ESTIMATION OF CONSOLIDATION SETTLEMENTS CAUSED BY GROUNDWATER DRAINAGE AT ULUS-KEÇİÖREN SUBWAY PROJECT

A THESIS SUBMITTED TO THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES OF MIDDLE EAST TECHNICAL UNIVERSITY

ΒY

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN GEOLOGICAL ENGINEERING

APRIL 2006

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ABSTRACT

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April 2006, 189 pages

Prediction of ground settlements have always been a big challenge for the engineers that are responsible for the design of subway tunnel projects. Since ground settlement is a crucial concept directly affecting the successfulness of a project, it must be taken seriously and should be accurately estimated. Consolidation settlements in the close proximity of Ulus-Keçiören Subway project due to groundwater drainage is the focus of this study. In this sense, the necessary data about the project characteristics and the site conditions were collected thru project descriptions and the geotechnical investigations conducted at the project site. Utilizing the generated database analytical calculations were carried out to predict the settlements. Upon completion of this stage of analysis several of the locations were numerically modeled for further investigation. Numerical analysis was conducted at four sections by using Plaxis, to determine the amount of expected displacements and the resulting groundwater situation. Despite of the differences between these two methods the resulting settlement estimations displayed consistency.

Keywords: consolidation settlement, subway tunnel, groundwater drainage

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ULUS-KEÇİÖREN METRO GÜZERGAHINDA YERALTISUYU DRENAJI NEDENİYLE OLUŞAN KONSOLİDASYON OTURMALARININ BELİRLENMESİ

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Nisan 2006, 189 sayfa

Metro tünel projelerinden sorumlu mühendisler için yüzey oturmaları her zaman önemli bir sorun teşkil etmektedir. Bu sorun, projenin başarısını doğrudan etkilemesi nedeniyle, ciddi olarak ele alınmalı ve hassas bir şekilde hesaplanmalıdır. Ulus-Keçiören Metro projesinde yeraltısuyu drenajı nedeniyle meydana gelebilecek konsolidasyon oturmaları bu çalışmanın odak noktasıdır. Bu amaçla, proje tanımlamaları ve jeoteknik araştırmalar dikkate alınarak proje özellikleri ve arazi koşulları hakkında gerekli bilgiler elde edilmiştir. Oluşturulan veritabanı yardımıyla olası oturma miktarları analitik olarak hesaplanmıştır. Analizin bu aşaması tamamlandıktan sonra belirlenen kesimler sayısal analizlerle ayrıntılı olarak incelenmiştir. Seçilen dört adet kesitte Plaxis programı ile olası deplasman değerleri ve yeraltısuyu son durumu belirlenmiştir. Bu iki yöntem arasındaki farklılıklara rağmen bulunan tahmini oturma miktarlarında tutarlılık gözlenmiştir.

Anahtar Kelimeler: konsolidasyon oturmaları, metro tüneli, yeraltısuyu drenajı

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In loving memory of my dear Grandmother, Gönül Çağlayaner

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to Prof. Dr. Vedat Doyuran for his valuable and encouraging guidance. He is the person that introduced me to geology a very long time ago and it has been a great experience to work with him ever since.

Special thanks to Dr. Nihat Işık that has provided valuable support on the numerical aspects of this dissertation. Thanks are also due to Yüksel Proje for providing the data that this study is based on.

I am very grateful to my beloved friends, Günseli – Deniz Akınç and Musa Yılmaz for standing beside me. Their great help and caring motivation made it a lot easier to complete this challenging work. I am also very thankful to Dr. Şahnaz Tiğrek for the useful academic advice she passed on.

Most importantly, I would like to thank my wonderful family that has always been there for me. I will always feel indebted for the unlimited love and constant support they have provided.

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CHAPTER 1

INTRODUCTION

1.1. Purpose and Scope

The focus of this study is to estimate the consolidation settlements taking place at the Ulus-Keçiören Subway Project. By calculating the settlements using different approaches, this study aims to investigate the efficiency of a variety of methods for calculating consolidation caused by groundwater drainage. During the course of the study a feedback of related theoretical information will also be provided.

Different approaches to be used for the estimation of consolidation settlements include both analytical and numerical methods. In this sense a range of information about the concept of consolidation and methods of settlement calculation are given as well as a detailed overview of the Ulus-Keçiören Subway project. The calculations were carried out by the help of both theoretical information such as commonly used concepts, equations determined by well known previous studies, textbook materials and data retrieved from the site such as the borehole logs, groundwater records, related laboratory test results and monitoring data.

Even though tunneling projects may cause many short and long term disturbances, this study specifically emphasizes the settlements due to the drawdown of groundwater table. This thesis discusses

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consolidation settlement calculations under the influence of Ulus-Keçiören project and suggests ways to improve the performance of these calculations where possible.

The settlement calculation process is the usual work to be done in every subway tunneling project and it is done in an intensive manner for the sake of the project. Every factor is carefully taken into consideration in detail within the project works to ensure an engineering accomplishment that will be satisfactory for its lifetime. Meanwhile the outcome of this study will represent an alternative point of view regarding the calculation methods and will be crucially important depending on the density of the population and constructions at the project area.

1.2. Location of the Study Area

The study area is the Ulus-Keçiören subway route which is 9685 meters long and stretched between two of Ankara's most densely populated and intensely constructed neighborhoods. As seen in Figure 1.1 the subway route initiates from Gençlik Parkı and intersects Istanbul Road reaching Kazım Karabekir Avenue. Following this avenue it then passes through Fatih and Kızlar Pınarı Boulevards. After Gökçek Park it reaches the Dutluk Crossroad. From there the route leads to Gazino Crossroad through Nuri Pamir Boulevard and ends at Aksaray Avenue. There are 9 subway stations that will be built along this route. Most of these stations are named after the areas which they will be built in and they are: Ulus, ASKİ, Dışkapı, Meteroloji, Belediye, Mecidiye, Kuyubaşı, Dutluk and Gazino stations.



Figure 1.1 Location of the study area (Yüksel Proje, 2003)

Starting from Ulus the topography of the area is mostly flat (approximately at 850 meters of elevation) or slightly undulated. Towards the end where the project approaches the Keçiören area, topography starts rising steeply to higher elevation. Since the project is located at where could be referred to as the heart of the city of Ankara, the construction stage is undergoing major challenges. The project works have to be performed under extreme care in order not to damage any of the surrounding structures above ground or service infrastructure founded below the ground as well as not to interfere with daily lives of the population within the vicinity of the neighborhood. Passing by many important residential and commercial areas the project will have a major affect on the city of Ankara. Even though the project is designed to make this a positive one, a minor mistake in the engineering applications can cause a mess in this critical area. Upon completion the project will provide a huge percentage of population the practicality of public mass transit which is not as well developed as it is supposed to be.

1.3. Previous Studies

Construction of urban tunnels in soft soil or weak rocks requires meticulous considerations in terms of geotechnical site investigations, construction methods, types of tunnel boring machines, tunnel support systems, groundwater control measures, instrumentation and monitoring of surface subsidence and the subsequent impact on nearby buildings and services. Among the considerations, the most important aspect is the control of surface subsidence and types of ground deformation to minimize any damage to surrounding constructions and disturbance to the population in the environment.

As infrastructure, buildings and services stretch through the densely populated and scarcely limited land space, the engineering projects that are to be designed at these areas should yield minimum disturbance to the routine daily lives of the city both while under construction and when providing service. In order to fully understand the extent of disturbance due to tunneling in such tight conditions, a comprehensive knowledge of the deformation caused by tunneling is essential. For settlement prediction purposes many empirical methods have been developed over the years through various field studies and experiences to predict the settlement caused by tunneling in soft ground by Peck (1969), Attewell *et al.* (1986), New and O'Reilly (1991) and semi-empirical methods were also developed by Lo *et al.* (1984). Some important considerations for the prediction of settlement are presented in their solutions but may not provide assistance in obtaining the ultimate result. Analytical methods could also be utilized, however, the characteristics of the soil profile and the site conditions regarding that specific design have to be acknowledged. Continuous research and advancement in technology towards tunneling works will inevitably lead to safer and both economically and environmentally efficient construction process (Tan and Ranjith, 2003).

There exist several approaches that are readily used in prediction of ground deformations associated with tunneling. Namely, analytical methods and numerical methods are commonly used in practice and the selection of the appropriate method depends on the complexity of the problem (Loganathan and Poulos, 1998).

Among many factors creating ground subsidence, one of the main causes is the consolidation process due to reduction of groundwater level. Even relatively small groundwater drainage will very rapidly reduce the pore pressure within a medium of soft clay deposits. This will then initiate a consolidation process gradually progressing upwards through the deposit. Such a consolidation process can lead to large ground deformations severely damaging any structure above this deposit. Even a tunnel project of average size may produce settlements up to 30-40 cm along the tunnel and settlements could be observed as far as 500 m from the tunnel alignment (Karlsrud, 2001).

Limiting of the surface settlement caused by tunneling in shallow and soft ground is of utmost importance for any tunnel engineer. While creating a solution for this great challenge one must not overlook the complexities of tunneling. Many problems can arise if settlements caused by tunneling are considered to be only vertically troubling. Studies and field works have shown numerous times that tunneling also causes lateral deformation and the longitudinal movement of the ground at the sides and ahead of the tunnel face, respectively. As a result of scarcity of research conducted to understand the longitudinal behavior of the ground along tunnel axis, very limited information is acknowledged to grasp longitudinal settlement. Attewell and Woodman (1982) overcame the difficulties of field studies, equipment installation and intensive monitoring by the help of an assumption which later on led to a model. Studies involving the lateral movement of the ground due to tunneling were comparatively more extensive. An empirical equation by Norgrove et al. (1979) is an aid to relate the subsurface settlement to the lateral deformations (Tan and Ranjith, 2003).

Proper drainage of groundwater during tunnel construction is essential because of three main reasons. First is to prevent an adverse internal environment. Tunnels and underground openings are subject to strict requirements to obtain a safe and dry internal environment for various reasons. In most of the cases such requirements do not allow presence of water on internal walls, the roof or on the ground of the tunnel. Only by controlled drainage of the excess groundwater, the effective work medium and the safety of both the equipment and the personnel inside the confined tunnel space can be assured. Another necessity is to prevent unacceptable impact on the external environment.

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Tunneling introduces the risk of imposing adverse impacts to the surrounding environment by lowering the groundwater table, which may cause settlements of buildings and other surface structures in urban areas and disturb the balance of the natural lakes, ponds and recreational areas in the neighborhood. Findings by Kveldsvik *et al.* (2001) deeply emphasize the vulnerability of such natural areas and address the potential ecological consequences. Keeping the tunnel or underground opening dry should also be expected when hydrodynamic containment must be maintained. Of course such watertight tunneling is needed in particular cases of storage and disposal for leakage prevention (Grøv, 2001).

Throughout the world, land subsidence due to large amounts of fluid withdrawal has occurred in numerous regions and has been extensively investigated both quantitatively and qualitatively by many researchers. Such subsidence is explained by the consolidation of sedimentary deposits as the result of increasing effective stress (Bell et al., 1986). Pratt and Johnson (1926) demonstrated that land subsidence resulted directly from lowering of the piezometric surface due to fluid extraction. Poland and Davis (1969) showed that the centers of subsidence in the Santa Clara valley, California, coincided with the centers of major pumping and development of subsidence increasingly occurred with the continuing groundwater utilization. In addition, Abidin et al. (2001) have proven that excessive groundwater extraction in Jakarta caused a serious land subsidence incident. Karlsrud (2001) included a valuable study to the literature by emphasizing that the water leakage that takes place during tunneling under urban areas of the Oslo region possessed a great subsidence threat. Furthermore, Chen et al. (2003) have shown that land subsidence in Suzhou City was strongly related with groundwater exploitation through a complex aquifer system.

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All these researches have enlightened the future of tunnel engineering clearing the obstacles of ground settlement. Many studies are being carried out to have an advanced understanding about this subject and overwhelming discoveries takes place. Prior to conducting this study, a broad vision is gained through a detailed review of this source.

