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M.Sc. in Civil Engineering

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REPUBLIC OF TURKEY

GAZİANTEP UNIVERSITY

GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCES

MEASUREMENTS OF SETTLEMENTS FOR JET GROUTING COLUMNS

M.Sc. THESIS

IN

CIVIL ENGINEERING

BY

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Supervisor

Assoc. Prof. Dr. Hamza GÜLLÜ

Co-Supervisor

Prof. Dr. Hanifi ÇANAKÇI

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I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Hanifi ÇANAKÇI
Head of Department

This is to certify that we have read this thesis and that in our consensus/majority opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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I hereby declare that the thesis is written in line with the academic and ethical rules and that all the literature data used are included in the thesis by indicating reference.

Batuhan DÜZ

ABSTRACT

MEASUREMENTS OF SETTLEMENTS FOR JET GROUTING COLUMNS

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M. Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. Hamza GÜLLÜ
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As a result of rapid increase in the density of population, people have been directed to areas that may be problematic in geotechnical terms. Although there may appear many problems in geotechnical terms, one of the problems is settlement that occurs more than expected in Structures due to increased load. Ground improvement methods have been developed in order to solve these problems, and jet grouting method has become the most preferred method since it is generally advantageous. In general, developments in geotechnical sense are obtained empirically. Considering the Earth and its structure, homogeneously mixed types of ground prevent the use of strict rules and formulas. The most important feature of the engineering phenomenon is the comparison of the applied and received values with the formulas developed. In this study, jet grout ground columns application was applied in a residential project in the central district of Sivas province; and after the application, the group piles were analyzed while calculating settlements of pile group quantities with the Koerner and Partos (1974) Method and H. G. Poulos a E. H. Davis Method in the literature. Then, starting from the ground level, some points in the architectural plan were placed in the coordinate system, and the reading values were taken in 3-month periods for the control of the settlements under load. In the data obtained, the values in the literature were compared with the measured values in the application area, and differences were found. As a result of this study that was conducted with the thought of control experiment, it was considered that there might be some deviation errors in the formulas. The aim of this study was to contribute to future studies and to improve the literature more comprehensively.

Keywords: Geotechnical, Jet Grouting Columns, Pile Group, Settlement

ÖZET

JET GROUT KOLONLARININ OTURMA ÖLÇÜMÜ

DÜZ, Batuhan
Yüksek Lisans Tezi, İnşaat Mühendisliği
Danışman: Doç. Dr. Hamza GÜLLÜ
Yardımcı Danışman: Prof. Dr. Hanifi ÇANAKÇI
Haziran 2019
106 sayfa

Nüfus yoğunluğunun hızla artması sonucu insanlar geoteknik anlamda sorunlu olabilecek alanlara yönelmiştir. Geoteknik anlamda birçok problemle karşılaşılabilse de sorunlardan biri bina yapımında yük artışına bağlı olarak beklenenden fazla settlement gerçekleşmesidir. Bu sorunların giderilmesi amacıyla Ground Improvementyöntemleri geliştirilmiştir ve genel anlamda avantajlı olduğu için jet grouting method en çok tercih edilen yöntem haline gelmiştir. Genel olarak geoteknik anlamda ki gelişmeler ampirik olarak elde edilir. Yeryüzü ve yapısı göz önünde bulundurulduğunda homojen olarak karışabilen zemin çeşitleri kesin kurallar ve formüller kullanılmasına mani olmaktadır. Geliştirilen formüller ile yerinde uygulanan ve alınan değerlerin karşılaştırılması mühendislik olgusunun en belirgin özelliğidir. Bu çalışma da Sivas ili Merkez ilçesinde bir konut projesinde jet grout Ground Improvementmethodne başvuruldu ve uygulamadan sonra literatürde yer alan Koerner and Partos (1974) metodu ile ayrıca H.G Poulos a E.H. Davis yöntemlerine göre grup kazık etkisiyle settlement miktarları hesaplanarak analiz edildi. Daha sonra Foundation den başlayarak mimari planda bazı noktalar koordinat sistemine yerleştirilerek yük altında settlement miktarlarının kontrolü için 3'er aylık periyotlar halinde osanda değerleri alındı. Elde edilen verilerde literatürdeki hesaplamalar ile uygulama alanında ölçülen değerler kıyaslandı ve arasında differentlıklar olduğu saptandı. Kontrol deneyi düşüncesiyle yapılan bu çalışma sonucunda formüllerde bulunan değerlerde bazı sapma hatalarının olabileceği düşünüldü. Yapılan bu çalışmanın daha sonraki çalışmalara katkıda bulunması ve literatürün daha kapsamlı bir şekilde iyileştirilmesi amaçlandı.

Anahtar Kelimeler: Geoteknik, Jet Grout columnları, Grup Kazık, Settlement

Dedicated to

My most precious asset in life, my family,

To my brother, my wife and my mother who never abstained from supporting me,

To my dear relatives and friends ...

ACKNOWLEDGEMENTS

I would like to thank and express my endless gratitude to; Dear Assoc. Prof. Hamza GÜLLÜ, who was my supervisor, and Prof. Dr. Hanifi ÇANAKÇI, who was my co-supervisor, as the academicians of Gaziantep University and who shared all their knowledge with me, who never abstained from supporting me and who had great efforts in my thesis throughout my thesis study.

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LIST OF SYMBOLS

$\mathbf{N_{30}}$	SPT blow count
$\mathbf{N_{60}}$	SPT blow count
$\mathbf{D_f}$	Depth of Foundation
$\mathbf{C_N}$	Depth Correction Factor
$\mathbf{V_p}$	Compactional Wave Velocity
$\mathbf{V_s}$	Transverse Wave Velocity
$\boldsymbol{\phi}$	Internal Frictional Angle
$\boldsymbol{\gamma}$	Unit Weight
B	Foundation length
L	Foundation width
$\mathbf{K_s}$	Bedding Coefficient
D	Jet Grout Column Diameter
$\mathbf{Q_{ult}}$	Ultimate Bearing Capacity

LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
USCS	Unified Soil Classification System
GWT	Ground Water Table

CHAPTER I
INTRODUCTION

As a result of the fast spread of the population of the world, urban centers remained insufficient; and engineering areas were limited. Due to this need, people were directed to build settlements in different areas. The structures that exist in the present situation have shifted to swamp areas, dump sites, artificial filling areas, and sometimes, with the desire of humans, to pleasing areas of the sea, rivers and creeks in aesthetical terms from old and deeply-settled areas. In general, the problems related to grounds may be eliminated by studies that aim to solve them. Ground problems may appear in many areas. When the main points are considered, it may be mentioned that in the essence of engineering designs, the load that is carried by the structure is transferred directly to the ground. It is possible that the ground has not experienced the load before the construction, and therefore, the bearing-capacity problem may occur, shortly, the ground may not be ready for the load. Another problem faced in this respect is the settlement problem. The settlement amount of the ground when the load is faced is calculated according to the theoretical formulas used today; however, the settlement may be higher than the expected amount according to the ground structure. The liquefaction problem may appear before us as another problem. Liquefaction is the condition in which the sliding waves, which occur during an earthquake, act in completely liquid form as a result of the transition from solid state into liquid state after the earthquake affects the grains in the structure of the ground. Although it has many varieties, in general terms, liquefaction can be mentioned as above. These are most-frequently faced problems, and there are many problems in addition to these.

With the development of technology, many methods were developed to solve these problems in geotechnical terms. These Ground Improvement Methods, which were produced and implemented, were developed by considering the ground characteristics as a solution to the problems, which might be faced in the future. Some of the Ground Improvement Methods that are implemented by many countries, especially our country are raker pile, fore pile, mini pile, and jet grout column application. Although it is possible to apply all of these methods according to the purpose of application, jet grout columns are the most frequently implemented Ground Improvement Method since it has many advantages.

The Jet Grout Method, which was first used in Japan, has been applied in many countries with the developing technology. In general, developments in geotechnical sense are obtained in an empirical manner. When the ground and its structure are considered, the use of exact rules and formulas is prevented due to the variety of ground types that can be mixed homogeneously. It is absolutely more important that the control of the work is carried out rather than determining whether the improvement serves its purpose. No matter how must advanced a technology is, using machines, making required adjustments, and all kinds of installation are of human origin, and therefore, it is open to mistakes. In addition to this, even if everything is implemented perfectly, the investigation of the implementation provides explanatory information about many subjects. Comparing the developed formulas and the values obtained in an implementation is the most important feature of the engineering phenomenon.

The purpose and scope of the present thesis involves the comparison of the Jet Grout Columns according to the formula and rules that are used in the literature, and the net measurements that are made in the field following jet grout production. The thesis includes the expected total settlement amount and the strength of the piles as a group to carry the structure according to the ground and pile characteristics of the structure of the jet grout columns applied in the central county of Sivas province. The use of different methods and solution methods together with the geotechnical software programs that was developed, and the measurements that will be made in the field will provide detailed data on the area where the study will be conducted, and will be the source for future implementations.

CHAPTER II

THE STATUS OF THE FIELD AND THE GROUND PROFILE

2.1 Introduction of the Construction Site

The Study Area is located in the central county of Sivas. The topographic slope of the Study Area is straight or close to straight. The continental climate, which is the typical climate of Central Anatolia, is dominant in the area. In this respect, summers are dry and hot, winters are cold and rainy in the study area, which is extremely poor in terms of vegetation.

The Study Area has not been exposed to any natural disasters like landslide, avalanche, water pressure, and rockfalls; and transportation is shown in the side location map.

Figure 2.1 Side Location Map of the Study Area

2.2 General Geology

In regional terms, the general geological characteristics of the outcropping units in Sivas and surroundings are summarized based on (Ayaz et al.1996).

a.Hafik Formation (Th)

Hafik Formation is named after the area and is specific to this region, which has the name “Hafik”. The unit is observed with its dominant gypsum lithology in many parts of the Study Area. Gypsum, which includes intermediary levels of fine clay and sand, have the quality of being mostly massive and secondary. In the lower parts of this gypsum, which is defined as the Hafik Member, there is the Boynuzözü Member, which is represented by sandstone and conglomerate lithology; and Celalli Member, which is represented by shallow marine limestone and mudstone lithology on the upper part. The Boynuzozü Member, the Hafik Member, and the Celalli Member are transitive in lateral and vertical directions. While Hafik Formation comes with an angular unconformity onto the Selimiye Formation, it also has a tectonic contact on the Lower Miocene-aged Apa Formation and Mid-Upper Miocene-aged Tatlıcak Formation. In the geological compilation study of Sivas region, the age of the Oligo-Miocene age, which covered a wide age range, was given to the Hafik Formation, which is also divided into lower age hosts to eliminate the difficulties in correlation.

b.Boynuzözü Member (Thb)

The unit starts with coarse gravel in the base level, and consists of red conglomerate-sandstone alternation that includes gypsum-limestone intermediary level and/or lenses. The lithology of the stack mostly consists of 1-6 cm polygenic sand and gravels and 25-30 cm block-size rocks in lower rates. The thickness of the stack varies between 5-10 m and 275 m (Çubuk et al., 1994).

c.Hafik Member (Thj)

Hafik Member consists of dominating massive vision with thin-middle-layered gypsum in some areas. Gypsum has the quality of being secondary gypsum in general. The gypsum that rises with salt tectonics is seen on the younger units. The thickness of the unit varies between 40-50 m to 475 m (Çubuk et al., 1994). In many studies that have been conducted until our present day, the Oligo-Miocene wide age range has been accepted for this gypsum, whose site has widely been discussed.

d.Celalli Member (Thc)

Celalli Member is represented by green mudstone and reef limestones with dirty-white color. The lithology of the unit is observed as algal limestone-mudstone at the bottom in the western side of Celalli with mudstone-sandstone alternates and marine character at the upper levels; however, the opposite of valid near Ağılkaya. Celalli Member is observed with alternating mudstone-sandstone at the bottom near Ağılkaya and with alternating 50-60 m algal limestone-mudstone at the upper levels (Çubuk et al., 1994). Celalli Member stretches towards Oligocene-aged Selimiye Formation in Ziyaret Tepe in the west with an angular unconformity, and stretches towards Hafik Member gypsum in Ağılkaya Village. The unit shows lateral and vertical transitions with Hafik Member and Boynuzözü Member. Its thickness was observed around 250-450 m by Stick et al. (1994).

e.Tatlıcak Formation (Tta)

Tatlıcak Formation consists mostly of conglomerate, sandstone, claystone, limestone and siltstone, and is divided into three members, which are; Kızılcakışla Member, Höyüklü Member and Savcun Member. Kızılcakışla Member comes towards Hafik Formation below with an angular unconformity, and is overlapped tectonically by the gypsies of Hafik Formation. The age of the formation was accepted as Middle-Upper Miocene.

f.Kızılcakışla Member (Ttak)

Kızılcakışla Member consists of red conglomerates and sandstones that have fine gray limestone bands. Höyüklü Member comes to this member with conformity.

g.Höyüklü Member (Ttah)

Höyüklü Member consists of gray sandstones that contain fine gray limestone levels. Savcun Members comes to it in a conforming way.

h.Savcun Member (Ttas)

The unit consists of alternating green claystone, sandstone, siltstone and marn that have thin coal levels. Savcun Member comes to Höyüklü Member in a conforming way, and is overlapped tectonically by Hafik Formation gypsum.

ı. Incesu Formation (Ti)

Incesu Formation was divided into two members as the Derindere Conglomerate Member and Porsuk Limestone Member by Yılmaz (1980). Incesu Formation is in the position that presents the widest spread area of Sivas Tersiyer Basin. The thickness of the unit was measured as approximately 300 m in the geological sections that were taken by Çubuk et al. (1994)

i.Derindere Member (Tid)

This unit consists of alternating conglomerate, sandstone, claystone and marl. Derindere Conglomerate Member consists of alternating conglomerate, sandstone and claystone bedded horizontally. The conglomerate is adhered loosely with clayey, sandy and partly chalky binders, and the grains are rounded at medium level and are poorly sorted. The sandstones also show a poorly sorted appearance. The claystone of this member is observed to be specific towards the upper levels of the stack. Although the thickness of the Derindere Conglomerate Member changes, it was measured as approximately 250-260 m in surrounding Sıcak Çermik area. The age of Derindere Conglomerate Member was measured to be at Upper Miocene according to the fossil contents in the samples taken by Yalçınlar (1955).

j.Porsuk Limestone Member (Tip)

Porsuk Limestone Member is represented dominantly by occasional limestone and sandy limestone and clayey limestone lithology, and corresponds to the highest level of Incesu Formation. Porsun Limestone Member is horizontally layered, and has gray and yellowish limestones at lower levels, and there are clayey limestones at the upper parts, and on the top, there are whiteish-gray limestones. The thickness of Porsuk Limestone Member, which comes towards the Derindere Member with a gradual transition, is approximately 40-50 m, although it has local variations. No characteristic fossils were detected in the samples that were taken from the Porsuk Limestone Member. However, the fact that this unit comes towards the Upper Miocene Derindere Member with a gradual transition shows that the unit may be of Lower Pliocene age. The Porsuk Limestone Member is a lacustrine environment sediment product.

k.Traverten (Qt)

Formed based on former or current carbonated water outlets, travertines generally spread on the Incesu Formation with a non-compliant surface. These travertines that are mostly precipitated by hot waters are seen in yellow, cream and white colors. Based on geological structure and drilling data, it was determined that travertines were formed as a result of the emergence of Akdağ metamorphic, marbles, and the dissolution of the mostly hot carbon dioxide waters. The age of the travertines was accepted as being Quaternary.

l. Alluvion (Qal)

In general, alluvions are in the form of fine-to-medium-material sediments. The thicknesses of the alluvions that are seen along suitable sedimentation environments like rivers, streams and valleys is variable. The thickness of alluvions may increase as high as 20-30 m in Kalın River, 25-35 m in Yıldız River and 30-40 m in Kızılırmak. Alluvions are of Quaternary-age, and show as the youngest units together with travertines.

Structural Geology: Sivas Tertiary Basin is of great importance in terms of having the data that will shed light on Paleotectonic and Neotectonic periods of Turkey. The basement of Sivas Tertiary Basin, which is on the south of the Northern Anatolian Fault, forms the Akdağmadeni Metamorphites in the west and north-west, while it is formed by the Tekeli Mountain Mixture (Yılmaz, 1980) in the north, the Refahiye Ophiolitic Mixture in the east and southeast (Aktimur et al., 1988), and Munzur Limestone (Özgül, 1981) and Hınzır Mountain Metamorphites in the south (Erkan et al., 1978). Sivas Basin started to be form at the beginning of the Eocene, and remained under the influence of a K-G-directional pressure regime in late Eocene. Because of these pressures, various folds and fractures developed in the area. Oligo-Miocene-aged gypsum caused gypsum tectonics in the area. According to the Turkey Earthquake Zone Map, which entered into force after the decision No. 96/8109 of Ministerial Cabinet on April 18, 1996, the construction area is located in the III Level earthquake zone.

Tectonic: The Tertiary Basin in and around Sivas is of great importance in terms of having the data that may shed light on the Paleotectonic and Neotectonic period of Turkey. The basement of the Sivas Tertiary Basin, which is at the south of the North Anatolian Fault consists of the Akdağmadeni Metamorphites in the west and north-west, and Tekelidağı Mixture (Yılmaz, 1980) in the north, the Refahiye Ophiolitic Mixture in the east and southeast (Aktimur et al., 1988), and Munzur Limestone (Özgül, 1981), and the Hınzır Mountain Metamorphic in the south (Erkan et al., 1978) (Inan et al, 1993).

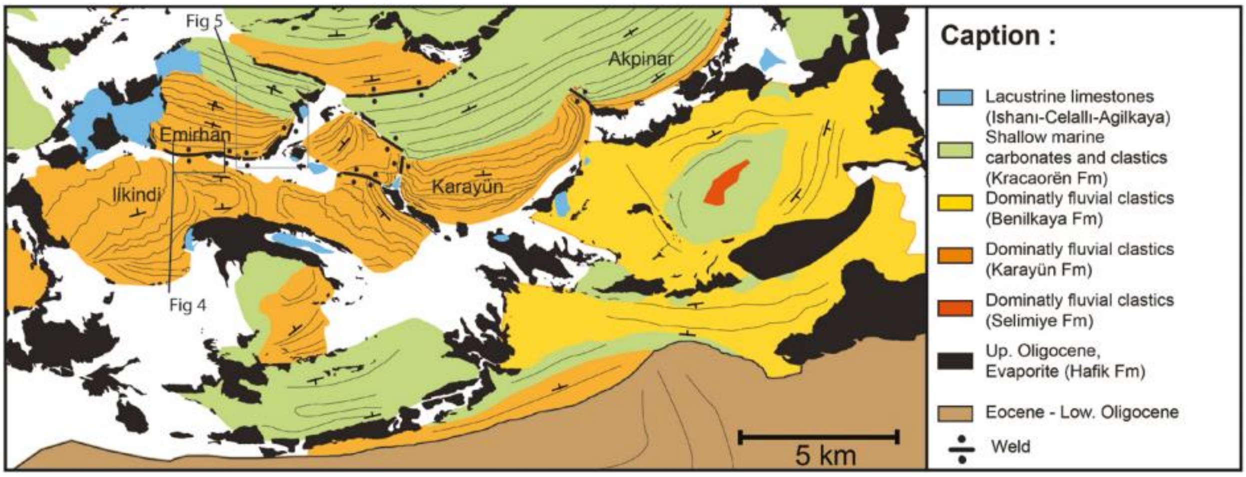


Figure 2.2. Simplified tectonic belts and active faults of Sivas and surroundings (Yılmaz, 1980).

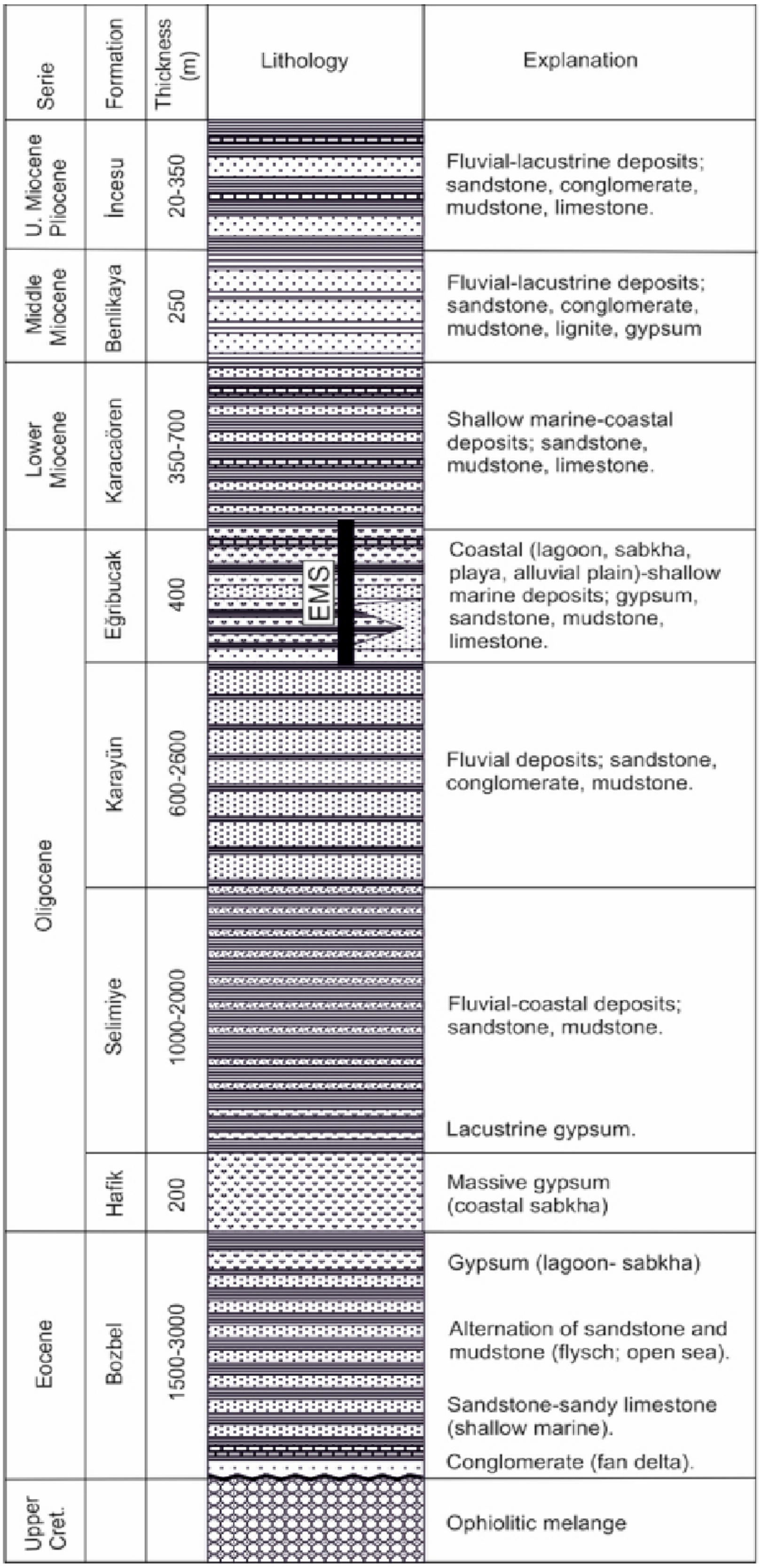


Figure 2.3. Generalized stratigraphic vertical section of Sivas and surroundings (Kurtman, 1993; Pelin, 1977; Öztürk, 1979; Yılmaz 1980).

2.1.Field Works and Experiments

2.3.1 Bore holes

A total of 160m drilling works were carried out in the Study Area, and 8 of them were in 20.0 m depth in the project, which is located in Sivas, Central County, Kardesler Neighborhood, 5495 Island, I38D01C Plot, 30th parcel. The drillings were made as dry boring with the MSUltra D500-equivalent hydraulic machine mounted on a truck. During drilling works, bore system (Flight Augier) with 3-inch diameter was used. The boring logs are given below.

