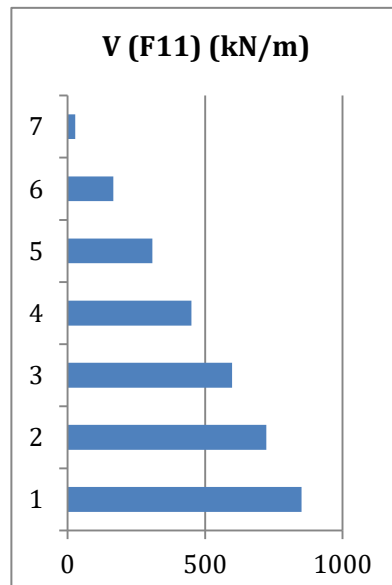
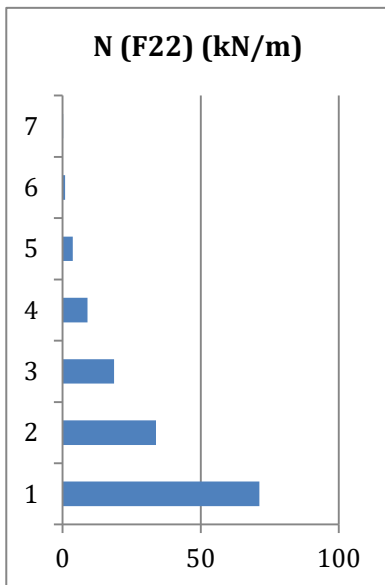
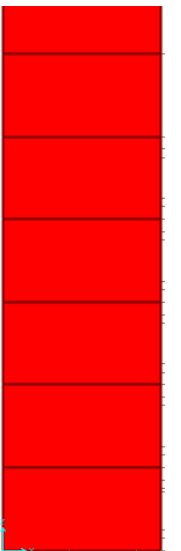
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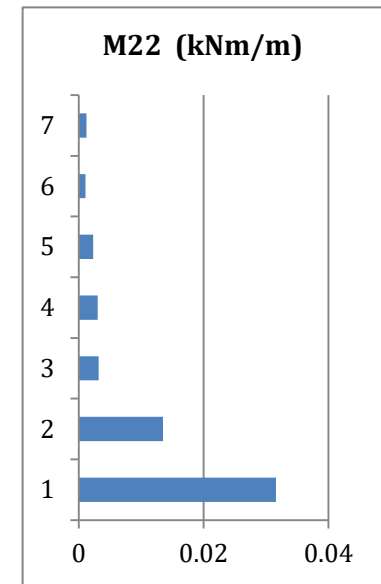
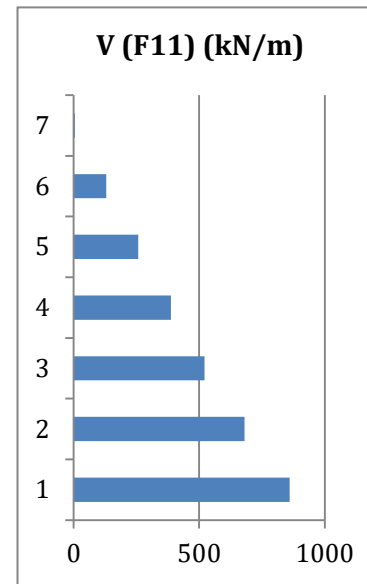
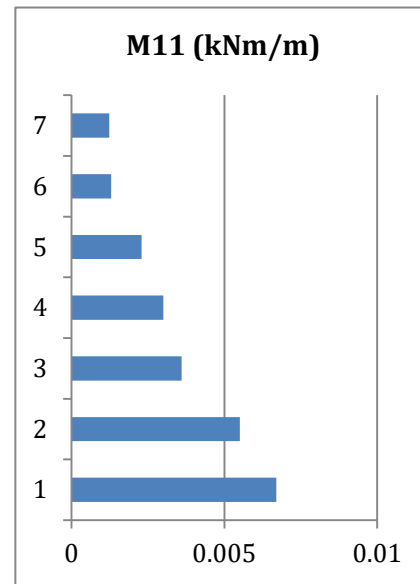
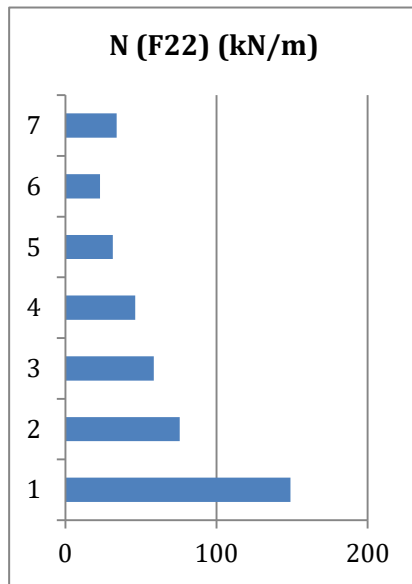
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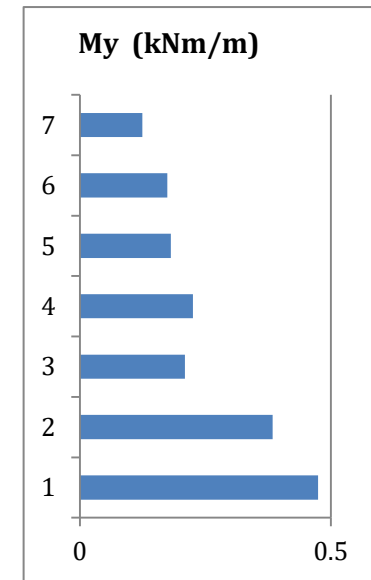
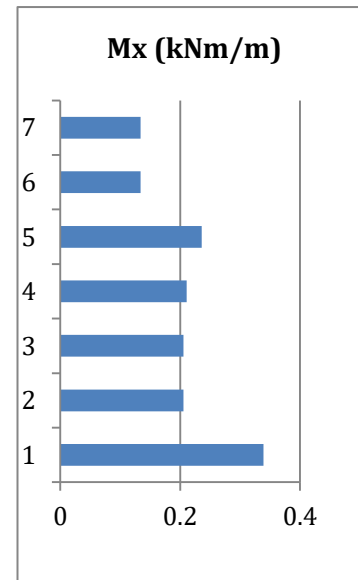
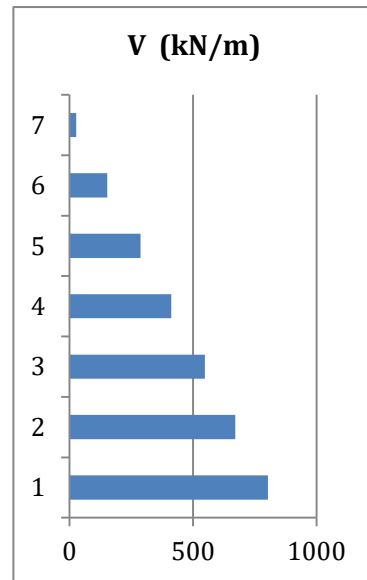
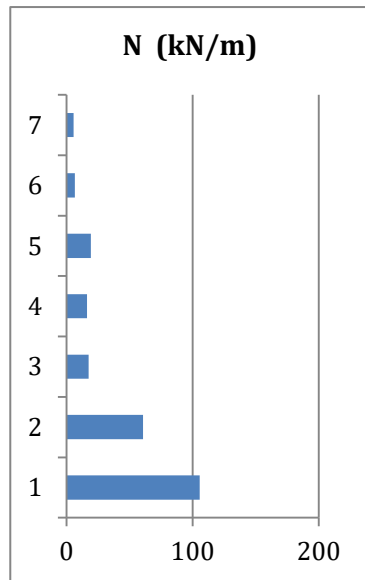
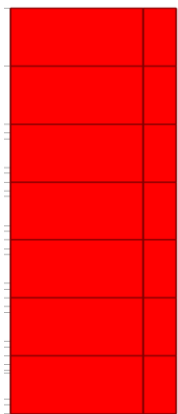
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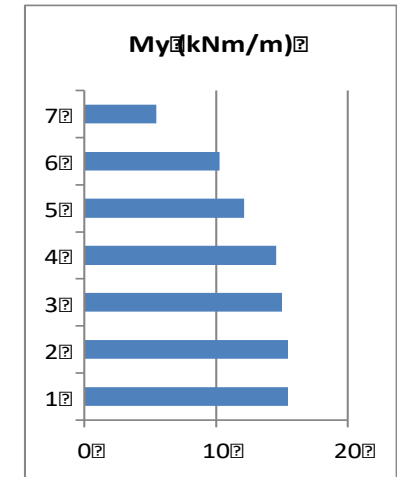
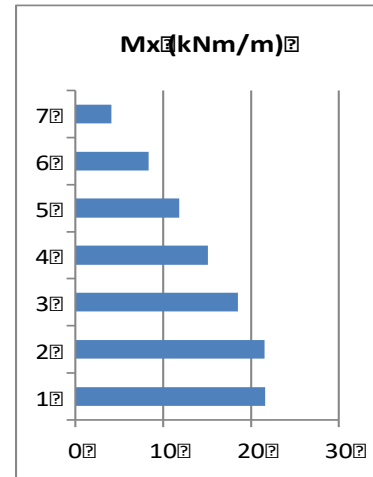
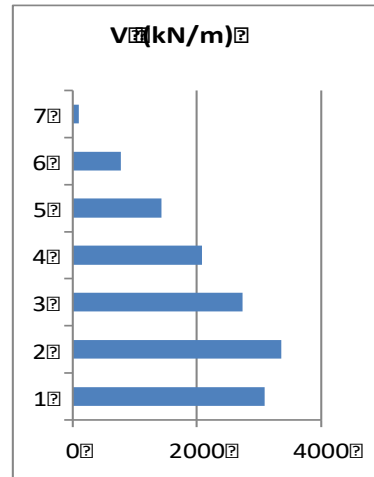
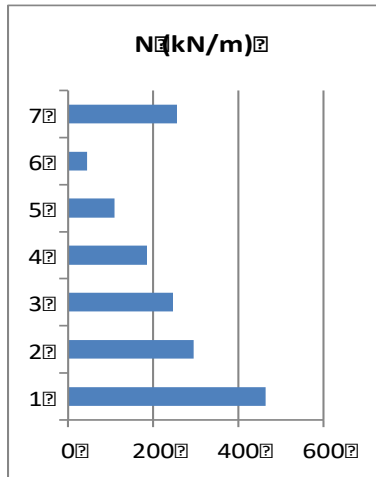
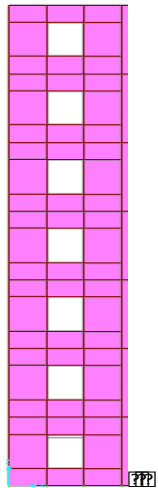
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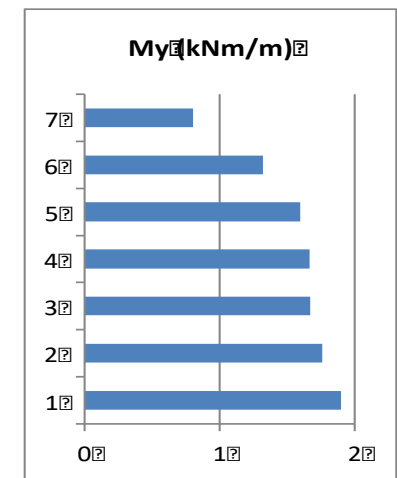
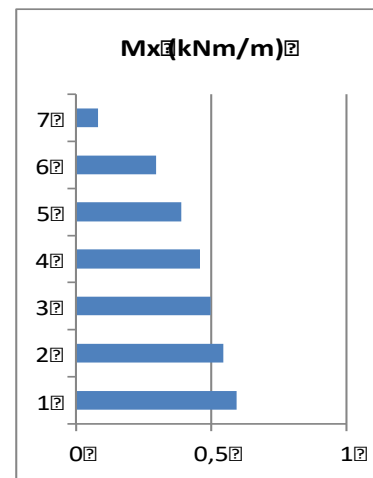
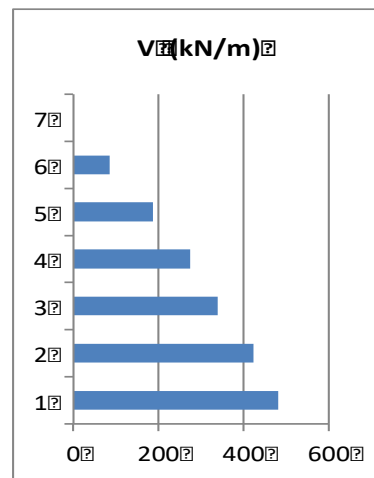
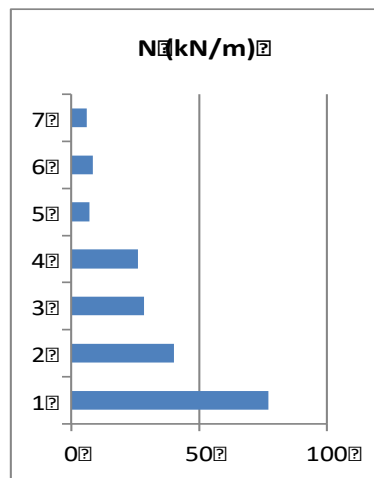
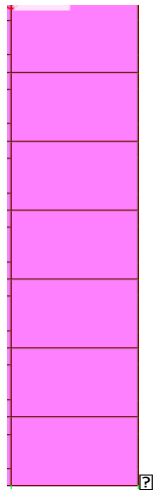
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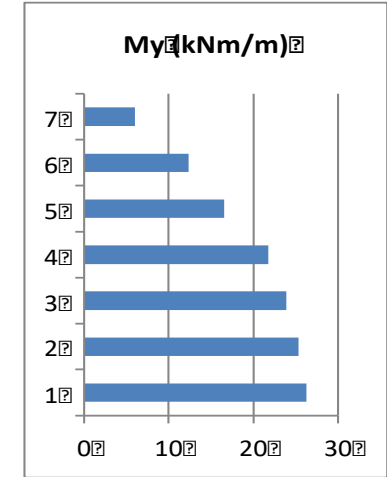