CHAPTER 2

ULUS – KEÇİÖREN SUBWAY PROJECT

2.1. **Project Characteristics**

Other than being a very vital public mass transport project for the city of Ankara, the construction of Ulus – Keçiören Subway project also inherits properties that are strikingly unique. A total of approximately 10 kilometers of excavation is being made where a maximum inclination of 3.5% is reached. Five kilometers of this excavation, which is within a volcanic series, is done by explosives using New Australian Tunneling Method (NATM). As of January 2006 excavation within the volcanic series neared completion and another 4 kilometers of the project will be excavated mostly in Ankara Clay and alluvial deposits through cut-andcover and tunneling. In the original project, construction of 900 meters of the alignment is planned as using cut-and-cover technique at three different locations. The rest of excavation will be completed in the form of tunnel with nine subway stations that were mentioned earlier. All these different construction methods are selected through careful evaluation of site conditions. construction effectiveness. cost estimations. environmental impact and other important engineering concerns.

2.1.1. New Austrian Tunneling Method (NATM)

2.1.1.1. Historical Perspective

NATM is evolved as a result of experiences gained in Austrian Alpine tunneling conditions as the name implies. It was developed extensively between the years of 1957 – 1965 by the contribution of many tunneling pioneers. In 1958 Brunner patented "Shotcrete Method"-Runserau H.E.P Project-Squeezing ground, shotcrete application. After this, Mueller developed systematic deformation measuring system in 1960. The year of 1962 marks the time when Rabcewicz first used the term "NATM". Two years later in 1964 NATM achieved worldwide recognition. A number of events which are important in the development of NATM are summarized chronologically in Table 2.1.

2.1.1.2. Definition

NATM aims stable and economic tunnel support systems where the main idea is to utilize the geological stress of the surrounding mass (soil or rock) to stabilize the tunnel itself. The NATM has been particularly successful in conditions where complex geological features are anticipated or indeed encountered and which cause uncertainties in the prognosis of the rock mass behavior. Success in improving tunnel stability is achieved by shortening the length of the round which in fact reduces rate of advance, but improves tunnel stability extremely. The definition made by the Austrian Society of Engineers and Architects states that, the NATM "…constitutes a method where the surrounding rock or soil formations of a tunnel are integrated into an overall ring-like support structure. Thus the supporting formations will themselves be part

Table 2.1 Series of events that lead to NATM in chronological of	rder
(Karakuş and Fowell, 2004)	

Year	Development
1811	Invention of circular shield by Brunel
1848	First attempt to use fast-setting mortar by Wejwanow
1872	Replacement of timber by steel support by Rziha
1908	Invention of revolver shotcrete machine by Akeley
1914	First application of shotcrete in coal mines, Denver
1948	Introduction of dual-lining system by Rabcewicz
1954	Use of shotcrete to stabilize squeezing ground in tunneling by Brunner
1955	Development of ground anchoring by Rabcewicz
1960	Recognition of the importance of a systematic measuring system by Mueller
1962	Rabcewicz introduced the New Austrian Tunneling Method in a lecture to the XIII Geomechanics Colloquium in Salzburg
1964	English form of the term NATM first appeared in literature produced by Rabcewicz
1969	First urban NATM application in soft ground (Frankfurt am Main subway)
1980	Redefinition of NATM due to conflict existing in the literature by the Austrian National Committee on Underground Construction of the International Tunneling Association (ITA)

of this supporting structure." A more recent definition is given by Sauer (1988) claiming that NATM is: "...A method of producing underground space by using all available means to develop the maximum self-supporting capacity of the rock or soil itself to provide the stability of the underground opening." The NATM is actually an approach of philosopy, rather than a set of excavation and support techniques, integrating the principles of the behaviour of rock masses under load and monitoring the performance of underground construction during construction.

2.1.1.3. General Concepts of NATM

There exist twenty two principles of NATM in total. Among these principles seven important features play a major role in NATM:

- 1. Mobilization of rock mass strength
- 2. Shotcrete protection
- 3. Rock mass deformation and load measurements
- 4. Flexible supports
- 5. Invert closing
- 6. Tunneling contract agreements
- 7. Rock mass classification: Determining support measures

Example:

A1 – No support required (may be random local supports); fullface or top heading and bench in large excavation profiles; drill and blast

A2 – Shotcrete and random rockbolts; top heading (2.5-3.5m) and bench (4.00m); drill and blast

B1 – Shotcrete and systematic bolting; top heading (2.0-3.0m) and bench (4.00m); drill and blast

B2 – Shotcrete, systematic bolting, forepoling; top heading (1.5-2.5m), bench (3.5m); smooth blasting, roadheaders if rock masses are sensitive to vibrations

C1 – Shotcrete, systematic bolting, forepoling, steel ribs; top heading (1.0-1.5m), bench (2.0m), invert arch (100-150m); smooth blasting or rockheader or tunnel excavator

C2 – Shotcrete, systematic bolting, forepoling, steel ribs; top heading (1.2m), side galleries may be required, bench (2.0m), invert arch (25-50m); smooth blasting or rockheader or tunnel excavator

L1 – Shotcrete, forepoling or lagging ribs; top heading (1.5m), bench (3.0m), invert arch (100-150m); tunnel excavator

L2 – Shotcrete, forepoling or lagging ribs; top heading (1.5m), bench (2.0m), invert arch (24-50m); tunnel excavator



Figure 2.1 Photo image showing shotcrete application

Rock Class	Austrian Standard	Classification after
Description	ÖNORM B2203	Rabcewicz-Parcher
A1 Stable	1 Stable	
A2 Slightly overbreaking	2 Afterbreaking	
B1 Friable	3 Slightly friable	II Friable
B2 Heavily friable	4 Friable or slightly pressure exerting	III Heavily friable
C1 Pressure exerting	5 Heavily friable or pressure exerting	IV Pressure exerting
C2 Heavily pressure	6 Heavily pressure	V Heavily pressure
exerting	exerting	exerting or flowing
L1 Loose ground, highly		
cohesive		
L2 Loose ground, low		
cohesive		

 Table 2.2 Rock classification system for NATM

The NATM is a widely used method that avoids an extensive supporting system by making effective use of the inherent ground strength and the strengthening of existing ground by shotcrete and rock bolts.



Figure 2.2 Support measures according to rock mass classification (Doyuran, 2000)

2.1.2. Types of Tunnel Cross-sections

Ahead of construction phase of such a remarkable project the information from geological and geotechnical investigations must be fully acknowledged. Geological and geotechnical reports should be reviewed and the geotechnical profile of the tunnel should be studied well. The sections with different rock classes, the critical zones (pressured, weak or fractured rock mass, thin overburden, groundwater problem, fault, etc.) should be determined and corresponding excavation method and support measures should be defined. This is a vital work when it comes to make comparisons between the evaluations made in the reports and the

observations (geological mapping and geotechnical measurements) made at the tunnel face as the excavation proceeds.

Along the tunnel alignment there are six different types of tunnel cross-sections designed according to the project requirements, site conditions and the criteria above. They all serve different sorts of purposes and possess unique systems of support measures (Figure 2.3) with changing intensities.



Figure 2.3 Support measures: steel ribs, shotcrete, wire mesh, and concrete lining

2.1.2.1. Approach Tunnel

The subway excavation is carried out at approximately 20-35 meters of depth from the ground surface. In this case an approach tunnel is required to transport machinery and workers to the tunnel face. Excavation is done by blasting and progress length is around 2 meters in average. Support measures to be installed are steel ribs, wire mesh, shotcrete (20-25 cm in thickness) and rock bolts depending on the rock class in which tunnel is excavated. Besides of the approach tunnels there are two shaft accesses (Dutluk and Gazino shafts) connected to the main-line tunnel (Figure 2.4).



Figure 2.4 A view of the shaft access

2.1.2.2. Main-line (Base) Tunnel

As the name implies, this type of tunnel cross-section is the one that subway line and necessary appliances are situated in. The excavation is done in two stages during which the top and bottom halves of the tunnel face are blasted consequently. The diameter is 6.10 meters.



Figure 2.5 Main-line (Base) tunnel cross-section (Türkerler-Limak, 2004)

Steel ribs, two layers of wire mesh, systematic rock bolting and 20-25 cm thick reinforced shotcrete are designed as immediate support completed with a finishing concrete lining of 40 cm of thickness (Figure 2.5). Intensity of support application is determined according to the conditions of the excavated material and the frequency of structures on the ground surface.

2.1.2.3. Turnout Tunnel

It is the most critical kind of tunnel cross-section since the two tubes containing their own subway lines are combined together within the same single cross-section. It is the biggest cross-section designed in the project with 15 meters of diameter (Figure 2.6). Since a huge portion of material is excavated, the support installation is upgraded and improved. Same types of support measures with the main-line tunnel are applied more intensively including 25 cm of reinforced shotcrete and 50 cm of concrete lining.





There are various excavation methods in NATM according to the split of excavation face such as full face, bench cut, top/bottom drift and sectional excavation. Since this tunnel profile requires excavation of a sizable tunnel face and plays an important role in the project its excavation is planned in six phases (Figure 2.7).



Figure 2.7 Six phase excavation method

Excavation Phase 1 progresses further than all the other phases having the most distant excavation face. Excavation faces 2, 3, 4 and 5 reach a closer distance. Phase 6 is the excavation with least progress (Figure 2.8). This method of excavation enhances the security measures for tunnel construction as well as the long-term safety factor of the project.


Figure 2.8 Photo images illustrating six phase excavation: close and far

2.1.2.4. Station Platform Tunnel

This tunnel profile is the one that encapsulates the subway line together with a platform area that provides space for passengers of the mass transit unit (Figures 2.9 and 2.10). In order to fit all of these, a large excavation area of 9 meters of diameter is designed and the excavation is planned as two stages.



Figure 2.9 Platform tunnel cross-section (Türkerler-Limak, 2004)



Figure 2.10 A view from the platform tunnel cross-section

2.1.2.5. Connection Tunnel

They are the tunnels that provide a link between the two separate subway lines. It is a smaller elliptical tunnel profile that is planned to offer opportunities for maintenance and emergency cases (Figure 2.11). The excavation of this type of cross-section should be done only after the bigger tunnel profiles fulfill their deformation limit. Support systems include use of wire mesh, shotcrete of 20 cm thickness and 50 cm of concrete lining.



Figure 2.11 Connection tunnel cross-section (Türkerler-Limak, 2004)



Figure 2.12 Photo image showing connection tunnel

2.1.2.6. Staircase Inclined Tunnel

These profiles are used for the excavations that create direct access from ground surface to the tunnel level. They obviously require shallow overburden thickness to enable the public reaching the subway system with less effort. Among the variety of support measures are wire mesh, shotcrete and concrete lining applications.



STAIRCASE TUNNEL TYPICAL CROSS SECTION

Figure 2.13 Staircase inclined tunnel cross-section (Türkerler-Limak, 2004)

2.1.3. Investigation Works

2.1.3.1. Geological Mapping

A geological map of 1/5000 scale was prepared by Yüksel Proje (2003) for the region where Ulus – Keçiören Subway is located. Utilizing

the input from boreholes drilled, a geological profile is also prepared (Appendix A).

2.1.3.2. Borehole Investigations

Due to dense settlement only limited rock outcrops could be observed along the subway alignment. Thus, in order to reveal the geology along the route a number of boreholes were planned. A total of 1938.35 meters of drilling was made by Yüksel Proje (2005) in 67 boreholes to figure out the type, thickness, contact relationships, geological and geotechnical properties of lithological units present along the Ulus – Keçiören Subway route. Details regarding these boreholes are given in Appendix B.



Figure 2.14 Photo image showing borehole drill

In order to define the soil profile along the subway route, disturbed (SPT) and undisturbed samples were collected. In every borehole Standard Penetration Tests (SPT) and Pressuremeter tests were conducted at every 1.5 meters of depth in soil units. To determine the permeability of soil and rock units Constant Head Permeability and Water Pressure tests were conducted. SPT samples that are retrieved were prevented from exposure by plastic bags and undisturbed (UD) samples were sealed off by paraffin. Rock core samples were stored, with regard to the order, in wooden core boxes. All the samples were investigated further in soil mechanics laboratory of Yüksel Proje.