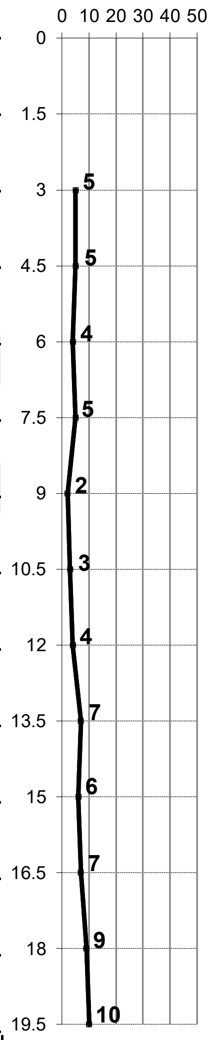
KÇ MÜHENDİSLİK İnşaat Sondaj Harita ve Müşavirlik Hizmetleri	**BORING LOG**		Page 1 / 1
	Drille	MUSTAFA YURTOGLU Document:19866120150040739389	**Borehole** SK-1

Project Name	*HİLMİ GÜL VE MÜŞT.*	Hole Diameter (mm)	*30 mm*
Boring Location	*SİVAS/MERKEZ İ38D01C/5495/30*	Groundwater (m)	*0*
Chainage	-	Casing Depth (m)	*10*
Boring Depth (m)	*13*	Start-Finish Date	*09/09/2016*
Elevation (m)	-	Coordinate(North)	-
D.Ring&Met.	*ROTARY*	Coordinate(East)	-

SPT Standard Peneration Test	UD Undisturbed Sample	P Pressuremeter Test
D Disturbed Sample	K Core Sample	VS Vane Shear Test

BORING DEPTH (m)	SAMPLE TYPE	RUN LENGTH (m)	STANDARD PENETRATION TEST: NUMBER OF BLOWS 0-15 cm	15-30 cm	30-45cm	PROFILE	GEOTECHNICAL DESCRIPTION	STRENGTH	WEATHERING	FRACTURE (30cm)	% (TCR) / T.Core R.	% (SCR) / S.CORE R.	RQD %	SAMPLE NO.
1.5		1.5	-	-	-		SANDY CLAY							
3	SPT+UD	1.5	2	2	3									
4.5	SPT	1.5	1	2	3									
6	SPT	1.5	1	2	2		CLAYEY GRAVEL							
7.5	SPT	1.5	2	2	3									
9	SPT	1.5	2	1	1									
10.5	SPT	1.5	2	1	2									
12	SPT	1.5	2	2	2									
13.5	SPT	1.5	3	3	4									
15	SPT	1.5	2	3	3									
16.5	SPT	1.5	2	3	4									
18	SPT	1.5	2	3	6									
19.5	SPT	1.5	3	4	6									
20														

GRAPH



STRENTGTH	WEATHERING	FINE GRAINED	COARSE GRAINED
I - STRONG	I - FRESH	N : 0-2 VERY SOFT	N:0-4 VERY LOOSE
II - MEDIUM STRONG	II - SLIGHTLY WEATHERED	N : 3-4 SOFT	N:5-10 LOOSE
III - MEDIUM WEAK	III - MEDIUM WEATHERED	N : 5-8 MEDIUM STIFF	N:11-30 MEDIUM DENSE
IV- WEAK	IV - HIGHLY WEATHERED	N : 9-15 STIFF	N:31 - 50 DENSE
V - VERY WEAK	V - COMPLETELY WEATHERED	N : 16-30 VERY STIFF	N> 50 VERY DENSE
		N > 30 HARD	

ROCK QUALITY DEFINITION (RQD)	FRACTURES - 30 cm	PROPORTIONS	
% 0 -25 VERY POOR	1 WIDE (W)	% 5 SLIGHTLY	% 5 SLIGHTLY
% 25 - 50 POOR	1-2 MODERATE (M)	% 5 -15 LITTLE	% 5 - 20 LITTLE
% 50 - 75 FAIR	2 -10 CLOSE (CL)	% 15 - 35 VERY	% 20 - 50 VERY MUCH
% 75 - 90 GOOD	10 -20 INTENSE (I)	% 35< AND	
% EXCELLENT	> 20 CRUSHED (Cr)		

ÖMER KAYA JEOLOJİ MÜHENDİSİ Oda No:18304	CONTROL

Figure 2.4 SK-1

KÇMÜHENDİSLİK İnşaat Sondaj Harita ve Müşavirlik Hizmetleri	**BORING LOG**		**Page** 1 / 1
	Drille	MUSTAFA YURTOGLU Document:19866120150040739389	**Borehole** SK-2

Project Name	*Göksu Süit*	**Hole Diameter (mm)**	*30 mm*
Boring Location	*SİVAS/MERKEZ İ38D01C/5495/30*	**Groundwater (m)**	*0*
Chainage	-	**Casing Depth (m)**	*10*
Boring Depth (m)	*13*	**Start-Finish Date**	*09/09/2016*
Elevation (m)	-	**Coordinate(North)**	-
D.Ring&Met.	*ROTARY*	**Coordinate(East)**	-

SPT Standard Peneration Test	**UD** Undisturbed Sample	**P** Pressuremeter Test
D Disturbed Sample	**K** Core Sample	**VS** Vane Shear Test

BORING DEPTH (m)	SAMPLE TYPE	RUN LENGTH (m)	STANDARD PENETRATION TEST NUMBER OF BLOWS 0-15 cm	15-30 cm	30-45cm	GRAPH 0 10 20 30 40 50	PROFILE	GEOTECHNICAL DESCRIPTION	STRENGTH	WEATHERING	FRACTURE (30cm)	% (TCR) / T.Core R.	% (SCR) / S.CORE R.	RQD %	SAMPLE NO.
1.5		1.5	-	-	-			SANDY CLAY							
3	SPT+UD	1.5	1	2	3	5									
4.5	SPT	1.5	2	2	4	6									
6	SPT	1.5	2	3	3	6									
7.5	SPT	1.5	1	2	2	4		CLAYEY GRAVEL							
9	SPT	1.5	2	3	3	6									
10.5	SPT	1.5	2	2	3	5									
12	SPT	1.5	1	1	2	3									
13.5	SPT	1.5	3	4	4	8									
15	SPT	1.5	3	2	4	6									
16.5	SPT	1.5	3	4	5	9									
18	SPT	1.5	3	4	3	7									
19.5	SPT	1.5	4	4	6	10									
20															

STRENTGTH	WEATHERING	FINE GRAINED	COARSE GRAINED
I - STRONG	I - FRESH	N : 0-2 VERY SOFT	N:0-4 VERY LOOSE
II - MEDIUM STRONG	II - SLIGHTLY WEATHERED	N : 3-4 SOFT	N:5-10 LOOSE
III - MEDIUM WEAK	III - MEDIUM WEATHERED	N : 5-8 MEDIUM STIFF	N:11-30 MEDIUM DENSE
IV- WEAK	IV - HIGHLY WEATHERED	N : 9-15 STIFF	N:31 - 50 DENSE
V - VERY WEAK	V - COMPLETELY WEATHERED	N : 16-30 VERY STIFF	N> 50 VERY DENSE
		N > 30 HARD	

ROCK QUALITY DEFINITION (RQD)	FRACTURES - 30 cm	PROPORTIONS	
% 0 -25 VERY POOR	1 WIDE (W)	% 5 SLIGHTLY	% 5 SLIGHTLY
% 25 - 50 POOR	1-2 MODERATE (M)	% 5 -15 LITTLE	% 5 - 20 LITTLE
% 50 - 75 FAIR	2 -10 CLOSE (CL)	% 15 - 35 VERY	% 20 - 50 VERY MUCH
% 75 - 90 GOOD	10 -20 INTENSE (I)	% 35< AND	
% EXCELLENT	> 20 CRUSHED (Cr)		

ÖMER KAYA GEOLOGICAL ENGINEER	CONTROL

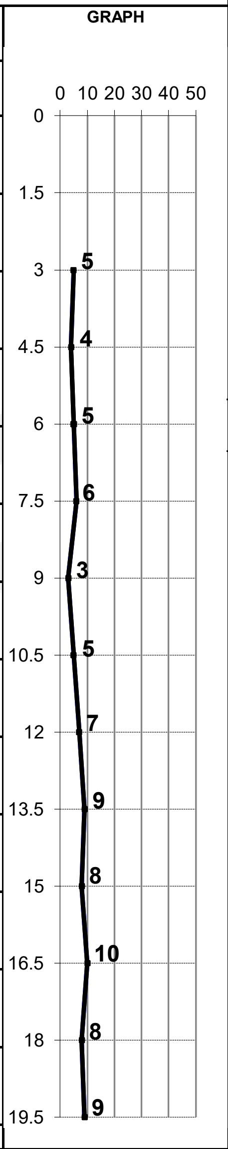
Figure 2-5 SK-2

KÇ MÜHENDİSLİK İnşaat Sondaj Harita ve Müşavirlik Hizmetleri	BORING LOG		Page 1 / 1
	Drille	MUSTAFA YURTOGLU Document:19866120150040739389	Borehole SK-3

Project Name	*HİLMİ GÜL VE MÜŞT.*	**Hole Diameter (mm)**	*30 mm*
Boring Location	*SİVAS/MERKEZ 389/3977/13*	**Groundwater (m)**	*0*
Chainage	-	**Casing Depth (m)**	*10*
Boring Depth (m)	*13*	**Start-Finish Date**	*09/09/2016*
Elevation (m)	*0*	**Coordinate(North)**	*0*
D.Ring&Met.	*ROTARY*	**Coordinate(East)**	*0*

SPT Standard Peneration Test	**UD** Undisturbed Sample	**P** Pressuremeter Test
D Disturbed Sample	**K** Core Sample	**VS** Vane Shear Test

BORING DEPTH (m)	SAMPLE TYPE	RUN LENGTH (m)	STANDARD PENETRATION TEST – NUMBER OF BLOWS 0-15 cm	15-30 cm	30-45cm	GRAPH	PROFILE	GEOTECHNICAL DESCRIPTION	STRENGTH	WEATHERING	FRACTURE (30cm)	% (TCR) / T.Core R.	% (SCR) / S.CORE R.	RQD %	SAMPLE NO.
1.5			0	0	0										
3			2	2	3			SANDY CLAY							
4.5			1	2	2										
6			2	2	3										
7.5			2	3	3										
9			1	1	2			CLAYEY GRAVEL							
10.5			2	3	2										
12			2	3	4										
13.5			3	4	5										
15			2	3	5										
16.5			4	4	6										
18			3	4	4										
19.5			3	4	5										
20															



STRENTGTH	WEATHERING	FINE GRAINED	COARSE GRAINED
I - STRONG	I - FRESH	N : 0-2 VERY SOFT	N:0-4 VERY LOOSE
II - MEDIUM STRONG	II - SLIGHTLY WEATHERED	N : 3-4 SOFT	N:5-10 LOOSE
III - MEDIUM WEAK	III - MEDIUM WEATHERED	N : 5-8 MEDIUM STIFF	N:11-30 MEDIUM DENSE
IV- WEAK	IV - HIGHLY WEATHERED	N : 9-15 STIFF	N:31 - 50 DENSE
V - VERY WEAK	V - COMPLETELY WEATHERED	N : 16-30 VERY STIFF	N> 50 VERY DENSE
		N > 30 HARD	

ROCK QUALITY DEFINITION (RQD)	FRACTURES - 30 cm	PROPORTIONS	
% 0 -25 VERY POOR	1 WIDE (W)	% 5 SLIGHTLY	% 5 SLIGHTLY
% 25 - 50 POOR	1-2 MODERATE (M)	% 5 -15 LITTLE	% 5 - 20 LITTLE
% 50 - 75 FAIR	2 -10 CLOSE (CL)	% 15 - 35 VERY	% 20 - 50 VERY MUCH
% 75 - 90 GOOD	10 -20 INTENSE (I)	% 35< AND	
% EXCELLENT	> 20 CRUSHED (Cr)		

ÖMER KAYA JEOLOJİ MÜHENDİSİ Oda No:18304	KONTROL

Figure 2.6. SK-2

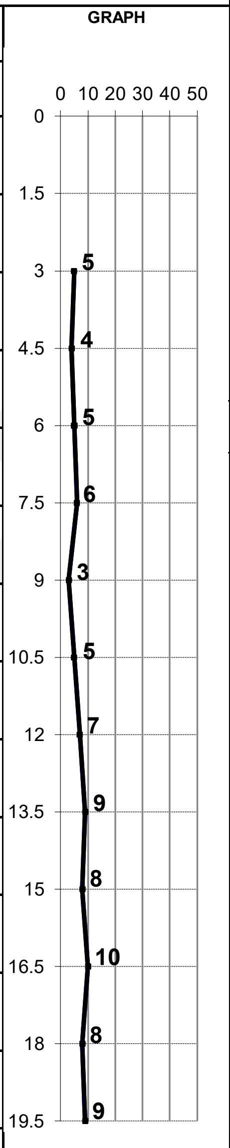
KÇ MÜHENDİSLİK İnşaat Sondaj Harita ve Müşavirlik Hizmetleri	**BORING LOG**		**Page** 1 / 1
	Drille	MUSTAFA YURTOĞLU Document:19866120150040739389	**Borehole** SK-3

Project Name	*HİLMİ GÜL VE MÜŞT.*	**Hole Diameter (mm)**	*30 mm*
Boring Location	*SİVAS/MERKEZ 389/3977/13*	**Groundwater (m)**	*0*
Chainage	-	**Casing Depth (m)**	*10*
Boring Depth (m)	*13*	**Start-Finish Date**	*09/09/2016*
Elevation (m)	*0*	**Coordinate(North)**	*0*
D.Ring&Met.	*ROTARY*	**Coordinate(East)**	*0*

SPT Standard Peneration Test	**UD** Undisturbed Sample	**P** Pressuremeter Test
D Disturbed Sample	**K** Core Sample	**VS** Vane Shear Test

BORING DEPTH (m)	SAMPLE TYPE	RUN LENGTH (m)	STANDARD PENETRATION TEST – NUMBER OF BLOWS 0-15 cm	15-30 cm	30-45cm	GEOTECHNICAL DESCRIPTION	STRENGTH	WEATHERING	FRACTURE (30cm)	% (TCR) / T.Core R.	% (SCR) / S.CORE R.	RQD %	SAMPLE NO.
1.5			0	0	0	SANDY CLAY							
3			2	2	3								
4.5			1	2	2								
6			2	2	3								
7.5			2	3	3	CLAYEY GRAVEL							
9			1	1	2								
10.5			2	3	2								
12			2	3	4								
13.5			3	4	5								
15			2	3	5								
16.5			4	4	6								
18			3	4	4								
19.5			3	4	5								
20													

GRAPH



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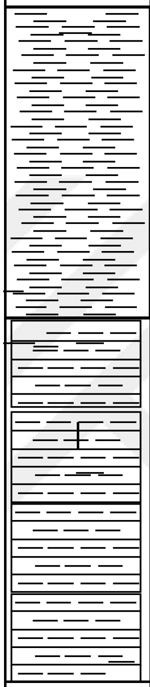


STRENTGTH	WEATHERING	FINE GRAINED	COARSE GRAINED
I - STRONG	I - FRESH	N : 0-2 VERY SOFT	N:0-4 VERY LOOSE
II - MEDIUM STRONG	II - SLIGHTLY WEATHERED	N : 3-4 SOFT	N:5-10 LOOSE
III - MEDIUM WEAK	III - MEDIUM WEATHERED	N : 5-8 MEDIUM STIFF	N:11-30 MEDIUM DENSE
IV- WEAK	IV - HIGHLY WEATHERED	N : 9-15 STIFF	N:31 - 50 DENSE
V - VERY WEAK	V - COMPLETELY WEATHERED	N : 16-30 VERY STIFF	N> 50 VERY DENSE
		N > 30 HARD	

ROCK QUALITY DEFINITION (RQD)	FRACTURES - 30 cm	PROPORTIONS	
% 0 -25 VERY POOR	1 WIDE (W)	% 5 SLIGHTLY	% 5 SLIGHTLY
% 25 - 50 POOR	1-2 MODERATE (M)	% 5 -15 LITTLE	% 5 - 20 LITTLE
% 50 - 75 FAIR	2 -10 CLOSE (CL)	% 15 - 35 VERY	% 20 - 50 VERY MUCH
% 75 - 90 GOOD	10 -20 INTENSE (I)	% 35< AND	
% EXCELLENT	> 20 CRUSHED (Cr)		

ÖMER KAYA JEOLOJİ MÜHENDİSİ Oda No:18304	KONTROL

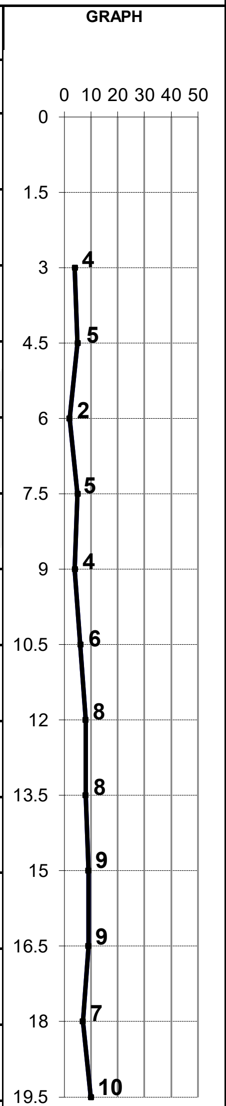
Figure 2.7. SK-3

BORING LOG				Page	1 / 1
Drille	MUSTAFA YURTOGLU Document:19866120150040739389			Borehole	SK-4

Project Name	*HİLMİ GÜL VE MÜŞT.*	Hole Diameter (mm)	*30 mm*
Boring Location	*SİVAS/MERKEZ 389/3977/13*	Groundwater (m)	*0*
Chainage	-	Casing Depth (m)	*10*
Boring Depth (m)	*13*	Start-Finish Date	*09/09/2016*
Elevation (m)	*0*	Coordinate(North)	*0*
D.Ring&Met.	*ROTARY*	Coordinate(East)	*0*

SPT Standard Peneration Test	UD Undisturbed Sample	P Pressuremeter Test
D Disturbed Sample	K Core Sample	VS Vane Shear Test

BORING DEPTH (m)	SAMPLE TYPE	RUN LENGTH (m)	STANDARD PENETRATION TEST – NUMBER OF BLOWS 0-15 cm	15-30 cm	30-45cm	GRAPH	PROFILE	GEOTECHNICAL DESCRIPTION	STRENGTH	WEATHERING	FRACTURE (30cm)	% (TCR) / T.Core R.	% (SCR) / S.CORE R.	RQD %	SAMPLE NO.
1.5			0	0	0			SANDY CLAY							
3			1	2	2										
4.5			2	3	2										
6			1	1	1			CLAYEY GRAVEL							
7.5			2	2	3										
9			1	2	2										
10.5			1	3	3										
12			1	3	5										
13.5			2	3	5										
15			3	3	6										
16.5			3	4	5										
18			4	3	4										
19.5			4	4	6										
20															



STRENTGTH	WEATHERING	FINE GRAINED	COARSE GRAINED
I - STRONG	I - FRESH	N : 0-2 VERY SOFT	N:0-4 VERY LOOSE
II - MEDIUM STRONG	II - SLIGHTLY WEATHERED	N : 3-4 SOFT	N:5-10 LOOSE
III - MEDIUM WEAK	III - MEDIUM WEATHERED	N : 5-8 MEDIUM STIFF	N:11-30 MEDIUM DENSE
IV- WEAK	IV - HIGHLY WEATHERED	N : 9-15 STIFF	N:31 - 50 DENSE
V - VERY WEAK	V - COMPLETELY WEATHERED	N : 16-30 VERY STIFF	N> 50 VERY DENSE
		N > 30 HARD	

ROCK QUALITY DEFINITION (RQD)	FRACTURES - 30 cm	PROPORTIONS	
% 0 -25 VERY POOR	1 WIDE (W)	% 5 SLIGHTLY	% 5 SLIGHTLY
% 25 - 50 POOR	1-2 MODERATE (M)	% 5 -15 LITTLE	% 5 - 20 LITTLE
% 50 - 75 FAIR	2 -10 CLOSE (CL)	% 15 - 35 VERY	% 20 - 50 VERY MUCH
% 75 - 90 GOOD	10 -20 INTENSE (I)	% 35< AND	
% EXCELLENT	> 20 CRUSHED (Cr)		

ÖMER KAYA JEOLOJİ MÜHENDİSİ Oda No:18304	KONTROL

Figure 2.8. SK-4

2.3.2 Underground and Above-ground Waters

In the borings, the underground water level was detected at -3.0 meters depth. It must be kept in mind that the underground water level may increase at a significant level that might affect the grounds due to seasonal rainfall regimes.

2.3.3 SPT Experiments and Results

To measure the strength of the ground in the area, the SPT test was applied at 1.5 m intervals in the drilling, and representative samples were taken. The test was applied as counting the number of the blows (N) that must be performed to let the 15 cm-long part of the sampler enter into the ground by dropping a sledgehammer weighing 63.5 kg onto rods from a height of 760 mm with a sampler that is attached to the sounding rods and that may be separated into two parts in the middle with a brazen inner tube in it. The number of the blows (N) obtained from the first 15-cm hits of the tube were not taken into account. Here, the purpose was to bypass the disturbed part of the ground at the bottom of the well. After the first 15-cm work, the tube was hit into the ground as 30 cm, and the total blow count was recorded for 30 cm. The blow count (N30) that was recorded was considered as the result of the experiment.

In the SPT Relations, the SPT N value is an index of the ground behavior, as it is the case in many tests, and does not yield the engineering parameters of the ground directly. But the engineering parameters of the ground can be detected with some empirical relations. These are;

Relative Compactness (Dr), Internal Friction Angle (Φ), Undrained Cohesive Coefficient (Cu), Deformation modules (mv, Es, G), Liquefaction potential, Settlement calculation, Bearing power, Pile capacity.

SK-1 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	-	-	-		60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	2	2	3	5	60	1	1	0.75	1	3.8	58	28	1.85	6.9	9.3
3	4.5	19.3	1	2	3	5	60	1	1	0.85	1	4.3	87	42	1.51	6.4	7.6
4	6.0	19,3	1	2	2	4	60	1	1	0.95	1	3.8	116	56	1.31	5.0	5.2
5	7.5	19.3	2	2	3	5	60	1	1	0.95	1	4.8	145	70	1.17	5.6	5.9
6	9.0	19.3	2	1	1	2	60	1	1	0.95	1	1.9	174	84	1.07	2.0	2.1
7	10.5	19.3	2	1	2	3	60	1	1	1	1	3.0	203	98	0.99	3.0	3.0
8	12.0	19.3	2	2	2	4	60	1	1	1	1	4.0	232	112	0.93	3.7	3.7
9	13.5	19.3	3	3	4	7	60	1	1	1	1	7.0	261	126	0.87	6.1	6.1
10	15.0	19.3	2	3	3	6	60	1	1	1	1	6.0	290	140	0.83	5.0	5.0
11	16.5	19.3	2	3	4	7	60	1	1	1	1	7.0	318	153	0.79	5.5	5.5
12	18.0	19.3	2	3	6	9	60	1	1	1	1	9.0	347	167	0.76	6.8	6.8
13	19.5	19.3	3	4	6	10	60	1	1	1	1	10.0	376	181	0.73	7.3	7.3

SK-2 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	-	-	-	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	1	2	3	5	60	1	1	0.75	1	3.8	58	28	1.85	6.9	9.3
3	4.5	19.3	2	2	4	5	60	1	1	0.85	1	5.1	87	42	1.51	7.7	9.1
4	6.0	19,3	2	3	3	6	60	1	1	0.95	1	5.7	116	56	1.31	7.5	7.9
5	7.5	19.3	1	2	2	4	60	1	1	0.95	1	3.8	145	70	1.17	4.5	4.7
6	9.0	19.3	2	3	3	6	60	1	1	0.95	1	5.7	174	84	1.07	6.1	6.4
7	10.5	19.3	2	2	3	5	60	1	1	1	1	5.0	203	98	0.99	5.0	5.0
8	12.0	19.3	1	1	2	3	60	1	1	1	1	3.0	232	112	0.93	2.8	2.8
9	13.5	19.3	3	4	4	8	60	1	1	1	1	8.0	261	126	0.87	7.0	7.0
10	15.0	19.3	3	2	4	6	60	1	1	1	1	6.0	290	140	0.83	5.0	5.0
11	16.5	19.3	3	4	5	9	60	1	1	1	1	9.0	318	153	0.79	7.1	7.1
12	18.0	19.3	3	4	3	7	60	1	1	1	1	7.0	347	167	0.76	5.3	5.3
13	19.5	19.3	4	4	6	10	60	1	1	1	1	10.0	376	181	0.73	7.3	7.3

SK-3 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	0	0	0	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	2	2	3	5	60	1	1	0.75	1	3.8	58	28	1.85	6.9	9.3
3	4.5	19.3	2	2	3	5	60	1	1	0.85	1	3.4	87	42	1.51	5.1	6.1
4	6.0	19,3	2	2	3	5	60	1	1	0.95	1	4.8	116	56	1.31	6.2	6.6
5	7.5	19.3	2	3	3	6	60	1	1	0.95	1	5.7	145	70	1.17	6.7	7.0
6	9.0	19.3	1	1	2	3	60	1	1	0.95	1	2.9	174	84	1.07	3.0	3.2
7	10.5	19.3	2	3	4	7	60	1	1	1	1	5.0	203	98	0.99	5.0	5.0
8	12.0	19.3	2	3	4	7	60	1	1	1	1	7.0	232	112	0.93	6.5	6.5
9	13.5	19.3	3	4	5	9	60	1	1	1	1	9.0	261	126	0.87	7.9	7.9
10	15.0	19.3	2	3	5	8	60	1	1	1	1	8.0	290	140	0.83	6.6	6.6
11	16.5	19.3	4	4	6	10	60	1	1	1	1	10.0	318	153	0.79	7.9	7.9
12	18.0	19.3	3	4	4	8	60	1	1	1	1	8.0	347	167	0.76	6.1	6.1
13	19.5	19.3	3	4	5	9	60	1	1	1	1	9.0	376	181	0.73	6.5	6.5

SK-4 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	0	0	0	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	1	2	2	4	60	1	1	0.75	1	3.0	58	28	1.85	5.6	7.4
3	4.5	19.3	2	3	2	5	60	1	1	0.85	1	4.3	87	42	1.51	6.4	7.6
4	6.0	19,3	1	1	1	2	60	1	1	0.95	1	1.9	116	56	1.31	2.5	2.6
5	7.5	19.3	2	2	3	5	60	1	1	0.95	1	4.8	145	70	1.17	5.6	5.9
6	9.0	19.3	1	2	2	4	60	1	1	0.95	1	3.8	174	84	1.07	4.1	4.3
7	10.5	19.3	1	3	3	6	60	1	1	1	1	6.0	203	98	0.99	5.9	5.9
8	12.0	19.3	1	3	5	8	60	1	1	1	1	8.0	232	112	0.93	7.4	7.4
9	13.5	19.3	2	3	5	8	60	1	1	1	1	8.0	261	126	0.87	7.0	7.0
10	15.0	19.3	3	3	6	9	60	1	1	1	1	9.0	290	140	0.83	7.5	7.5
11	16.5	19.3	3	4	5	9	60	1	1	1	1	9.0	318	153	0.79	7.1	7.1
12	18.0	19.3	4	3	4	7	60	1	1	1	1	7.0	347	167	0.76	5.3	5.3
13	19.5	19.3	4	4	6	10	60	1	1	1	1	10.0	376	181	0.73	7.3	7.3