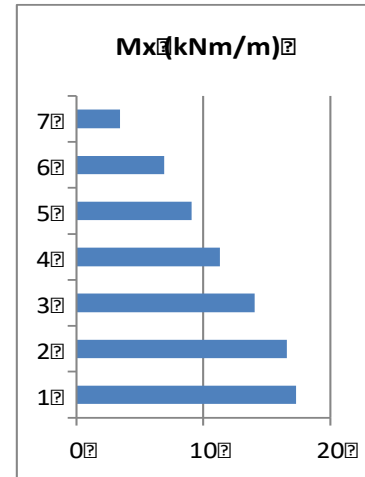
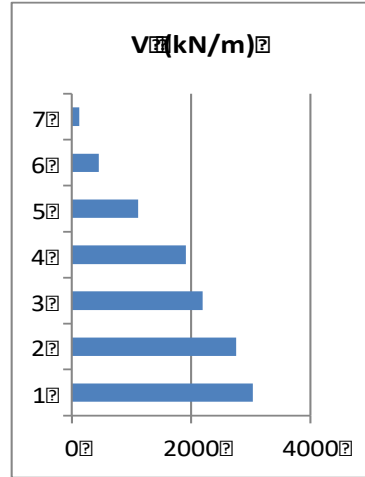
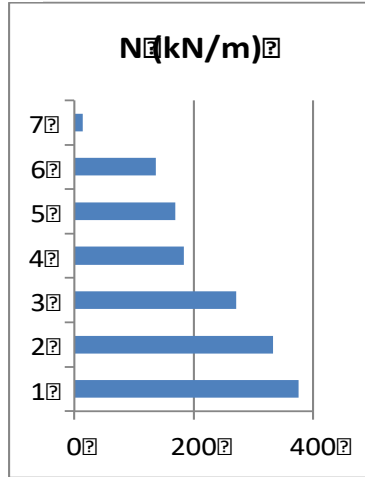
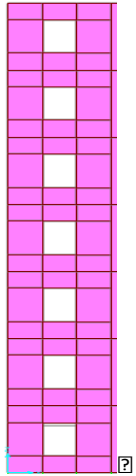
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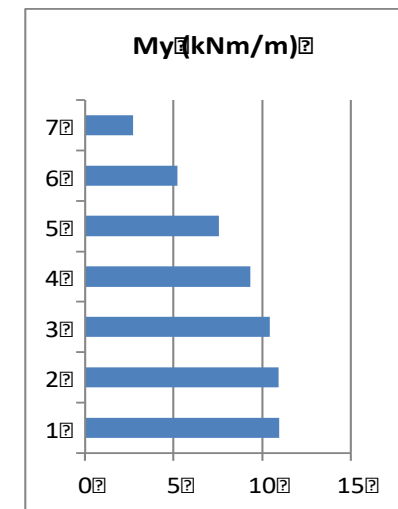
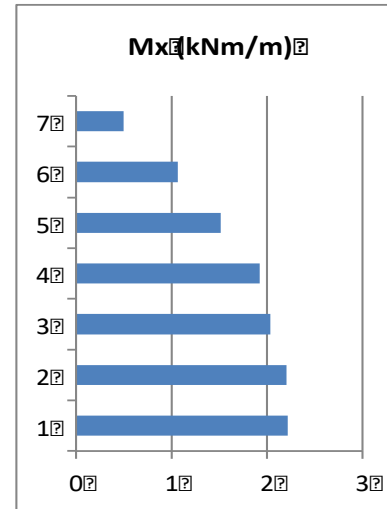
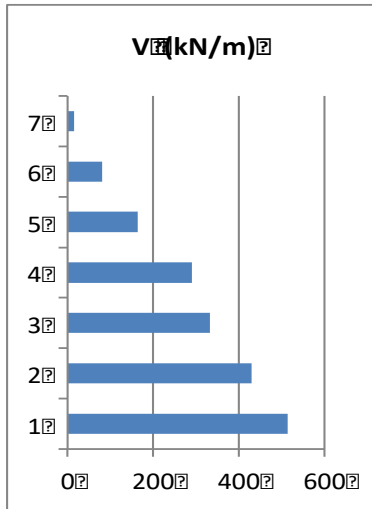
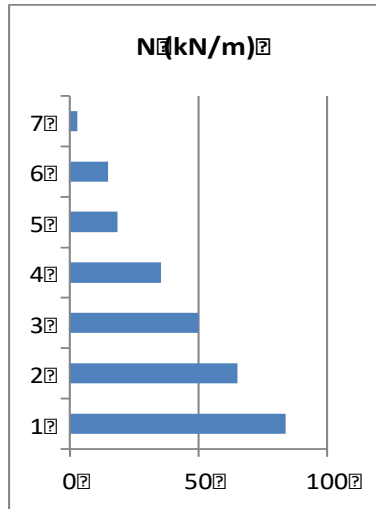
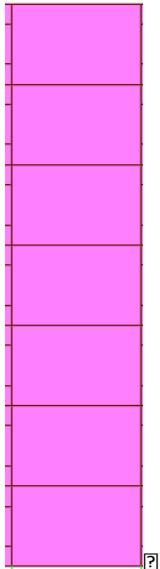
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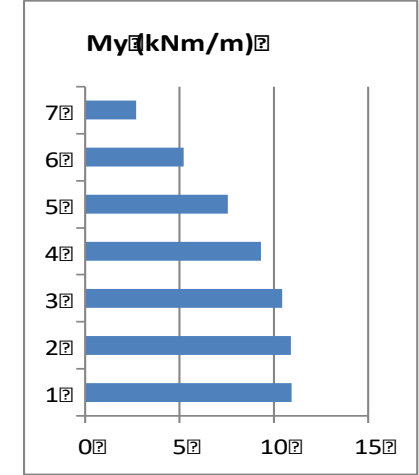
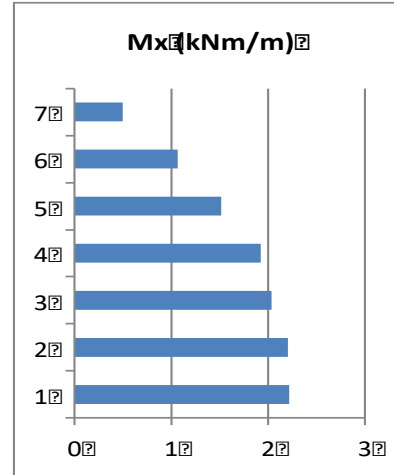
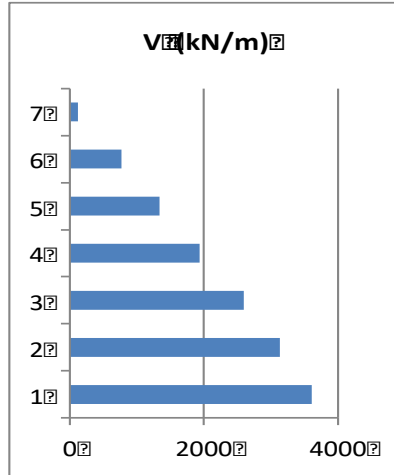
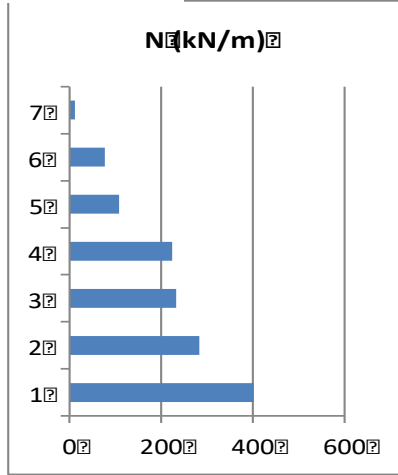
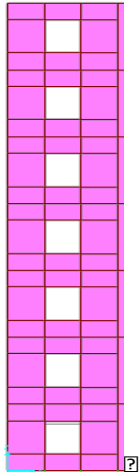
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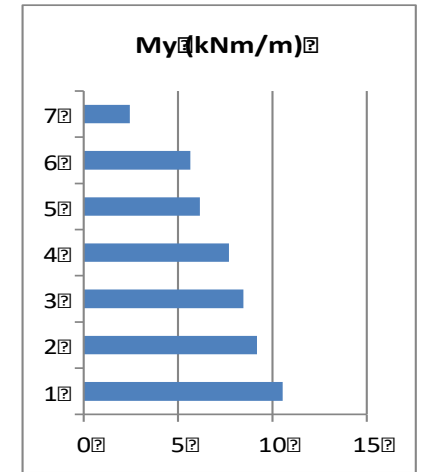
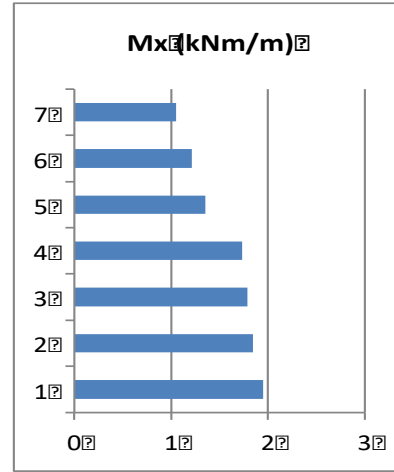
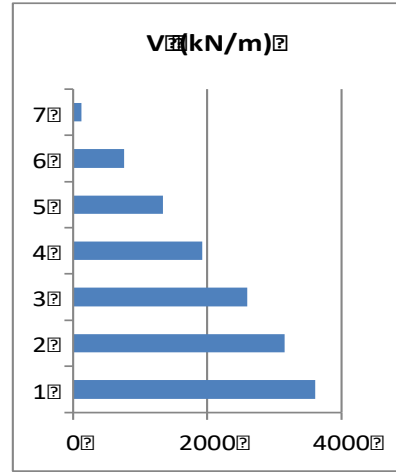
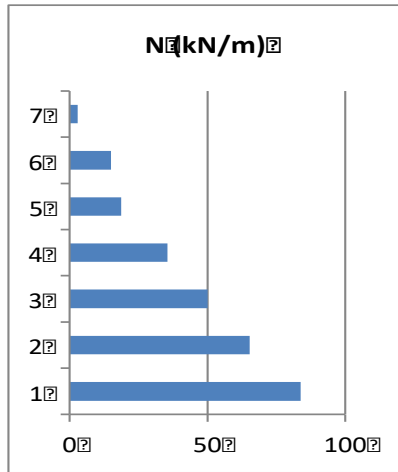
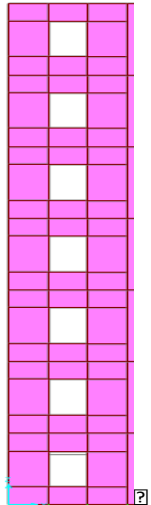
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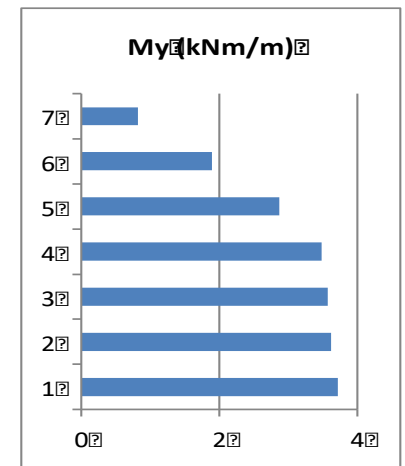
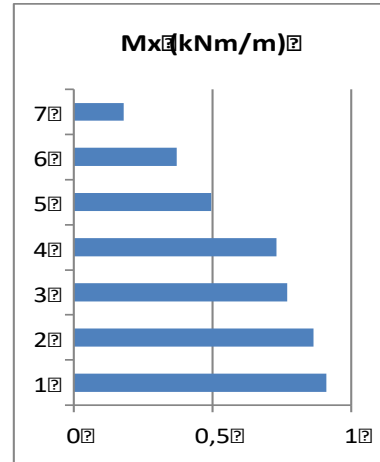
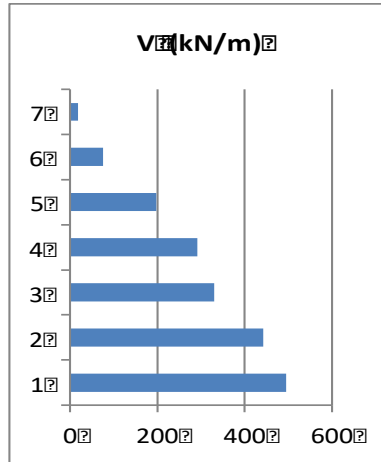
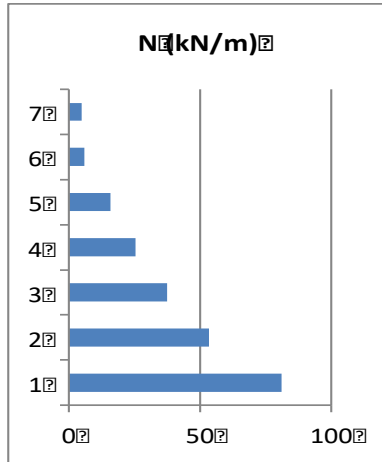
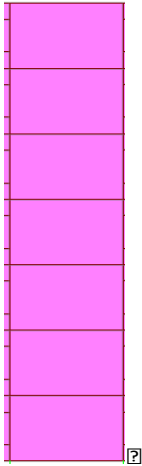