Figure 2.15 SPT Sampler, sample bag and pressuremeter device

For groundwater level monitoring, a perforated PVC pipes were installed into the boreholes. Groundwater level measurements are taken regularly on a monthly and for some periods, on a biweekly basis.

2.2. Geology

2.2.1. General Geology

Main geological units exposed in and at close vicinity of Ankara are: Dikmen Formation, Alacaatli Formation, Hançili Formation, volcanic series, Akhöyük Formation, Etimesgut Formation and alluvial deposits.





Figure 2.16 Geological map of Ankara (Akyürek et al., 1997)

Primary rock unit in the area is the Dikmen Formation of Paleozoic - Triassic age which lithologically consists of schist and greywacke with occasional limestone blocks. Alacaatlı Formation is mostly represented by limestones which crop out at Alacaatli, Balikuyumcu, Dereköy and Deveci villages. It also contains marl, claystone, sandstone and occasional sand - gravel layers. Along the subway route, Hançili Formation is represented by sandstone, siltstone and tuff alternations. This formation is closely associated with the volcanites of the same age. Inside the city, near Ankara Citadel, Keçiören, Mamak and north of Yenimahalle a volcanic series of Miocene age is observed. This volcanic series contains andesite, dacite, basalt, tuff and agglomerate. The Akhöyük Formation consists of an alternation of claystone, marl and clay. Etimesgut Formation of Pliocene age is a clay based combination of lacustrine deposits and river deposits. It consists of silty clay and gravelly, sandy clay. It is also referred to as "Ankara Clay". Alluvial deposits are seen along the major stream valleys.

2.2.2. Geology of the Study Area

The units exposing along Ulus – Keçiören Subway route include Hançili Formation, Volcanic Series, Ankara Clay and alluvial deposits. In urban environment most of these units are concealed with artificial fill, asphalt paved roads and buildings. Within the total length of subway route the sedimentary units cover 5650 meters (approximately 58%) and volcanic series cover the remaining 4035 meters (approximately 42%).

2.2.2.1. Hançili Formation

Hançili Formation is deposited in streams and lakes in a terrestrial environment (lake being the dominant environment of deposition) in which alluvial fans are developed at the margins. It is formed by the alternation of clayey limestone, marl, siltstone, sandstone, conglomerate and tuff of Miocene age. At parts it may contain bituminous shale and gypsum. During this alternation, dominant rock type changes locally. Andesite sills have been observed. Clay-limestone and marl are white to yellowish-white, thin-to-medium bedded and alternates with siltstone and sandstone. Siltstone is grey, weakly cemented, thinly layered and shows lamination. Conglomerate and sandstone are yellowish-grey, weakly cemented and do not show obvious layering. This formation was first named by Akyürek *et al.* (1980). Along the subway route it is encountered in few boreholes and constitutes about 2% of the subway route.

2.2.2.2. Volcanic Series

The volcanic series is dominantly composed of andesite, basalt, tuff and agglomerate. The agglomerate is white, grey and red colored and it contains andesite, dacite, and basalt fragments of different sizes within a tuffaceous matrix. Layering is barely observed. Andesite is usually red, pink, grey and black. Basalt is black and dark brown. It is vesicular and shows flow structure. These volcanic units show a chaotic mixture causing sudden changes in lithology as the excavation proceeds through the tunnel.

2.2.2.3. Ankara Clay

It is dominantly composed of silty and/or sandy clays with occasional sand and gravel lenses. They are deposited within the floodplains of ancient streams. Even though fine-grained deposits are dominant the sand and gravel lenses represent ancient river channels. Outcropping between Etlik Avenue and Turgut Özal Boulevard, the Ankara Clay is of Pliocene age. It is basically silty clay and gravelly, sandy clay that is red, brown and beige, fissured, contains carbonate concretions, partly has layers of sand and gravel, either low or high in plasticity, very stiff and over-consolidated. Its mineralogical composition is directly controlled by the bedrock from which they are derived. For instance, montmorillonite originates from volcanic rocks; whereas chlorite is a weathering by-product of schist and greywacke. The sand and gravel lenses within the unit range between sandy gravel, clayey sand or clayey, sandy gravel. The Ankara Clay could be found at a 20% portion of the subway route.

2.2.2.4. Alluvial Deposits

They are observed within the Ankara Creek and its tributaries. The deposits are composed of sandy-silty clay, clayey sand, clayey-sandy gravel. The clayey fractions possess medium to high plasticity. The color of the alluvial deposits is grayish-brown.

The clayey portion of these deposits possesses a great potential of causing consolidation settlement. Presence of clay is surely the main reason of consideration but high plasticity, water content, permeability to allow drainage and other properties put this unit forward more than Ankara Clay. Hence this unit stands at the focal point of this study in terms of consolidation settlements. It is the main sedimentary unit representing the 36% of the subway route.

2.2.2.5. Artificial Fill

Covers the project area almost completely. It is originated by the excavations made for the constructions of neighboring structures and other dumped material. It occurs mostly as a thin layer and its engineering aspects are at an unsatisfactory level.

2.3. Hydrogeology

The groundwater conditions of Ulus – Keçiören Subway Project are very closely related with the hydrogeological properties of the existing geological units that were mentioned earlier.

Within the volcanic series, the agglomerate unit has low permeability. Andesite and dacite units allow groundwater movement depending on the fractures they bear. Tuff that is found in lenses within these rocks is usually not permeable at all. Ankara Clay, which is an impermeable unit, possesses some residual groundwater at the sandgravel lenses it contains. Depending on size of these lenses and place where they are situated, groundwater could be found at different levels of depth even though a groundwater table is not established generally throughout this unit. Since these sand-gravel lenses are separate from each other, during excavation of Ankara Clay it could be either completely dry or groundwater could be encountered at some parts. The river channels and their branches that intersect the tunnel route have accumulated alluvial deposits represented by sandy clay, clayey sand and clayey-sandy gravel. These units contain groundwater, are highly permeable and because of these reasons they are expected to have a negative effect on the excavation process.



Figure 2.17 Groundwater drainage

As an underground excavation is under operation it is guite difficult to predict the amount of groundwater draining from fractured rocks. This difficulty could be explained by three main aspects. Firstly, it is quite a challenge to determine the hydrogeological aspects of rocks that are to be passed on tunnel route. Even though geological mapping studies, investigational borehole drillings, permeability and water pressure tests reveal some information about subsurface geology and the hydraulic properties of the rocks, the distance between boreholes cause lack of data that is critical for investigation. This lack of data is a result of the heterogeneity of the soil profile. Besides the subway route passes through a residential area covering a great portion of the project area which makes examination of surface geology impossible. A second factor is the difficulty of modeling groundwater drainage due to specific soil conditions at the site. The complexities are much more at a medium of fractured rocks than a porous medium of soil. The last aspect is the inadequacy of field works. The number of field tests conducted to define hydraulic properties of rocks is not enough and the medium made up of rocks is way too heterogeneous. Therefore, the input from these field tests will not go further than giving a general idea because it cannot

provide the numerical support that is sufficient for any kind of model. For example, it is even not possible to pick an average value of hydraulic conductivity in a case when different permeability levels are read from two different depths of the same borehole. In a medium of alluvial soil the hydraulic properties are relatively more homogeneous even if it is known to be a mix of clay, silt, sand and gravel showing lateral and vertical transitions.

Uncontrolled groundwater flow is one of the worst geotechnical problems the tunneling operation may cause. Sudden discharge of groundwater through a tunnel face inside a medium of saturated, highly jointed and fractured rock is a critical problem. Hence the potential groundwater discharges that may occur should be elaborated at the design stage. If predicted early, certain drainage precautions can be taken for such groundwater discharges. Depending on the inclination of the excavation base the drainage could happen by gravity otherwise pumping will be necessary. In any case prior acknowledgement of the amount of groundwater to be drained is needed. Amount of discharge could usually be decreased by the use of impervious barriers or injection applications. But these measures may not provide any guarantee for stopping leakage through tunnel face or the invert.

2.3.1. Distribution of Hydraulic Conductivity Values

A total of 73 constant head permeability tests and 41 water pressure tests were conducted inside the boreholes drilled during the investigation works. Water pressure tests were done in the volcanic series and the Hançili formation, whereas in the alluvial deposits and Ankara clay constant head permeability tests were performed. The alluvial deposits consist of clay, silty clay, clayey silt, gravelly clay, clayey silty sand, sandy gravel, gravelly sand. They are observed in the form of layers and lenses having lateral and vertical transitions. The hydraulic conductivity values range between $1.5*10^{-7}$ m/sec and $6.4*10^{-4}$ m/sec. In Figure 2.18, the distribution of hydraulic conductivity values derived from the permeability tests is displayed (Doyuran, 2005). The value of 10^{-8} m/sec indicates impervious unit. It can be observed from the normal distribution of the values that the average hydraulic conductivity concentrates around 10^{-6} m/sec. The average is assumed to be $3.3*10^{-6}$ m/sec.



Figure 2.18 Distribution of hydraulic conductivity in alluvial deposits (Doyuran, 2005)

For Ankara clay 17 out of 22 permeability tests yielded no seepage and hence they are regarded as impervious. In three of these tests the hydraulic conductivity is estimated as 10^{-7} m/sec, in one of them 6.7×10^{-6} m/sec and in another one 1.17×10^{-5} m/sec were found. Ankara clay could be considered to be impervious depending on these values. Other construction projects carried out in Ankara clay yielded insignificant amount of groundwater drainage.

A total of 39 water pressure tests were conducted in andesite, dacite, agglomerate and tuff units, also called volcanic series. Tests results suggested that the permeability ranges between 1.17 and 25 Lugeon. As it is proved in Figure 2.19, there is no correlation between Lugeon and RQD values. This is a result of anisotropic - heterogeneous medium fractured rocks create and the limitations of water pressure test.



Figure 2.19 Comparison of water pressure test results and RQD values (Doyuran, 2005)

The results of the water pressure tests depend on the aperture size of discontinuities as well as their orientation. RQD being related with

frequency of fractures does not suggest that it is directly related to permeability. A rock mass that bears many narrow aperture discontinuities has a low permeability as well as RQD. Therefore the aperture of discontinuities is more important rather than frequency. As seen in Figure 2.19, the 24 out of 25 test results are over 25 Lugeons indicating that the discontinuities from the rocks forming the volcanic series happen to have wide apertures. Hence occasional high discharge of groundwater is to be expected when excavating the tunnel in this zone. The hydraulic conductivity in volcanic series is around 4*10⁻⁷ m/sec (Doyuran, 2005).

2.3.2. Groundwater Levels

Starting from 01.10.2003, groundwater level at every drilled borehole on the subway route was measured on a monthly basis. The measurements revealed that groundwater table is positioned at a depth usually shallower than 10 meters. The hydrostatic pressure on the crown of tunnel is less than 3 atm., at some parts as low as 1-1.5 atm. The initiation of tunnel excavation works triggered groundwater drainage either as leakage or low-medium discharge drainage causing significant drop of groundwater table in boreholes near the tunnel face.



Figure 2.20 Changes in groundwater level in the boreholes due to drainage during tunnel construction (Doyuran, 2005)

2.3.3. Groundwater Drainage in Tunnel

The region where subway route passes through alluvial deposits of Ankara Stream the hydraulic conductivity is $3.3*10^{-6}$ m/sec and the hydraulic burden (depth from groundwater table to the invert of tunnel) changes between 20 and 2 meters. The excavation works have not yet started at this region so groundwater drainage did not occur. Possible groundwater drainage per unit length of the tunnel is calculated by using static levels and the results are displayed in Figure 2.21. This figure also shows that in worst case the expected level of drainage is 350 m³/day (~4lt/sec) for one meter of excavated portion (Doyuran, 2005). This amount will surely go down as the hydraulic burden decreases with time. Of course such a furious drainage will not be achieved since the excavation can not be completed in a single day. The drainage situation for the worst case is shown in Figure 2.22. In this case the drainage will

start around 350 m³/day, descending rapidly by time and reaching a stable flow regime with low discharge (Doyuran, 2005). Both Figures 2.21 and 2.22 prove that groundwater drainage will be in controllable limit even when no precautions are taken. On the other hand, the sand-gravel lenses existing in alluvial deposits may cause remarkable discharge occasionally.