SK-5 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	$\sigma_{vc'}$ (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	0	0	0	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	1	2	2	4	60	1	1	0.75	1	3.0	58	28	1.85	5.6	7.4
3	4.5	19.3	1	2	4	6	60	1	1	0.85	1	5.1	87	42	1.51	7.7	9.1
4	6.0	19,3	1	1	2	3	60	1	1	0.95	1	2.9	116	56	1.31	3.7	3.9
5	7.5	19.3	1	2	2	4	60	1	1	0.95	1	3.8	145	70	1.17	4.5	4.7
6	9.0	19.3	2	2	3	5	60	1	1	0.95	1	4.8	174	84	1.07	5.1	5.3
7	10.5	19.3	2	3	4	7	60	1	1	1	1	7.0	203	98	0.99	6.9	6.9
8	12.0	19.3	2	2	4	6	60	1	1	1	1	6.0	232	112	0.93	5.6	5.6
9	13.5	19.3	1	2	5	7	60	1	1	1	1	7.0	261	126	0.87	6.1	6.1
10	15.0	19.3	2	3	2	5	60	1	1	1	1	5.0	290	140	0.83	4.1	4.1
11	16.5	19.3	4	4	4	8	60	1	1	1	1	8.0	318	153	0.79	6.3	6.3
12	18.0	19.3	4	5	4	9	60	1	1	1	1	9.0	347	167	0.76	6.8	6.8
13	19.5	19.3	4	5	6	11	60	1	1	1	1	11.0	376	181	0.73	8.0	8.0

SK-6 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	$\sigma_{vc'}$ (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	0	0	0	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	1	2	2	4	60	1	1	0.75	1	3.0	58	28	1.85	5.6	7.4
3	4.5	19.3	1	2	4	6	60	1	1	0.85	1	5.1	87	42	1.51	7.7	9.1
4	6.0	19,3	1	1	2	3	60	1	1	0.95	1	2.9	116	56	1.31	3.7	3.9
5	7.5	19.3	1	2	2	4	60	1	1	0.95	1	3.8	145	70	1.17	4.5	4.7
6	9.0	19.3	2	2	3	5	60	1	1	0.95	1	4.8	174	84	1.07	5.1	5.3
7	10.5	19.3	2	3	4	7	60	1	1	1	1	7.0	203	98	0.99	6.9	6.9
8	12.0	19.3	2	2	4	6	60	1	1	1	1	6.0	232	112	0.93	5.6	5.6
9	13.5	19.3	1	2	5	7	60	1	1	1	1	7.0	261	126	0.87	6.1	6.1
10	15.0	19.3	2	3	2	5	60	1	1	1	1	5.0	290	140	0.83	4.1	4.1
11	16.5	19.3	4	4	4	8	60	1	1	1	1	8.0	318	153	0.79	6.3	6.3
12	18.0	19.3	4	5	4	9	60	1	1	1	1	9.0	347	167	0.76	6.8	6.8
13	19.5	19.3	4	5	6	11	60	1	1	1	1	11.0	376	181	0.73	8.0	8.0

SK-7 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	0	0	0	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	3	2	3	5	60	1	1	0.75	1	3,8	58	28	1.85	6.9	9.3
3	4.5	19.3	3	2	1	3	60	1	1	0.85	1	2.6	87	42	1.51	3.9	4.5
4	6.0	19,3	3	1	1	2	60	1	1	0.95	1	1.9	116	56	1.31	2.5	2.6
5	7.5	19.3	3	1	2	3	60	1	1	0.95	1	2.9	145	70	1.17	3.3	3.5
6	9.0	19.3	2	2	1	3	60	1	1	0.95	1	2.9	174	84	1.07	3.0	3.2
7	10.5	19.3	2	3	2	5	60	1	1	1	1	5.0	203	98	0.99	5.0	5.0
8	12.0	19.3	2	2	3	5	60	1	1	1	1	5.0	232	112	0.93	4.6	4.6
9	13.5	19.3	3	2	1	3	60	1	1	1	1	3.0	261	126	0.87	2.6	2.6
10	15.0	19.3	2	2	3	5	60	1	1	1	1	5.0	290	140	0.83	4.1	4.1
11	16.5	19.3	2	3	4	7	60	1	1	1	1	7.0	318	153	0.79	5.5	5.5
12	18.0	19.3	2	4	4	8	60	1	1	1	1	8.0	347	167	0.76	6.1	6.1
13	19.5	19.3	3	4	7	11	60	1	1	1	1	11.0	376	181	0.73	8.0	8.0

SK-8 SPT CORRECTIONS

SPT Sample No	Depth (m)	Weight per unit volume (kN/m^3)	N1	N2	N3	N30	Energy Ratio (ER)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N')_{60}$	N'
1	1.5	19.3	0	0	0	0	60	1	1	0.75	1	0.0	29	14	2.62	0.0	0.0
2	3.0	19.3	3	2	1	3	60	1	1	0.75	1	2.3	58	28	1.85	4.2	5.6
3	4.5	19.3	3	2	2	4	60	1	1	0.85	1	3.4	87	42	1.51	5.1	6.1
4	6.0	19,3	2	1	1	3	60	1	1	0.95	1	2.9	116	56	1.31	3.7	3.9
5	7.5	19.3	2	2	1	3	60	1	1	0.95	1	2.9	145	70	1.17	3.3	3.5
6	9.0	19.3	3	2	2	4	60	1	1	0.95	1	3.8	174	84	1.07	4.1	4.3
7	10.5	19.3	2	3	2	5	60	1	1	1	1	5.0	203	98	0.99	5.0	5.0
8	12.0	19.3	4	2	2	4	60	1	1	1	1	4.0	232	112	0.93	3.7	3.7
9	13.5	19.3	3	2	2	4	60	1	1	1	1	4.0	261	126	0.87	3.5	3.5
10	15.0	19.3	4	3	3	6	60	1	1	1	1	6.0	290	140	0.83	5.0	5.0
11	16.5	19.3	3	2	3	5	60	1	1	1	1	5.0	318	153	0.79	3.9	3.9
12	18.0	19.3	2	3	4	6	60	1	1	1	1	6.0	347	167	0.76	4.5	4.5
13	19.5	19.3	3	4	4	8	60	1	1	1	1	8.0	376	181	0.73	5.8	5.8

Figure 2.9. Standard penetration experiment corrections

In $N_{60} = N_{30}*(ER/60).C_L. C_S. C_D$, ER = Energy Ratio (The energy ratio for the donut sledgehammer is ER=60), Rod Length (C_L), Internal Tube (C_S) and Well Diameter (C_D) corrections. It is seen in the literature that Tokimatsu and Yoshimi [1983] Relation is used commonly after Cover Tension Corrections (CN), Energy Ratio, Rod Length, Well Diameter and Tube Corrections.

The corrected number of the blows in the $N_{60} = CN*N_{60}$ expression are given in the table below.

Explanation: The results of the SPT Test must be corrected by taking the area and the equipment used into consideration by using the most appropriate one among the SPT Sledgehammer Yield Area (Clayton, 1990) Well Diameter and Rod Length Corrections Skempton (1986) Inner Coating Corrections (Bowles, 1988) as follows..

Table 2.1 SPT Sledgehammer Yields (Clayton, 1990)

Sledgehammer Type	Sledgehammer Yield, E_r (%)
Donut	0.45–0.85
Safety	0.60–0.73
Automatic	0.55–0.60

Corrections with Rod Length (C_R), Inner Tube (C_S) and Well Diameter (C_B)

Table 2.2 Correction Factors for N60

<table>
<tr><th>Correction Type</th><th>Definition</th><th colspan="2">Correction Factor</th></tr>
<tr><td rowspan="4">Rod Length (m) under the blow block (Skempton, 1986)</td><td>> 10</td><td>1.0</td><td rowspan="4">C_R</td></tr>
<tr><td>Between 6 and 10</td><td>0.95</td></tr>
<tr><td>Between 4 and 6</td><td>0.85</td></tr>
<tr><td>Between 3 and 4</td><td>0.75</td></tr>
<tr><td rowspan="3">Well Diameter Corrections (Skempton, 1986)</td><td>65-115</td><td>1.0</td><td rowspan="3">C_B</td></tr>
<tr><td>150</td><td>1,05</td></tr>
<tr><td>200</td><td>1,15</td></tr>
<tr><td rowspan="3">Inner Coating Corrections (Bowles, 1988)</td><td>When Inner Coating is not used</td><td>1.0</td><td rowspan="3">C_S</td></tr>
<tr><td>When Inner Coating is not used: tight sand, clay</td><td>1.05</td></tr>
<tr><td>When Inner Coating is used: loose sand</td><td>1.15</td></tr>
</table>

Table 2.3 Seal load correction factors (CN) (Carter and Bentley, 1991) (N.B. 1kg/cm2=98.0665 ≈ 100 kPa)

<table>
<tr><th>Reference</th><th colspan="2">Correction Factor (C_N)</th><th>C_NSeal load pressure unit (σ_v')</th></tr>
<tr><td>Peck et al. (1974)</td><td colspan="2">$C_N = 0.77 \cdot \log_{10} \frac{20}{\sigma_v^{'}}$</td><td>kg/cm^2 or ton/ft^2</td></tr>
<tr><td>Tokimatsu And Yoshimi (1983) (The equation also used by Skempton (1986) for “excessively consolidated fine sands”)</td><td colspan="2">$C_N = \frac{1.7}{0.7 + \sigma_v^{'}}$</td><td>kg/cm^2 or ton/ft^2</td></tr>
<tr><td>Liao and Whitman (1986)</td><td colspan="2">$C_N = \sqrt{\frac{1}{\sigma_v^{'}}}$</td><td>kg/cm^2 or ton/ft^2</td></tr>
<tr><td rowspan="2">Skempton (1986) (For normally consolidated grounds)</td><td>$C_N = \frac{2}{1 + \sigma'_V}$</td><td>Relative Density (D_r) Between 40 – 60%</td><td>kg/cm^2 or ton/ft^2</td></tr>
<tr><td>$C_N = \frac{3}{2 + \sigma'_V}$</td><td>Relative Density (D_r) Between 60 – 80%</td><td>kg/cm^2 or ton/ft^2</td></tr>
</table>

2.3.4 Geophysical Works

Seismic Method

The crust of the Earth consists of layers of rocks that have different physical properties like elastic velocity and density. Seismic Method is the process of determining the geological structure of the underground with various calculation methods from seismograms after recording the vibrations that occur as a result of the energy generated by an energy source on the Earth (dynamite, falling weights, etc.) with seismometers (geophones) and recorders placed at certain points away from the point where the energy is released.

2.3.4.1 Seismic Refraction Method

The Seismic Refraction Method is based on the measurement of the travel times of the seismic waves that are broken on the interfaces of the soil layers that have different diffusion rates. Since refraction waves travel at a suitable path in the ground, their frequency contents are between 5-25 Hz compared to the reflection waves. This method is generally applied according to the selected spread lengths depending on the depth of the research with alternating shots; and the horizontal-vertical stratification of the ground is defined according to the Vertical (Vp) and Transverse (Vs) wave velocities. The elastic-dynamic parameters of the ground are calculated with these empirical relations.

Multichannel Analysis Method of Vertical Waves

Many problems might appear in urban areas when the sliding wave velocity is determined with classical seismic methods. These are;

- High noise level
- The need fora ide receptive series
- Absorption of seismic energy in sediment layers
- Low speed zones

For this reason, the method of analyzing the surface waves of active and passive origin is used to determine the sliding wave velocity in recent years.

The diffusion rates of the Rayleigh Waves, which are a kind of surface waves, are very close to the sliding wave velocity (V_R= 0.87 and 0.96 V_S, depending on the Poison Ratio). This relation is important for using the sliding wave for various purposes in geotechnical studies (determining the shear modulus, magnification and liquefaction analysis, etc.).

The Rayleigh Wave depends on the P Compaction and S Sliding wave rates. Slip and compressive wave velocities vary with depth in heterogeneous environments. For this reason, different frequency components of the Rayleigh Wave show different phase velocities, which is called dispersion. To switch to the Sliding Wave Velocity Depth Model from the Dispersion Curve, Reverse Solution Operations are used.

Operation steps in multichannel analysis method of the surface waves;

1. Multichannel data collection
2. Forming the Dispersion Curve

3. Creating the Sliding Wave-Depth Section with Reverse Solution Operation.

Selecting parameters is very important in Surface Wave Analysis during field data collection. Important points that must be considered are that data collection must be carried out with minimum noise, and at least half of the distance between the initial receiver (offset distance) research depth and the source must be selected.

Seismic Refraction

The P wave that is measured in seismic studies is the measurement of the resistance of the material to compaction and expansion. The S wave is the measurement of the resistance of the material to form disruptions or torsion. For example, the S wave does not diffuse in liquids. Since seismic velocities are dependent on the elastic properties of the medium completely, dynamic parameters can be written as the function of seismic velocities. The dynamic and elastic parameter values obtained from the study area are shown with dark colors in the corresponding places in the following tables. Elastic and dynamic parameters are given in detail. The Soil Study was carried out to determine the S-wave velocities from the geophysical studies conducted in the context of the Soil Survey Report, and the Doremi Device and 12 Geophones at 4.5 hertz were made use of.

Table 2.4 Ground class according to seismic velocities

Ground class	**Definition**	**Features**
A	Hard rock	VS>800
B	Very firm sand-conglomerate-hard clay	360<Vs<800
C	Firm medium-firm sand conglomerate-hard clay	180<Vs<360
D	**Loose medium-firm weak ground**	**180>Vs**

Density (d-gr/cm^3): The density value used in the relations is calculated with the following formula depending on the Vp velocity. $P=d=0.31*Vp^{0,25}$ (gr/cm^3).

Density (gr/cm³)	Definition
<1.20	Very low
1.20-1.40	Low
1.40-1.90	**Medium**
1.90-2.20	High
›2.20	Very high

<u>Seismic Velocity Ratio: (V_p/V_s)</u>: This defines the compactness and liquefaction of the ground. If there is a geological unit that is suitable for liquefaction in the medium, V_P / V_S ratio is considered in case there are underground waters, and if this ratio is bigger than 3, the probability of liquefaction is high.

Table 2.5 The compactness of the medium according to the VP/VS ratio

Ground Class	V_p/V_s
Z1	1.5-2.0
Z2	2.0-2.5
Z3	2.5-3.0
Z4	**3.0-10.0**

<u>Poisson Ratio (v)</u>: This ratio gives the rate of transverse fraction to longitudinal elongation. The Poisson Ratio for solid objects is ca. 0.25, and varies between 0 and 0.5 for various media. In media like water, this ratio closes to 0.5 limit. In aquatic media, the Poisson Ratio will increase since the V_s velocity will decrease. The Poisson Ration non-dimensional. $P=(V_p^2-2*V_s^2)/(2*V_p^2-2*V_s^2)$.

Table 2.6 Poisson Vp/Vs ratio and ground compactness table

Poisson Ratio	Compactness	V_p/V_s
0.5	Slosh-liquid	∞
0.4-0.49	Very loose	∞-2.49
0.3-0.39	**Loose**	**2.49-1.87**
0.20-0.29	Compact solid	1.87-1.71
0.1-0.19	Solid	1.71-1.5
0-0.09	Firm rock	1.5-1.41

<u>Maximum Shear Modulus (Shear Flex Resistance G_{max})</u>: This is the flexible torsional strength that indicates the flexion of the ground under shear force; and defines the torsion warping due to earthquake waves or lateral pressure separations. Since liquids do not have shear resistance, this parameter is zero. The higher the shear modulus, the greater the resistance of the formation to shear tensions, in other words, to horizontal forces (horizontal earthquake load). Its unit is kg/cm^2. $G_{max} = \rho * Vs^2 * 100$.

Table 2.7 The strength of the ground or rocks according to shear modulus values

Shear Modulus (kg/cm^2)	**Resistance**
<400	Very weak
400-1500	Weak
1500-3000	**Medium**
3000-10000	Firm
> 10000	Very firm

<u>Young Module (Dynamic Elasticity-Axial Flexion Resistance-E)</u>: This is the rate of vertical axis stress to vertical axial warping, and defines the warping of the ground under vertical pressure; therefore, it shows the warping of the ground in the case of a vertical load-loading or vertical lifting off the ground. If the flexion resistance of the medium is large, the shape change of the ground under stress becomes small. The unit is kg/cm^2. According to the values obtained so far, the ground types are given below. $E = G*(3*Vp^2 - 4*Vs^2)/(Vp^2 - Vs^2)$.

Table 2.8 The strength of the ground or the rocks according to Elasticity Module Values

Young Module (Kg/cm^2)	**Resistance**
< 1000	Very weak
1000–5000	Weak
5000–10000	**Medium**
10000-30000	Firm
> 30000	Very firm

Bulk Module (Wrapping Flex Resistance-k): This is the measurement of a mass under a pressure wrapping it. This defines the compaction of the grains or spaces under geological and environmental pressure. Its unit is kg/cm^2. $K=E/[3(1-2v)]$.

Table 2.9 The strength of the ground or the rocks according to the Bulk Module values

Bulk Module (m, Kg/cm^2)	**Compaction**
< 400	Very low
400–10000	Low
10000–40000	**Medium**
40000-100000	High
> 100000	Very high

Ground Bed Coefficient-Ko: The Bulk Module is made use of in calculating the Ground Bed Coefficient. The Ground Bed Coefficient is calculated with the following formula. $Ko=2.49*(1/(1/K*1009))^{1,26}*7$ (ton/m^3).

Ground Bearing Capacity (kg/cm^2): This shows the utmost capacity that may be carried by 1 cm^2 of the ground.

$d =0.31*V_p^{0,25}$ $q_{limit}=dVs/100(kg/cm^2)$

$q_{em}=dVs^2/100Vp(kg/cm^2)$.

Ground Settlement: This gives the maximum settlement amount of the ground under load based on seismic velocities.

$G_s=(qu/E)*1500$

Elasticity Module (E): This is the unit deformation ratio of the stress applied to an object, and reflects the firmness and hardness (i.e. the solidness) of the ground in mechanical terms.

Shear Modulus (G): This reflects the shear of the ground units due to lateral forces, in other words, the strength of the ground against lateral forces like earthquakes.

Table 2.10 Ground classes and comparisons

Ground Class	Ground Classification	Vs(m/s)	Vp/Vs	Gkg/cm^2	Edkg/cm^2
Zl	Very firm hard	>700	1,5-2,0	>10000	>30000
Z2	Firm-hard	400-700	2,0-2,5	3000-10000	10000- 30000
Z3	Medium firm-Broken	200-400	2,5-3,0	600-3000	2000- 1 0000
Z4	**Loose-Soft**	**<200**	**3-10**	**<600**	**<1700**

Ground Dominant Vibration Period (To-sn): Ground Dominant Vibration Period (To) is calculated by making use of the V_s wave velocity and seismic layer thicknesses and with the following formula: To=((4h1Vs)+4(30-h)V_s). Its unit is second, and shows that dominant period of the ground in dynamic status. When the Ground Dominant Period and the natural period of the engineering structure are the same in an earthquake (i.e. in resonance status), the damage is enormous.

To ensure the Structure-ground harmony, it is necessary to move the Structure Natural Period away from the Dominant Period of Ground. As the adequate removal criterion, it may be suggested that the Structure Period must be between 0.67 and 1.5 times of the Ground Dominant Period. Sliding Wave Velocity is one of the key parameters in ground works. It affects a Structure that is in contact with the ground during an earthquake because of its high amplitude. Sliding Wave has an important place in shear modulus, magnification and liquefaction analysis, which are important parameters.

Longitudinal Wave Velocities V_p (m/s): Longitudinal Wave Velocities occur if the material has resistance against compaction and expansion. The arrival times of the longitudinal wave signal output were measured at the receivers (geophones), and their velocities were calculated by drawing the graphic that corresponded to the receiving ranges.

2.3.4.2 Geophysical Study Field Applications and Evaluation Seismic Method

The 5-Layout Seismic Refraction 5-Layout Surface Wave Multichannel Analysis (MASW) Method was applied in the Study Area. The study depth was considered as 30 m. The MASW Method as applied to eliminate the deficiencies of the seismic break (long opening requirement, high noise level, etc.) and to detect the sliding wave velocity more accurately. Active source was employed in the present study, and flat and reverse shots were made.

The data that were obtained from the field works were evaluated by using the SeisImager Package Program, and the V_P (Compactional Wave Velocity), V_S (Sliding Wave Velocity) and S-wave velocity profile were obtained. The graphics and the screenshots of the evaluations are given in detail.

Table 2.11 The parameters obtained from seismic velocities

EXPLANATION	DEFINITION
Ground Type according to Seismic Velocities	D
Density	MEDIUM
Ground Type according to Seismic Velocity Ratio	Z4
Poisson Ratio	LOOSE
G-max Shear Modulus	MEDIUM
E-Young Module	MEDIUM
Bulk Module	MEDIUM

CHAPTER III
ENGINEERING ANALYSES AND EVALUATIONS

3.1.General Information on the Structure

The construction site, which is the subject of the present study, was registered in Sivas Province, Central County, Kardesler Neighborhood, 5495 Island, I38D01C Plot, 30^{th} Parcel in the Title Deed, and construction was built for housing purposes. The structure consists of Basement + Ground + 4 normal grounds. The structure is ca. 35.0 x 54.0 m in size; and the total area of it is ca. 1890.0 m^2. The foundation system is 70.0 cm-thick base slab foundation; and the foundation base elevation is constructed in a gradual manner as -5.81 m under the pool, and as -3.96 meters in other parts.

Dimensions: 35.0 m x 54.0 m

Foundation Depth (D_f) :-3.96m and -5.81 m; and the calculations were made based on D_f=-3.96 m.

Conveyed superstructure load: 10 700 t

Base Slab Foundation Weight: 3300 t

Total Net Load: 14000 t

As specified, the total load coming from the structure and the foundation was taken from the static projects, and was used in all necessary calculations.

3.2.Examining the Structure-Ground Relation

The Shear Box Test was done on the Undisturbed (UD) Samples taken from bore holes to determine the shear strength parameters (C, Ø and γn), and the Safe Bearing Value of the ground was calculated according to the analysis result. In the light of these data that were obtained as a result of the analyses that were made at Soil Mechanics Laboratory, the Bearing Value was calculated according to the formula of Terzaghi $\{qd = K1.C.\ Nc + \gamma 1\ .\ Df\ .\ Nq + K2.\ \gamma 2\ .\ B\ .\ N\gamma\}$.

q_d: Ultimate Bearing Capacity of the Foundation C: Cohesion of the foundation

D_f: Foundation depth (m) B: Foundation width (m)

γ_1: Volume weight of the unit over the foundation level

γ_2: Volume weight of the unit below the foundation level

N_c , N_q , N_γ: The Bearing Value factors found by making use of the friction angle of the foundation.

Φ = Shear Strength Angle qs=qd / Gs, Safety Coefficient (Gs = 3) (Gs: Safe Bearing Value)

Table 3.1 Type of the structure safety coefficient

Class	**Type of the Structure**	**Project Features**	**SAFETY COEFFICIENT**	
			Adequate ground study is fulfilled	**Adequate ground study is not fulfilled**
A	Railway bridge Warehouses Blast Furnaces Supporting Walls Silos	The biggest design load is used. Unstableness results in very bad outcomes.	3,0	4,0
B	Highway Bridges Light industry structures and business buildings	The biggest design load is used if necessary. Unstableness results in serious outcomes.	2,5	3,5
C	**Apartment buildings** **Official buildings**	**The biggest design load is not applied.**	2,0	**3,0**

Table 3.2 Bearing capacity coefficient

Table3.2 Bearing Capacity Coefficients Vesic, 1973)							
	N_c	N_q	N		N_c	N_q	N
0	5.7	1	0	**25**	25.13	12.72	8.34
1	6	1.1	0.01	**26**	27.09	14.21	9.84
2	6.3	1.22	0.04	**27**	29.24	15.9	11.62
3	6.62	1.35	0.06	**28**	31.61	17.81	13.7
4	6.97	1.49	0.1	**29**	34.24	19.98	16.18
5	7.34	1.64	0.14	**30**	37.16	22.46	19.13
6	7.73	1.81	0.2	**31**	40.41	25.28	22.65
7	8.15	2	0.27	**32**	44.04	28.52	26.87
8	8.6	2.21	0.35	**33**	48.09	32.33	31.94
9	9.09	2.44	0.44	**34**	52.64	36.5	38.04
10	9.61	2.69	0.56	**35**	57.75	41.44	45.41
11	10.16	2.98	0.69	**36**	65.53	47.16	54.36
12	10.76	3.29	0.85	**37**	70.01	53.8	65.27
13	11.41	3.63	1.04	**38**	77.5	61.55	78.61
14	12.11	4.02	1.26	**39**	85.97	70.61	95.03
15	12.86	4.45	1.52	**40**	95.66	81.27	115.31
16	13.68	4.92	1.82	**41**	106.81	93.85	140.51
17	14.6	5.45	2.18	**42**	119.67	108.75	171.99
18	15.12	6.04	2.59	**43**	134.58	126.50	211.56
19	16.56	6.7	3.07	**44**	151.95	147.74	261.60
20	17.69	7.44	3.64	**45**	172.28	173.28	325.34
21	18.92	8.26	4.31	**46**	196.22	204.19	407.11
22	20.27	9.19	5.09	**47**	224.55	241.80	512.84
23	21.25	10.23	6	**48**	258.28	287.85	650.67
24	23.36	11.4	7.08	**49**	298.71	344.63	831.99

Table 3.3 Form coefficient

K1 & K2 Calculation Formulas				
Figure Coefficient	**Strips**	**Rectangular (B<L)**	**Square (B=L)**	**Circle L=B=D**
K_1	**1**	**1+(0,2.(B/L))**	**1.2**	**1.2**
K_2	**0.5**	**0,5-(0,1.(B/L))**	**0.4**	**0.3**

Table 3.4 Bearing Capacity Analysis

LABORATORY DATA (c, ϕ) FOR RAFT FOUNDATION BEARING CAPACITY ANALYSIS		
Formula Parameters		**Data Entry**
c	Cohesion (kPa)	**28**
ϕ	Internal Friction Angle (degree)	**6**
D_f	Foundation Depth (m)	**3.5**
B	Foundation Width (m)	**35**
L	Foundation Length (m)	**54**
γ_1	Unit weight of the ground above the foundation base $(kN/m^3)^2$	**12.8**
γ_2	Unit weight of the ground under the foundation base $(kN/m^3)^2$	**12.8**
K_1	Foundation base shape coefficient	**1.13**
K_2	Foundation base shape coefficient	**0.44**
N_c	Bearing force coefficient	**7.73**
N_q	Bearing force coefficient	**1.81**
N_γ	Bearing force coefficient	**0.20**
GS	Safety coefficient	**3**

qd = K1 * c *Nc + γ1 *Df* Nq + K2 * Nγ *B* γ2		
Terzaghi (1943) Formula		
Results	**kPa**	**kg/cm²**
Foundation Bearing Capacity (q_d)	364.58	3.72
Net Bearing Capacity (q_d')	319.78	3.26
Foundation Safe Bearing Capacity (q_e)	121.53	1.24

Bed Coefficient (ks)

Table 3.5 Bed Module Ranges(Sub-ground reaction module ranges BOWLES J. 1996).