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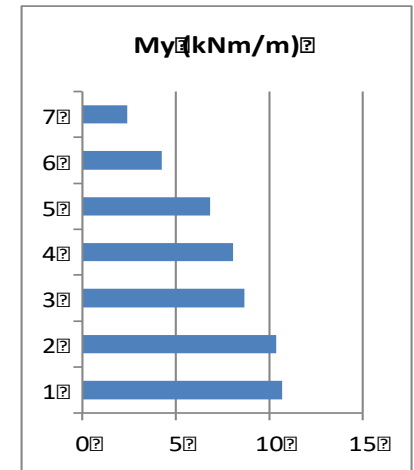
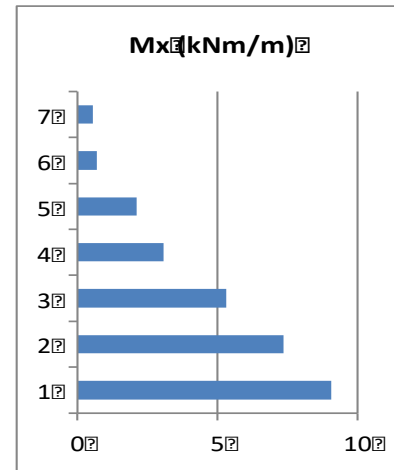
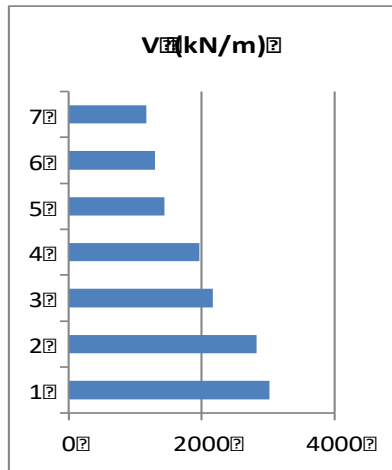
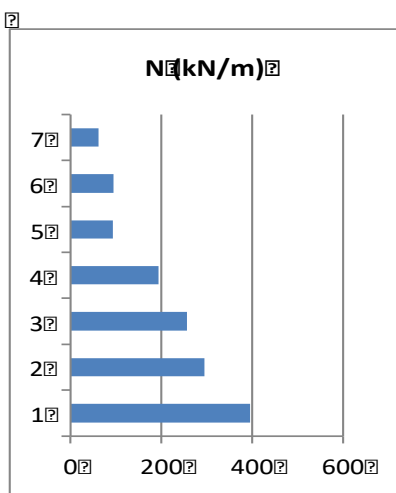
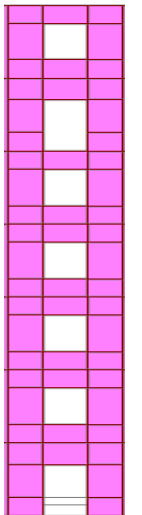
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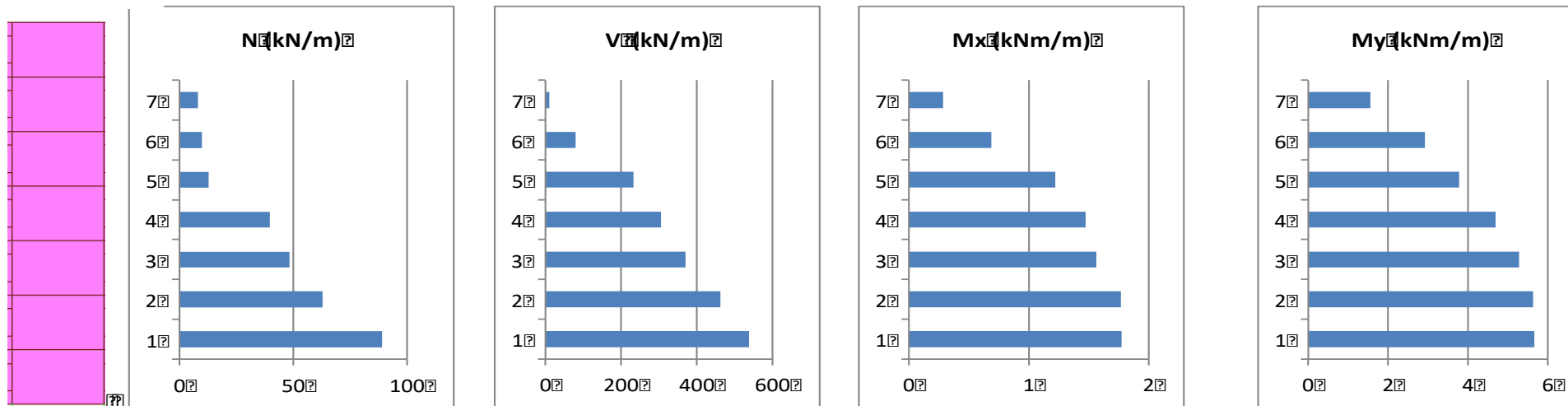
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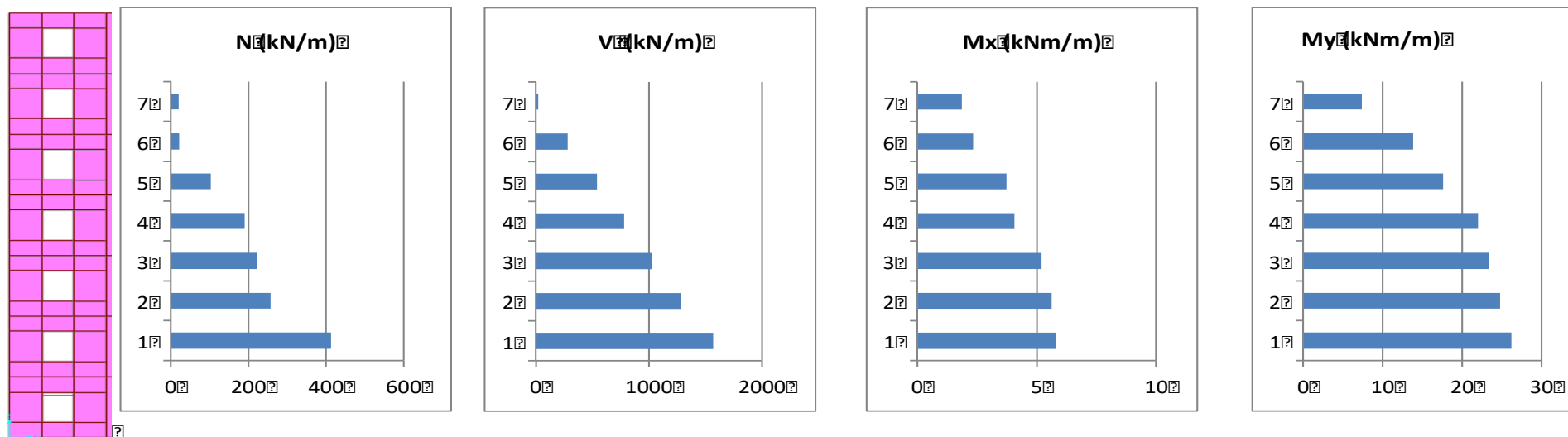
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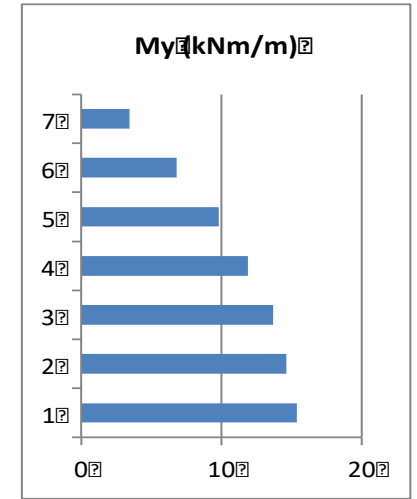
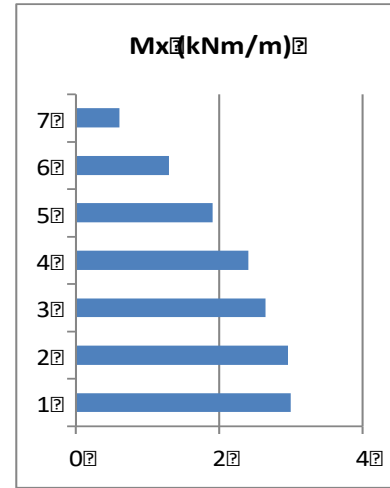
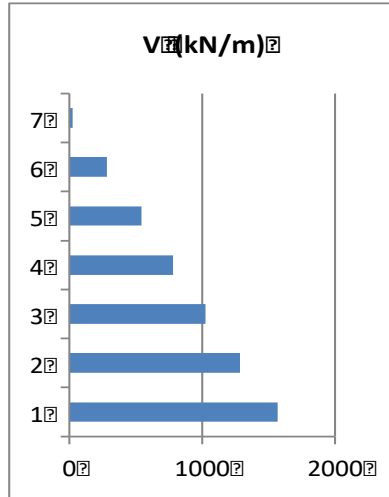
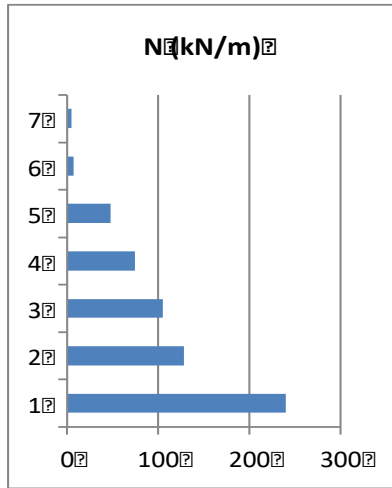
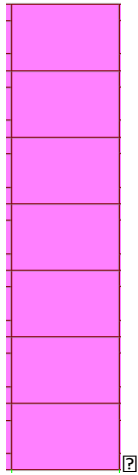
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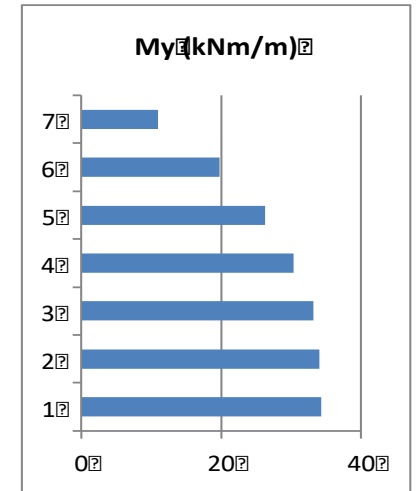
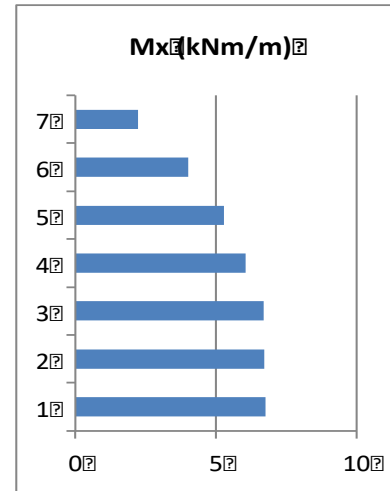
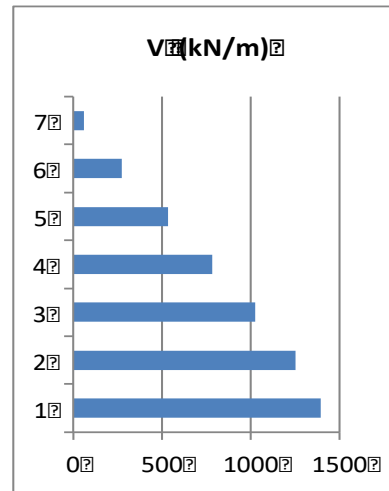
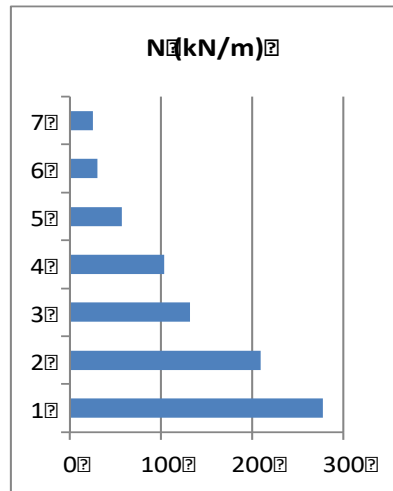
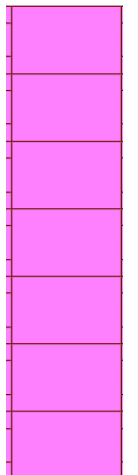
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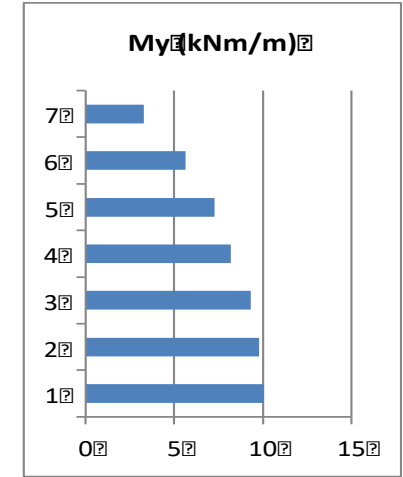
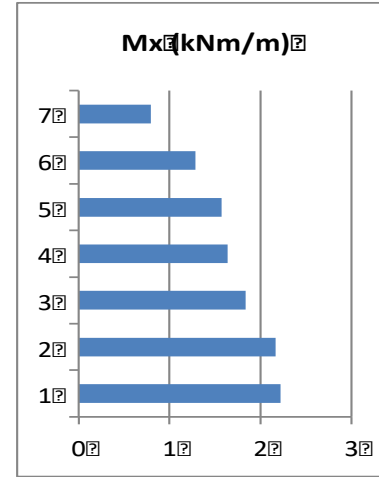
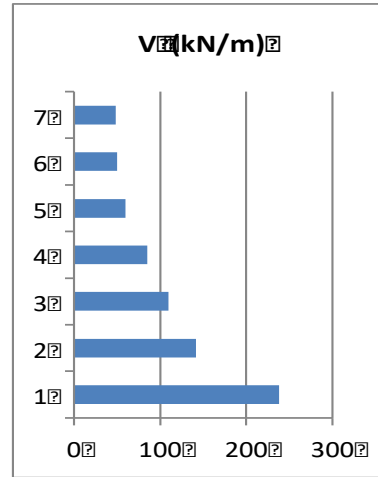
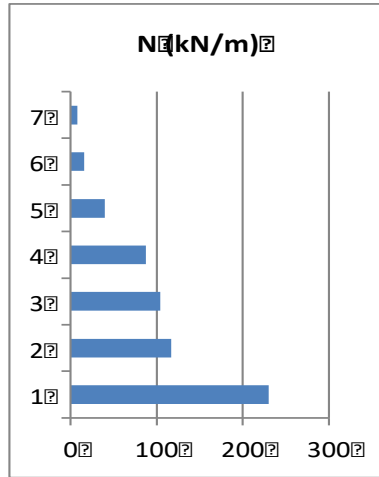
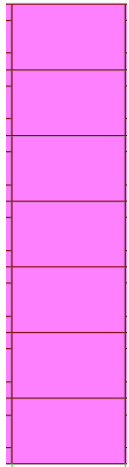
3E-E