Figure 2.21 Groundwater drainage with respect to hydraulic head and the radius of influence (Doyuran, 2005)



Figure 2.22 Change of groundwater drainage with time (Doyuran, 2005)

As mentioned earlier in regions where tunnel excavations will be carried out in Ankara clay, no critical groundwater drainage is expected given that the unit is impermeable.



Figure 2.23 Groundwater inflow to the tunnel

2.3.4. General Evaluation

Among the geological units that form a foundation to the subway tunnel construction, the alluvial deposits and the volcanic series show aquifer properties, therefore, having a great hydrogeological importance. Through these units leakage and low-discharge draining is expected. However, this can be controlled by necessary precautions and will diminish to an insignificantly low level with time. The problems that may occur in alluvial deposits might be sand boils, caused by loss of groundwater equilibrium in presence of cohesionless soil at the excavation base. The other one is consolidation settlement due to drainage which reduces the pore water pressure within a soft clay deposit. The buildings that are located on top of this bed of clay deposit may have serious settlement damage in the long term. Groundwater drainage when excavating the volcanic series will cause a decrease in the pore water pressure at the clay/rock interface. This will trigger the consolidation process to take action starting from this interface and progressing up through the clay bed. These occurrences should be carefully monitored since they may result in cracks on the buildings of the surrounding area. In addition, the flushing of fine particle deposits during groundwater drainage should be prevented. The drainage should be kept at a low and stable level if possible.

From the tunneling point of view, it is well known that the presence of groundwater has a negative effect on rock mass properties and behavior. Hence this fact shouldn't be ignored when rock mass classes are being assigned and when temporary support measures are being designed.

2.4. Geotechnical Investigations

To acquire detailed information about the site conditions, geotechnical investigations of wide coverage are used. These investigations are divided as field studies and laboratory tests. All these investigations helped to distinguish the units that need concentrating on from a geotechnical point of view. They also aided the analysis process which will be explained later on.

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2.4.1. In-situ Investigations

Best way to gather important information about a project is to get observations out in the field where project is located. It involves all the techniques and inquiries that can be used to gain knowledge on a particular location. It proves even better results once ground investigation starts, during when in depth investigation of subsurface material is done.

For this purpose, a total of 67 boreholes were drilled through almost 2 kilometers of soil by Yüksel Proje. Observations made by supervising engineers were recorded on logs producing a detailed lithological profile. Sampling allowed further research about the material and in-situ testing helped with useful and highly precise data from the actual site in its natural state. A variety of in-situ tests provide feedback at the same time as drilling continues. These tests include standard penetration tests and pressuremeter tests. Both of these tests clearly present compressibility, resistivity and deformation behaviors by determining corresponding properties of the soil material.

The investigations in the field showed that artificial fill should be disregarded. It has also proven that deeply seated Ankara Clay is highly resistant for consolidation. The alluvial layer that was found at around the mid-borehole depth displayed high-sustainability of settlement. The risk imposing zones in the project area were also noticed during the course of these studies.

2.4.2. Laboratory Investigations

The soil mechanics laboratory of Yüksel Proje put a great effort in determining important material parameters from the project area. The

samples received were carefully investigated by reliable methods. Main investigations carried out were:

- Sieve analyses
- Determination of Atterberg limits
- Uniaxial compression tests
- Triaxial compression tests
- Consolidation tests

Data provided by these studies were collected within a database created especially for the purpose of consolidation analysis. Evidently, instead of Ankara Clay, the alluvial deposits were once again proven to bear a potential of consolidation.

2.5. Recent Developments

Despite the fact that the construction phase already began, some major changes were applied to the original project. As much as it was surprising to come across such execution, the modifications were improving and necessary for the current situation.

The most important change made was about the tunnel alignment. The original project suggested that the subway line would extend from Keçiören to Ulus. However, the tunnel route was then re-directed in the direction of Tandoğan starting from Mecidiye subway station. Therefore, the new alignment connects Keçiören – Tandoğan to each other even though the name of the project remains as Ulus – Keçiören Subway Project. Due to this remarkable deviation from the initial state of the project, almost all aspects of the design needed an up-to-date evaluation. This re-evaluation process included the method of tunneling as well. According to the latest investigation works, newly designated project area was assessed and tunneling by TBM (Tunnel Boring Machine) was selected as a suitable conduct rather than cut-and-cover method. The TBM for soft ground tunneling would especially be active at the newly added portion of the alignment where alluvial deposits are abundant. This way the groundwater drainage that was expected to take place on an extensive scale is avoided even before it began because of the fact that support measures are applied right after excavation when tunneling with TBM.



Figure 2.24 Open excavation for tunnel portal approach at Dışkapı.



Figure 2.25 Assemblage of TBM at Dışkapı station



Figure 2.26 TBM and segment installation

Even though these series of changes were appropriate for the sake of the project, modifications of such proportions must be completed before project initiation and during the planning stage when the project is being designed. The change of plans also took its tool on the scope of this dissertation by costing much more time and effort. The findings of this thesis study would be relevant and useful in the case when a tunneling methodology that allows groundwater drainage (such as cut-and-cover method) is adopted.

CHAPTER 3

THEORETICAL BACKGROUND ON CONSOLIDATION

3.1. General Definition

It is a well-known fact that any material tends to reform and change its shape due to application of load. A steel rod will extend under the influence of tensile stress whereas a cement block will shorten as a result of compressive pressure. Stress changes have a similar effect upon soil. When this behavior is investigated it is understood that some deformations are reversible with the removal of load whereas others show a permanent effect on the material under influence. It is called an elastic deformation if recovery is observed. Plastic deformation, on the other hand, is the type which strain lasts. Only a small portion of ground deformation inhabits elastic behavior. In addition to this, the historical record of the soil is a factor when deformation behavior is to be estimated, meaning that a previously applied stress somehow leaves a trace property on the soil. All these characteristics make investigation of ground deformation an extremely complicated problem which can not be always expressed by mathematical equations (Özaydın, 1997).

The process of consolidation is the gradual reduction in volume of a saturated low permeability soil due to drainage of pore water. It continues until the excess pore water pressure is completely dissipated. Consolidation settlement is the vertical displacement of the ground surface corresponding to the volume change at any stage of the consolidation process (Craig, 2004). There are three main reasons for consolidation:

- compaction of soil particles
- compaction of air/water in the pores
- extrusion of air/water in the pores

Soil particles are usually composed of solid minerals which resist to yield a noticeable amount of compaction. Since the compressibility of water is also negligible, the compaction of pore water within a fully saturated soil will not be able to contribute to the consolidation either. All is left for consolidation to occur is the last option (pore water extrusion) which brings soil particles closer and decreases the overall volume of the soil. Hence the main reason for consolidation (especially in saturated soil) is the dissipation of pore water. Consolidation is the process of soil mechanics in which reduction of volume occurs and varying levels of permeability within a soil profile brings in the time factor. In order to investigate the consolidation of soil, stress-deformation-time relationships must be elaborated (Özaydın, 1997).

The process of swelling, on the other hand, is the gradual increase in the volume of a soil under negative excess pore water pressure proving that it is the reverse of consolidation. Consolidation settlement may occur due to a structure built over a layer of saturated clay or by lowering of groundwater table whereas excavation of saturated clay may result in heaving (reverse of settlement) which will cause swelling of clay at the bottom of the excavation (Craig, 2004).

3.2. Consolidation (Oedometer) Test

The properties of a soil during one-dimensional consolidation or swelling can be determined by means of the consolidation test carried out by oedometer device (Figure 3.1).



Figure 3.1 Schematic diagram of consolidation test apparatus (Das, 2002)

The test specimen is disc-shaped (6.35 cm. in diameter and 2.54 cm. in height) and placed inside a metal ring and lying between two porous stones. The porous stone on top is fixed below a loading cap through which pressure can be applied to the specimen. The whole assembly is situated in an open cell filled with water to which the pore water in the specimen has free access. The confining ring imposes a condition of zero lateral strain on the specimen. The ring must have a smooth and polished surface at the inner face for reduced side friction. The compression level of the specimen under pressure is measured by use of a dial gauge. According to the standardized test procedure load on

the specimen is applied (initial pressure depending on the type of soil) followed by a sequence of pressures each being double the previous value. Each pressure is maintained for a 24 hour period during which compression readings are taken at suitable intervals. These are usually $\frac{1}{4}$, $\frac{1}{2}$, 1, 2, 5, 10, 30 min.; 1, 2, 4, 8, 24 hr. intervals (Figure 3.2). At the end of the increment period the excess pore water pressure has completely dissipated and the applied pressure is equal to the vertical effective stress within the specimen. This way it is possible to determine the consolidation settlement caused by various incremental loadings (Cernica, 1995).



Figure 3.2 Photo image of oedometer device in soil mechanics laboratory of Yüksel Proje

Based on laboratory tests, a graph can be plotted showing the variation of the void ratio e at the end of consolidation against the corresponding stress p. The nature of change in void ratio (e) to stress (log p) is displayed in Figure 3.3.



Figure 3.3 An example of e – log p curve for soft clay

After the desired consolidation pressure is reached, the specimen is gradually unloaded allowing it to swell. The variation of e against log p during this unloading period is also plotted in Figure 3.3.

From the e-log p graph, three parameters can be derived that will be essential in calculation of settlement. These are:

a. Pre-consolidation pressure, p_c: It is the maximum past effective overburden pressure to which the soil has been subjected. A simple graphical procedure was proposed by Casagrande (1936) to obtain this value from e-log p curve. Comparing this value to the applied pressure will reveal the level of consolidation (overconsolidation ratio, OCR). Soil deposits are found in either normally consolidated or overconsolidated state in the nature. The soil is normally

consolidated if the present effective overburden pressure p_0 is equal to pre-consolidation pressure ($p_0=p_c$) and overconsolidated if present effective overburden pressure is less than pre-consolidation pressure ($p_0 < p_c$).

- b. Compression index, C_c: It is the slope of linear portion of the loading curve. The value determined from curve may be somewhat different from that encountered in the field primarily due to the remolding of soil. It can vary widely based on soil type and there are some empirical correlations that have been suggested.
- Swelling Index, C_s: It is important in the consolidation settlement estimation for overconsolidated clays. It is about 1/4 to 1/5 of the compression index in most cases.

At the time of compression, soil structure goes through continuous changes and the clay does not bounce back to the original structure after the expansion. Studies have shown that overconsolidated clay will be much less compressible than normally consolidated form. There are two parameters that represent the compressibility of the clay, one of which is the compression index that is described above. The other is the coefficient of volume compressibility, m_v , which is defined as the change in volume per unit volume per unit increase in effective stress. It is not a constant value for particular soil but instead it depends on the stress range over which it is calculated (Das, 2002).

3.3. The Principle of Effective Stress

The best way to visualize the structure of the soil is as a skeleton of solid particles enclosing continuous voids that contain either water or air. The volume of soil skeleton can change due to rearrangement of soil particles. In fully saturated soil a reduction of volume is only possible by escape of water (which is incompressible) from the voids. The concept of effective stress is introduced when this water extrusion occurs.

In 1923, Terzaghi presented the principle of effective stress and the importance of the forces transmitted through the soil skeleton was recognized. The principle is a relationship based on experimental data, applies only to fully saturated soils and involves following three stresses:

- a. the total normal stress, σ : the force per unit area on a plane within the soil mass
- b. the pore water pressure, u : the pressure of water in voids
- c. the effective normal stress, σ : the stress transmitted through the soil skeleton only.