GROUND	**Ks, ton/ m^3**
Loose sand	480 – 1600
Medium firm sand	960 – 8000
Firm sand	640 – 12800
Clayey medium firm sand	3200 – 8000
Silty medium firm sand	2400 – 4800

Clayey ground:	
$q_u \leq 200$ kPa	1200 – 2400
200 < $q_u \leq$ 800 kPa	**2400 – 4800**
$q_u > 800$ kPa	> 4800

The average of the vertical bed coefficient value must be taken as ks=2450 t/m³ in the safe side by considering all the boreholes as the bases to be used in foundation calculations. This value is the average of the values suggested for grounds by B. M. Das (Foundation Engineering) and Bowles (Foundation Engineering) in their Foundation Engineering books.

3.3.Ground Profile

The Standard Penetration Test (SPT), which was done during the boring work, the number of the blows, the ground classification data according to the Combined Ground Classification System-USCS and according to the laboratory results are given below.

Table 3.6 Ground profiles

SK1	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 6.00 m	Clayey Gravel (GC)
6.00 – 7.50 m	Clayey SAND (SC)
7.50 – 9.00 m	Clayey Gravel (GC)
9.00 – 13.50 m	Clayey SAND (SC)
13.50 – 19.50 m	Clayey Gravel (GC)

SK 2	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 4.50 m	Clayey SAND (SC)
4.50 – 9.00 m	CLAY with high plasticity (CH)
9.00 – 20.00 m	Clayey Gravel (GC)

SK 3	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 20.00 m	Clayey SAND (SC)

SK 4	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 20.00 m	Clayey SAND (SC)

SK 5	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 7.50 m	Clayey SAND (SC)
7.50 – 20.00 m	Clayey Gravel (GC)

SK 6	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 12.00 m	Clayey SAND (SC)
12.00 – 20.00 m	Clayey Gravel (GC)

SK 7	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 6.00 m	Clayey SAND (SC)
6.00 – 13.50 m	Clayey Gravel (GC)
13.50 – 15.00 m	Well-graded Clayey Gravel (GW-GC)
15.00 – 20.00 m	Clayey Gravel (GC)

SK 8	
Depth	**Soil Type**
0.00 – 0.30 m	Vegetable soil
0.30 – 3.00 m	Clayey SAND (SC)
3.00 – 20.00 m	Clayey Gravel (GC)

3.4.Liquefaction and Lateral Spreading Analysis and Evaluation

Liquefaction may be defined as the conversion of the layer into liquid form as a result of the layers such as water-saturated fine-grain sand and silt increasing the pore water pressure value during earthquake vibrations as a result of effective lateral stress being zero.

In the study area, a depth of maximum 20.00 meters was bored, and the groundwater was found at 3.00 meters. It was calculated that the units that constituted the foundation had liquefaction risk between 8.00 meters and 10.00 meters. Geotechnical reports must be prepared and necessary improvement work must be performed to eliminate the liquefaction problem between the abovementioned meters.

3.5.Settlement-Swelling and Mounting Potential Evaluation

Such settlements consist of the deformation in the ground with the vertical load that occur due to the load on the ground with the water in it. These s-deformations are valid for clayey and silty grounds. Sudden and consolidation settlement cannot be separated in sandy grounds. Because the water in water-saturated sands will be removed rapidly when the ground is under load and the sand is not among impermeable layers. This will cause that settlement will occur very quickly.

Consolidation Settlement: Hd: The layer at H thickness double-side permeable (drainage distance)

This is found with the following formula:

Cv : Consolidation coefficient T : Consolidation

$\rho c = \gamma \ . \ \Delta H$

If ΔH; $\Delta H = mv \ . \ \Delta\rho \ . \ H$ (ρi may be ignored in practice, and it is assumed that $\gamma = 1$)

It is calculated as $\rho = \rho i + \rho c$ $\rho = mv \ . \ \Delta\rho \ . \ H$

ΔH =Total settlement mv = Volume compaction coefficient

Δp = Medium pressure increase in the ground H = Thickness of compressible ground layer

Z = Foundation depth

Table 3.7 Allowable settlement amount according to Skempton

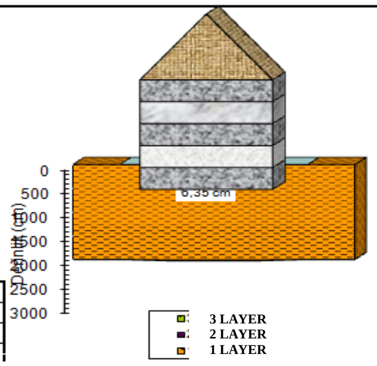
	Ground Type	**Total Settlement**	**Different Settlement**
Single Footings	Clay	6 cm	4 cm
	Sand	4 cm	2,5 cm
Base Slab Foundation	**Clay**	10 cm	**4 cm**
	Sand	6 cm	**2,5 cm**

CONSOLIDATION SETTLEMENT CALCULATION

PROJECT NAME	HİLMİ GÜL VE MÜŞT
PROVINCE	SİVAS
DISTRICT	MERKEZ
PARCEL NO	VAS/MERKEZ İ38D01C/5495/
NO OF	5
	SK-1

Pressure Intervals Kg/cm²		Volumetric Compression Coefficient (cm2/kg) 1 Layer	2. Layer	3. Layer
0,01	0,5	0,0185		
0,5	1	0,0068		
1	2	0,0039		
2	4	0,0053		
4	8	0,0054		

LAYERS	Depth (m)	γn	Gs	γk
1 LAYER	20	1,93	2,7	1,58
2 LAYER				
3 LAYER				



SETTLEMENT CALCULATION

DATA	
$q_{surface}$	1,24 kg/cm2
Df	3,50 m
t	1,0 m
γn1	1,93 ton/m3
YSS Der	3,00 m
B	35,0 m
L	54,0 m
Foundation Type	RECTANGULAR
Settlement Depth	20,00 m
Nih Sett. Depth	20,00 m

RESULTS

As per the Data

Total Settlement 6,346 cm

Depth m	γn	Gs	γk	γd	P Middle	Δσz	ΔP	Mv	H Settlement
4,00	1,93	2,70	1,58	1,99	0,398	0,562	0,285	0,0185	1,040
5,00	1,93	2,70	1,58	1,99	0,497	0,560	0,382	0,0185	1,036
6,00	1,93	2,70	1,58	1,99	0,597	0,556	0,477	0,0185	1,029
7,00	1,93	2,70	1,58	1,99	0,696	0,551	0,572	0,0068	0,375
8,00	1,93	2,70	1,58	1,99	0,796	0,544	0,664	0,0068	0,370
9,00	1,93	2,70	1,58	1,99	0,895	0,536	0,756	0,0068	0,364
10,00	1,93	2,70	1,58	1,99	0,995	0,526	0,846	0,0068	0,358
11,00	1,93	2,70	1,58	1,99	1,094	0,516	0,934	0,0068	0,351
12,00	1,93	2,70	1,58	1,99	1,194	0,504	1,022	0,0039	0,196
13,00	1,93	2,70	1,58	1,99	1,293	0,491	1,109	0,0039	0,192
14,00	1,93	2,70	1,58	1,99	1,393	0,478	1,195	0,0039	0,186
15,00	1,93	2,70	1,58	1,99	1,492	0,464	1,281	0,0039	0,181
16,00	1,93	2,70	1,58	1,99	1,592	0,450	1,366	0,0039	0,176
17,00	1,93	2,70	1,58	1,99	1,691	0,436	1,452	0,0039	0,170
18,00	1,93	2,70	1,58	1,99	1,791	0,422	1,537	0,0039	0,164
19,00	1,93	2,70	1,58	1,99	1,890	0,408	1,622	0,0039	0,159

Figure 3.1 The settlement calculation according to the consolidation experiment data made on the SK-1UD sample

3.6.Foundation Ground Geotechnical Evaluation

To examine the ground of the structure-foundation that consisted of B+Z+4N that is planned to be built in Sivas City, Central County, Kardeşler Neighborhood, 5495 Island, I38D01C Plot, 30 Parcel, a total of 160.0 meter boring was carried out as SK1, SK2, SK3 and SK4 borings in February 2016, and SK5, SK6, SK7 and SK8 borings in April 2016 at 8 borings at 20.0 meter depth. As stated previously, the underground water was found at -3.0-meter depth in the boring works. The Boring Logs were given in previous sections.

The construction site in question consists of Clayey SAND (SC) and Clayey CONGLOMERATE (GC) layers; and continues with the alternation of these layers. The average blow count of the standard penetration test in 20.0 m depth from the ground surface was determined as SPT N30 = 5. This value shows that the ground does not show a reliable structure in terms of the carrying power and settlement of the ground along the 20.0 m depth from the surface. However; the fact that the underground water level being close to the surface makes the already unreliable structure even more unfavorable in terms of the carrying power and settlement. After 20.0 m depth, there is a layer in which the SPT N30 blow counts increased at ca. 8.0 m thickness (20.0 - 28.0 m.) and SPT N30 blow count increased (16, 24, 32, 39, 42). In addition, it was calculated that there was a liquefaction risk at the layer between 8,00 meters and 10,00 meters in the units that constitute the foundation, and this calculation was presented in earlier sections. To eliminate the liquefaction problem between the meters mentioned, it is recommended that a geotechnical report is prepared and necessary improvement works are made. In the light of all these evaluations, the layer mentioned may be adequate to provide the carrying capacity and settlement criteria of the structural loads. For this reason, the structural loads mentioned will be transferred to a layer that is more robust through Ground Improvement Methods. When the advantages of the improvement methods are considered, Jet Grout Column Method is prominent. Jet Grout Columns will cover the superstructure loads through edge-carrying capacities. It must be considered that the ground masses around the columns will also be improved with the Jet Grout Column construction. General Ground Improvement methods and the preferred method i.e. Jet Grout Column Method will be described in a detailed way in the following section.

CHAPTER IV
GENERAL VIEW ON GROUND IMPROVEMENT METHODS

Ground Improvement Methods will be examined in 4 groups as recommended by Hausmann (1990). The methods and relevant principles are as follows:

1) Mechanics,

2) Hydraulics,

3) Physical and Chemical Improvement,

4) Improvement by Using Equipment.

The purpose is to increase the shear strength of the ground, to improve ground behavior under important loads, to reduce settlement, to prevent sudden liquefaction that may occur at the time of earthquake, etc. by applying these methods (Gökçer Özen, 2018). In this respect, the Ground Improvement and Geotechnical Works in Istanbul will be evaluated.

4.1.Mechanic Improvement:

Mechanic Improvement is the process of the compaction of the ground by external forces. It may be considered as identical to compaction. In actual fact, this process, which may also be called as the pulse of short-time mechanical forces, sudden heavy loading is applied for a short duration or the ground is crushed under a certain load. The target is the compaction of the ground by reducing the air volume in the ground or bringing together the grains of the ground in a closer manner with these methods. There are several methods and equipment like static, vibrating or blow-cylinders that involve superficial compaction techniques for the layers close to the surface; dynamic compaction, Rapid Impact Compaction (RIC), Vibro Compaction and Vibro Displacement (stone column) techniques for deeper layers (Osman Murat Sarsılmaz, 2017).

4.2.Hydraulic Improvement:

Hydraulic Improvement involves the dehydration techniques of the ground. In general sense, dehydration may be defined as the improvement of the ground by reducing the underground water level. Free void water is pushed to come out of the ground through drainage or well. In grounds that have large grains, this process is performed as drawing water from boring hole with pumps to reduce the underground water level; in grounds that have thin grains, long-term external forces (preloading) or electric forces (electro-kinetic stabilization). Traditional techniques made use of the development of geosynthetics like vertical drains. Diaphragm walls, plain walls, and geomembranes may be named in this group in which pressurized air is used in caisson and tunnel to remove the ground water.

4.3.Physical and Chemical Improvement:

Improvement may be carried out by adding additives to the physical mixture and to the ground blocks or to the layers on the surface in a form that will constitute columns in deep areas. Natural grounds, industrial by-products, cement material that react with the ground and chemical materials may be used as additives. The additive may be added to the ground surface or may be injected to the voids in the ground. The ground may also be improved by heating or freezing. Heating causes a permanent change in the mineral structure of the ground by vaporizing the water in it; and freezing strengthens the bond between the grains of the ground by freezing the water in the ground. In this part, injection, stabilization and deep mixing methods are dealt with. Especially in recent years, jet grout column application, which is applied widely in our country, is included in this group, and is dealt with in detail in further parts of the study.

4.4.Improvement by using Reinforcement:

“Reinforced ground” is the strengthening of the ground by using materials that can resist against tensile strength and that are in interaction with the ground (with friction and/or adhesion forces). Fibers, stripes, bars or nets may be placed in the ground to give it tensile strength.

Reinforcement may be applied with nailing and anchorage in the field, or may be carried out in the form of retaining wall, concrete, steel or synthetic elements that capture the ground and ensure stabilization.

Under this title, there are ground reinforcement, Mechanic Stabilized Reinforced retaining structures, Reinforced Slopes, Ground Nails, Anchorages and use of geocomposite as reinforcement.

Ground Improvement Methods were classified in different ways by different researchers. Some researchers classified these methods according to the type of the ground, some according to both the type of the ground and the additive to be used, an some according to the changes on the improvement method or the method.

The classification of Varaksin (2010), who made a classification according to the use of additive and the type of the ground, is given in Table 4.1.

Table 4.1. Classification of Ground Improvement Methods (Varaksin, S., 2010)

Category	**Method**	**Principle**
A.Soil improvement method without additives for non-cohesive grounds or fillers	**A1. Dynamic Compaction**	A heavy mass is dropped from the air to the granular ground and the soil is compressed.
	A2. Vibro-compaction	A vibratory catheter is lowered onto the granular ground and the ground is compressed.
	A3. Explosive Compaction	The granular ground is compressed by using shock waves and vibration created by the explosives.
	A4. Compaction under Electric Current	The granular ground is compressed by using the energy generated by shock waves and ultra- high voltage electrical signal.
	A5. Surface Compaction	The surface of the ground or filler is compressed to a shallow depth by utilizing various compaction tools.

B.Without additives Cohesive Soil Improvement Methods	**B1. Ground Replacement**	It is the process of dumping a bad ground with a good ground or rock. It can be used to reduce the pressure of the load of ground with some light materials.
	B2. Pre-loading using the filler (including the use of drains).	It is ensured that the ground is pre-consolidated by filling without building the structure on a consolidated ground. Thus, the ground will be less consolidated when facing with the subsequent loads (settles)
	B3. Pre-loading using Vacuum (Including Mixed Filling and Vacuum Filling)	It is ensured that the ground is pre-consolidated by applying a vacuum pressure up to 90kPa to the ground. Thus, the ground will be less consolidated when facing with the subsequent loads (settles)
	B4. Dynamic Compaction with advanced drainage or vacuum	As in Method A1 but as it is applied into a cohesive ground, horizontal and vertical drains are used to reduce the pore water pressure during the compaction.
	B5. Electro-osmosis or electro-kinetic consolidation	When direct current is applied to the ground, water or solution is allowed to flow from the anode to the cathode.
	B6. Thermal stabilization (by heating or freezing)	The mechanical, physical properties of the ground can be improved temporarily or permanently by filling or heating the ground.
	B7. Compaction with Hydro-explosion	A compressible (collapsible) soil is allowed to be compressed by heating it through the borehole and deep blasting.

Category	Method	Principle
C. Improvement Methods by adding support materials	**C1. Vibro-Displacement or Stone Column**	The vibrofloat is lowered into the soft, fine-grained soil and the column-shaped bore-hole is filled with gravel or sand fills.
	C2. Dynamic Displacement	Aggregates are pushed into the ground with a high energy dynamic pulse to form a column. The fills can be sand, gravel, stone or remnants of proper debris.
	C3. Sand Compression Piles	Sand is placed into the ground through a protective pipe; squeezed and compressed by vibration, dynamic pulse or static stimulation to have the form of a column.
	C4. Geotextile Coated Columns	Sand is placed into the ground through a closed-end cylindrical geotextile to have the form of a column.
	C5. Rigid piles or composite base additives	Rigid, semi-rigid piles or columns are prefabricated and applied into the ground or to the ground, in order to make soft soil gaining strength.
	C6. Geo-synthetically Supported Column or Pile Supported Berm	Pile, Rigid or semi-rigid columns and Geo-synthetically reinforced columns are used to stabilize the pavements and reduce the settlement.
	C7. Microbial Method	By utilizing microbial material; increasing the resistivity of the soil and decreasing the permeability of the soil is provided.

	C8. Other Methods	Non-traditional methods. Making sand piles with blasting, Using bamboo, wood-like natural materials.
D.Ground Improvement Methods by Using Injection Type Additives	**D1. Chemical Injection**	Two or more chemical solutions react in the ground cavities and precipitate in gel or solid form, thereby reducing the strength and permeability of the soil.
	D2. Mixing Methods	Cement, lime or other binding material is added into the weak ground and mixed
	D3. Jet Injection	Columns and panels are formed by injection into the ground with high speed jets.
	D4. Compaction Injection	Injections such as highly stucco plaster are injected into different ground areas to ensure that the loose ground is tightened or the seated ground is raised.
	D5. Compensation Injection	Medium-High viscosity suspension is injected between the excavation ground and the building underground, and the settlement of the structure due to the excavation is reduced or eliminated.
E.Soil Improvement Methods Equipped with;	**E1. Geo-synthetically or mechanically stabilized grounds**	By using a variety of steel or geo-synthetically stabilized materials, the shear strength of the grounds of the road ground, foundations, shelters, slopes or abutment structures are increased.
	E2. Ground anchor	The stability of the slope or abutment structures is adjusted

	or nail	by using the built-in nail or the tensile strength of the anchor.
	E3. Biological Methods of planting	The plant root reinforcement provides slope stability.

Soil Type	Stabilization Method
Various Fillers	
Shallow	Excavation / Backfill
Deep	Dynamic Compaction
	Sand Columns
Organic Soils	
Shallow	Excavation / Backfill
	Geotextile
Deep	Surcharge
	Geotextile
	Sand Columns
Layered	Surcharge
	Blasting Technique
	Dynamic Compaction
	Compaction Injection
	Sand Columns, Stone Columns
Soft Clay	
Shallow	Excavation / Backfill
	Geotextile

Deep	Surcharge
	Geo-synthetics
	Sand or Lime Columns
Layered	Surcharge
	Dynamic Compaction
	Compaction injection
	Sand or Lime Columns
Clay	
	Mechanical Stabilization
	Lime Stabilization
	Thermal (Freezing), Electro-Osmosis
	Geo-synthetics
	Drainage
	Lime or Cement Columns
Loose Silts	
Shallow	Excavation / Backfill
	Salt Additive
	Dynamic Compaction
Deep	Surcharge
	Stone Columns
	Electro-Osmosis
	Vacuum Wells
Loose Sands	
Shallow	Cement or Bitumen Stabilization
	Dynamic Compaction
Deep	Vibro-flotation

	Vibro-compaction
	Dynamic Compaction
	Stone Columns
	Injection
	Drainage Wells
	Thermal (Freezing)
Inflatable Soils	Dynamic Compaction
	Stone Columns
	Drainage
	Injection
Swellable Soils	Lime Stabilization
	Drainage
	Cement, fly ash, salt, chemical additives
	Injection
Cracked Rocks	Injection
	Anchors
	Shotcrete
	Sub-surface horizontal Drainage

As a result of long-term studies, and when compared to the Ground Improvement methods used, it was decided to apply the Jet Grout Column Method. Each feature of the method is discussed in detail in the next part.

CHAPTER IV
JET GROUTING METHOD

Jet Grouting Method has been an effective method used in solving many geotechnical problems. The diversity in jet grout systems allows users to apply different production parameters to perform design criteria (Burke, G., K., 2004). Jet Grout Method is applied for the purpose of improving the ground or to transfer the structural loads to deeper layers. This technique brings great design functionality in many different ground conditions and in a wide range of application fields. These include the structures to which the brecaet excavation is applied and for which safety is important (Fırat, A. T. 2001). When Jet Grout Method is compared with other classical methods, its advantages come to the forefront clearly. The injection methods that are used today cannot change the ground features at a great deal; however, the Jet Grout Method is applied by mixing the ground with cement grout. In this case, a part of the ground exchanges places with the cement grout, and has a direct and positive effect on the ground strength (Fırat, A. T., 2001).

5.1.History of Jet Grouting Method

The first Jet Grouting Method applications started in the middle age by utilizing the erosion effect of the water; and were generally accepted in the mining sector at that time (Doğanışık S.K). Although the foundation works for Jet Grouting Method were carried out in the UK in 1950s, the first practical development of a real Jet Grouting Method was realized in Japan (Doğanışık S.K.). At first, this technology was intended to deform and separate the untreated or partially treated grounds with the effect of water jet. Then, cement-based chemical mixtures were sent to the separated matrices, and thus, impermeable regions were obtained. Following this step, Jet Grouting Method was used in creating thin sealing screens (Moseley, M., P. and Kirsch., K., 2004).

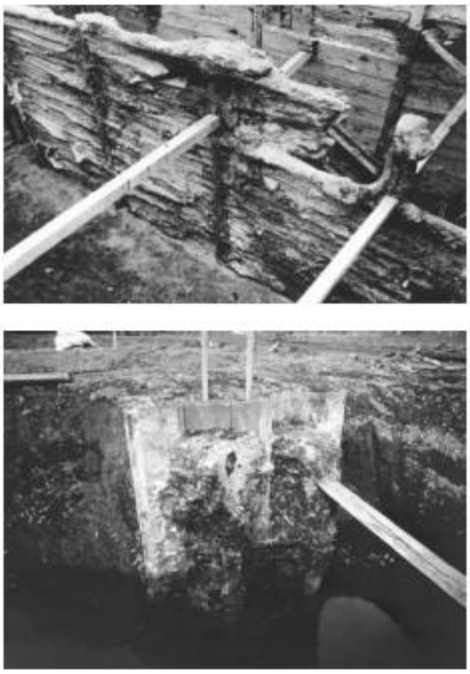


Figure 5.1 Exposed jet grout panels (Moseley, M., P. and Kirsch., K., 2004)

Since early 1970s, the Rotational Jet Grouting System was developed as a result of the lack of adequate thickness of the paneled jet grouting system and due to the low resistance of the fracture resistors in Japan (Moseley, M., P. and Kirsch., K., 2004).

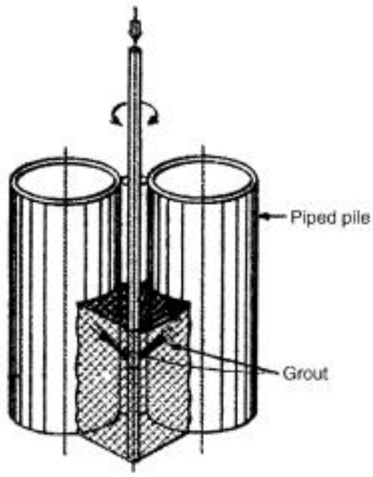


Figure 5.2 Jet grout sealing between piles (Moseley, M., P. and Kirsch., K., 2004)

The Jet Grouting System spread worldwide with the licenses of geotechnical contractor groups particularly in Germany, Italy, France, Singapore and Brazil by the end of 1970s (Xanthakos, P., P., Abramson, L., W. and Bruce., D., A., 1994).

5.2.Jet Grouting Method Foundation Principles

Jet Grouting application is divided into two main titles as drilling and injection. Firstly, drilling is started with a certain diameter (e.g. 120 mm). The drilling is made until the suitable depth that is determined in the project. After the drilling reaches the specified elevation, high-pressure cement injection (400-600 bar) is started from the horizontal injection nozzles located on the monitor. Meanwhile, the rods and the monitor, which is connected to the rods, are gradually drawn to the upper elevations with equal times by rotation. The injection that is at high pressure and velocity coming from the nozzles ensures the formation of cementitious ground mass by mixing with the ground with the help of the rotation of the rod monitor system. When this mechanism continues to be drawn towards the upper elevations, this results in cylindrical jet grout columns on the ground (Warner, J., 2004).

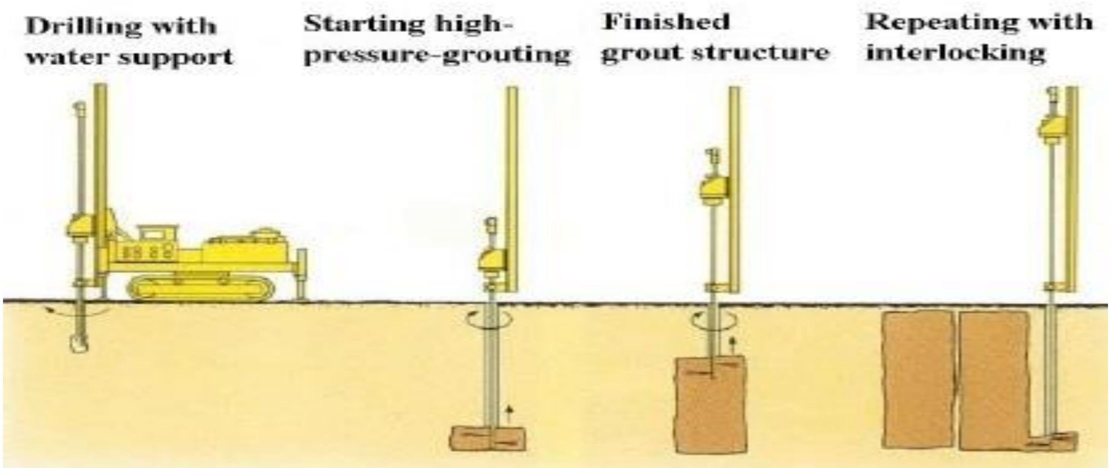


Figure 5.3 Execution of a jet grout body (http://www.es-en.com.tr/muhendislikalani/jeoteknik-zemin-etudu/zemin-etudu-zemin-

Forming effective jet grout column diameter and resistances varies according to the ground features and jet grout application parameters (Bell, 2003).