3E-I

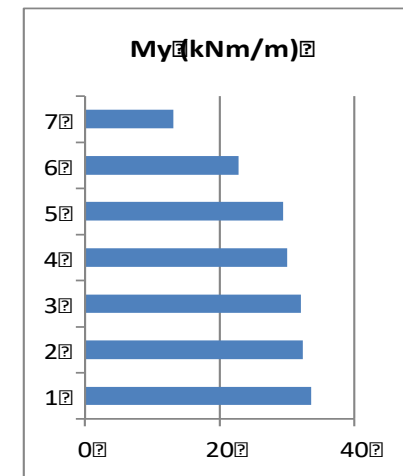
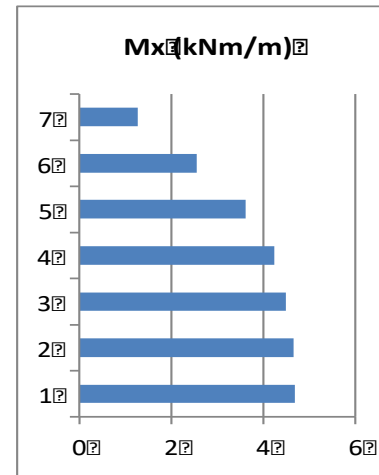
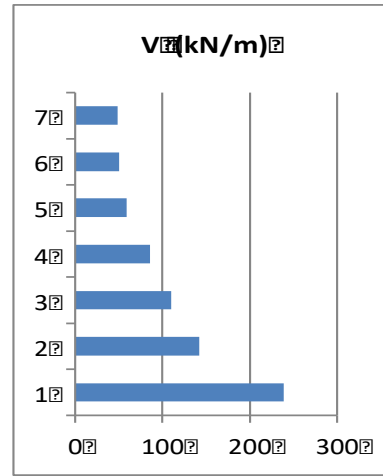
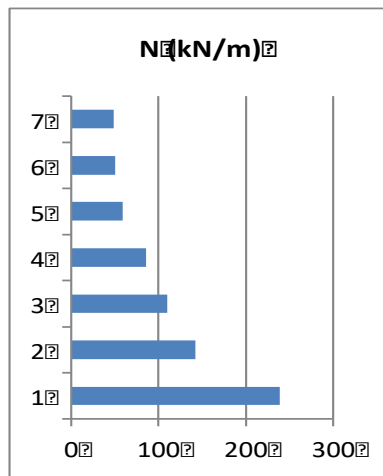
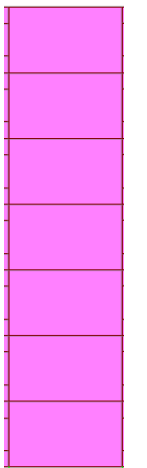