The relationship is:
$$\sigma = \sigma' + u$$
 (3.1)

An increase in pore water pressure results in transient flow of pore water towards free-draining zone. This drainage continues until steadystate pore water pressure is reached. The increase in pore water pressure above this value is referred to as excess pore water pressure. Dissipation is the reduction of the excess pore water pressure back to steady-state value and the soil is considered to be in drained condition. As pore water drainage takes place (for reasons such as, external loading, tunnel excavation, groundwater pumping, etc.), the reducing space can not be replaced by air and particles of soil are taking up new positions. With the dissipating excess pore water pressure, loading will be entirely carried by the soil skeleton increasing the vertical effective stress, making the soil particles become more condensed and therefore creating a reduction in volume. This is the process of consolidation settlement and the time taken to complete this process depends on the permeability of the soil. Since this behavior usually occurs in cohesive soil such as clay which has low permeability and slow drain ability, consolidation could become a long-term process with more than one phase.

The mechanics of consolidation could be explained by the aid of a basic analogy. Figure 3.4 displays a spring inside a water-filled cylinder with a valve fitted piston on top. Spring represents the compressible soil skeleton, water in the cylinder is the pore water and the valve deploys the permeability of soil. With the valve closed (Figure 3.4 a - b), a load placed on top will not move the piston because the water is incompressible. This situation corresponds to the undrained condition in the soil. However, with the valve opened (Figure 3.4 c - d), the water will be forced out allowing the piston to move and squeeze the spring to which the load is transmitted to. Increase in the load on spring will represent the decrease in pore water pressure. Load will be totally lifted by the piston and the spring, resembling the drained condition in the soil. Load carried by the spring is assumed to be the effective normal stress in the soil and the movement of the piston is the change in volume of the soil (Craig, 2004).



Figure 3.4 Consolidation analogy

3.4. Terzaghi's Theory of One-Dimensional Consolidation

This theory relates three main quantities:

- a. excess pore water pressure (u)
- b. depth (z)
- c. time (t)

It is based on following assumptions:

- 1. The soil is homogeneous
- 2. The soil is fully saturated
- 3. The solid particles and water are incompressible
- 4. Compression and flow are one-dimensional (vertical)
- 5. Strains are small
- 6. Darcy's Law is valid at all hydraulic gradients
- Both the coefficient of permeability and the coefficient of volume compressibility remain constant throughout the process
- 8. There is a unique relationship between void ratio and effective stress that is independent of time.

When these assumptions are investigated carefully some shortcomings of the theory become evident. From studies made it is known that a deviation from Darcy' Law exists at low hydraulic gradients. Considering assumption 7, the coefficient of permeability and the coefficient of volume compressibility both decrease during consolidation since the void ratio and effective stress have a non-linear relationship. On the other hand, it is a reasonable assumption for the case of small stress increments. The most important limitations of Terzaghi's theory arise from
the eighth assumption. There is evidence from the experimental results that the void ratio-effective stress relationship is dependent of time.

3.5. Calculation of Consolidation Settlement

3.5.1. One Dimensional Method

The value of either the coefficient of volume compressibility or the compression index is required for the estimation of consolidation settlement. Assuming that the settlement is one-dimensional, the condition of zero lateral strain applies within the clay layer. The decrease in volume per unit volume of clay is expressed as below in terms of void ratio:

$$\frac{\Delta V}{V_0} = \frac{e_0 - e}{1 + e_0}$$
(3.2)

Considering that in the absence of lateral strain, decrease in volume is equal to reduction in thickness per unit thickness. The settlement of a layer with thickness H is given by:

$$S_c = m_v \Delta \sigma' H \tag{3.3}$$

Here, $\Delta \sigma'$ (stands for the change in effective stress) and m_v are assumed to be constant with depth (Craig, 2004).



Figure 3.5 e – log p curve for normally consolidated clay (Das, 2002)

For normally consolidated clay the e-log p curve will be like in Figure 3.5, therefore, the equation for calculating consolidation settlement is:

$$S = \frac{C_c H}{1 + e_0} \log \frac{p_0 + \Delta p}{p_0}$$
(3.4)

For overconsolidated clay, however, the e-log p curve has two different sections as shown in Figure 3.6 and the settlement calculation depends on Δp value (Das, 2002). In the case where the summation of initial effective overburden pressure and average pressure increase on the clay layer (which reveals the pressure in final condition) is less than pre-consolidation pressure ($p_o + \Delta p < p_c$), the settlement is calculated by:

$$S = \frac{C_s H}{1 + e_0} \log \frac{p_0 + \Delta p}{p_0} \quad \text{(Case 1)}$$
(3.5)



Figure 3.6 e – log p curve for overconsolidated clay (Das, 2002)

The other case suggests that if $p_o < p_c < p_o + \Delta p$, then:

$$S = \frac{C_c H}{1 + e_0} \log \frac{p_c}{p_0} + \frac{C_c H}{1 + e_0} \log \frac{p_0 + \Delta p}{p_c} \quad \text{(Case 2)}$$
(3.6)

3.5.2. Skempton-Bjerrum Method

The equation from the preceding section are based on onedimensional laboratory consolidation tests using representative samples of the clay and the underlying assumption is that the increase of pore water pressure is equal to the increase of stress at any given depth. In the field, however, this assumption will fail to comply. In practice significant lateral strain will occur and the initial excess pore water pressure will depend on the in-situ stress conditions. In such cases where lateral strain exists, there will be an immediate settlement in addition to the consolidation settlement. Skempton-Bjerrum modification for consolidation settlement calculation takes this into account by the aid of a pore water pressure parameter and settlement ratio (Craig, 2004).

3.5.3. Stress Path Method

As the name implies this is a method which recognizes the stress path followed up to the final state of stress as a factor that affects the resulting soil deformation. The method relies on the correct selection of typical soil elements and on the test specimens capable of truly representing the in-situ material regardless of having a sound principle. It is considered complex and time consuming due to the triaxial techniques involved in running the correct stress paths (Craig, 2004).

3.6. Average Degree of Consolidation

As explained earlier, gradual dissipation of the excess pore water pressure causes an increase in effective stress which induces settlement. To be able to estimate the degree of consolidation of a clay layer at some time t after the consolidation process initiated, the rate of dissipation of the excess pore water pressure should be known (Das, 2002).

The average degree of consolidation of clay layer is defined by:

$$U = \frac{S_t}{S_{\text{max}}}$$
(3.7)

where S_t : settlement of a clay layer at time t

S_{max} : maximum consolidation settlement that the clay will undergo

3.7. Consolidation Behavior in Natural Soil Deposits

Different types of soil display consolidation behaviors totally unique to themselves. Even though the general form of strain-stress curves derived from oedometer tests are similar, the rate and amount of consolidation show variations depending on soil types. Furthermore, this process is under the effect of stress history, degree of compaction and internal structure of soil together with the errors that happen during retrieval of test specimen and its preparations for laboratory test. All these factors should be taken into account when investigating the consolidation behavior of natural soil deposits.

3.7.1. Compaction of Sand

Sandy soil performs a sudden reaction to consolidation and immediate compaction occurs. In the field, the compaction and corresponding settlements happen and are completed in the construction phase. Most important factor that affects consolidation of sand is the degree of compaction. The difficulty of undisturbed sample retrieval enforces the necessity of artificial specimen (prepared with the same sand in field) production to be used in oedometer test. The results should be satisfactory, but it should not be forgotten that the sand layers in the field may hold differences. Especially in the situations which involve cemented or collapsible sandy soil the test would create misleading results. Silty soil usually has a similar behavior to that of fine sand as well. It is more suitable to take undisturbed samples from silty soil making it available for laboratory investigation.

3.7.2. Consolidation of Clay

Consolidation is a slow process depending on time when it comes to clay. In order to get the exact field behavior, the experiments should be carefully conducted on undisturbed samples. Compaction of clay is highly affected by its loading history. It just so happens that in some cases the present effective vertical stress (weight of the overlying layers) turns out to be less than a stress applied in the past. In such cases the clay layer has been consolidated beforehand but then the additional stress is diminished. As mentioned before, this type of soil is called preconsolidated or overconsolidated clay. If the present overburden pressure is actually the biggest consolidation pressure experienced then the clay layer is said to be normally consolidated. The $e - \log p$ curve gives a better idea about the consolidation behavior of this type of soil.

CHAPTER 4

ANALYSIS

In this section the consolidation settlement due to groundwater drainage at the alluvial sections of the Ulus-Keçiören subway project is predicted. Before beginning the whole process, detailed information regarding the site and the project were gathered to establish an understanding of the whole situation. Furthermore, certain assumptions were made to provide the analysis with the ability to closely represent the actual site parameters and project characteristics. The calculations were carried out according to the analytical equations and basic principles suggested by widely accepted conventional methods that are in existence for quite a long time. These calculated values of settlement are then verified by the aid of a numerical model constructed in the Plaxis program.

4.1. Database Generation

In order to initiate the analysis, the cases that match the conditions of this study were picked and the required data were gathered through the results of a variety of tests. The following steps point out to the creation of a data source that aided the settlement calculations.

4.1.1. Borehole Selection

As mentioned previously, the sedimentary strata for which consolidation settlement is expected are the layers of alluvial deposits. These alluvial materials contain gravel, sand and silt deposits, which are intercalated with the soft clay deposits and/or embedded within a clayey matrix. This geological unit is more vulnerable to settlements than the Ankara clay which is a hard, stiff, relatively impervious silty clay which occasional contains gravelly and sandy lenses. Moreover, the investigation is based on settlement solely caused by the groundwater withdrawal from the clayey relatively permeable units. In order to be able to assess the settlement process in the alluvial deposits, related data should be gathered from in-situ and laboratory tests which will provide detailed information about the characteristics of the layer that is expected to experience consolidation settlement. Sometimes it is not possible to conduct such detailed investigation at the preliminary stages of a project and so, the in-situ tests and samples may not be available from the corresponding material. Throughout subsurface exploration works that were conducted in the project area it has been a challenge to find the ones fitting the above conditions. This difficulty was due to unavailability of the undisturbed samples, presence of alluvial deposits at only certain parts of the project and the fact that project works are still under way during the preparation of this study. Therefore, only some of the boreholes drilled on the route in between Tandoğan and Mecidiye stations suited the case above. These boreholes are: TA-1, TA-2, TA-3, TA-4, TA-5, TA-6, TA-7, TA-8, TA-9, TA-21, TA-23, TA-24, TA-25, UK-7, UK-8, UK-12A and UK-18A1 (Yüksel Proje, 2003). A total of 17 boreholes (Appendix C) which are unique according to following criteria:

- located at certain topographical elevations
- variable depths to groundwater table

- variable thickness of the compressible layers
- variable composition of the alluvial deposits
- uneven lowering of the groundwater table
- variable overburden thickness above the tunnel
- availability and/or lack of in-situ tests and samples for laboratory analysis

All of these criteria mentioned above provided the study with the versatility of input data. This way, the method of settlement estimation had to be defined and adjusted separately for every other case which, obviously, proposed a fine challenge to incorporate accurate results.

4.1.2. Measuring the Depth to Groundwater Table

As mentioned earlier, monitoring of the depth to groundwater is a routine check within the coverage of project works. The PVC pipes installed upon the completion of boreholes make it possible to keep track of groundwater level fluctuations on a regular basis. Depth to groundwater table is measured within every geotechnical borehole.

4.1.3. Defining the Tunnel Depth

The thickness of overburden changes at every other location depending on topography, project requirements (gradient of the tunnel) and the type of tunnel cross-section. The geological cross-section (Appendix A) that is prepared together with the compiled project report (Yüksel Proje, 2003) presents the topographical elevation at the surface and the elevation of top of rail that is to be installed inside the tunnel. The depth of the tunnel is revealed by simply the subtraction of these two values from one another. The groundwater table depth will be assumed to drop down to invert level as the drainage process continues.

4.1.4. Determining Unit Weight Values of Soil

Natural unit weight and specific gravity values were available in the geotechnical reports (Yüksel Proje, 2005). In addition to the natural unit weight, the value of saturated unit weight is also necessary during the course of analysis. Saturated unit weight can be calculated effortlessly if initial void ratio (e_o) and specific gravity (G_s) values of the material are known:

$$\gamma_{sat} = \left(\frac{G_s + e_0}{1 + e_0}\right) \gamma_{water}$$
(4.1)

With the saturated and natural unit weight values for every undisturbed sample it is possible to perform effective stress analysis.