5.2.1.Jet grouting application equipment

Jet grout column is formed by employing some machinery and auxiliary equipment, which are explained as sub-titles (Doğanışık, S.K.), and are shown in Figure 5.4

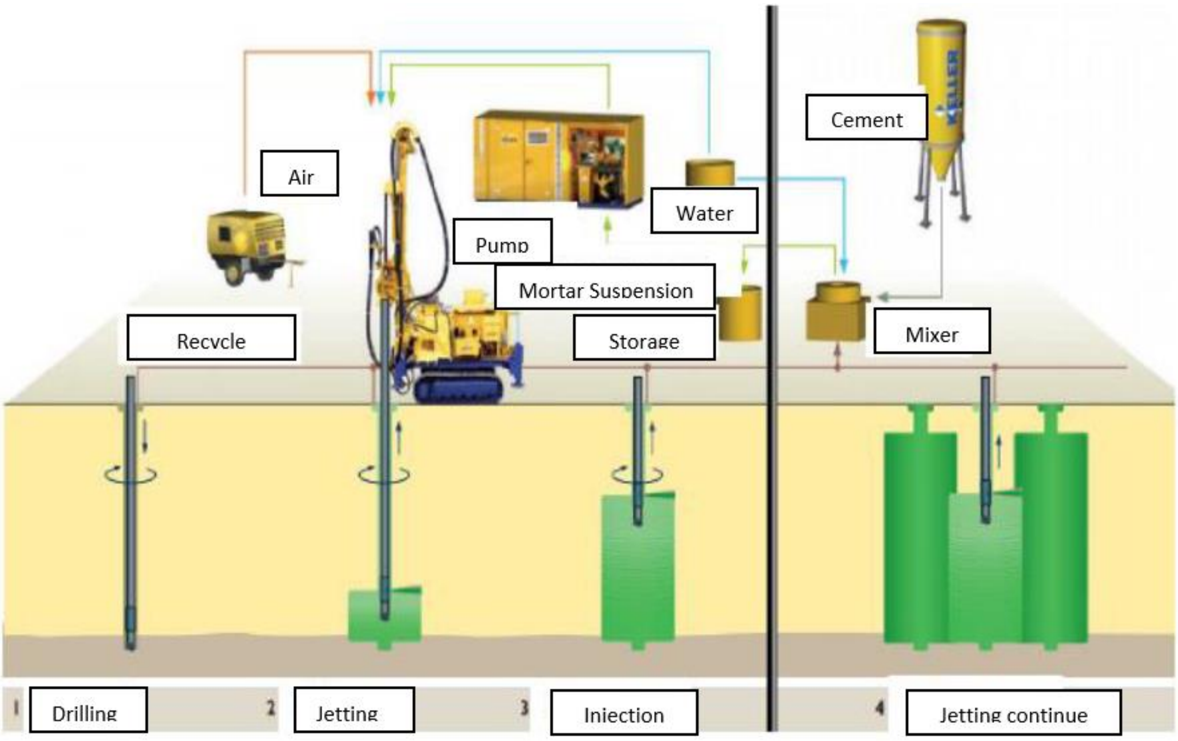


Figure 5.4 Jet grout drilling machine

5.2.1.1.Drilling machines

It is the machine that performs the drilling that has the capacity to form the Jet grout column project depth (Figure 5.5).

Drilling machines;

- consist of hydraulic systems,
- have the pallets that enable them to move in maximum level in problematic field conditions,
- have the torque and pressure power at adequate level.

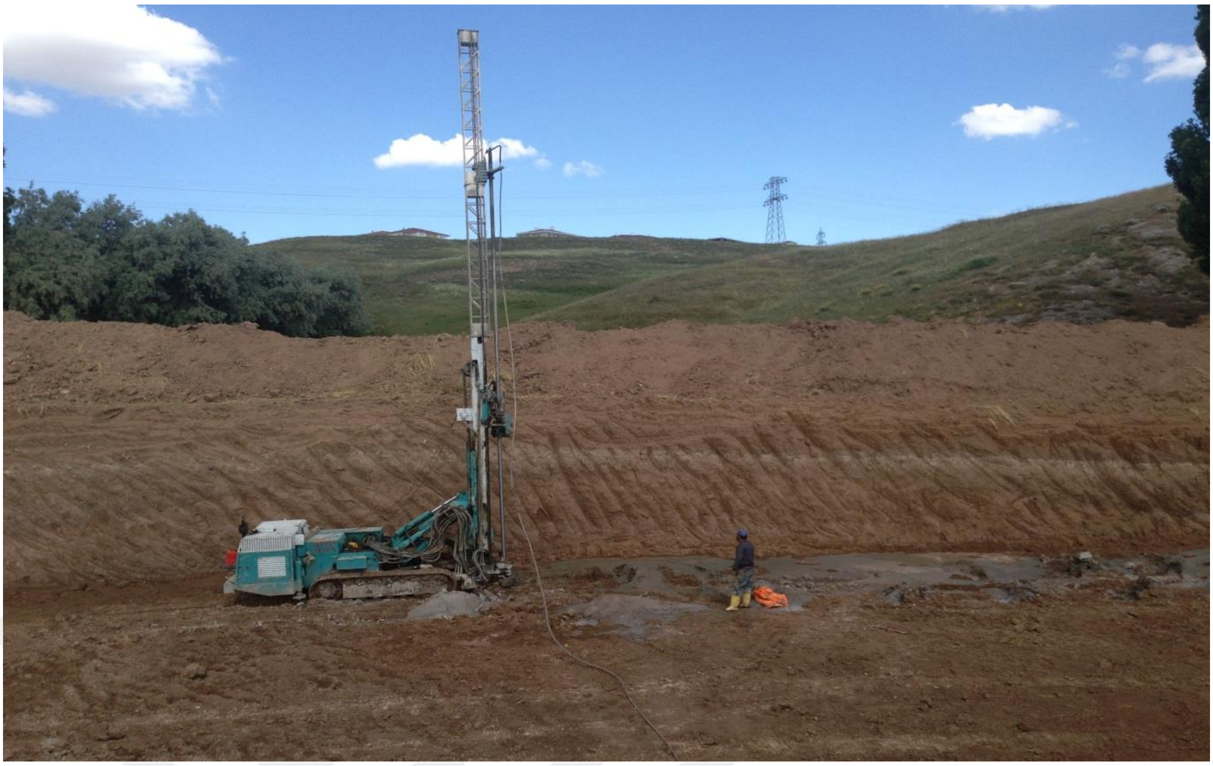


Figure 5.5 Jet grout drilling machine

5.2.1.2. Injection pump

It is a machine, which separates the ground by sending high-pressure cement injection to it at up to 600 bars. The continuity of the pressure that is produced is ensured by the coordinated suction-pressure of the piston equipment. A view from the jet grout injection pump is given in Figure 5.6 (Doğanışık S.K.).

Figure 5.6 Injection pump

5.2.1.3. Injection preparation unit

Injection preparation unit is a mixer consisting of mixer and resting boilers and prepares cement injection at desired water-cement ration according to jet grout project. The cement injection prepared is transferred firstly from the mixer tank to the resting tank, and is then absorbed by the injection pump and pumped to the ground with high pressure. The injection preparation unit is shown in Figure 5.7 (Doğanışık S.K.).

Figure 5.7 Injection preparation unit

5.2.1.4. Water head and rods

Water head is a kind of adapter that has a movable head ensuring the connection between the hoses that endures high-pressure connected to the injection pump in one end with the rods that are connected to the boring machine. Rods are the equipment that passes water while drilling, and that passes high-pressure injection during jet grout column formation ensuring that monitor adapter set and drill tip descend to the desired project elevations (Doğanışık S.K.).

5.2.1.5.Monitor adapter set

The Monitor adapter set constitutes the final ring of the water head and the rod system that are connected to the drilling machine. It is the equipment that constitutes the jet grout column and is also the last equipment for injection exit. The drill tip is attached to the tip of this equipment. The monitor may consist of single, double or triple walls which is also the case in water head and rods according to the jet grout column creating methods. The adapter set is shown in Figure 5.8 (G. N. Karahan, 2016).

Figure 5.8 Monitor adapter set

The monitor has the following features (TS EN 12716, 2002):

For the single system, there is one or more circular sprayers which make the injection grout into jelly. These sprayers may be at the same level or at different levels enabling alternated work. For the dual (air) system, one or more double sprayers (at the same or different levels, at angles that allow alternating operation) are used for simultaneous jet formation of air and injection syrup. The air sprayer wraps the circular grout sprayer like a ring.

One or more sprayers placed deeply to send cement mixture with one or more sprayer to make the water become a grout for the dual (water) system.

For the triple system, one or more double sprayers in simultaneous jetting process of water and air, and one or more normal sprayers placed in a deeper area to ensure that injection grout is sent.

5.2.2. Jet grout application stages

5.2.2.1 Drilling stage

The drilling type is selected depending on the characteristics of the ground that will be drilled. In the first stage, the ground is drilled to the foreseen depth for trial purposes. During the drilling, the well mouth is preferred to be over the ground water level to facilitate working (Askay, A 2002).

Various fluids like water, air, bentonite suspension or cement-water mixture are employed to facilitate the drilling process, cool the tip set and prepare the ground for injection during drilling. As the tip set, clay bits are usually employed on soft grounds, trichome bits on hard grounds, and pistol bits on blocky grounds. As a drilling rod, special production drill rods that are resistant to high pressure are employed in the coupling sleeves (felt) that are resistant to 600-700 bar pressure (Askay, A 2002).

Figure 5.9 The water used in drilling process (Kimpritis, T, 2013)

5.2.2.2 Injection stage

Injection mixture is prepared by mixing specific amount of water and cement in the mixer. The water/cement ratio is adjusted according to the application area (Küsin, C.C., 2009).

Impermeability and filling of fine cracks are realized with bentonite mixture. The injection mixture that is prepared in the mixer is taken to the resting element, and is given to the well with a pump through hoses. When the drilling process is completed, the cement grout that consists of water/cement mixture is drawn to the ground with a very big pressure depending on the grout density, and the drilling rod is pulled up. In this way, the high-speed injection mixture corrodes the ground and disrupts its structure and therefore enters the cement, which creates a circular column on the ground. The rotational movement of the drilling rod creates a structure that consists of vertical columns that are combined with a predetermined and constant velocity pull movement (Xanthakos, P., P., Abramson, L., W. and Bruce., D., A., 1994).

During the pressure injection, it is considered normal that a certain amount of ground material exceeds around the drilling set. This shows that there is no excessive pressure in the ground that is mixed with the grout. Because when there is excessive pressure, the excessive pressure escapes out from the void that is formed due to the diameter difference drilled with drilling rod diameter. The amount of the material escaping out during pressure injection depends on the permeability and type of ground. This amount will be more on clayey soils, and less in sandy-conglomerate grounds, and may be 10% of the amount injected (Xanthakos, P., P., Abramson, L., W. and Bruce., D., A (1994)). In jet grout application, the injection is given in Figure 2.10; and Figure 5.11 shows an image of the jet grout application at the end of the injection.

Figure 5.10 Injection stage

Figure 5.11 An image of the jet grout application after the injection

5.2.3 Methods and types of Jet Grout Colum formation

In practice, Jet Grout Columns are created by employing 3 different foundation methods, which are single-fluid system (Jet1), double-fluid system (Jet2) and triple-fluid system (Jet3) (Figure 2.12). Although these methods are similar to each other as basis, the this that differentiates them is the additional apparatus and mixture materials. The decision of the materials to be used is made based on the features of the ground, the purpose of the application, the application area, the diameter, pressure resistance the volume and permeability of the application area that will be improved (Fırat, A. T. 2001).

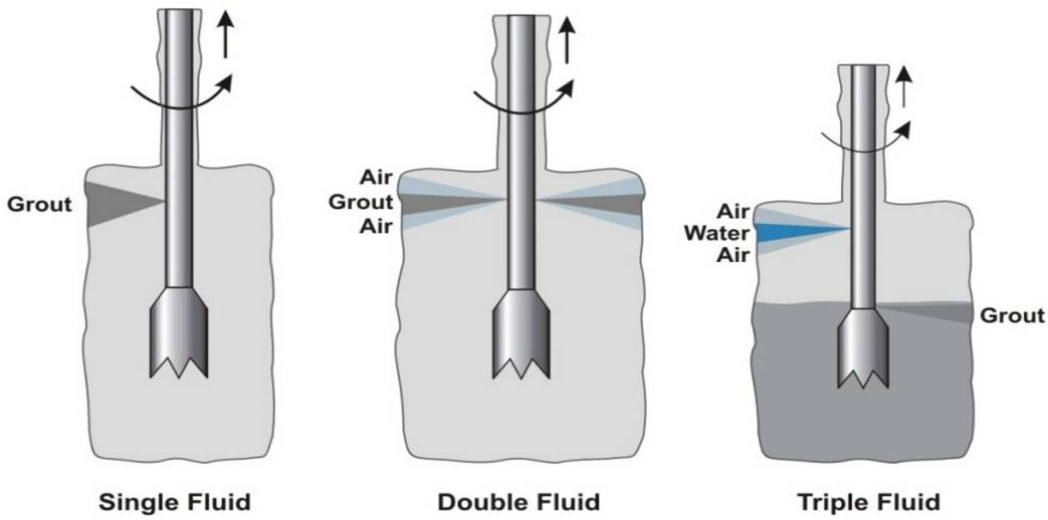


Figure 5.12 Jet grout systems

5.2.3.1 Single-fluid method

The single-fluid grout method is the simplest one among the three systems. The reason for this is that there is one single jet flow that performs the duty of cutting, corrosion, removal and mixing the injection. There are one or more spray nozzles in which the injection is transferred with 2.0 - 4.0 mm diameters close to the lower end. The number of nozzles is directly proportional to the loss of energy, the number of spray nozzles being small also decreases the energy loss. Most of the energy loss in low-diameter nozzles stems from the cloud that is formed by the injection in the output area of the nozzle. This clouding event decreases in relatively large diameter nozzles (Fırat, A. T. 2001).

It is mostly preferred for applications in horizontal manufacturing, especially in tunnel construction applications, and in ground conditions that have liquefaction potential because air is not used in it. In the single-fluid method, the residual material that is taken to the surface is less than in other jet grout systems (Melegary, C. and Garassino, A. L., 1997).

In the single-fluid (Jet1) system, the column may be formed up to 50-120 cm diameter in loose, non-cohesive granular grounds; and up to 40-80 cm diameter on cohesive grounds. The elements that have the highest strength are formed with the single-fluid system in granular soils. The void rates in grounds are less than in other two systems (Melegary, C. and Garassino, A. L., 1997).

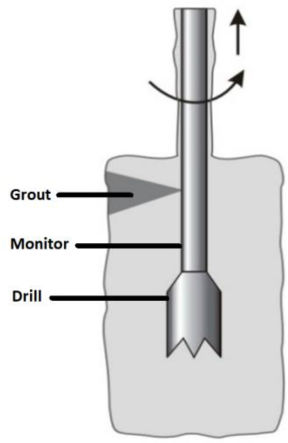


Figure 5.13 Single fluid system (Kaymakcı, S. 2014)

5.2.3.2 Double-fluid method

A second rod set is placed in the equipment that is employed in Jet1 Method. Another name is Jet2. As it is the case in Jet1 Method, the cement injection passes through the jet grout pump and the innermost wall. The air produced by the compressor at a pressure of 5-12 bar is dragged through the void between the outer wall and the inner wall. Air and cement injection meet the ground at the same time from the spray nozzles. As the air flow that is sent reduces the kinetic energy friction losses partly in the Jet2 method, the column diameters created with this method are 55-75% larger than in the Jet1 Method. an application of this method was made by Ichihashi et al., 1992 (Doğanışık S.K., 2014).

In Jet2 Method, an air nozzle is usually employed. The reason for this is the difficulty in detecting that one of the two airways is obstructed by the operator. Such a situation makes it difficult to attain the target of the method. In addition, the diameters of the nozzles used in Jet2 may be larger than Jet1 (2.5-4.5 mm). For this reason, in Jet2 method, the risk of breaking ground is less (Stark, T., D., Axtell, P., J., Lewis, J., R., Dillon, J., C., Empson, W., B., Topi, J., E. and Walberg , F., C., 2009).

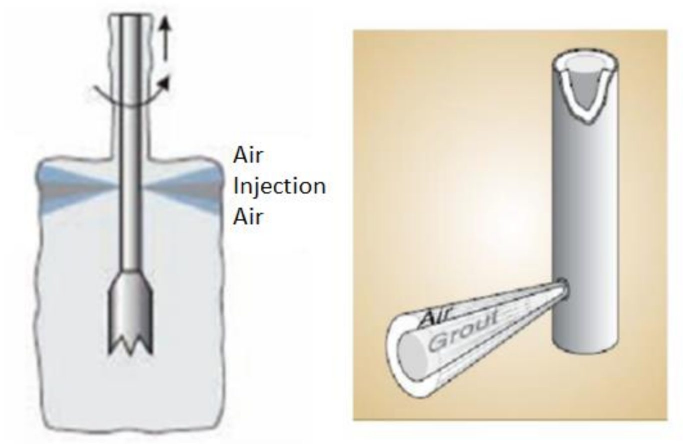


Figure 5.14 Double fluid system (Stark, T., D., Axtell, P., J., Lewis, J., R., Dillon, J., C., Empson, W., B., Topi, J., E. and Walberg, F., C., 2009)

5.2.2.3 Triple-fluid method

This method is applied by passing the cement injection, water and air through the three-wall drilling equipment. High pressure cement injection sent from jet grout pump passes through the innermost rod, water sent with 30-80 bar pressure passes through the gap between the inner wall and the medium wall, and air is sent through the void between the medium wall and the outer wall at 5-12 bar pressure. The air and water are sent from the same spray nozzle, and cement injection is sent from a different spray nozzle to the ground. The spray nozzle that is employed for injection on the monitor is below the spray nozzle where air and water come out. It is possible to obtain jet grout columns up to two meters in diameter by using the Jet3 Method (Xanthakos, P., P., Abramson, L., W. and Bruce., D., A., 1994).

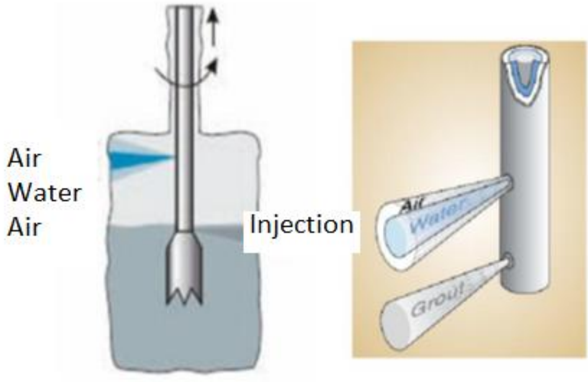


Figure 5.15 Triple fluid system(Stark, T., D., Axtell, P., J., Lewis, J., R., Dillon, J., C., Empson, W., B., Topi, J., E. and Walberg, F., C., 2009)

The triple-fluid system is usually the system with which the largest elements are created. Column-type jet grout elements are created up to 300 cm diameter in non-cohesive grounds, and up to 150 cm diameter in non-cohesive grounds with this system (Fırat, A. T. 2001).

5.2.3.4 Super jet system

To improve larger areas faster with lower cost, the super jet grout method was developed. When super jet grout columns are formed, the cement-mortar and air mixture is applied at high pressure by employing rods with large diameters (Figure 5.15). With this method, super jet grout columns may be created up to 3-5 m diameter (Figure 5.16). the rods rotate more slowly and are pulled up more slowly compared to other methods of super jet grout method (G.N. Karahan, 2016).

In super jet grout column production, the operating parameters may be as 3-4 rpm, 7 mm/minute drawing speed, and 40 MPa injection pressure up to 4 m diameter. In columns created with the Super Jet Grout Method, it was observed that column sections grew. This method is used in water control in horizontal grounds, liquefiable layer stabilization, impermeable curtain-wall construction, and in supporting excavation lateral sides (Bell, K.R., 2003).

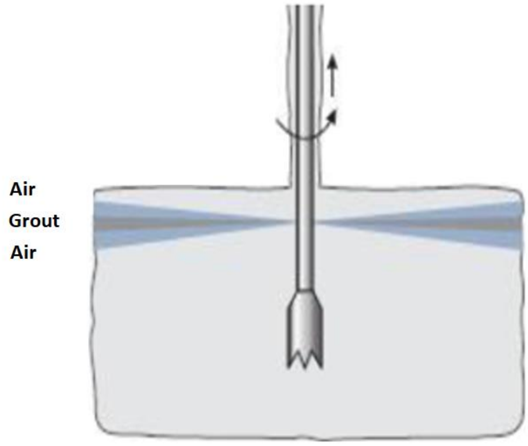


Figure 5.16 Super jet grout technique (Küsin, C.C.2009).

Figure 5.17 Columns created with the Super Jet Grout Method

5.2.3.5 Comparisons of Jet Grout Systems

Comparison of different jet grout systems in terms of the ground type, column diameter and pressure resistance is given in Table 5.1 (G.N. Karahan, 2016).

Table 5.1 Comparison of different jet grout systems (Sağlamer, A., Düzceer, R., Gökalp, A. and Yılmaz, E., 2002)

Jet Type	Soil Type	Column Diameter	JG Column Strength
Jet 1	Sand and Pebble Clay	60 cm – 90 cm (up to 110 cm) 60 cm – 90 cm	70-250 kg/cm^2 20-100 kg/cm^2
Jet 2	Sand and Pebble Clay	60 cm – 180 cm (up to 300 cm) 90 cm – 180 cm	35-140 kg/cm^2 10-70 kg/cm^2
Jet 3	Sand and Pebble Clay	150 cm – 240 cm 90 cm – 180 cm	35-105 kg/cm^2 10-50 kg/cm^2

Although the column diameter is lower in Jet 1 system than in Jet 2 and Jet 3, the strength of the jet grout column is higher. In Jet Grout Column formation, mixing of air with cement mortar in Jet 2, and mixing of water, air and cement mortar in Jet 3 causes that the strength of the jet grout column decreases, and the diameter increases (G.N. Karahan, 2016).

5.2.4 Jet grout column production parameters

The working parameters in the production of jet grout column are selected according to the type of the applied ground, the condition of the layers, the desired diameter of the column, longitudinal carrying capacity of the column, and the application method. Some of these parameters are injection pressure, water/cement rate, rod drawing speed, rod rotation speed, nozzle count and nozzle diameter (Doğanışık, S.K.).

In practice, before the production of jet grout columns, trial columns are created by selecting different design parameters. As a result of examining these columns, suitable design parameters for the ground structure are defined, and desired column diameter and pressure strength are ensured as continuity in the columns with defined production parameters. An exemplary implementation of these parameters is given in Table 5.2, which also shows the applicable ranges of the production parameters for JET 1, JET 2 and JET 3 Methods (Ergun, U. 2007).

Table 5.2 Jet Grout Production Parameters (Durgunoğlu, H., T., 2004).

Method	Injection Type	Pressure (bar)	No. of Nozzles and Diameters (mm)	Pulling Speed (cm/min)	Rotation Speed (rev/min)	Water/cement ratio	Pump Capacity (L/m)
JET 1	Cement	400-550	1-2 x 1.8-5.0	15-100	5-15	1.0 – 1.5	70-600
JET 2	Cement	400-550	1-2 x 1.8-5.0	10-30	4-8	1.0 – 1.5	70-600
	Air	10-12	-	10-30	-		4000-10000
JET 3	Cement	50-100	1-2 x 4.0-5.0	6-15	4-8	1.2 – 1.5	80-200
	Air	10-12	-	6-15	-		4000-10000
	Water	-	-	6-15	-		40-100

As high-pressure injection is sent into the ground in the Jet Grout Method, the high abrasion energy that emerged at the moment the high-pressure injection comes into contact with the ground allows the formation of the columns. In case the resulting high abrasion energy is tried in different ground types and in different production methods, the change graphics between the applied jet energy and the column diameter are given in Figure 5.18 and 5.19. In this context, it was observed that the diameters of the columns formed with the increasing jet energy also increase in logarithmic terms. However, to create larger-diameter columns with JET1 method in soft clays, more energy is needed compared to JET 2 Method. This means that JET 2 Method uses less energy in soft clays to produce larger diameter columns. However, in soft clays, JET1 Method is more economical if small diameter columns are designed.