3E-J

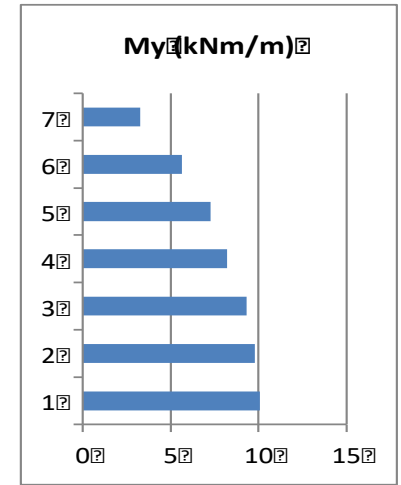
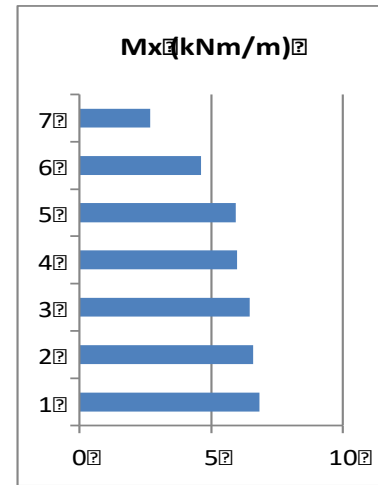
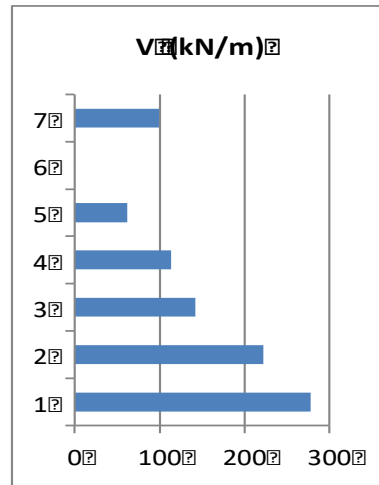
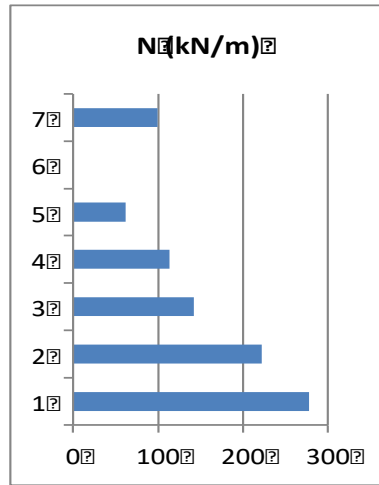
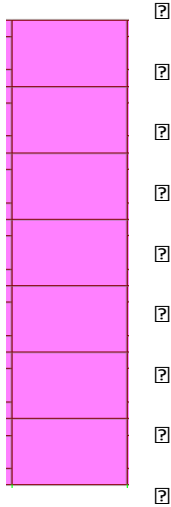