4.1.5. Calculation of Pre-consolidation Pressure

Pre-consolidation pressure can be obtained from $e - \log p$ curve by use of a simple graphical procedure (Casagrande, 1936). Firstly, the point of inflection where the sharpest curvature is achieved on $e - \log p$ curve is found. This point is shown on Figure 4.1 as point O. Next, two lines passing through this point are plotted, one being a tangent to $e - \log p$ p curve (line OB in Figure 4.1) and the other being a horizontal line (line OA in Figure 4.1). Then, the straight line portion of the $e - \log p$ curve is produced. The point where these two lines intersect with the bisector of the two lines on point O shows the value of pre-consolidation pressure. This point is displayed as point D on Figure 4.1.



Figure 4.1 Calculation of pre-consolidation pressure from e – log p curve (Das, 2002)

This procedure was applied to the entire consolidation test results received from the 17 boreholes that are used in the analysis. The values of pre-consolidation pressure created an opinion whether the soil at respective boreholes is normally consolidated or overconsolidated.

4.1.6. Calculating the Compression Index

Another important value derived from the $e - \log p$ curve is the compression index (C_c). It is the slope of the linear portion of the loading curve on $e - \log p$ plot. Therefore, according to Figure 4.1 the following expression is obtained.

$$C_{c} = \frac{e_{1} - e_{2}}{\log p_{2} - \log p_{1}} = \frac{e_{1} - e_{2}}{\log \left(\frac{p_{2}}{p_{1}}\right)}$$
(4.2)

4.1.7. Calculating the Swelling Index

Similar to that of compression index, the swelling index (c_s) is also found using $e - \log p$ plot. The attention this time is diverted to the unloading part of the $e - \log p$ curve (Figure 4.1) and it is again the slope that gives the value of swelling index:

$$C_{s} = \frac{e_{3} - e_{4}}{\log p_{4} - \log p_{3}} = \frac{e_{3} - e_{4}}{\log \left(\frac{p_{4}}{p_{3}}\right)}$$
(4.3)

The final overall form of the database that is created to be used in settlement calculations is tabulated below.

	Ро	eo	Yn	γs	Cc	Cs	OCR
TA-1 UD-1	115	1.24	16.60	17.08	0.270	0.0660	1.00
TA-1 UD-2	110	0.95	17.50	17.76	0.200	0.0500	1.00
TA-2 UD-1	110	0,87	18.30	18.78	0.200	0.0330	1.00
TA-3 UD-1	110	0.94	18.10	18.51	0.200	0.0170	1.00
TA-4 UD-1	120	1.51	15.60	16.18	0.400	0.0830	1.33
TA-4 UD-2	100	1.01	17.10	17.57	0.270	0.0330	1.00
TA-4 UD-3	150	0.82	18.10	18.60	0.166	0.0330	1.00
TA-5 UD-1	110	1.37	16.20	16.89	0.370	0.0660	1.22
TA-5 UD-2	120	0.90	18.00	18.43	0.200	0.0330	1.10
TA-6 UD-1	170	0.97	17.60	18.13	0.230	0.0500	1.79
TA-6 UD-2	160	1.01	17.70	18.11	0.230	0.0500	1.14
TA-7 UD-1	110	0.92	17.90	18.50	0.133	0.0330	1.16
TA-7 UD-2	100	0.99	17.70	18.24	0.166	0.0330	1.00
TA-8 UD-1	120	1.15	16.80	17.48	0.200	0.0500	1.71
TA-9 UD-1	190	1.27	15.90	16.64	0.166	0.0330	1.90
TA-21 UD-1	200	0.76	18.10	18.67	0.133	0.0330	1.18
TA-21 UD-2	125	1.07	17.30	17.82	0.166	0.0330	1.00
TA-23 UD-1	90	1.32	16.20	16.79	0.330	0.0660	1.00
TA-23 UD-2	125	0.99	17.38	17.99	0.200	0.0500	1.00
TA-24 UD-1	95	1.51	15.70	16.34	0.430	0.0660	1.06
TA-24 UD-2	160	1.17	17.40	18.22	0.166	0.0660	1.28
TA-24 UD-3	145	0.90	17.60	18.12	0.133	0.0330	1.00
TA-25 UD-1	105	1.34	16.50	17.06	0.330	0.0660	1.00
TA-25 UD-2	160	1.15	17.30	17.70	0.200	0.0660	1.30
UK-7 UD-1	200	0.96	18.10	18.17	0.270	0.0330	1.05
UK-8 UD-1	140	0.85	18.60	18.45	0.166	0.0660	1.00
UK-12A UD-1	200	0.82	17.80	18.27	0.100	0.0166	2.67
UK-18A1 UD-1	155	1.05	17.20	17.71	0.200	0.0500	1.00

 Table 4.1 Database generated for settlement calculation

4.2. Calculation of Consolidation Settlement

Following the tabulation of the source data the situation is now available for the calculation of consolidation settlements. However, a few more adjustments in relation to the systematic of calculations are essential for the benefit of the case. Utilizing the compressible layer for calculations, considering the context of the material, adapting the input from in-situ and laboratory tests, choosing the correct analytical approach and integrating the necessary parameters are parts of the problem which are dealt with as explained in this section.

4.2.1. Division of Compressible Layer into Sub-layers

First of all, borehole logs were elaborated and prepared for the analysis. Due to the requirements and the sake of the study, the compressible layer is the portion of the alluvial deposits submerged in groundwater. Therefore, it extends from the top level of the saturated alluvial deposits, to until either the bottom of the tunnel cross-section or the bottom level of the alluvial deposits whichever is reached first. This complete layer will not be available for the analysis as a whole and must be divided into sub-layers. The reason is that, at greater depths through the alluvial clay layer the tendency to induce settlement increases rapidly in a logarithmic fashion. Increasing stress conditions have an important role in this together with groundwater conditions and characteristics of the soil. The layer itself may also go through some structural changes that will provoke (collapsibility, high plasticity, fissures, etc.) or inhibit (cementation, carbonated zones, high stiffness, etc.) occurrence of settlement. Hence the layer that is expected to be under the influence of consolidation is divided into sub-layers of approximately 1.5-2.0 meter thickness. This way the layer in consideration will be represented more precisely by means of corresponding geological properties and stress conditions attained separately to each sub-layer. The parameters recovered from in-situ and laboratory tests are also assigned to the sublayers at those borehole depths where the test was conducted or the sample was taken.

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4.2.2. Settlement Calculations at Sand-Gravel Deposits

Alluvial deposits found at the project area often contain cohesionless material that shows only a small amount of long-term settlement. Although clay deposits dominate the unit in general, it would be unwise to exclude levels of sand and gravel from the settlement calculation process.

The performance of such deposits in the consolidation test cannot be trusted and using results from such applications will cause a serious flaw affecting the reliability of the study. Instead the in-situ tests are quite valuable to provide the data needed for analysis.

In this study, the layers of alluvial deposits that contain sand and gravel were included in the analysis stage by use of the Standard Penetration Test (SPT) results. This type of in-situ testing is a practical, economical and a very common technique of sub-surface exploration that has been applied in the project whenever possible. The N values resulted from SPT tests give idea about other properties of the soil as well.

Since these sorts of deposits are not practical for an attempt to produce a decent $e - \log p$ curve, the values for C_c and C_s are not available. In this case the settlement equations based on these two coefficients are not suitable. Instead, the utilized equation is the one that links settlement to the coefficient of volume compressibility which could be defined by using SPT values. For this purpose, a computer program converting SPT values into many other soil parameters was used. The program is called S.P.T. correlations v.1.2, a simple application to use (Figure 4.2).

S.P.T. corre	elations v.1.2			- ×
Field SPT value - SPT: 14				
	Test depth	7.50 (m) G.W.T. 6.80 (m)	q 100.00	(kPa) B 1.50 (m)
N60 - N1,60	Dr of sands	Friction angle Es Su for cohesive V	s Settlements on sand	s 🛛 Bearing capacity i 🔇 🔪
Simplified ca	lculation	Reference	N60 N1,60	
Ce	0.75	Peck & Bazaraa (1969) : 0.86	10.5 9.0	
		Peck et al. (1974) : 0.89	9.3	
		Tokimatsu & Yoshimi (1983) : 0.80	8.4	
Cr	1.00	Liao & Whitman (1986) : 0.84	8.8	
ch 🗌	1.00	Skempton (1986) :	0.7	
	1.00	Fine sands of medium Dr : 0.83	8.7	
Cs	1.00	Overconsol, fine sands 0.80	9.2	
Ca	0.90	$0.4 \le C_D \le 1.7 (1996 \otimes 1998 \text{ NCEER})$		
Cbf	0.95			
Cc	1.00			

Figure 4.2 S.P.T. correlations program

During the course of the analysis as the sand-gravel layers are encountered, raw N values observed at the field are corrected into $N_{1,60}$. Three types of corrections are applied to these N values. The first one is the energy correction, C_e. Since it is a procedure applied worldwide, this correction is needed to standardize every different application. In Turkey a donut hammer released by a pulley mechanism of two turns on cathead is usually used and therefore the hammer efficiency (E_m) value is equal to 0.45. To get the corrected value of N_{1,60}:

$$N_{1,60} = N_{field} \, \frac{E_m}{0.6} \tag{4.4}$$

All that is needed in the program is to input the energy correction value which is the ratio of hammer efficiency to 0.6 and that is 0.75 in this case (Figure 4.2). The other correction which is the overburden correction (Liao and Whitman, 1986) is automatically applied by the program when testing depth and depth to groundwater table is entered. Further corrections could be manually inserted such as rod length factor C_R , borehole diameter factor C_B , sampling method factor C_S and others. After all of these corrections are executed, $N_{1,60}$ value is available depending on the preferred method.

Every N_{field} values found in the alluvial sand-gravel layers are converted into N_{1,60} values as explained above and the value proposed according to appropriate methodology is taken into account. Then, the average of these N_{1,60} values are re-entered into the program to get the corresponding value of deformation modulus, E_s. It is critical to pick the correct methodology that will propose a more reasonable value with respect to the material composition. In the course of the analysis, deformation modulus of artificial fill was found by the method of Bowles (1996) for gravelly sand. For sandy soil preferred method for determining E_s value was Tan *et al.* (1991) for clayey sand (Figure 4.3).

¥ S.P.T. correlations v.1.2 − ×					
501:9					
					-
Test depth 7.50 (m) G.W.T. 6.80 (m) q 100.00 (kPa) B 1.50 (m)					
N60 - N1,60 Dr of sands	Friction angle Es	Su for cohesive	Vs Settlements o	n sands 🔰 Bearing capacity i 🔇	
Soil type	Reference		Es (kPa) - (Nspt)	Es (kPa) - (N60)	
Normally consolidated	Tan et al. (1991)	1	12000	10900	
sands		2	32958	28754	
	from Bowles (1996)		21000	18254	
			54000	40800	
	Japanese Design Stand	ards	23400	17680	
Saturated sands	from Bowles (1996)		6000	5450	
Overconsolidated sands	D' Appolonia et al. (197	0)	49450	47140	
	Tan et al. (1991)		24750	23100	
O.C.R. 2.00		* based on 1	16971	15415	
		* based on 2	46610	40664	
Gravelly sand	Tan et al. (1991)		9000	7680	
	from Bowles (1996)		18000	15360	
Clayey sand	Tan et al. (1991)		7680	6976	
Silty sand	Tan et al. (1991)		4500	3840	
				🛛 🔀 Clos	е

Figure 4.3 Determining E_s value by S.P.T. correlations program

The deformation modulus is an important parameter because when inverted, it provides the coefficient of volume compressibility ($m_v = 1 / E_s$) which is a direct component of the settlement equation (3.3) mentioned earlier. Deformation modulus retrieved from the program (E_s) is assumed to be equal to E_{oed} , the modulus of deformation derived from oedometer test. Multiplying $1/E_s$ with layer thickness H and change in effective stress between initial and final conditions $\Delta \sigma$ will give the amount of settlement that will be experienced in that particular layer.