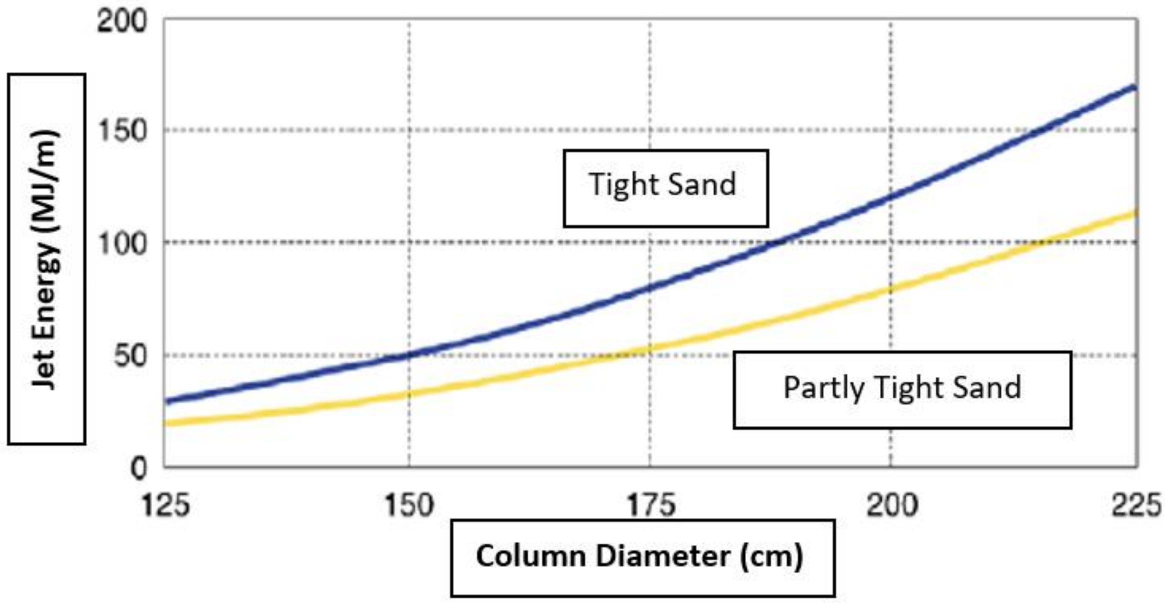


Figure 5.18 The jet energy-column diameter relation in non-cohesive grounds (Soletanche Bachy)

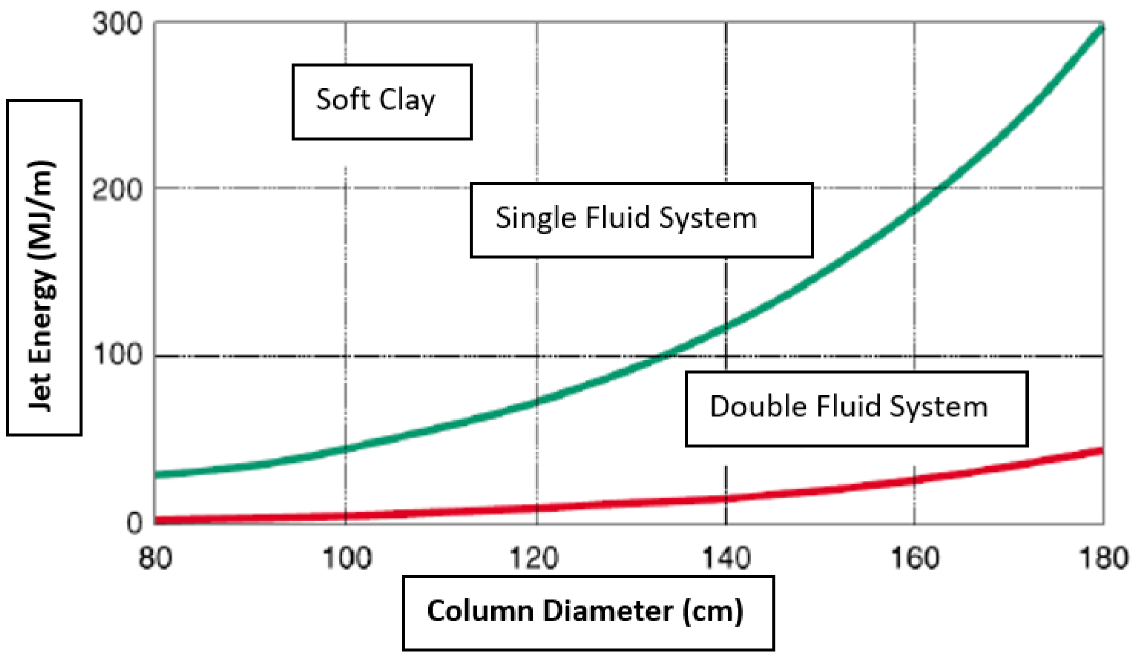


Figure 5.19 The jet energy-column diameter relation in cohesive grounds (Soletanche Bachy)

The characteristics of Jet Grout Method and the factors that affect the parameters are listed by Cippo and Tornaghi as follows (Cippo, P. A., Tornaghi, R0).

- Ground profile and hydrogeologic data

- Ground cohesion and density values and the CPT and SPT results that were applied to obtain these

- Water content of non-cohesive grounds, and grain diameter distribution; Atterberg limits in cohesive grounds
- Results of the experiment done to determine the permeability, resistance, etc. of the injection mixture and ground - injection

- Results of the field experiments and checks on the columns that are produced.

5.2.4.1 Injection pressure

The most important parameter to obtain the targeted column diameters is the injection pressure. Generally, the increase in column diameter is obtained with an increase in injection pressure. The increase in the energy that is used may not always result in an increase in the desired column diameter. The reason for this is the cohesion and construction time. As the cohesion of the ground decreases, and as the coarse grain rate increases, the column diameter also increases (Melegary, C. and Garassino, A.L., 1997) - (Kutzner, C., 1996).
The fact that between 5% and 10% of the injection pressure will be lost in hoses, drive rods and spray nozzles must be considered. The injection pressure is classified as follows (Kutzner, C., 1996).

 Low: 200-250 bar pressure
 Medium: 300-400 bar pressure
 High: 400-700 bar pressure

The relation between jet grout column diameter and injection pressure in different grounds is given in Figure 5.20.

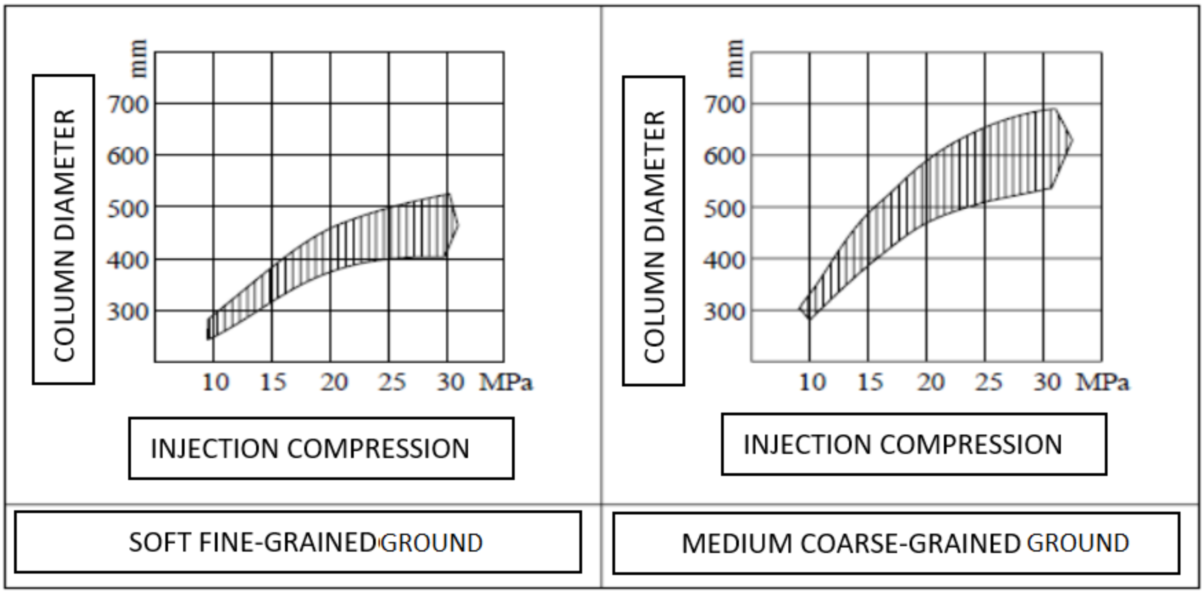


Figure 5.20 The relation between jet grout column diameter and injection pressure (Xanthakos, P., P., Abramson, L., W. and Bruce., D., A., 1994)

5.2.4.2 Injection mortar

Injection mortar is formed by obtaining water and cement mixture in the foundation at 1:1 and 1:1.5 rate. Its specific density varies between 1410-1570 kg/m^3. Research shows that as the water/cement rate increases, the compressive strength of the column decreases. These rates may vary according to the ground type in the application area and to the final pressure strengths targeted in the columns according to the selected jet grout production method. Usually, the cement varies between 300 and 750 kg/m^3 in 1m^3 improved soil. It is also possible to add some additives to the injection mixture. For example, the addition of 1-3% sodium silicate when there is high-flow ground water may be recommended to accelerate the mixture or different chemical resin types or flying ash may be added to the mixing water at an amount of the cement percentage to save the cement and reduce the costs (Ibrahim Hakkı ERKAN).

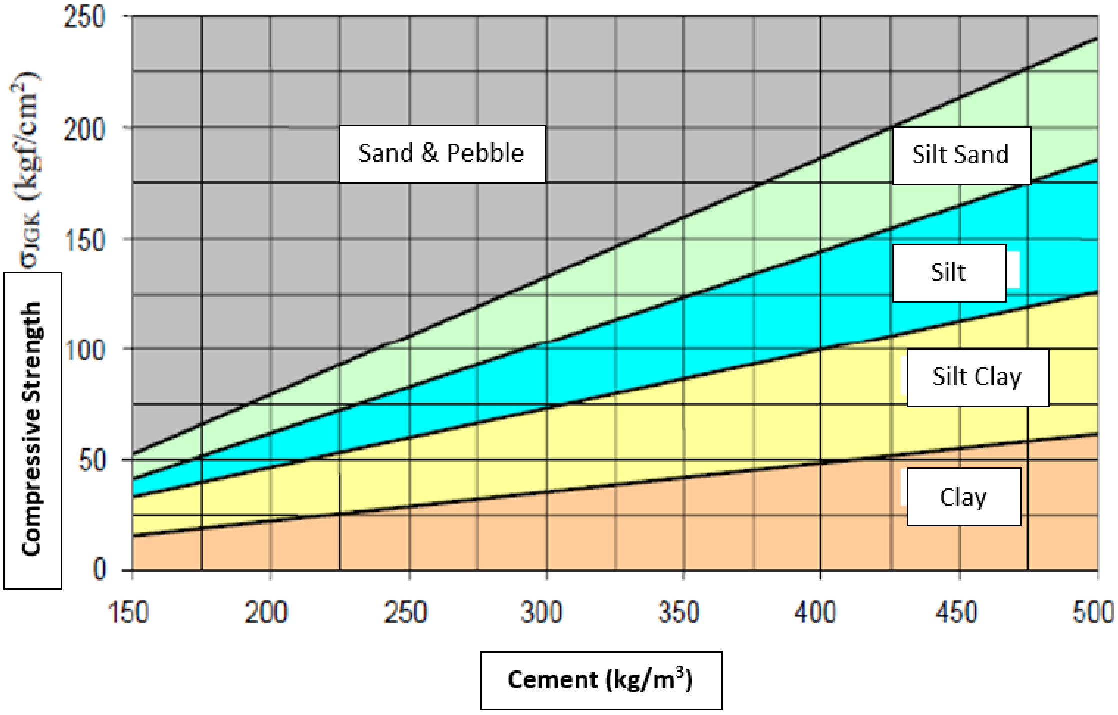


Figure 5.21 Cement amounts for compressive strength in various ground types (Öz, M.Y.,2015)

In grounds with organic content, the cement dosage may be increased from 450 kg/m^3 to 700 kg/m^3. The reason for this is that a part of the cement used is spent on the neutralization of the acids in the organic medium that removes the binding feature of the cement from the medium. The ground that is improved with the Jet Grout Method also improves the properties of the untreated ground next to it. With this second effect of the jet grout method, the SPT values of the improved ground increase at a rate of 20-25% when compared to the natural ground (and compared to the result of other improvement methods) **(**Öz, M.Y., 2015).

5.2.4.3 Spray Tip (nozzle)

One of the Jet grout column creation parameters is the nozzles. As the injection mortar is sprayed through the nozzles at a certain pressure, the nozzle diameter and number are significant. For a correct implementation, the number of the nozzles and their diameters must be considered well (G. N. Karahan, 2016).

The nozzles are on the monitor that is located at the edge of the injection rod. The number of nozzles varies according to the jet injection application used. For example, although there is one or two nozzles in the single-fluid jet application, there is usually one nozzle in the double-fluid jet application. The spread of the injection mortar increases as the nozzle diameter becomes smaller. If the number of the nozzles is low, the energy loss becomes also low (Melegary, C. and Garassino, A.L., 1997) - (Okyay, S., 1987) - (Shibazaki, M. and Ohta, S., 1982).

The diameters of the nozzles range between 1.5 and 8 mm. The schematic drawing of single-hole nozzle form is shown in Figure 3.3. The single-hole nozzle image is given in Figure 3.4.

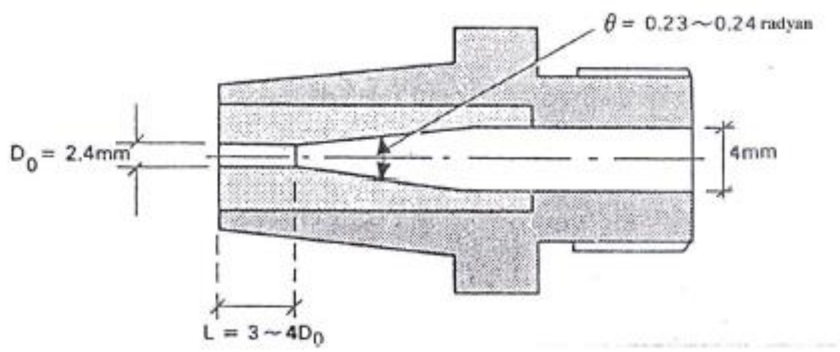


Figure 5. 22 Single-hole nozzle (Shibazaki, M. and Ohta, S.,1982)

Figure 5.23 Single-hole nozzle (Çınar, H., 2014)

The connection between the flow debit and energy and injection pressure of the injection mortar is provided by the nozzles. The spread of the injection mortar increases as the nozzle diameter decreases (Shibazaki, M. and Ohta, S., 1982). This relation is shown in Table 5.4.

Table 5.3 The relation between nozzle diameter and injection mortar

(Melegary, C. and Garassino, A.L., 1997)

WATER/CEMENT=1					INJECTION SPECIFIC GRAVITY = 1.52						
Nozzle Diameter (mm)	**1.4**	**1.6**	**1.8**	**2**	**2.4**	**2.8**	**3**	**3.2**	**3.5**	**4**	**4.5**
INJECTION OUTPUT THROUGH NOZZLES											
PRESSURE	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**	**L/m**
300	**18**	**24**	**30**	**37**	**53**	**73**	**83**	**95**	**114**	**148**	**188**
350	**20**	**26**	**32**	**40**	**58**	**79**	**90**	**103**	**123**	**160**	**203**
400	**21**	**27**	**35**	**43**	**62**	**84**	**96**	**110**	**131**	**171**	**217**
450	**22**	**29**	**37**	**45**	**65**	**89**	**102**	**116**	**139**	**182**	**230**
500	**23**	**31**	**45**	**48**	**69**	**94**	**108**	**123**	**147**	**192**	**242**
550	**25**	**32**	**48**	**50**	**72**	**98**	**113**	**129**	**154**	**201**	**254**
600	**26**	**34**	**69**	**52**	**76**	**103**	**118**	**134**	**161**	**210**	**266**

5.2.4.4 Rotation and drawing speed

To obtain the jet grout columns in required diameter and feature and to mix the ground and injection mortar in a proper manner, the rod must rotate slowly and the mixture must be drawn at a rate that may obtain a homogeneous mixture. For this reason, the rotation and drawing speed parameters of the rod become important factors (Melegary, C. and Garassino, A. L., 1997).

The time of the drawing of the sprayer body depends on the type of the ground and the quality of the injected material. In general, more time is required to have a good mixture in cohesive grounds and to have an effective separation on the ground. The amount of the drawing of the spraying body changes between 2 and 8 cm in steps. As a result of past experiences, it was decided that the most suitable drawing speed was 4 cm. In addition to these, as the drawing speed is related closely with the Jet 1, Jet 2 and Jet 3 systems determined for improvement, Jet 2 and Jet 3 systems will need more time. The reason for this is that the column is formed in larger diameters, in other words, the volume of the ground to be improved is large. In general, the rotation speed changes between 10 and 20 rpm, and is rarely rotated at 30 rpm (Okyay, S., 1987) – (Melegary, C. and Garassino, A.L.,1997).

5.2.4.5 Usage area of Jet Grout Columns

Jet Grout Columns are employed in many fields in geotechnical engineering and serve different purposes that increase day by day due to their ease of use. Some of the usage areas of Jet Grout Columns are listed below (Durgunoglu, H., T., 2004).

- Bearing strength and settlement control as pressure element for vertical loads under foundation
- Bearing strength and deformation control as pressure element especially under high and vertical spread loads and under foundation
- Carrying filling loads under approach fillings in bridges, settlement control under filling, and avoiding wall friction and transfer in edge-feet piles
- Creating weight-type retaining structures and taking lateral ground thrust in excavations
- As back prop exposed to vertical bending by equipping with reinforcement in excavations
- Between carrier elements in excavations where underground water level is high and the soil is permeable
- As reinforcement element to ensure stability in slopes or with weight coffer dam
- For the purpose of improving the ground over tunnels that are close to surfaces that open to soft surfaces
- To create a bearing umbrella on the tunnel section before excavation in the tunnel and in front of the panel in tunnels that are close to surfaces that open to soft surfaces
- To control the horizontal loads that will come to the foundations with piles in places with low liquefaction in heavy and important structures together with piles
- To increase safety number against liquefaction risk, to limit the vertical and horizontal deformations that may occur due to earthquake by carrying some of the stress
- Impermeability curtains
- To reinforce the body of dams and in impermeability

5.2.4.6 Advantages and Disadvantages of Jet Grout Method

The benefits of Jet Grout Technique may be summarized as follows (Melegary, C. and Garassino, A.L.,1997).

 Access is easy to the production team and equipment.

 It may be applied in almost all ground types.

 Large diameter columns (500 ~ 3000 mm) may be produced drilling a hole with a small diameter in the ground (100 mm).

 It helps to stabilize the ground without causing environmental pollution.

 It provides faster, easier and safer production compared to other methods.

 No vibrations are produced that damage sensitive structures, production may be carried out in areas close to such structures.

 It allows working even in small areas.

 It is economically feasible, it has an amount ca. 0.5 to 1% of the total building cost.

 Columns are produced in desired depth.

 If the excavation depth is high, and if the elevation of the foundation is unfavorable, the construction period is reduced at a rate of 30~60% according to the conditions of the area.

The disadvantages of the Jet Grout Technique may be summarized as follows (Melegary, C. and Garassino, A. L., 1997).

 Injection flow must be followed from the injection point to the surface. If there are obstructions in the injection route, this will cause the ground to swell, and as a result, there will be no columns.

 The diameters of the columns may be different depending on the type of ground.

 Very high pressures may reduce the bearing capacity of the ground, and cause unwanted settlements.

 In two-fluid and triple-fluid systems, inclinations may be formed between 10º~20º. The reason for this limitation is that it is desired that the crumpled material in the ground is removed with the sprayed air.

 Bedding might occur in two-fluid and triple-fluid methods, and the elements created in low-permeability grounds may have low strength values. In such grounds, as water cannot be removed from the improved ground, high water/cement rates may be obtained, columns with low elasticity modulus and low strength columns are produced compared to the single-fluid system.

- A completely simple design method has not been developed yet, and for this reason, experience gains importance.
- There is a high probability of mistakes since the human factor plays an important role in the production process.

CHAPTER VI
APPLICATION OF JET GROUT TO THE PROJECT AND MEASUREMENT PLANNING

6.1 Calculation and Projecting of Jet Grout Column

The purpose in Jet Grout Design is to increase the bearing capacity of the clayey sand and clayey conglomerate ground, which is weak; and to ensure that the settlements remain in the permissible limits. The sizes of the jet grout columns were taken as 16m to transfer the loads from the foundation to the deeper and relatively more sound ground. The number of the blows (N) that were obtained from SPT experiments in the field were used in the design. The average value of the SPT N pulse numbers of 20.0 meters during the Depth is approximately 5 as of the ground surface. However, the level of groundwater being high makes it even more unfavorable to cover the criteria of bearing strength and settlements. The allowable net pressure value in calculating the bearing strength based on SPT experiment done for N30ort=5 and 50.0 mm for the settlement was obtained as 3.8 t/m^2. When the load of the superstructure (including the weight of the foundation) and the structure foundation depth are taken into consideration, excavation weight to stay on the safe side was calculated and evaluated by considering the fact that the structure in question cannot be carried by the profile (to find a ground profile that is not suitable in terms of bearing strength along the depth). Qg is found by subtracting the effective weight of the ground excavated on the pile group from the total load on the piles. (Das, B.M. 1984) When this relation is considered, the general information about the structure may be summarized as follows;

Dimensions : 35.0 m x 54.0 m

Foundation Depth (D_f) :-3.96m and -5.81 m; the calculations were made according to Df=-3.96 m.

Transferred superstructure load: 10 700 t

Base Slab Foundation Weight: 3300 t

Excavation Weight: 6735 t

Total Net Load: 7265 t

Ground Safety Stress calculation is given according to the

The calculation of ground safety stress according to SPT values was given by Bowles (1996).

$$q_a = \frac{N_{55}}{0.08}\left(\frac{\Delta H_a}{25}\right) * 1 + \frac{D_f}{3B}$$

$$1 + \frac{D_f}{3B} \leq 1.33$$

$$\left[1 + \frac{3.96}{3 * 35}\right] = 1.038 \leq 1.33$$

$$q_a = \frac{4}{0.08}\left(\frac{50}{25}\right) * \left[1 + \frac{3.96}{3*35}\right] = 103.0 \text{ kPa} = 10.3 \text{ t/m}^2$$

$C_w = 0.54$

$q_a^* = 0.54 * 10.3 = 7.56 \text{ t/m}^2$

$q_u = q_a * GS$

$q_u = 7.56 \text{ t/m}^2 \; 2.0 = 11.1 \text{ t/m}^2$

$q_{net} = q_u - \gamma * D_f$

$q_{net} = 11.1 \text{ t/m}^2 - 0.9 * 3.96 = 7.56 \text{ t/m}^2$

$q_{net,a} = q_{net}/GS\ (= 2.0)$

$q_{net,a} = 3.8 \text{ t/m}^2$

This is the formula that defines the relation between the SPT-N value and the internal friction angle (ϕ) and was proposed by Shioi and Fukui (1982) according to Japanese Railway Standard.

ϕ = 0.36 N70 + 27

Shioi and Fukui (1982)

$\phi = 0.36*((45/70)*16) + 27 = 30.7 \approx 31$

$Q_u = q_u * A_{uç}$

Auc= π * 0.802 / 4 = 0.503 m2

For ϕ =31° Nc = 40.41 Nq = 25.28 Nγ = 22.65

qu = 1.3 c Nc + γ L Nq + 0.3 D γ Nγ

For a D = 80 cm pile;

q_u = 0 + (19 – 10.0) * 21 * 25.28 + 0.3 * 0.8 * (19.0-10.0) * 22.65 = 4826.84 kN/m2

$Q_d = Q_u$ = 4826.84 * 0.503 = 2430.0kN

121 pcs □80 cm jet column with $Q_{servis} = Q_d / G_s$ = 2430.0 / 3.0 = 809.0kN = 80.0 ton

4 x 4 meter grid will be produced according to the following diagram. With this application, one jet column will transfer a load of 80.0 tons.

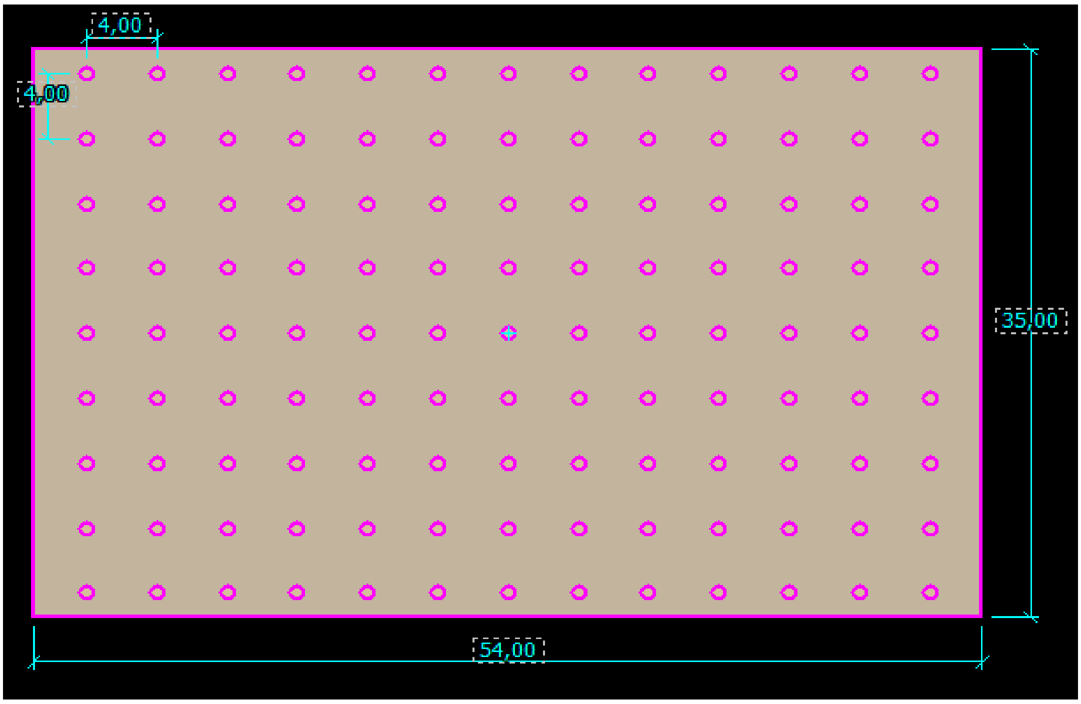


Figure 6.1 Jet Grout Column design section 1

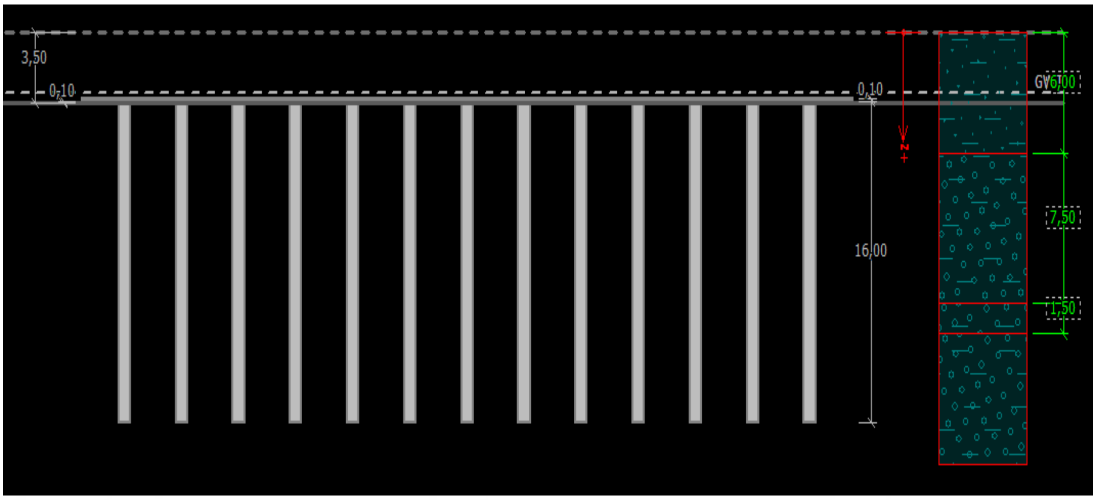


Figure 6.2 Jet Grout Column design section 2

After Jet Grout Column application, the bearing strength and settlement calculation are given below.