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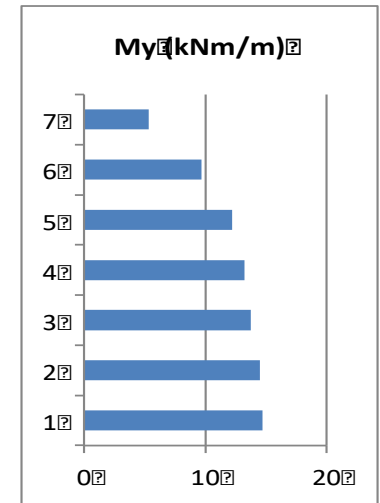
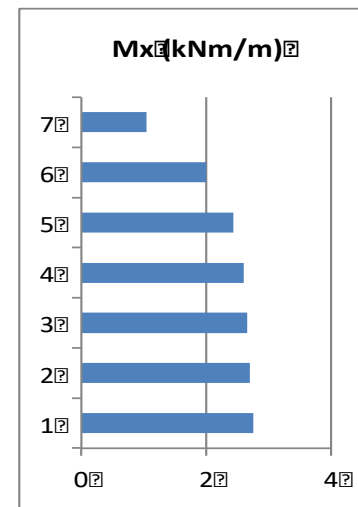
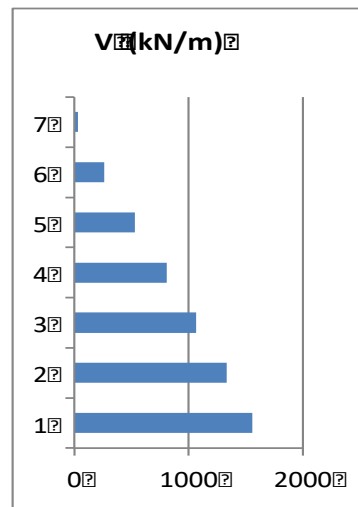
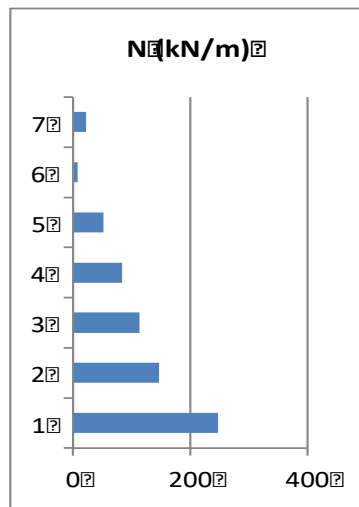
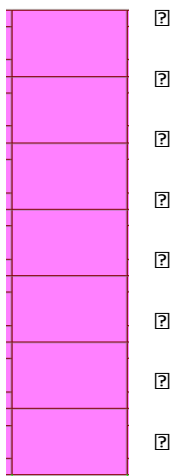
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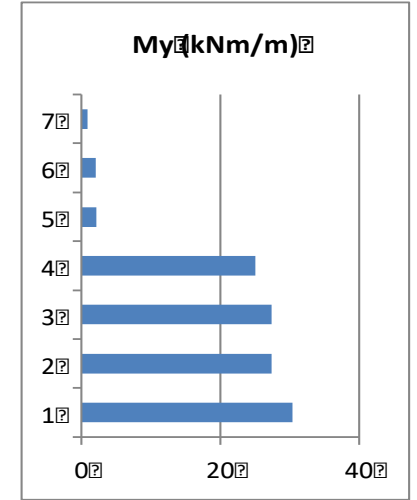
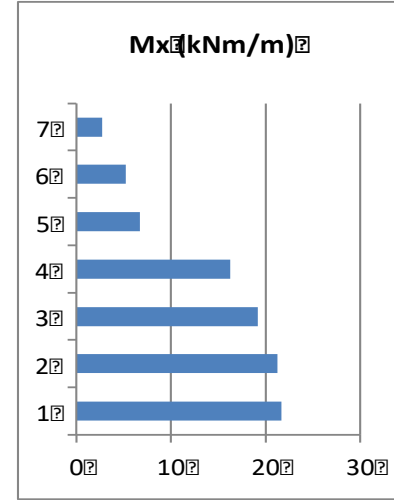
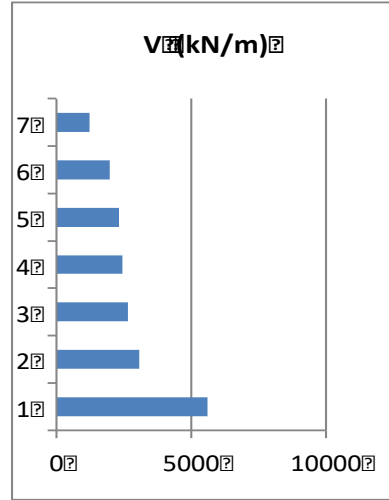
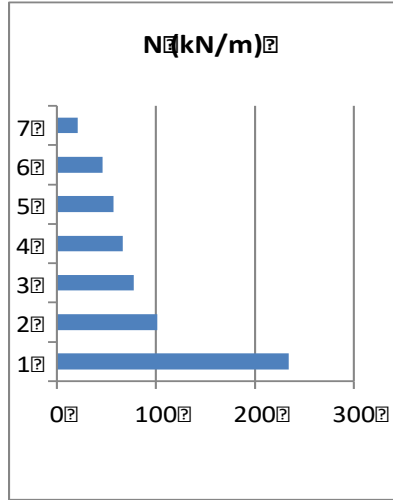
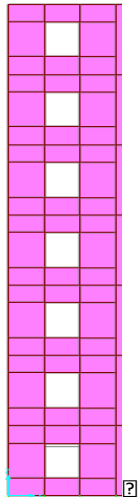
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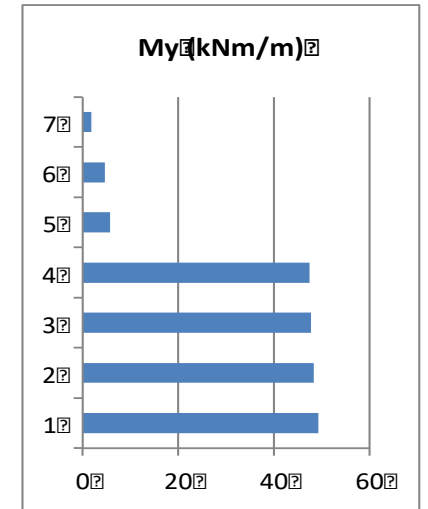
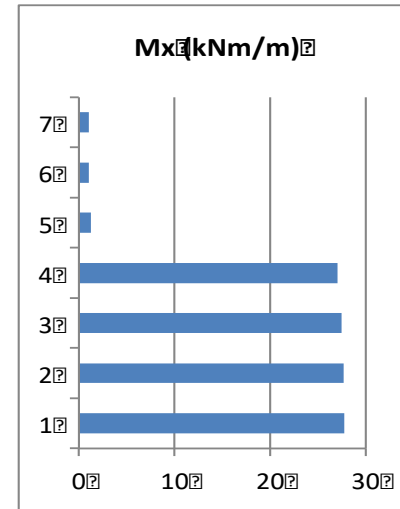
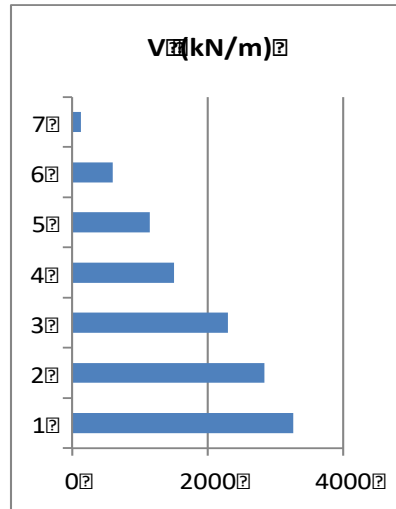
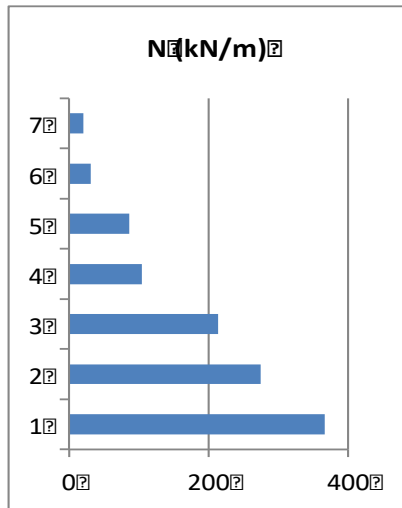
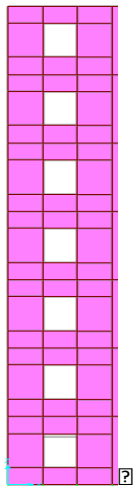
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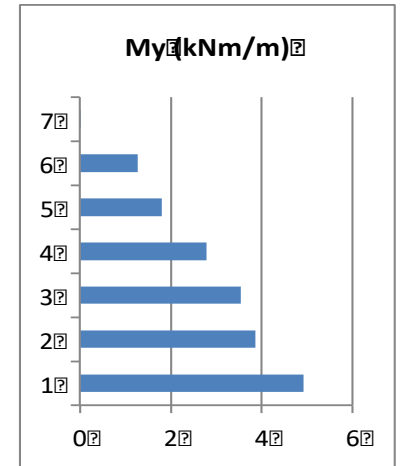
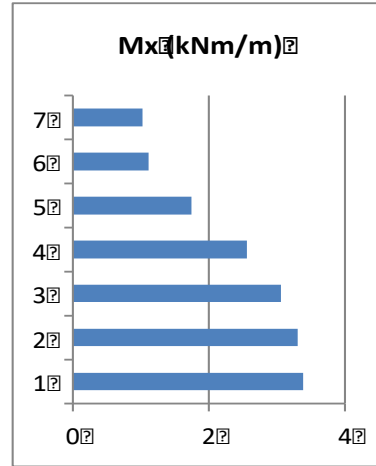
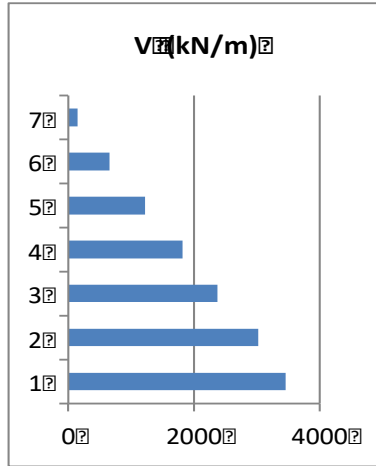
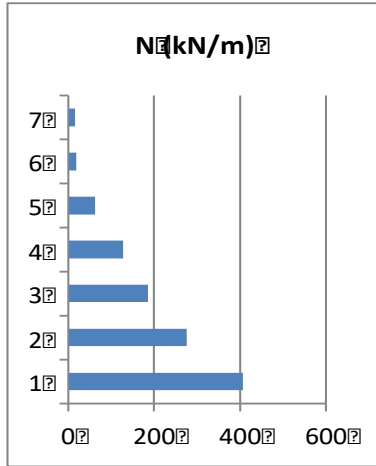
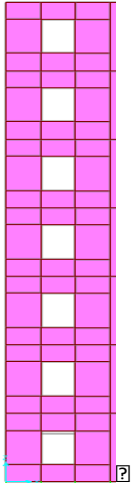
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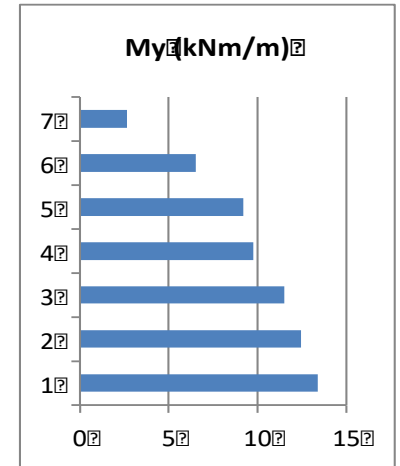
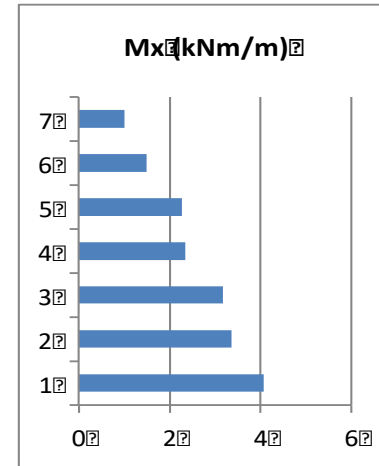
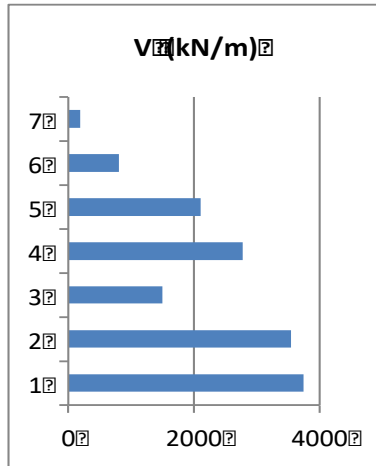
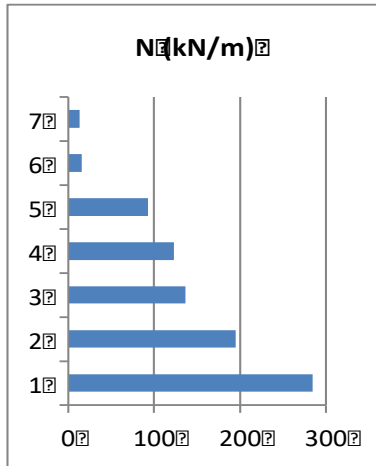
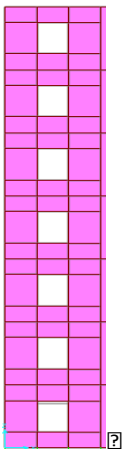
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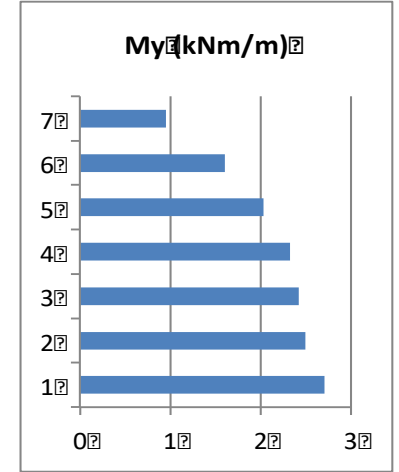
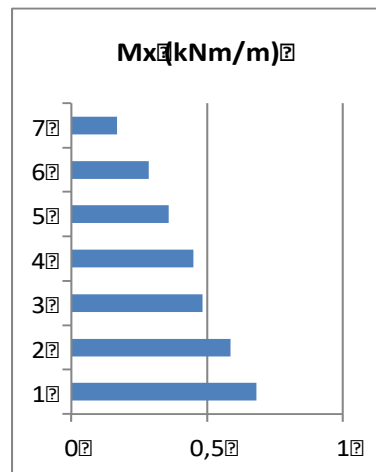
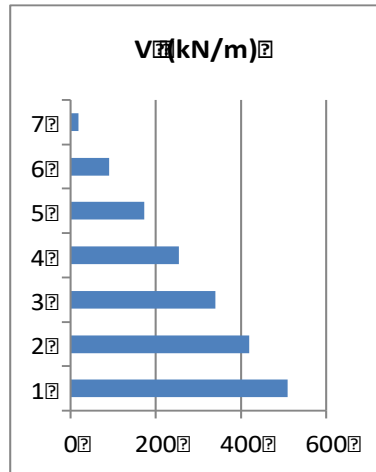
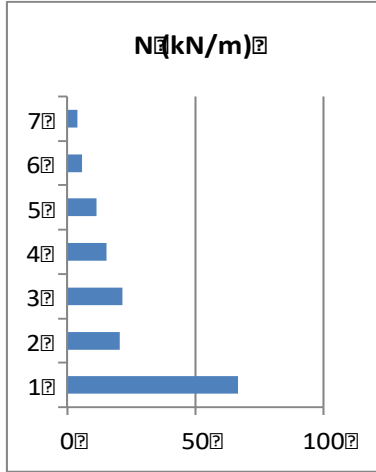
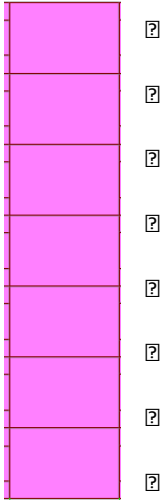
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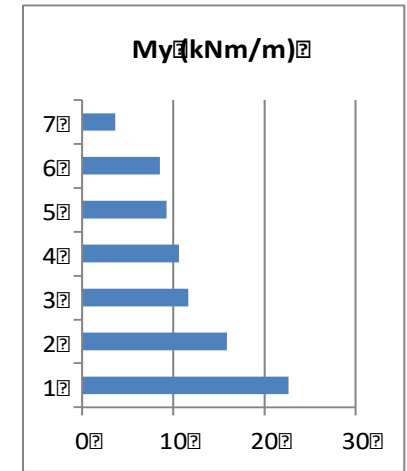
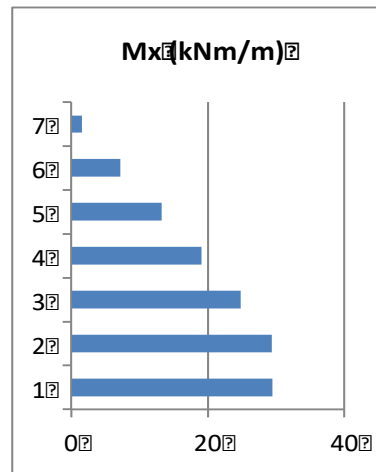
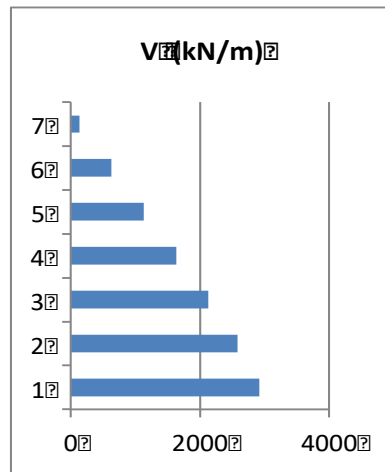
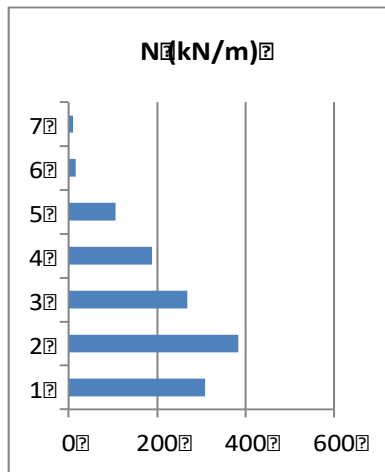
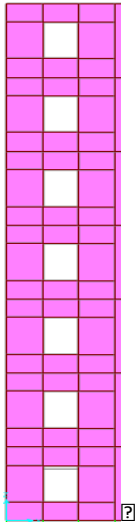
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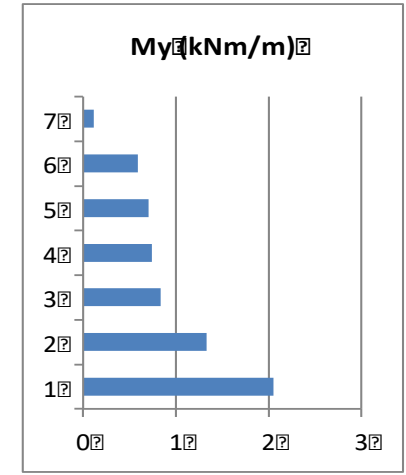
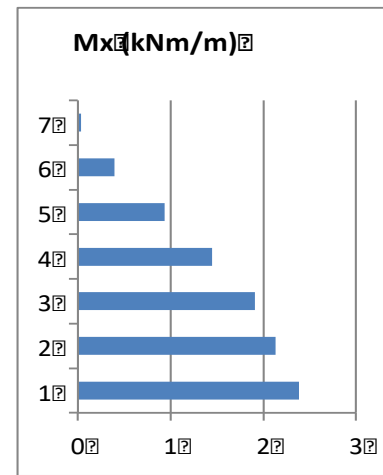
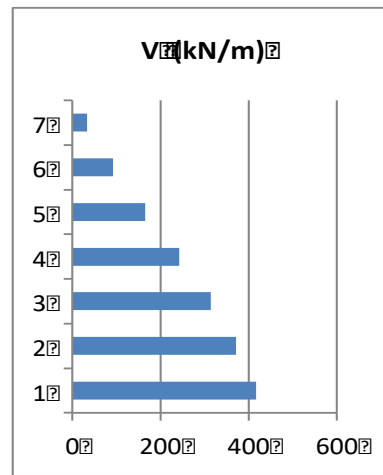
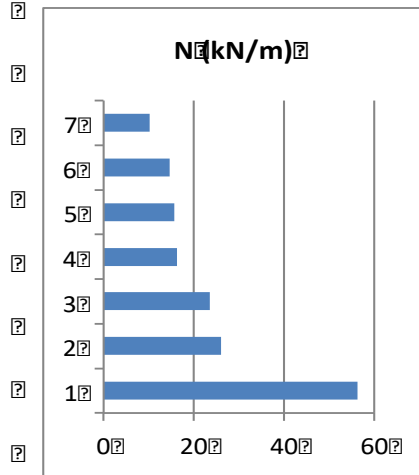
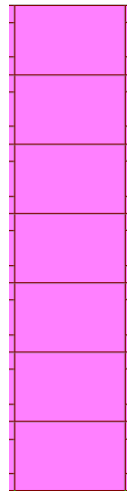
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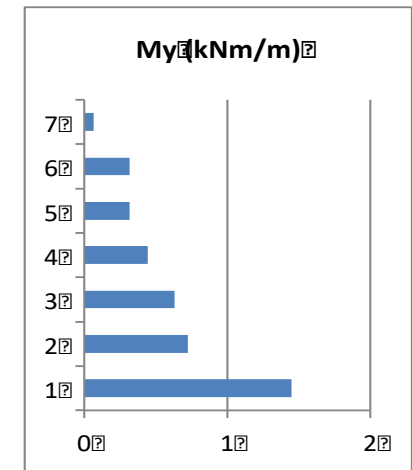
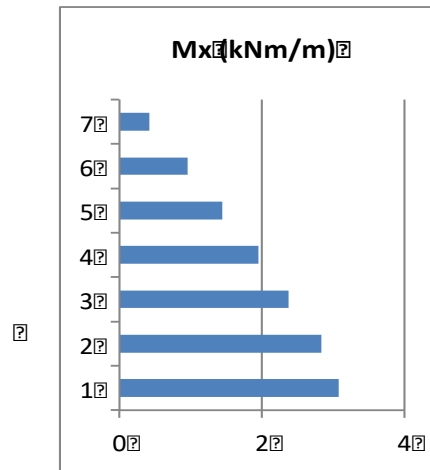
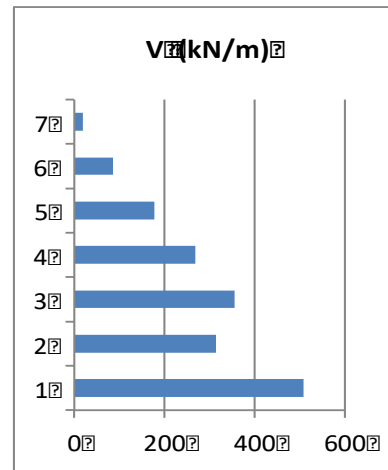
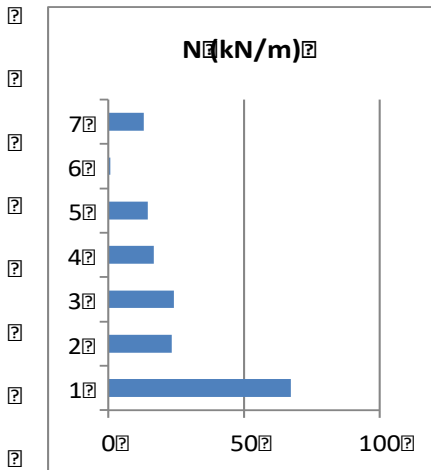
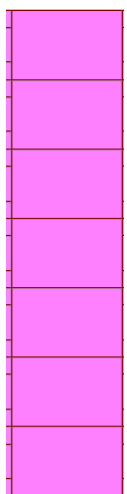
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5H-J