4.2.3. Settlement Calculations at Silt-Clay Deposits

Alluvial deposits found at the project area are mostly composed of cohesive silty-clay matrix. Undisturbed sampling in this type of soil is a very convenient method of getting useful and uncorrupt information about the compressibility of the material. Further investigation carried out at soil mechanics laboratories often provide vital information like Atterberg limits, swelling potential, cohesion level and much more.

In this case the attention is on $e - \log p$ graphs which are plotted according to the results of oedometer tests conducted on undisturbed samples. As stated before, the $e - \log p$ graphs hold very important information about settlement process. The indices of compression and swelling summarize the consolidation behavior of the soil under investigation. In the analysis, these indices are utilized together with preconsolidation pressure and settlement calculation is completed by the designated equation that is appropriate for the conditions.

For such deposits the first step is to determine the properties of soil. Natural and saturated unit weight values (γ_n , γ_s), pre-consolidation pressure (p_c), initial void ratio (e_0) and indices of compression and swelling (C_c , C_s) are all noted in the database. These values are assigned to each sub-layer and are driven from the tests on closest undisturbed sample within the borehole. First step was to determine the initial stress condition at the midpoint of each sub-layer. Comparing this to preconsolidation pressure revealed the ratio of consolidation. There were cases where initial effective stress was greater than pre-consolidation pressure but since this is not possible these two values were assumed to be equal and the layer was assumed to be normally consolidated. This situation will be emphasized on upcoming discussions section. The next step is to calculate the effective stress (using γ_n) after groundwater is

drained completely to the tunnel level. Considering these two effective stress values one of the three equations is picked and the amount of settlement is estimated by the help of either C_c , C_s or both.

4.3. Assumptions

A number of assumptions were adopted for improving the analysis by means of practicality, flexibility and precision. These are listed below:

- Settlements that occur due to ground loss are not included in the analysis. Any other reason than groundwater drainage is excluded while calculating settlement.
- The change in effective stress during the process of consolidation and its effects are ignored. Only initial and final effective stresses were taken into consideration.
- In accordance with the initial project the tunnel construction method is assumed to be cut-and-cover. Tunneling methods that inhibit water drainage into the tunnel excavation (such as TBM) are not considered even if they are to be selected with a modification in the project.
- The groundwater drainage is predefined as a sudden incident. The study is completely independent of time since the main objective is to find the ultimate amount of settlements that the project area is potentially exposed to under special circumstances. Therefore, permeability and time parameters are not applied in this study.
- The relationship between the groundwater drainage and the consolidation process is ignored. In reality the consolidation occurs during the groundwater drainage but in this study the drainage is assumed to occur first and only then, the

consolidation is to take place with new conditions under affect.

- The drop in groundwater table is assumed to fall down to the invert level. In actual case, this drop may be interrupted due to some lithological occurrences or decrease in rate of groundwater drainage.
- The groundwater table is considered to stay horizontal during the lowering and even after the drainage is complete. It is an assumption made for not complicating the analysis.
- The settlements were estimated with respect to the tunnel axis only. Settlements away from the axis were not taken into account.
- Possible deformations on the tunnel surface and uneven settlements are assumed not to happen.
- Once the sub-layers are defined, they are all considered to be homogeneous elements on their own.
- Very low values of pre-consolidation pressure are ignored and the corresponding material is assumed to be normally consolidated.
- Investigation works conducted in the field and the soil mechanics laboratory are assumed to be by the standards with minimum error possible.

4.4. Numerical Analysis

When analyzing the settlement due to groundwater drainage at the project area analytical concepts were primarily utilized. This process is possibly under the effect of numerous errors that started to accumulate from the time the project works began. The systematic of analysis brought the need for specific assumptions that also set some limitations on the concluding results. Even though the analytical approaches that were used in this study are trustworthy methods with long history of application, their results should be confirmed by another approach that will use different criteria and a different path to get the results out of the identical conditions simulated in a numerical medium.

For this purpose, numerical models of several sections located at the project area were constructed by Plaxis. In this section the operating principles, preparation of the model, considered facts and the calculation process will be explained.

4.4.1. General Definition for Plaxis Program

Development of PLAXIS began in 1987 at the Technical University of Delft as an initiative of the Dutch Department of Public Works and Water Management. The initial brief was to develop a simple finite element code for the analysis of river embankments on the soft soils of the lowlands of Holland. In subsequent year, Plaxis was extended to cover most other areas of geotechnical engineering. Because of continuously growing activities, a company named PLAXIS BV was formed in 1993.

PLAXIS v. 7.2 (1998) is a finite element program for plane strain and axisymmetric modeling of soil and rock behavior. Plaxis has a fully automatic mesh generation, allowing for a virtually infinite number of 6node and 15-node elements, based on graphical input of soil layers. The models can contain both drained and undrained layers. For undrained layers, excess pore pressures are calculated and elasto-plastic consolidation analysis may be carried out. Large deformations may be analyzed by means of an updated mesh (Lagrangian) calculation. Using this option, the finite element mesh is continuously updated during the calculation. For some situations, a conventional small strain analysis may show a significant change of geometry. In these situations, it is advisable to perform a more accurate Updated Lagrangian calculation (PLAXIS, 1998).

Main goal of Plaxis is to fulfill its intention to provide a practical analysis tool for use by geotechnical engineers who are not necessarily numerical specialists. Most of the time practical engineers find non-linear finite element computations cumbersome and too time-consuming for regular analyses. This issue was addressed through intense research and development and theoretically sound computational procedures encapsulated in a logical and easy-to-use shell is designed. As a result, Plaxis came to use as a world-wide numerical code in practical applications.

Plaxis is the finite element package specifically intended for the analysis of deformation and stability in geotechnical engineering projects. Geotechnical applications require advanced constitutive models for the simulation of the non-linear and time-dependent behavior of soils. In addition, since soil is multi-phase material, special procedures are required to deal with numerous cases arise in the complicated nature of the soil.

Although the modeling of the soil itself is an important issue, many geotechnical engineering projects involve the modeling of structures and the interaction between the structures and the soil. Plaxis is equipped with exclusive features to deal with the numerous complexities of geotechnical structures. A brief summary of the important features of the program is given below.

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4.4.1.1. Graphical Input of Geometry Models

The input of soil layers, structures, construction stages, loads and boundary conditions are based on convenient drawing procedures (CAD), which allows a detailed and accurate modeling of real situations to be achieved. From this geometry model a finite element mesh is automatically generated.

4.4.1.2. Automatic Mesh Generation

Plaxis allows for fully automated generation of unstructured finite element meshes with options for global and local mesh refinement. The mesh generator is a special version of the Triangle generator.

4.4.1.3. High-order Elements

High order elements are available to enable a smooth distribution of stresses in the soil and an accurate prediction of failure loads. In addition to the quadratic 6-node triangular elements 15-node cubic strain triangles are available (and preferred for this study) which perform extremely well in axisymmetric analyses.

4.4.1.4. Beams

Special beam elements are used to model the bending of retaining walls, tunnel linings and other slender structures. The behavior of these elements is defined using flexural rigidity, a normal stiffness and an ultimate bending moment. A plastic hinge may develop for elastoplastic beams, as soon as the ultimate moment is mobilized. Beams may be used together with interfaces to perform highly realistic analyses of a large range of geotechnical structures.

4.4.1.5. Interfaces

These joint elements are needed for calculations involving soilstructure interaction. They may be used to simulate the thin zone of intensely shearing material at the contact of footings, piles, geotextiles, retaining walls, etc. Values of interface friction angle and adhesion that are not necessarily the same as the friction angle and cohesion of the surrounding soil, may be assigned to these elements.

4.4.1.6. Tunnels

Plaxis offers a convenient option to create circular and non-circular tunnels composed of arcs. Beams and interfaces may be added to model the tunnel lining and the interaction with the surrounding soil. Fully isoparametric elements are used to model the curved boundaries within the mesh. Different practical methods are implemented to analyze the deformations that occur due to the construction of the tunnel.

4.4.1.7. Mohr-Coulomb Model

This robust and basic non-linear model is based on soil parameters that are known in most practical situations but not all nonlinear features of soil behavior are included. The Mohr-Coulomb model may be used to compute realistic ultimate loads for circular footings, short piles, etc. It may also be used to calculate a safety factor using a "phi-c reduction" approach.

4.4.1.8. Advanced Soil Models

Plaxis offers a variety of soil models in addition to the Mohr-Coulomb model. To analyze accurately the logarithmic compression

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behavior of normally consolidated soft soils, a Cam-Clay type model is available. This is called the Soft Soil Model, the model which was utilized in this study. For stiffer soils, such as overconsolidated clays and sand, an elastoplastic type of hyperbolic model is available and it is the Hardening Soil Model.

4.4.1.9. Steady State Pore Pressure

Two alternative approaches exist for the generation of steady state pore pressures. Complex pore pressure distributions may be generated on the basis of a two-dimensional groundwater flow analysis. As an alternative for simpler situations, multi-linear pore pressure distributions can be directly generated on the basis of phreatic lines.

4.4.1.10. Excess Pore Pressures

Plaxis is equipped with the ability to distinguish between drained and undrained soils to model permeable sands as well as almost impermeable clays. Excess pore pressures are computed during plastic calculations when undrained soil layers are subjected to loads. Undrained loading situations are often decisive for the stability of geotechnical structures. In cases of insufficient stability, intermediate consolidation periods have to be introduced to reduce the excess pore pressures.

4.4.1.11. Automatic Load Stepping

Plaxis can be run in an automatic step-size and automatic time step selection mode. This avoids the need for users to select suitable load increments for plastic calculations by themselves and it guarantees an efficient calculation systems.

4.4.1.12. Staged Construction

It is possible to simulate construction and excavation processes by activating and deactivating clusters of elements. This procedure allows for a realistic assessment of stresses and displacements.

4.4.1.13. Updated Lagrangian Analysis

Using this option makes the finite element mesh to update continuously during the calculation. For some situations, a conventional small strain analysis may show a significant change of geometry. In these kinds of cases it is advised to perform more accurate Updated Lagrangian calculation, which is called an Updated Mesh analysis in Plaxis.

4.4.1.14. Consolidation

Although this feature was not accessed in the study, the decay of excess pore pressures with time can be computed in a consolidation analysis. A consolidation analysis requires the input of permeability coefficients in the various soil layers. Automatic time stepping procedures make the analysis precise and easy-to-use.

4.4.2. The Soft-Soil Model

The soil model is picked with respect to the geological aspects of the project section that is being concentrated on. In this study the soil model established is the soft-soil model. Some properties of Soft-Soil model are:

- Stress dependent stiffness (logarithmic compression behavior)
- Distinction between primary loading and unloadingreloading
- Memory for pre-consolidation stress
- Failure behavior according to the Mohr Coulomb criterion

In the Soft-Soil model, a logarithmic relation between the volumetric strain, ε_v , and the mean effective stress, p['], is assumed. For failure behavior modeling purposes, a linearly elastic – perfectly plastic Mohr Coulomb type yield function is introduced.

4.4.2.1. Parameters in the Soft-Soil Model

A number of parameters are needed to be defined so that the process could be initiated. These are:

a. Modified compression index, λ^* , determines the compressibility of the material in primary loading. It is in relation with normalized parameters by parameters below:

$$\lambda^* = \frac{C_c}{2.3(1+e)}$$
(4.5)

b. Modified swelling index, K^{*}, determines the compressibility of the material in unloading and subsequent reloading. Using Poisson's ratio for unloading / reloading, v_{ur}, it can be defined by:

$$\kappa^* \approx 1.3 \frac{1 - v_{ur}}{1 + v_{wr}} \frac{C_s}{1 + e}$$
 (4.6)

Both of these two parameters can be obtained from an isotropic compression test with unloading. Slope of the primary loading line gives the modified compression index, whereas the slope of the unloading (or swelling) line gives the modified swelling index (Figure 4.4).