Settlement Analysis:

The settlement that may occur in the column group following the ground improvement may be calculated with the help of the following formula that was proposed by Meyerhof.

$$S_{grup} = \frac{2q\sqrt{B_g}I}{N_{\text{düz}}}$$

$$S_{grup} = \frac{2 * 0.52\sqrt{35} * 0.35}{5} = 0.43\ in\c{c} = 12.5 \leq 25mm$$

Bearing capacity following ground improvements:

Average cohesion for 16 m: (c)= 18 kPa

Jet grout Parameters:

Jet grout cohesion (c_{jg})= 600 kPa

Jet grout column diameter (D)= 0.80 m

Jet grout column area (A_b)= 0.50 m^2

Square settlement= 4.00m x 4.00 m

Ground area= 6.25 m^2

Area ratio, a_s= 0.08

Improved ground cohesion:

$c_u = a_s * c_{jg} * + (1-a_s) * c_{zem}$

$c_u = 0.08*600+(1-0.08)*18$

$c_u = 64.5$ kPa

Final bearing capacity:

$q_{ult} = 5.7*c_u + q*Nq$

$q_{ult} = 5.7*64.5+65*1$

Ground safety stress:

$q_{allow} = 432.6/3 = 144$ kPa

For the purpose of having clearer results, the behaviors of the piles were foreseen as Group Pile Effect, and the following calculations were proceeded in this respect. Firstly, let us examine the Group Pile Effect as the sub-theme.

6.2 Group effect in Piles

Each element of the superstructure, which frequently necessitates a foundation, is supported in three or more pile groups. Pile groups are used instead of individual piles. Because, a single pile generally does not have adequate capacity.

The piles are distributed or placed vertically with care. If a building column was supported on one single pile, its geometric axes would coincide seldomly, and the eccentricity that results in this way would create unwanted moments and deviations in the column and the pile. However, if the column is supported by three or more piles, eccentricities like these become less important. Piles are seldom used alone. In general, a pile group consisting of at least three piles is formed. In this way, it becomes possible that the eccentric loads acting on the foundation are transported more safely.

Multiple piles bring safety, and in this way, the load of the structure is carried easily even if a pile is defective. The lateral soil compaction is larger during pile mounting. For this reason, surface friction capacity is bigger than the friction capacity of the individual pile. If the piles are designed in groups, they are planned with a certain geometric sequence (square, rectangular, circular or octagonal).

It is necessary that the distances among the piles in the pile group are chosen bigger than a certain value. Otherwise, there will be interference in the stress areas around the pile, which creates excessive settlements in the pile group. In practice, the distance between the pile centers is 2.5-3-fold the pile diameters (S=(2.5-3.0)D).D: the pile diameter. The piles are connected to each other with the pile cap in connection or without connection with the ground. If the pile cap is in contact with the ground, some part of the load is transferred to the ground directly.

The load capacity of the pile group is less than the value that is obtained by multiplying the bearing capacity of one pile and the number of the piles in the group because of the intervention of the stress areas. For this reason, “Group Effect Reduction” concept is made use of to calculate the bearing capacity of the pile group.

The following rules may be applied for group effect in pile groups: Group Effect Reduction is not carried out in compaction piles that are formed on granular grounds. Group Effect Reduction is not carried out in piles that are formed into solid clay. Group Effect Reduction is not carried out in edge piles. Group Effect Reduction must be carried out in piles that are formed in soft and water-saturated clay.

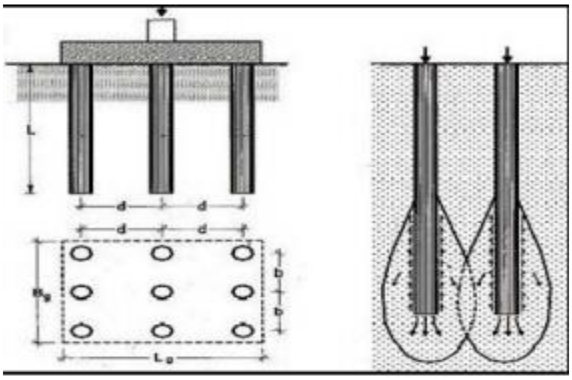


Figure 6.3 Group effect in piles (Braja M. Das., 2007)

6.3 Measurements

Measurements were taken from the same points by the map engineer from the beginning of the above-mentioned structure by considering the altitude of the area with 3-month intervals. In addition, the geographical coordinates of these points were recorded in the measurement device. The locations of these points are specific on the project by using the AUTOCAD Program, and are shown below;

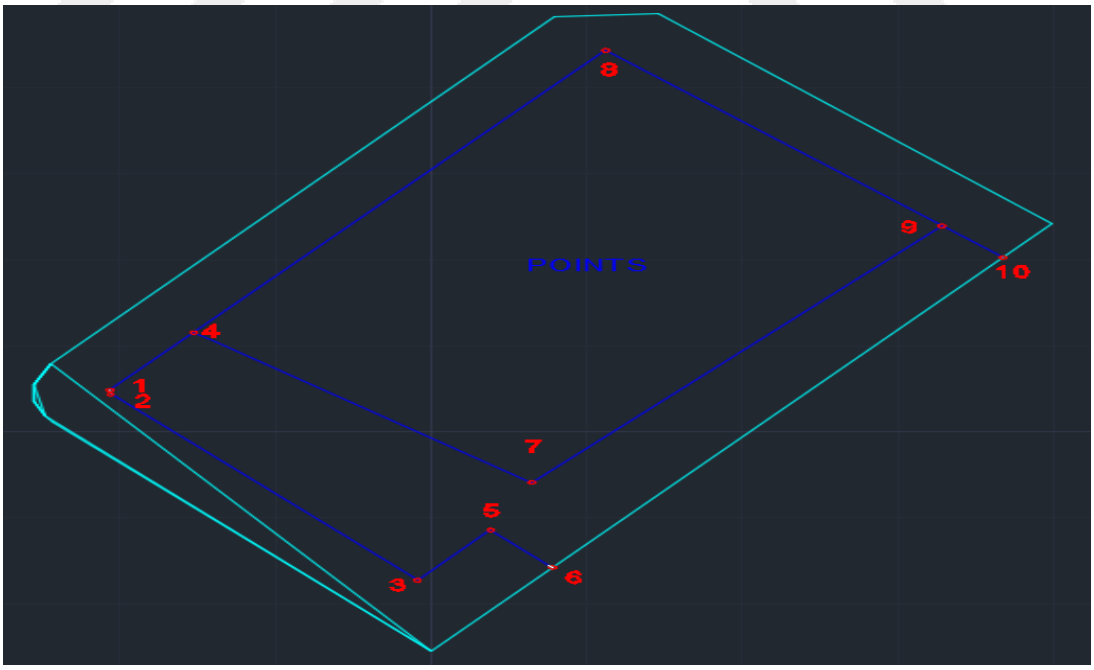


Figure 6.4 Points

The results of the measurements that were carried out with certain intervals at the determined points are stated on the project as shown below. To provide general information on points; the area which appear when we combine the points 4, 7, 8, 9 is the section of the building where it rises, in other words, these points are the corner points of the building. Points 1, 2, 3, 5, 6, 8, 10 are the building foundation corner points.

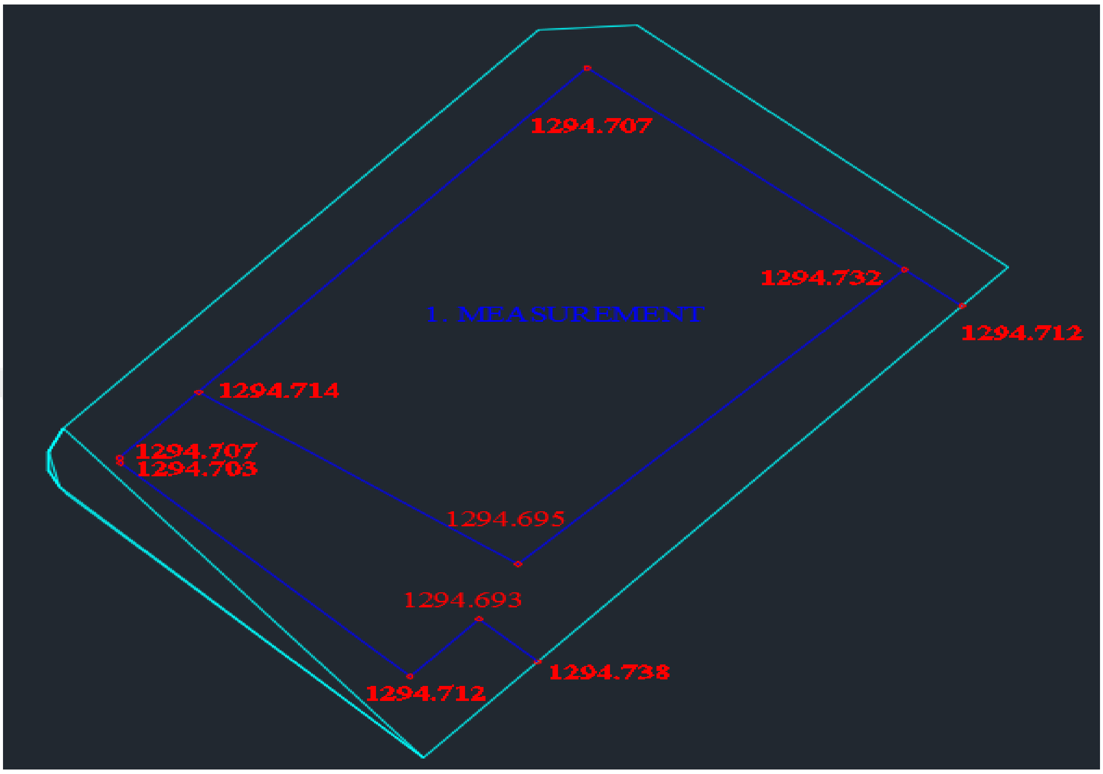


Figure 6.5 1. Measurement

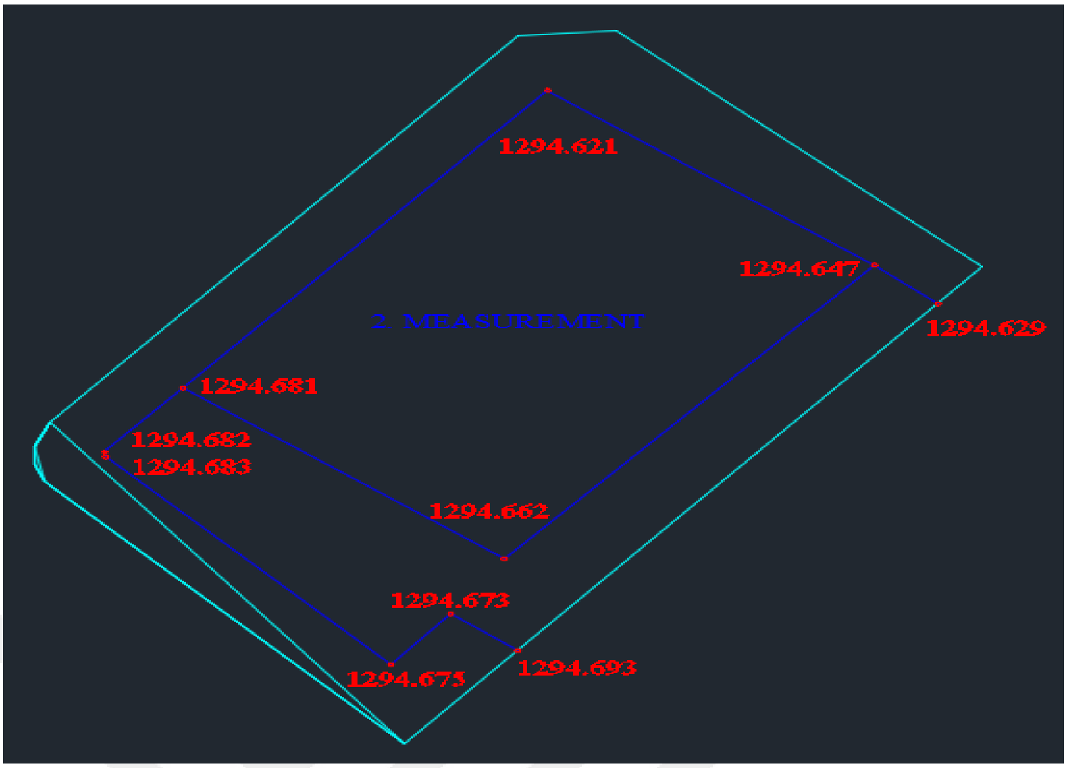


Figure 6.6 2. Measurement

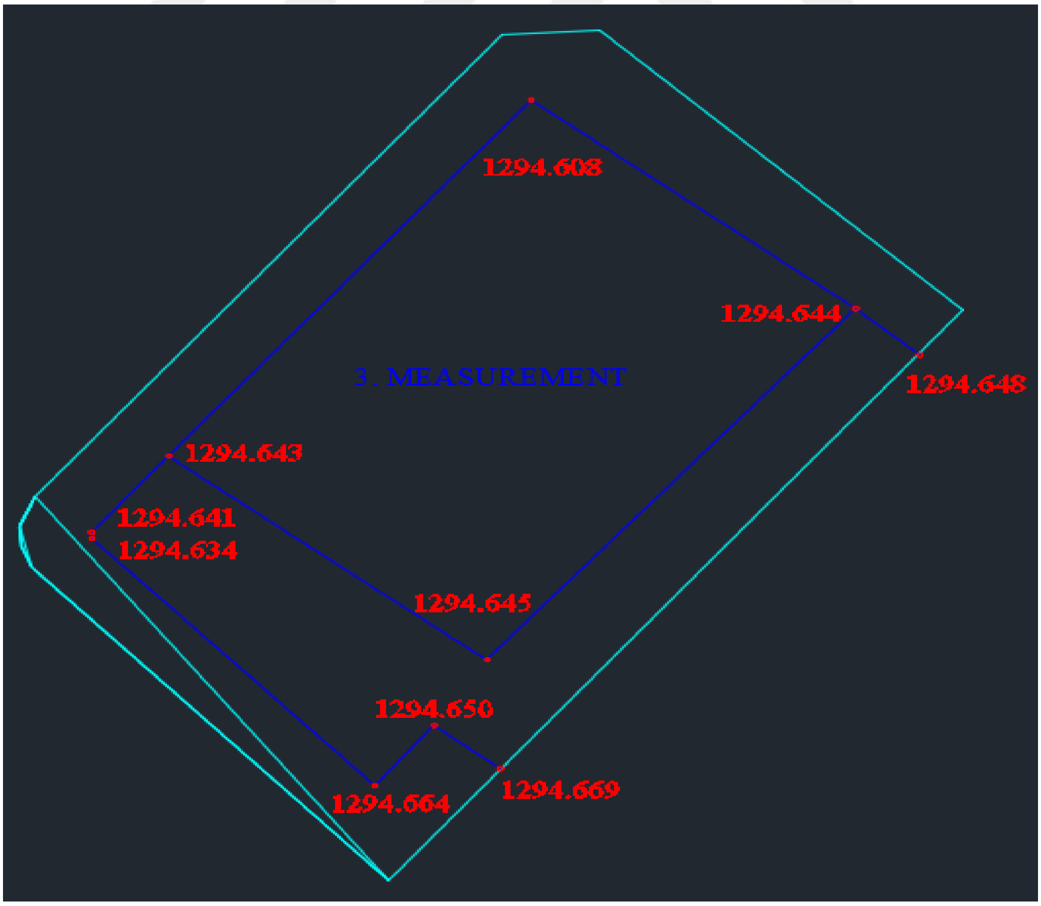


Figure 6.7 3. Measurement

According to the measurements made in the area, the data are shown as follows. Detailed information will be provided about the measurements that are used in the literature in the following section.

CHAPTER VII

RESULTS OF IN-SITU MEASUREMENTS OF SETTLEMENT AND DISCUSSIONS

We saw the measurement results from the structure that was checked at certain intervals in the previous section; now, let us examine the settlement calculations made according to different approaches to compare the data obtained from pile groups with several formulas in the literature.

7.1 Different Approaches for Pile Groups Settlement Calculations

7.1.1 Koerner and Partos (1974)

$\Delta H = \Delta H_{pile} + \Delta H_{cap}$

Pile load : 810 kN

Pile diameter: 800 mm

Pile length: 16 m

E_S : 70 MPa

s : 4 m

Use table 6.1 , since a load test indicates very little point movement for a working load of 810 Kn indicating the principal load mechanism must be skin resistance. Assume μ: 0.31 ≈ 0.3

L: 16 m s: 4 m

Take r = s/2

n = r / L 4/2*16 = 0.125 (use 0.10 to avoid interpolation)

With this n and several m = z/L values we obtain the following table for K_z at point midway between the 121 piles beneath the cap :

Table 7.1 Poisson ratio value

(−) = compression; $m = z/D$; $n = r/D$

m	n = 0.0	0.1	0.2	0.3	0.4	0.5	0.75	1.0	1.5	2.0
					Poisson ratio = 0.20					
1.0		−0.0960	−0.0936	−0.0897	−0.0846	−0.0785	−0.0614	−0.0448	−0.0208	−0.0089
1.1	−17.9689	−3.7753	−0.6188	−0.2238	−0.1332	−0.0999	−0.0659	−0.0467	−0.0222	−0.0099
1.2	−4.5510	−2.7458	−1.0005	−0.3987	−0.2056	−0.1325	−0.0724	−0.0490	−0.0236	−0.0110
1.3	−2.0609	−1.6287	−0.9233	−0.4798	−0.2672	−0.1681	−0.0811	−0.0520	−0.0249	−0.0119
1.4	−1.1858	−1.0328	−0.7330	−0.4652	−0.2926	−0.1930	−0.0905	−0.0555	−0.0263	−0.0129
1.5	−0.7782	−0.7153	−0.5682	−0.4114	−0.2875	−0.2025	−0.0985	−0.0592	−0.0277	−0.0138
1.6	−0.5548	−0.5238	−0.4457	−0.3518	−0.2664	−0.1997	−0.1038	−0.0625	−0.0290	−0.0147
1.7	−0.4188	−0.4018	−0.3569	−0.2984	−0.2399	−0.1893	−0.1061	−0.0651	−0.0303	−0.0156
1.8	−0.3294	−0.3193	−0.2918	−0.2539	−0.2133	−0.1755	−0.1057	−0.0668	−0.0315	−0.0164
1.9	−0.2673	−0.2609	−0.2431	−0.2177	−0.1890	−0.1606	−0.1033	−0.0675	−0.0325	−0.0172
2.0	−0.2222	−0.2180	−0.2060	−0.1883	−0.1676	−0.1462	−0.0995	−0.0673	−0.0334	−0.0179
					Poisson ratio = 0.30					
1.0		−0.1013	−0.0986	−0.0944	−0.0889	−0.0824	−0.0641	−0.0463	−0.0209	−0.0087
1.1	−19.3926	−3.9054	−0.5978	−0.2123	−0.1287	−0.0986	−0.0668	−0.0475	−0.0222	−0.0097
1.2	−4.9099	−2.9275	−1.0358	−0.4001	−0.2027	−0.1303	−0.0722	−0.0493	−0.0235	−0.0106
1.3	−2.2222	−1.7467	−0.9757	−0.4970	−0.2717	−0.1687	−0.0808	−0.0519	−0.0247	−0.0116
1.4	−1.2777	−1.1152	−0.7805	−0.4891	−0.3032	−0.1974	−0.0908	−0.0555	−0.0260	−0.0125
1.5	−0.8377	−0.7686	−0.6070	−0.4356	−0.3012	−0.2098	−0.0999	−0.0594	−0.0274	−0.0134
1.6	−0.598	−0.5626	−0.4768	−0.3738	−0.2809	−0.2086	−0.1063	−0.0631	−0.0288	−0.0143
1.7	−0.4500	−0.4312	−0.3819	−0.3177	−0.2538	−0.1988	−0.1094	−0.0661	−0.0302	−0.0152
1.8	−0.3536	−0.3424	−0.3122	−0.2706	−0.2262	−0.1849	−0.1096	−0.0682	−0.0315	−0.0161
1.9	−0.2866	−0.2795	−0.2600	−0.2321	−0.2006	−0.1697	−0.1076	−0.0693	−0.0326	−0.0169
2.0	−0.2380	−0.2333	−0.2201	−0.2007	−0.1780	−0.1547	−0.1039	−0.0694	−0.0336	−0.0177
					Poisson ratio = 0.40					
1.0		−0.1083	−0.1054	−0.1008	−0.0947	−0.0876	−0.0676	−0.0483	−0.0212	−0.0083
1.1	−21.2910	−4.0788	−0.5699	−0.1970	−0.1228	−0.0970	−0.0680	−0.0486	−0.0223	−0.0093
1.2	−5.3884	−3.1699	−1.0829	−0.4020	−0.1989	−0.1274	−0.0720	−0.0496	−0.0233	−0.0102
1.3	−2.4373	−1.9040	−1.0455	−0.5200	−0.2776	−0.1695	−0.0804	−0.0519	−0.0244	−0.0111
1.4	−1.4002	−1.2179	−0.8438	−0.5208	−0.3173	−0.2032	−0.0913	−0.0554	−0.0256	−0.0120
1.5	−0.9172	−0.8395	−0.6587	−0.4678	−0.3194	−0.2196	−0.1017	−0.0596	−0.0270	−0.0129
1.6	−0.6527	−0.6143	−0.5181	−0.4033	−0.3001	−0.2205	−0.1095	−0.0638	−0.0284	−0.0138
1.7	−0.4915	−0.4705	−0.4152	−0.3435	−0.2724	−0.2116	−0.1138	−0.0675	−0.0300	−0.0147
1.8	−0.3858	−0.3732	−0.3393	−0.2929	−0.2433	−0.1976	−0.1148	−0.0701	−0.0314	−0.0156
1.9	−0.3123	−0.3044	−0.2825	−0.2512	−0.2161	−0.1818	−0.1133	−0.0717	−0.0328	−0.0166
2.0	−0.2590	−0.2537	−0.2390	−0.2173	−0.1919	−0.1659	−0.1098	−0.0722	−0.0340	−0.0174

Table 7.2 Poisson ratio value

(−) = compression; $m = z/D$; $n = r/D$

m	n = 0.00	0.02	0.04	0.06	0.08	0.10	0.15	0.20	0.50	1.0	2.0
					Poisson ratio = 0.20						
1.0		−6.4703	−3.2374	−2.1592	−1.6202	−1.2962	−0.8630	−0.6445	−0.2300	−0.0690	−0.0081
1.1	−1.7781	−1.7342	−1.5944	−1.4178	−1.2418	−1.0850	−0.7953	−0.6138	−0.2283	−0.0730	−0.0096
1.2	−0.9015	−0.8789	−0.8576	−0.8269	−0.7882	−0.7446	−0.6317	−0.5307	−0.2231	−0.0759	−0.0111
1.3	−0.5968	−0.5799	−0.5725	−0.5629	−0.5500	−0.5340	−0.4867	−0.4355	−0.2138	−0.0779	−0.0125
1.4	−0.4569	−0.4288	−0.4241	−0.4201	−0.4142	−0.4068	−0.3838	−0.3562	−0.2010	−0.0789	−0.0139
1.5	−0.3482	−0.3359	−0.3334	−0.3313	−0.3282	−0.3242	−0.3113	−0.2952	−0.1862	−0.0790	−0.0152
1.6	−0.2922	−0.2726	−0.2716	−0.2707	−0.2689	−0.2666	−0.2589	−0.2487	−0.1708	−0.0784	−0.0165
1.7	−0.2518	−0.2304	−0.2287	−0.2274	−0.2261	−0.2247	−0.2195	−0.2127	−0.1559	−0.0770	−0.0175
1.8	−0.1772	−0.1953	−0.1949	−0.1942	−0.1936	−0.1925	−0.1891	−0.1844	−0.1420	−0.0750	−0.0185
1.9	−0.1648	−0.1702	−0.1698	−0.1687	−0.1682	−0.1675	−0.1650	−0.1616	−0.1295	−0.0727	−0.0193
2.0	−0.1461	−0.1482	−0.1486	−0.1480	−0.1478	−0.1473	−0.1455	−0.1429	−0.1180	−0.0700	−0.0201
					Poisson ratio = 0.30						
1.0		−6.8419	−3.4044	−2.2673	−1.6983	−1.3567	−0.8998	−0.6695	−0.2346	−0.0686	−0.0076
1.1	−1.9219	−1.8611	−1.7072	−1.5134	−1.3211	−1.1503	−0.8368	−0.6419	−0.2335	−0.0728	−0.0091
1.2	−0.9699	−0.9403	−0.9166	−0.8825	−0.8400	−0.7922	−0.6688	−0.5588	−0.2292	−0.0760	−0.0105
1.3	−0.6430	−0.6188	−0.6099	−0.5992	−0.5850	−0.5675	−0.5157	−0.4597	−0.2207	−0.0782	−0.0120
1.4	−0.4867	−0.4558	−0.4507	−0.4461	−0.4396	−0.4316	−0.4063	−0.3761	−0.2082	−0.0796	−0.0134
1.5	−0.3766	−0.3561	−0.3533	−0.3510	−0.3476	−0.3432	−0.3291	−0.3115	−0.1834	−0.0800	−0.0148
1.6	−0.3339	−0.2895	−0.2878	−0.2863	−0.2843	−0.2817	−0.2732	−0.2621	−0.1777	−0.0796	−0.0160
1.7	−0.2664	−0.2438	−0.2414	−0.2399	−0.2384	−0.2369	−0.2313	−0.2239	−0.1623	−0.0784	−0.0172
1.8	−0.2025	−0.2065	−0.2054	−0.2044	−0.2038	−0.2026	−0.1989	−0.1938	−0.1479	−0.0766	−0.0182
1.9	−0.1847	−0.1794	−0.1785	−0.1777	−0.1768	−0.1760	−0.1733	−0.1696	−0.1347	−0.0744	−0.0191
2.0	−0.1634	−0.1565	−0.1561	−0.1556	−0.1551	−0.1545	−0.1525	−0.1498	−0.1229	−0.0718	−0.0199
					Poisson ratio = 0.40						
1.0		−7.2744	−3.6270	−2.4110	−1.8026	−1.4373	−0.9488	−0.7029	−0.2407	−0.0681	−0.0069
1.1	−2.0931	−2.0296	−1.8574	−1.6409	−1.4266	−1.2372	−0.8921	−0.6794	−0.2404	−0.0725	−0.0083
1.2	−1.0486	−1.0209	−0.9947	−0.9567	−0.9091	−0.8556	−0.7181	−0.5964	−0.2373	−0.0760	−0.0098
1.3	−0.6922	−0.6694	−0.6598	−0.6476	−0.6318	−0.6122	−0.5543	−0.4921	−0.2298	−0.0787	−0.0113
1.4	−0.5347	−0.4922	−0.4860	−0.4807	−0.4735	−0.4645	−0.4362	−0.4026	−0.2178	−0.0805	−0.0128
1.5	−0.4020	−0.3823	−0.3798	−0.3771	−0.3734	−0.3684	−0.3527	−0.3332	−0.2029	−0.0813	−0.0142
1.6	−0.3440	−0.3096	−0.3083	−0.3068	−0.3045	−0.3017	−0.2922	−0.2800	−0.1868	−0.0812	−0.0155
1.7	−0.2943	−0.2606	−0.2580	−0.2564	−0.2549	−0.2531	−0.2469	−0.2387	−0.1708	−0.0803	−0.0167
1.8	−0.2114	−0.2207	−0.2189	−0.2181	−0.2174	−0.2161	−0.2119	−0.2063	−0.1558	−0.0787	−0.0178
1.9	−0.1782	−0.1907	−0.1904	−0.1890	−0.1881	−0.1873	−0.1843	−0.1802	−0.1419	−0.0766	−0.0188
2.0	−0.1741	−0.1660	−0.1658	−0.1652	−0.1648	−0.1642	−0.1620	−0.1590	−0.1294	−0.0741	−0.0196