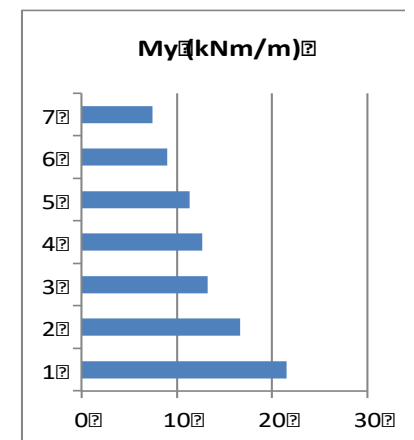
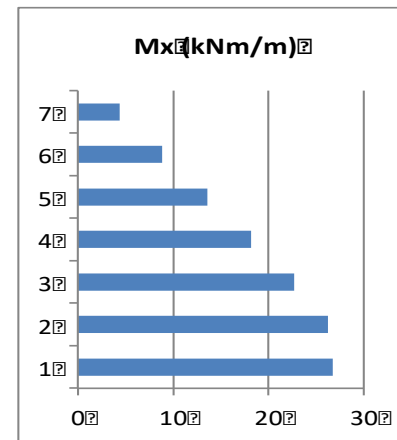
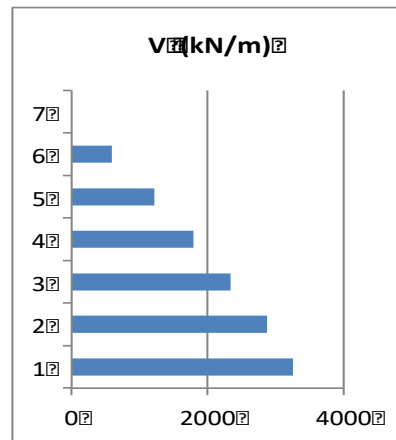
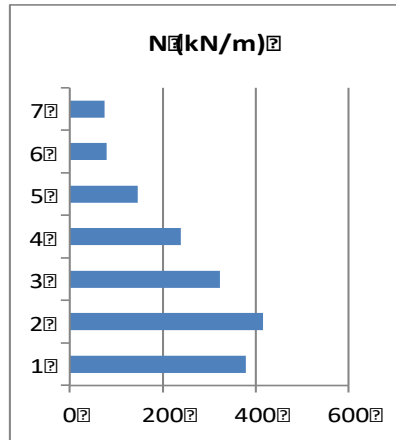
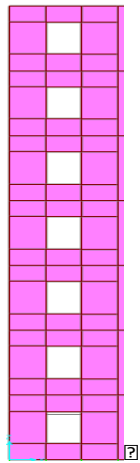


5P-R



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5R-U



5U-V

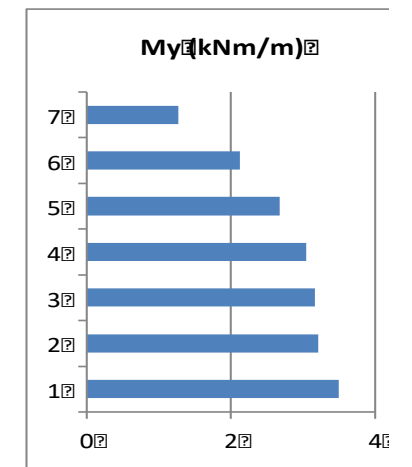
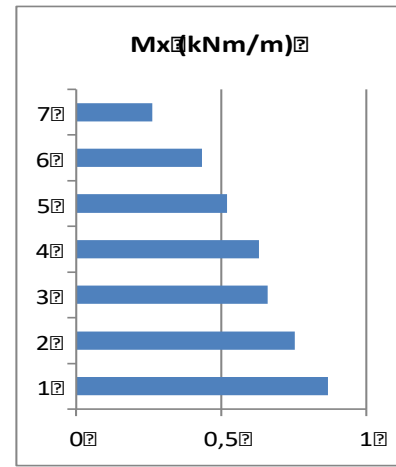
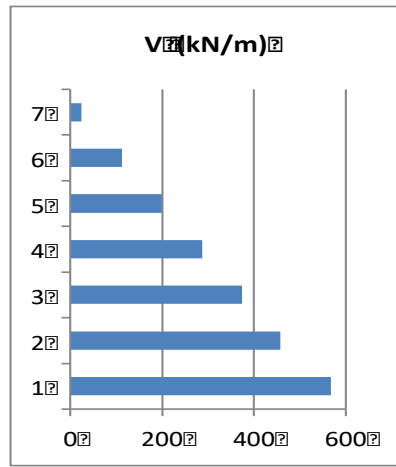
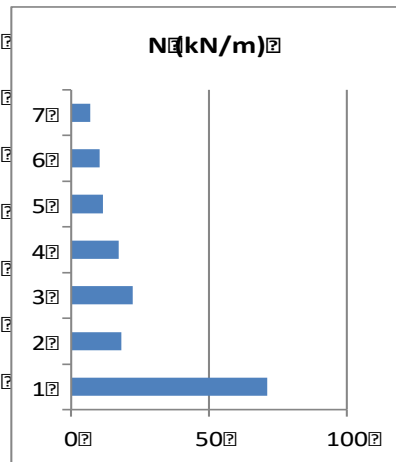
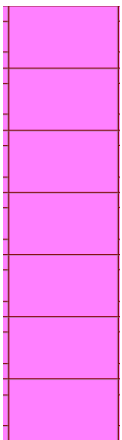


Figure 1. Internal forces (M, N, V) for walls

Table 9. Soil and interface properties.

Parameter	Name	Loam	Medium size sand	Coarse sand	Gravelly sand	Gravel	Loam	Unit
General								
Material model	<i>Model</i>	Hardening soil	Hardening soil	Hardening soil	Hardening soil	Hardening soil	Hardening soil	-
Type of material behaviour	<i>Type</i>	Drained	Drained	Drained	Drained	Drained	Drained	-
Soil unit weight above phreatic level	γ_{unsat}	18.7	19.2	19.7	20	20.5	19.3	kN/m ³
Soil unit weight below phreatic level	γ_{sat}	20.9	21.5	21.9	22	22	21.3	kN/m ³
Parameters								
Secant stiffness in standard drained triaxial test	E_{50}^{ref}	1.2 · 10 ⁴	1.7 x 10 ⁴	2.1 x 10 ⁴	2.1 x 10 ⁴	4.0 x 10 ⁴	1.35 x 10 ⁴	kN/m ²
Tangent stiffness for primary oedometer loading	E_{oed}^{ref}	8.0 · 10 ³	1.7 x 10 ⁴	2.1 x 10 ⁴	2.1 x 10 ⁴	4.0 x 10 ⁴	8.0 · 10 ³	kN/m ²
Unloading / reloading stiffness	E_{ur}^{ref}	3.6 · 10 ⁴	5.1 x 10 ⁴	6.3 x 10 ⁴	6.3 x 10 ⁴	1.27 x 10 ⁵	4.05 x 10 ⁴	kN/m ²
Power for stress-level dependency of stiffness	<i>m</i>	0.8	0.5	0.5	0.5	0.5	0.8	-
Cohesion	c_{ref}'	18	0.0	1.0	1.0	0.0	18	kN/m ²
Friction angle	φ'	22	35	38	38	28	22	°
Dilatancy angle	ψ	0.0	4.0	0.0	0.0	0.0	0.0	°
Poisson's ratio	ν_{ur}'	0.2	0.2	0.2	0.2	0.2	0.2	-
Groundwater								
Permeability in horizontal direction	k_x	0.6	4.23	9.2	12.95	23.9	0.6	m/day
Permeability in vertical direction	k_y	0.6	4.23	9.2	12.95	23.9	0.6	m/day
Interfaces								
Interface strength	—	Rigid	Rigid	Rigid	Rigid	Rigid	Rigid	-
Strength reduction factor inter.	R_{inter}	1.0	1.0	1.0	1.0	1.0	1.0	-
Consider gap closure	—	Yes	Yes	Yes	Yes	Yes	Yes	-
Initial								
K_0 determination	—	Automatic	Automatic	Automatic	Automatic	Automatic	Automatic	-
Over-consolidation ratio	<i>OCR</i>	1.0	1.0	1.0	1.0	1.0	1.0	-
Pre-overburden pressure	<i>POP</i>	25.0	0.0	0.0	0.0	0.0	25.0	kN/m ²

Table 10. Material properties of building (plate properties).

Parameter	Name	Basement wall	Basement floor/slab	Footing	Unit
Material type	Type	Elastic; Isotropic	Elastic; Isotropic	Elastic; Isotropic	-
Normal stiffness	<i>EA</i>	9.0 · 10 ⁶	1.2 · 10 ⁷	1.2 · 10 ⁷	kN/m
Flexural rigidity	<i>EI</i>	6.75 · 10 ⁴	1.6 · 10 ⁵	1.6 · 10 ⁵	kNm ² /m
Weight	<i>w</i>	10	20	20	kN/m/m
Poisson's ratio	ν	0.0	0.0	0.0	-

Table 11. Material properties of column (plate properties).

Parameter	Name	Column	Unit
Material type	<i>Type</i>	Elastic	-
Normal stiffness	<i>EA</i>	2.5 · 10 ⁶	kN
Spacing out of plane	<i>L_{spacing}</i>	3.0	m

Table 12. Material properties of micropiles.

Parameter	Name	Pile foundation	Unit
Young's modulus	<i>E</i>	3 · 10 ⁷	kN/m ²
Unit weight	γ	6.0	kN/m ³
Beam type	-	<i>Predefined</i>	—
Predefined beam type	-	Massive circular beam	—
Diameter	<i>Diameter</i>	1.5	m
Axial skin resistance	<i>Type</i>	<i>Linear</i>	—
Maximum traction allowed at the top of the embedded beam	$T_{skin,start,max}$	200	kN/m
Maximum traction allowed at the bottom of the embedded beam	$T_{skin,end,max}$	500	kN/m
Base resistance	F_{max}	1 · 10 ⁴	kN

Table 13. Material properties of retaining wall.

Parameter	Name	Value	Unit
Material type	<i>Type</i>	Elastic; Isotropic	-
Prevent punching	-	Yes	-
Normal stiffness	<i>EA</i>	$1.2 \cdot 10^7$	kN/m
Flexural rigidity	<i>EI</i>	$1.2 \cdot 10^5$	kNm ² /m
Weight	<i>w</i>	8.3	kN/m/m
Poisson's ratio	ν	0.15	-

Table 14. Material properties of strut.

Parameter	Name	Value	Unit
Type of behaviour	<i>Material type</i>	Elastic	-
Normal stiffness	<i>EA</i>	$2 \cdot 10^6$	kN
Spacing out of plane	<i>L_{spacing}</i>	5.0	m

Table 15. Maximum load per pile

Maximum load per pile				
	N/4+M1/1,1+M2/1,1	N/4+M1/1,1-M2/1,1	N/4+M2/1,1-M1/1,1	N/4-M2/1,1-M1/1,1
MAX	495,8710909	495,04945	502,8	502,0067

Table 16. Micropile Design

OD=	141	mm			
Wall thickness	9,5	mm			
ID=	122	mm			
Area of casing =	3922,645	mm ²			
F y-casing	241	MPa			
Reinforcing bar	32	mm			
Area of bar	803,84	mm ²			
F y-bar	520	MPa			
Grout Area	0	mm ²			
f' c-grout	34,5	MPa			
Fy-steel=min (F y-casing;F y-bar)	241	MPa			
P c-allow	535,368956	kN	>	502,8283636	kN
P t-allow	626,4955868	kN			
Grout Dia	191	mm			
Bond area of grout	27833,745	mm ²			
P c-allow	580,564177	kN	>	502,8283636	kN
P t-allow	229,89824	kN			
Type B					
Sand1 bond α	130	kPa	3,3-3=0,3	m	
Sand2 bond α	240	kPa		0,8	m
Gravel bond α	240	kPa		5	m
Loam Bond α	130	kPa			
SF	2				
Lb1*α	39				
Lb2*α	192				
Lb3*α	1200				
Lb4	-11,00769231				
Total Uncased Length	-4,91	m	Taking as 8 meters		
Total Cased Length	3	m			
P g-allow	503,571691	kN	>	502,8283636	kN

Table 17. Micropile Settlement

E steel	200000	MPa			
E grout	31000	MPa			
Grout Area (cased)	10880,1	mm ²	Bond area of grout	27833,745	mm ²
Area of steel (cased)	4726,485	mm ²	Area of bar	803,84	mm ²
Pile compression stiffness value					
EAcased	1282580,1	kN			
EA uncased	1023614,095	kN			
Elastic Displacement					
Cased L Δt	1,176133242	mm			
Uncased Δt	1,962686579	mm			

Table 18. Micropile to footing connection

P c-service	502,8283636			
f'c	35	MPa		
Pile Area	15606,585			
Plate width	254	mm		
Area of Plate	64516	mm ²		
Check Cone Shear	d2=	986,6809852	mm	
Diameter	286,6809852	287	mm	
Acp	699712,4027	mm ²		
Use ACI 349 Appendix B for cone shear				
P cone nominal strenght	1375,63	kN		
		5,916079783		
φ = 0.65 for unreinforced shear cone per ACI 349 Appendix B				
FS=1,53/0,65=2,35				
P cone allow	585,3744681	>	502,8283636	kN
Required Plate Thickness				
Bearing - comp	7,793855224	Mpa		
Mmax (10mm)	124,3996717	Nm		
Fy	345	MPa		
Fb	189,75	MPa		
Sx-req	655,5977428	mm ³		
t req	19,83327118	Taking as 20mm		

Table 19. Required welding size

Fu-welds	483	Mpa		
Minimum tensile strength of connected parts (ASTM A53 Grade B Casing)				
Fu-parts	414	MPa		
Fillet weld strength	111,78	MPa		
Top weld size, t weld	6,35	mm		
Stiffener plate size				
tstiff	12,7	mm		
Wstiff	100	mm		
Lstiff	150	mm		
Lweld	1191,94	Taking as 1200 mm		
Top weld strength, P weld-top	598,1521096	>	502,8283636	kN
Stiffener plate side weld strength, P weld-side	602,1968652	>	598,15	> 502,8283636

Table 20. Column design

Concrete f'_c	27,6 Mpa		4 ksi
Steel f_y	420 Mpa		60 ksi
Reinforcement ACI 10.9.1	0,01		
Design Load P_u	2562 kN		576 kip
Size of column	400		400 mm
Area	160000		
ϕ	0,65		
α	0,8		
A_g	243,0379747	in ²	248
One side	15,58967526	in	round up to 15,748 = 400mm
Slenderness			
k	1		
l_u	3000mm		118,11 in
r	4,7244		
$k \cdot l_u / r$	25	<	28
		Slenderness can be ignored	
P_n	886,1538462		
M_n	0,001769231		
e	1,99653E-06		
γ	0,65		
K_n	0,893300248		
R_n	1,13252E-07		
R4-60,7			
A_{st}	4,67300897	in ²	
# bars	8*#7		4,8 in ²
Tie Selection #3 as it less than #8			
Tie spacing	18	in	
	14	in	
	15,748	in	
Check			
Steell	1,935483871	%	>1%
Bar Spacing	4,6865		<6 in

Table 21. Footing design

f_c'	4	ksi			
D	5,66	in			
C	15,75	in			
ϕP_n	113	k			
ϕM_n	7,96	ft-k			
ϕV_n	15	k			
Clear	81	in			
Edge	16,9	in			
d	20	in			
g	@	12	in o.c., each way		
Flexureal capacity at column face					
Pile Spacing =		86,66			
Cap Edge Length =		126,1			
L_{1-1} =	156,1	in, length of section 1-1			
L_{2-2} =	126,1	in, length of section 2-2			
$M_{u, 1-1}$ =	38,0	ft-kips / ft, (to middle of cap elevation)			
$M_{u, 2-2}$ =	67,8	ft-kips / ft, (to middle of cap elevation)			
$\rho_{prov D}$ =	0,0042				
0,0032		<	$\rho_{prov D}$		
0,0206		>	$\rho_{prov D}$		
0,0024		<	$\rho_{prov D}$		
Check one way shear capacity at face of column and pile					
L_{3-3} =	61,5	in, length of section 3-3			
$V_{u, 2-2} = 2 (\phi P_n) / L_{2-2} =$		21,5	kips / ft	>	V_u
$V_{u, 3-3} = (\phi P_n) / L_{3-3} =$		22,1	kips / ft	>	V_u
$\phi V_c = 2 \phi b d (f_c')^{0.5} =$		22,8	kips / ft	>	V_u
where	$\phi =$	0,75	(ACI 318 21.2)		

Check column punching shear					
$\beta_c =$	1,00				
$b_1 =$	$(C + d) =$		36	in	
$b_2 =$	$(C + d) =$		36	in	
$b_0 =$	$2b_1 + 2b_2 =$		143	in	
$A_p =$	$b_0 d =$		2860	in ²	
	190	ksi	>	147	
Check single pile punching shear					
$\alpha =$	360,0	deg			
$b_1 =$	$(\alpha / 360) (D\pi / 4 + d) =$			24	in
$b_2 =$	$(\alpha / 360) (D\pi / 4 + d) =$			24	in
$b_0 =$	$(\alpha / 360) (D + d) \pi =$			81	in
$A_p =$	$b_0 d =$		1612	in ²	
	95		>	72	

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