Table 7.3 Poisson ratio

(−) = compression; $m = z/D$; $n = r/D$

m	n = 0.00	0.02	0.04	0.06	0.08	0.10	0.15	0.20	0.50	1.0	2.0
					Poisson ratio = 0.20						
1.0		−11.5315	−5.3127	−3.3023	−2.3263	−1.7582	1.0372	−0.7033	−0.1963	−0.0618	−0.0082
1.1	−2.8427	−2.7518	−2.4908	−2.1596	−1.8329	−1.5469	1.0359	−0.7346	−0.2074	−0.0656	−0.0096
1.2	−1.2853	−1.2541	−1.2158	−1.1620	−1.0930	−1.0162	0.8211	−0.6529	−0.2141	−0.0689	−0.0110
1.3	−0.7673	−0.7753	−0.7585	−0.7420	−0.7195	−0.6928	0.6142	−0.5312	−0.2139	−0.0717	−0.0123
1.4	−0.5937	−0.5450	−0.5343	−0.5267	−0.5181	−0.5063	0.4693	−0.4261	−0.2068	−0.0737	−0.0136
1.5	−0.4485	−0.4051	−0.4059	−0.4006	−0.3960	−0.3901	0.3704	−0.3460	−0.1947	−0.0750	−0.0148
1.6	−0.3635	−0.3201	−0.2326	−0.3183	−0.3154	−0.3123	0.3008	−0.2861	−0.1803	−0.0754	−0.0160
1.7	−0.3204	−0.2583	−0.2635	−0.2618	−0.2595	−0.2574	0.2503	−0.2408	−0.1652	−0.0750	−0.0170
1.8	−0.2533	−0.2222	−0.2239	−0.2206	−0.2181	−0.2166	0.2122	−0.2059	−0.1506	−0.0739	−0.0180
1.9	−0.2382	−0.1761	−0.1855	−0.1880	−0.1878	−0.1853	0.1827	−0.1782	−0.1371	−0.0722	−0.0188
2.0	−0.1767	−0.1643	−0.1648	−0.1630	−0.1631	−0.1614	0.1591	−0.1561	−0.1248	−0.0700	−0.0196
					Poisson ratio = 0.30						
1.0		−12.1310	−5.5765	−3.4591	−2.4320	−1.8346	1.0774	−0.7276	−0.1997	−0.0616	−0.0777
1.1	−3.0612	−2.9620	−2.6751	−2.3119	−1.9547	−1.6433	1.0908	−0.7680	−0.2115	−0.0654	−0.0090
1.2	−1.3821	−1.3465	−1.3052	−1.2465	−1.1706	−1.0864	0.8730	−0.6899	−0.2198	−0.0689	−0.0104
1.3	−0.8262	−0.8035	−0.8130	−0.7949	−0.7705	−0.7411	0.6548	−0.5639	−0.2212	−0.0720	−0.0117
1.4	−0.6194	−0.5827	−0.5722	−0.5630	−0.5540	−0.5410	0.5005	−0.4530	−0.2150	−0.0744	−0.0130
1.5	−0.5189	−0.4337	−0.4332	−0.4281	−0.4227	−0.4163	0.3946	−0.3679	−0.2033	−0.0760	−0.0143
1.6	−0.3841	−0.3415	−0.3449	−0.3395	−0.3361	−0.3327	0.3202	−0.3039	−0.1887	−0.0768	−0.0155
1.7	−0.3332	−0.2764	−0.2810	−0.2782	−0.2764	−0.2739	0.2660	−0.2556	−0.1732	−0.0767	−0.0166
1.8	−0.2837	−0.2268	−0.2381	−0.2347	−0.2319	−0.2300	0.2253	−0.2183	−0.1580	−0.0758	−0.0176
1.9	−0.2654	−0.1873	−0.1963	−0.1991	−0.1988	−0.1965	0.1937	−0.1887	−0.1439	−0.0742	−0.0186
2.0	−0.1872	−0.1730	−0.1744	−0.1732	−0.1725	−0.1714	0.1684	−0.1651	−0.1310	−0.0721	−0.0194
					Poisson ratio = 0.40						
1.0		−12.9304	−5.9282	−3.6683	−2.5729	−1.9365	1.1311	−0.7600	−0.2042	−0.0614	−0.0069
1.1	−3.3525	−3.2423	−2.9209	−2.5144	−2.1171	−1.7719	1.1641	−0.8125	−0.2170	−0.0652	−0.0083
1.2	−1.5030	−1.4712	−1.4255	−1.3588	−1.2742	−1.1800	0.9422	−0.7394	−0.2274	−0.0689	−0.0096
1.3	−0.8965	−0.9066	−0.8862	−0.8649	−0.8383	−0.8056	0.7089	−0.6076	−0.2308	−0.0723	−0.0109
1.4	−0.6753	−0.6350	−0.6222	−0.6120	−0.6018	−0.5874	0.5419	−0.4890	−0.2260	−0.0752	−0.0123
1.5	−0.5629	−0.4718	−0.4712	−0.4641	−0.4584	−0.4511	0.4270	−0.3971	−0.2147	−0.0773	−0.0136
1.6	−0.4198	−0.3701	−0.3730	−0.3672	−0.3642	−0.3600	0.3461	−0.3278	−0.1999	−0.0786	−0.0149
1.7	−0.3752	−0.2840	−0.3039	−0.3011	−0.2984	−0.2956	0.2870	−0.2754	−0.1838	−0.0788	−0.0161
1.8	−0.3158	−0.2496	−0.2575	−0.2530	−0.2497	−0.2479	0.2427	−0.2349	−0.1680	−0.0782	−0.0172
1.9	−0.2851	−0.2022	−0.2122	−0.2155	−0.2142	−0.2113	0.2083	−0.2028	−0.1530	−0.0769	−0.0182
2.0	−0.2012	−0.1929	−0.1878	−0.1854	−0.1850	−0.1837	0.1807	−0.1771	−0.1393	−0.0749	−0.0191

The average influence value in the zone L to 2L using the trapezoidal rule is

K_z = $\left[\left(\frac{1.87+0.17}{2}\right) + 1.64 + 1.08 + 0.74 + 0.54 + 0.41 + 0.33 + 0.27 + 0.23 + 0.19\right]$ = 6.4

$K_{z,av}$ = 6.4 / 10 = 0.64

Compute the average stress in depth *Lp* below pile and the corresponding settlement. Assume stress only from the 121 piles:

$$\sigma = \frac{121 * \mathrm{P} * \mathrm{Kz, av}}{L * L} = \frac{121 * 810 * 0.64}{16 * 16}$$

σ = 245 kPa

$$\Delta \mathrm{Hpile} = \frac{\sigma * L}{E} = \frac{245 * 16}{70}$$

ΔH_{pile}=56 mm

Calculations will be made by using 2 different methods for ΔH_{cap};

Method 1.

Total settlement is settlement of cap plus point movement just computed. Use Eq.(5-16a) for cap settlement.

B= 35.0 m   L=54.0 m

E= 70 MP

Table 7.4 E_s value

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$	$E_s = (2 \text{ to } 4)q_u$
	$= 7000\sqrt{N}$	$= 8000\sqrt{q_c}$
	$= 6000N$	— — —
	— — —	$E_s = 1.2(3D_r^2 + 2)q_c$
	‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	*$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$
		$e = 1.0 \quad F = 3.5$
		$e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$	$E_s = (6 \text{ to } 30)q_c$
	$E_{s(OCR)} \approx E_{s,nc}\sqrt{OCR}$	
Gravelly sand	$E_s = 1200(N + 6)$	
	$= 600(N + 6) \quad N \le 15$	
	$= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use §$E'_s = 2.5q_c$	
	$2500 < q_c < 5000$ use $E'_s = 4q_c + 5000$	
	where	
	$E'_s = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

Using Table 6.4 and *N=5* values given in the reference and weighting, one can obtain *E* = 6400 kPa. We will therefore use an average since the value of 300000 was arbitrarily doubled and may be somewhat too large.

E = (70000+6400)/2 = 38200 kPa

For *L/B* = 54/35 = 1.54 (use 2) and for *H* = 20 m from the boring log, we obtain *H/B'* =20(121)/35 = 70 (use 100 to avoid massive interpolation).

Table 7.5 Interpolation

N	$M = 1.0$	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.2	I_1 = 0.009	0.008	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.007
	I_2 = 0.041	0.042	0.042	0.042	0.042	0.042	0.043	0.043	0.043	0.043	0.043
0.4	0.033	0.032	0.031	0.030	0.029	0.028	0.028	0.027	0.027	0.027	0.027
	0.066	0.068	0.069	0.070	0.070	0.071	0.071	0.072	0.072	0.073	0.073
0.6	0.066	0.064	0.063	0.061	0.060	0.059	0.058	0.057	0.056	0.056	0.055
	0.079	0.081	0.083	0.085	0.087	0.088	0.089	0.090	0.091	0.091	0.092
0.8	0.104	0.102	0.100	0.098	0.096	0.095	0.093	0.092	0.091	0.090	0.089
	0.083	0.087	0.090	0.093	0.095	0.097	0.098	0.100	0.101	0.102	0.103
1.0	0.142	0.140	0.138	0.136	0.134	0.132	0.130	0.129	0.127	0.126	0.125
	0.083	0.088	0.091	0.095	0.098	0.100	0.102	0.104	0.106	0.108	0.109
1.5	0.224	0.224	0.224	0.223	0.222	0.220	0.219	0.217	0.216	0.214	0.213
	0.075	0.080	0.084	0.089	0.093	0.096	0.099	0.102	0.105	0.108	0.110
2.0	0.285	0.288	0.290	0.292	0.292	0.292	0.292	0.292	0.291	0.290	0.289
	0.064	0.069	0.074	0.078	0.083	0.086	0.090	0.094	0.097	0.100	0.102
3.0	0.363	0.372	0.379	0.384	0.389	0.393	0.396	0.398	0.400	0.401	0.402
	0.048	0.052	0.056	0.060	0.064	0.068	0.071	0.075	0.078	0.081	0.084
4.0	0.408	0.421	0.431	0.440	0.448	0.455	0.460	0.465	0.469	0.473	0.476
	0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.060	0.063	0.066	0.069
5.0	0.437	0.452	0.465	0.477	0.487	0.496	0.503	0.510	0.516	0.522	0.526
	0.031	0.034	0.036	0.039	0.042	0.045	0.048	0.050	0.053	0.055	0.058
6.0	0.457	0.474	0.489	0.502	0.514	0.524	0.534	0.542	0.550	0.557	0.563
	0.026	0.028	0.031	0.033	0.036	0.038	0.040	0.043	0.045	0.047	0.050
7.0	0.471	0.490	0.506	0.520	0.533	0.545	0.556	0.566	0.575	0.583	0.590
	0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.037	0.039	0.041	0.043
8.0	0.482	0.502	0.519	0.534	0.549	0.561	0.573	0.584	0.594	0.602	0.611
	0.020	0.022	0.023	0.025	0.027	0.029	0.031	0.033	0.035	0.036	0.038
9.0	0.491	0.511	0.529	0.545	0.560	0.574	0.587	0.598	0.609	0.618	0.627
	0.017	0.019	0.021	0.023	0.024	0.026	0.028	0.029	0.031	0.033	0.034
10.0	0.498	0.519	0.537	0.554	0.570	0.584	0.597	0.610	0.621	0.631	0.641
	0.016	0.017	0.019	0.020	0.022	0.023	0.025	0.027	0.028	0.030	0.031
20.0	0.529	0.553	0.575	0.595	0.614	0.631	0.647	0.662	0.677	0.690	0.702
	0.008	0.009	0.010	0.010	0.011	0.012	0.013	0.013	0.014	0.015	0.016
500.0	**0.560**	0.587	0.612	0.635	0.656	0.677	0.696	0.714	0.731	0.748	0.763
	0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001

$$I_s = I_1 + \frac{1 - 2(\mu)}{1 - \mu} * I_2$$

$$I_s = 0.720 + \frac{1 - 2(0.31)}{1 - 0.31} * 0.011 \qquad I_f = 1.0$$

I_s= 0.726

$$\Delta H = \Delta qB\frac{1-(\mu*\mu)}{E}*m*\mathrm{I}_s*\mathrm{I}_f$$

$$\Delta q = \frac{121P}{B*L}$$

$$\Delta q = \frac{121*810}{35*54}$$

$$\Delta q = 52kPa$$

Using m= 4 contributing corners and 1000 to obtain mm

$$\Delta \mathrm{Hcap} \ = 52(\frac{35}{2})(\frac{1-(0.31*0{,}31)}{38200}*4*0.726*1.0*1000$$

ΔH_{cap} =62.5 mm

$\Delta H = \Delta H_{pile} + \Delta H_{cap}$

ΔH = 56 mm + 62.5 mm = 118.5 mm

Method 2.

Settlement is computed as elastic shortening of pile + $\Delta H_{pile\ point}$. For a linear variation of P at top to P = P_{top} - ΔP where ΔP = 0.5 P, we have

$$\begin{aligned} e &= \int_0^{L_p} \epsilon\, dy = \frac{1}{AE}\int_0^{L_p}\left(P_o - \Delta P\frac{y}{L}\right)dy \\ &= \frac{1}{AE}\left(P_oL_p - \frac{\Delta PL_p}{2}\right) \\ &= \frac{L_p}{AE}\left(P_o - \frac{0.5}{2}P_o\right) = \frac{0.75P_oL_p}{AE} \end{aligned}$$

Taking Ec = 70 MPa for f_c = 35 MPa and A = 0.5024 m^2 for 0.8-m diameter pile, we have

$$e = \frac{0.75*P*L}{A*E}$$

$$e = \frac{0.75*810*16}{0.5024*70000}$$

e = 0.28 mm

$\Delta H_g = \Delta H_{pile} + e$ = 56 mm + 0.28 mm = 56.28 mm

To sum up;

ΔH_{pile} = 56 mm

ΔH_{cap} ; method 1 = 62.5 mm

Method 2 = 0.28 mm

Total expected settlement amount was calculated as;

56.28 mm and 118.5 mm.

7.1.2 H.G Poulos a E.H. Davis - Pile Foundations Analysis and Design

The Jet Grout Project design was analyzed below by using the GEO5 Program, and many other parameters including group pile settlements were also calculated.

Verification of pile group

Input data

Project

Task	: City of Sivas, Central County, Kardeşler Neighborhood, 5495 Island, I38D01C Plot, 30 Parcel
Descript.	: Göksu Suit Project
Author	: Batuhan Düz
Date	: 17.01.2019
Unit weight of water is considered as	: 9,81 kN/m^3

Settings

USA - Safety factor

Materials and standards

Concrete structures: ACI 318-11

Settlement

Analysis method :	Analysis using Oedometric Modulus
Restriction of influence zone :	by percentage of Sigma, Or
Coeff. of restriction of influence zone :	10,0 [%]

Pile group

Verification methodology :	Safety factors (ASD)

Safety factors			
Permanent design situation			
Safety factor :	SF_{cp} =	2,00	[–]

Soil parameters

Clayey sand (SC)

Unit weight :	γ	=	18,50	kN/m^3
Cohesion of soil :	c_u	=	28,00	kPa
Oedometric modulus :	E_{oed}	=	12,50	MPa
Saturated unit weight :	γ_{sat}	=	20,00	kN/m^3

Clayey gravel (GC)

Unit weight :	γ	=	19,50	kN/m^3
Cohesion of soil :	c_u	=	28,00	kPa
Oedometric modulus :	E_{oed}	=	67,50	MPa
Saturated unit weight :	γ_{sat}	=	21,00	kN/m^3

Construction

Width of pile cap	b_x	=	54,00	M
	b_y	=	35,00	M
Pile diameter	d	=	0,80	M
Number of piles	n_x	=	13	
	n_y	=	9	

Spacing of piles s_x = 4,00 M

s_y = 4,00 M

Geometry

Depth from ground surface h_z = 3,50 m

Pile head offset h = 0,10 m

Thickness of pile cap t = 0,10 m

Length of piles l = 16,00 m

Material of structure

Unit weight □ = 23,56 kN/m^3

Analysis of concrete structures carried out according to the standard ACI 318-11.

Concrete : Concrete ACI

Compressive strength f_{c}' = 5,00 MPa

Tensile-bending strength f_r = 1,39 MPa

Elasticity modulus E_{cm} = 10583,26 MPa

Shear modulus G = 4444,97 MPa

Longitudinal steel : A615/40

Tensile strength f_y = 1,39 MPa

Horizontal modulus of subsoil reaction

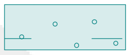
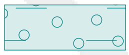
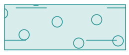
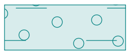
Depth [m]	k_h[MN/m3]
0.00	0.00
9.14	10.85
16.00	10.85

Determination of vertical springs

Shear modulus of subsoil reaction

Depth[m]	K_V [MN/m3]
0.00	0.00
9.14	10.85
16.00	10.85

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	6,00	Clayey sand (SC)	
2	7,50	Clayey gravel (GC)	
3	1,50	Clayey gravel (GC)	
4	-	Clayey gravel (GC)	

Load

No.	Load New	change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]	M_z [kNm]
1	YES		Load No. 1	Service	140000,00	0,00	0,00	0,00	0,00	0,00

Ground water table

The ground water table is at a depth of 3,00 m from the original terrain.

Global settings

Analysis type : analytical solution

Type of soil : cohesive soil

Settings of the stage of construction

Design situation : permanent

Verification No. 1

Analysis of bearing capacity - input data

Analysis carried out with an automatic selection of the most unfavorable load cases.

Analysis of bearing capacity of pile group in cohesive soils

Max. vertical force includes self-weight of pile cap.

Average undrained shear strength along the piles	c_{us}	=28,00	kPa
Undrained shear strength at base of pile group	c_{ub}	=28,00	kPa
Cohesion group bearing capacity factor	N_{cg}	=5,98	
Vertical bearing capacity of pile group	R_g	=413302,19	kN
Maximum vertical force	V_d	=146871,00	kN

Safety factor = 2,81 > 2,00

Vertical bearing capacity of pile group is SATISFACTORY

Verification No. 1

Analysis of settlement of pile group in cohesive soils

Max. vertical force includes self-weight of pile cap.

The depth of substitute found.	d	=10,60	m
Maximum service load	N	=146871	kN
Depth of influence zone	h	=14,61	m

Settlement of pile group s =8,2 mm

When all data are considered, different settlement results are obtained when calculations are made according to different formulas; to summarize these;

Table 1 Results of the formulas

Literature	Settlement (mm)
Koerner and Partos (1974)	0.056 0.118
H.G Poulos a E.H. Davis	0.008

CHAPTER VIII
CONCLUSION AND DISCUSSION

If the amount of settlement after the jet grouting method in the previous section are also tabulated below;

Table 8.1 Measurements summary

	1.Measurement	2.Measurement	3.Measurement	settlement(3-1) (m)
1.Point	1294,707	1294,682	1294,641	0,066
2.Point	1294,703	1294,683	1294,634	0,069
3.Point	1294,712	1294,675	1294,664	0,048
4.Point	1294,714	1294,681	1294,643	0,071
5.Point	1294,693	1294,673	1294,650	0,043
6.Point	1294,738	1294,693	1294,669	0,069
7.Point	1294,695	1294,662	1294,645	0,050
8.Point	1294,707	1294,707	1294,608	0,099
9.Point	1294,732	1294,732	1294,644	0,088
10.Point	1294,712	1294,712	1294,648	0,064

The most settlement points are Structure corner points as expected (4, 7, 8, 9). The results of the formula used in the literature are also tabulated ;

Table 8.2 Formulas results

Literature	Settlement (m)
Koerner and Partos (1974)	0,056 0,118
H.G Poulos a E.H. Davis	0,008

Settlement amounts are given below as time-settlement graphics;

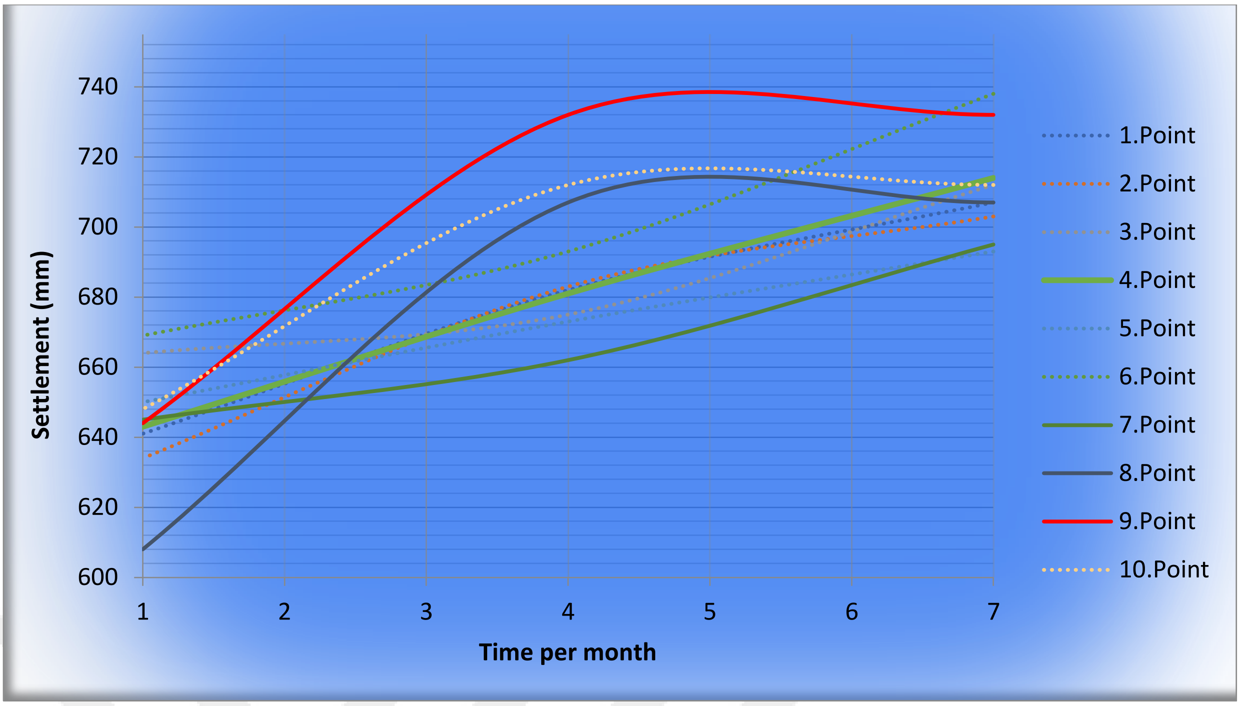


Figure 8.1: Settlement- Time Graph according points

The thick points are the corner points of the building where it rises.

To sum up, settlement quantities from measurements respectively 71 mm, 50 mm, 99 mm and 88 mm. When the formulas are taken into consideration, these values vary; according to principle of $\Delta H = \Delta H_{pile} + \Delta H_{cap}$ Koerner and Partos (1974) ΔH_{pile} amount was calculated 56 mm and also ΔH_{cap} amount 0,32 mm was calculated, besides ΔH amount is recorded as 56 mm because ΔH_{cap} amount is negligible. In the other method, the value of ΔH_{cap} was determined 62 mm. In short, the amount of ΔH was calculated as 56 mm in the first approach and 118 mm in the other approach. If we come to the H.G Poulos a E.H. Davis method, a difference of 8 mm in the GEO5 program when calculating a very low settlement on the spot when the difference in the formulas used for a long time has been supported that there may be serious deviation errors.

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