Figure 4.4 Logarithmic relation between volumetric strain-mean stress

- c. Cohesion, c, it has the dimension of stress. Entering a cohesion value will result in an elastic region that is partly located in the 'tension' zone of the stress space. Input of large cohesion value will simulate state of overconsolidation
- d. Friction angle, ϕ , specified in degrees and represents the increase of shear strength with effective stress level
- e. Dilatancy angle, ψ , for the type of materials that are analyzed by Soft-Soil model, dilatancy can generally be neglected

All these parameters are registered in the Soft-Soil model to set up Plaxis for specified analysis (Figure 4.5).

Soft soil model - Clay			×
General Parameters Interfaces			
Stiffness λ* (lambda*) : 0.100 κ* (kappa*) : 0.020	Strength ^C ref [∶] ¢ (phi) : ∳ (psi) :	1.000 27.000 0.000	kN/m ²
Next	<u>k</u>	<u>C</u> ancel	Advanced

Figure 4.5 Parameters tab for the Soft-Soil model

4.4.3. Selecting the Locations for Analyses

A total of four sections were selected to be analyzed in Plaxis. These sections were picked from locations where, depending on calculations discussed in previous chapter, maximum and minimum amount of consolidation settlements are expected and two other randomly selected locations. The locations chosen for section preparation are at boreholes TA-5 (for maximum estimated settlement), TA-9 (for minimum estimated settlement), TA-3 and TA-23 (both randomly selected). At these locations Plaxis is used to point out whether the calculations completed earlier are realistic enough.

4.4.4. Preparation of the Sections

Every section is geometrically identified to the program by a CAD. The geometry of mesh is defined in accordance with Mestat (1997), extending the lateral boundaries of each section six folds of the tunnel diameter and the bottom of the section as deep as five times the diameter of the tunnel.



Figure 4.6 Sketch of the section geometry

Geological units are defined lithologically in the same sequence at the exact depths with the borehole log and corresponding soil properties are noted. The information includes cohesion, internal friction angle, deformation modulus, overconsolidation ratio, modified indices of compression and swelling. Any structures that are constructed on ground surface are means of surcharge load and should be found in the prepared section. In the case of four locations that were picked for analysis, such constructions do not exist. This can be observed from the plan view of the project area in Figure 4.7 or at the actual site.



Figure 4.7 Plan view of section locations

The boundary conditions specified for these sections play an important role in the way the Plaxis processes. Nodes located at the bottom boundary of the section geometry are fixed in both x (horizontal) and y (vertical) directions. Nodes found at the side boundaries of the model sections are fixed in x direction and free in y direction to enable simulation of deformations. As far as the groundwater flow is concerned the nodes at the bottom border of geometry are closed to groundwater flow. Groundwater pressures due to total head are assigned to the nodes at the side boundaries of the model. This way, the position of the groundwater table is determined. Drain boundary condition is assigned to the nodes located at the tunnel opening. This allows the natural discharge of the groundwater into the tunnel excavation the same way as it happens in the actual case.

The selected borehole locations of TA-3, TA-23, TA-9 and TA-5 were represented by sections 1-to-4. All of the four sections prepared in Plaxis are displayed in Figures 4.8, 4.9, 4.10, and 4.11. The related properties mentioned before are also provided within the coverage of these sections. Table 4.2 shows number of elements and nodes for the numerical models of each section.

	Section-1 (TA-3)	Section-2 (TA-23)	Section-3 (TA-9)	Section-4 (TA-5)
Number of elements	1943	1814	2295	2413
Number of nodes	15791	21768	18727	19555

 Table 4.2 Information regarding numerical models of each section








4.4.5. Defining Stages of Analysis

Sections that are identified into the system are ready for analysis that should be designed as separate stages for Plaxis to process. The analyses take place in four stages:

- a. Stage-1: At this first stage in-situ effective stress generation takes place according to the initial site conditions and the parameters notified in the section.
- b. Stage-2: Tunnel excavation is implemented with reduced overburden load. In order to simulate the effect of tunnel face on the deformations that occur at the unsupported section of the tunnel, ∑M-Stage parameter was selected as 0.5 in value. The displacements that take place until the installation of support measures are taken into consideration with this reduction in the overburden pressure. At this stage no support is yet applied at the tunnel cross-section.
- c. Stage-3: The tunnel cross-section is integrated with the lining. Properties of the lining are stated in order to simulate the actual support systems. ∑M-Stage is set to 1 so that the overburden pressure is applied to its full extent. Regarding deformations are calculated. All of the displacements that are calculated until the end of this stage are the displacements that occurred due to tunneling.
- d. Stage-4: Since the study aims to determine the amount of deformations due to only groundwater drainage, the displacements that were calculated at previous stages are ignored and set to zero at the beginning of this stage. In this stage the lining that is

permeable allows groundwater to drain into the tunnel and the drained water is pumped out. From this point on, the stage mechanism works in two divisions, because Plaxis elaborates the consolidation and the drainage as independent (i.e. uncoupled) processes.

- i. The position of the groundwater table is estimated by taking a constant level of permeability for all soil materials found within the section. An overall approximate phreatic line representing the steady state groundwater level is generated with respect to the groundwater calculations.
- Upon completion of drainage the increase in effective stress and the resulting settlements formed in the final case are calculated.

Plaxis goes through specified computations following each phase and the final situation is displayed on screen so that the unique effects of that stage are clearly visible. Mesh geometry, boundary conditions, settlement of the ground surface, deformations at tunnel surface, position of groundwater table and groundwater flow tendencies can be all observed by the illustrations of Plaxis. Such concluding displays for each section and the numerical results will be presented next chapter.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1. Results of Analytical Approach

Consolidation settlement calculations done according to the analytical formulas suggested in the literature were quite straightforward once the site conditions were adapted to a basic systematic. The estimated settlements for every other borehole location are ranging from a few millimeters up to 20 centimeters. The arithmetical average of these estimations point out to 10 centimeters of average settlement. Amount of estimated settlements are available at Table 5.1. These values are also displayed in a histogram for better understanding of settlement distribution (Figure 5.1).

Borehole	Settlement (cm)
TA-1	13,13
TA-2	7,68
TA-3	11,47
TA-4	17,62
TA-5	21,34
TA-6	13,61
TA-7	11,16
TA-8	0,71
TA-9	0,124
TA-21	5,53
TA-23	19,21
TA-24	12,26
TA-25	5,85
UK-7	8,01
UK-8	10,99
UK-12A	3,29
UK-18A1	6,37

Table 5.1 Settlements estimated by analytical approach



Figure 5.1 The histogram showing settlement estimations

5.2. Results of Numerical Analysis

Plaxis produces graphical display of the results on the model section that was setup for analysis. These graphs visualize the steadystate groundwater level with associated pore pressure contours after the drainage of groundwater into the tunnel as well as the total displacement contours due to the process of consolidation. Extreme active pore pressures are: -443 kN/m² for Section-1, -532 kN/m² for Section-2, -634 kN/m² for Section-3 and -451 kN/m² for Section-4. Negative values indicate pressure. The amounts of maximum settlement are: 9.46 cm for Section-1, 22.31 cm for Section-2, 1.63 cm for Section-3 and 20.96 cm for Section-4. The resulting illustrations of four separately prepared sections are given in Figures 5.2 to 5.9.























Figure 5.8 Final position of groundwater table and pore pressure levels for Section-4



5.3. Discussions

The analysis portion of the study was under the effect of many complex factors. Defining the actual site conditions through limited quantity of investigation works was quite a challenge. To be able to run the analysis smoothly, one has to be greatly familiar with the related concepts.

In the analytical phase of the study too many restricting assumptions were adapted. The computations did not involve as many parameters as the numerical analysis where a lot more aspects of the consolidation process were considered. The results proposed by analytical methods show maximum settlement occurrence at the location of borehole TA-5. The numerical analysis, on the other hand, highlighted borehole TA-23 as the location with the settlement risk of highest magnitude.

As seen in Table 5.1, the settlement values predicted by analytical calculations vary from a few centimeters to more than 20 centimeters. Settlements of around 10-15 centimeters were observed at TA-1, TA-3, TA-6, TA-7, TA-24 and UK-8 locations mostly because the compressible layer is fairly thick and the groundwater table decline is allowed since the tunnel is constructed at a moderate depth. At TA-24, the layer which will settle is very thick but mostly composed of cohesionless soil with low settlement potential. At locations of TA-8 and TA-9 level of ground subsidence is almost zero because tunnel is at a shallower depth and hence the compressible layer is very thin. At TA-4, TA-5 and TA-23 settlements are over 15 centimeters because of the deeply seated tunnel and a thick, compressible layer consisting of soft clay soil. Approximately 5-to-10 centimeters of settlement is expected near UK-7, UK-12A, UK-18A1, TA-2, TA-21 and TA-25 locations because the layer which will

experience compression is mostly made up of cohesionless soil material that is reluctant for consolidation settlements. Especially at TA-2 the artificial fill layer which is incapable of consolidation is very thick and therefore the settlements are low.

The only odd thing about the results from numerical model was seen at the settlement contours of Section-3. Despite the fact that literature of tunnel engineering suggests maximum amount of ground deformations at the axis of the tunnel, in Section-3 this is not the case. The contours in Figure 5.7 must have appeared similar to those settlement graphs of other sections. Only reasonable explanation for this is that the thickness of compressible layer that was drained is smaller at the axis of the tunnel than at the sides. Therefore, a different sort of resulting graph was produced as Plaxis processed the case with respect to groundwater drainage only. This difference could also be explained by the fact that the tunnel in this section is seated at a shallow depth and therefore, the structure itself, could have provided a certain support to the overlying soil material.

It should be noted that the analyses from both approaches were unique in their own and were not meant to yield exact matching results of consolidation settlements. In accordance, the estimated settlement results of the analyses from the numerical approach were consistent with those from the previous phase with just 2-to-3 centimeters of difference (Table 5.2). The similarity of these results could be explained by the certainty of field parameters and use of the same database for analyses.

	Settlemen	ts
	Analytical Calculations (cm)	Numerical Model (cm)
TA-3	11.47	9.46
TA-5	21.34	20.96
TA-9	0.12	1.63
TA-23	19.21	22.31

 Table 5.2 Comparison of analytical and numerical results

The analytical approach is concerned with the initial effective stress conditions and the increase in effective stress caused by groundwater drainage to determine the degree of volumetric compression. The changes in effective stress that happen during the consolidation process are not involved. On the other hand, the numerical method considers the stress changes caused by tunnel excavation together with the effective stress discrepancies due to groundwater extrusion in a realistic manner. The numerical model involves the shear deformations in the analyses as well as the volumetric strains.

The estimated amount of settlements in this study may not conform with those that are to be observed in the actual field. The suggested ground displacements are the ultimate values and to achieve these values all the assumptions of the study must take place in the real case. Since most of these assumptions, that were used to bring practicality into the study, have no chance of occurrence, the actual measurements are (most probably) going to be less than the estimated values. The recent modifications made in the project already took effect. Hence, the expected groundwater drainage and estimated settlements will not be permitted.

The types of tunnel cross-sections will also have an important effect on the intention and magnitude of the settlements. The turnout tunnel cross-section requires the biggest excavation area where two subway lines are intersected and therefore it is the most critical crosssection. Station platform tunnel cross-sections also carry a huge importance with second largest diameter. Main-line (base) tunnels have great potential of causing settlements at increasing depths. Connection tunnels are smaller but still critical because they create tunnel crossings. Staircase inclined tunnel and approach tunnel cross-sections are the sections where risk of settlements is least since they are located at fairly shallower depths. Settling will also have a great tendency to happen at regions where geological situation gets complicated.

This study is prepared in accordance with the original project. If any modifications are to be deployed in the initial project, than the study should be taken as an independent case of its own. Creating a false opinion about the project is not among the intentions of the study since any problem such project would possibly face could be handled with an effective engineering practice.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Estimation of consolidation settlements is a crucial part of almost every subway project. The process involves many complexities such as achieving precision in representation of the site conditions and the difficulty in adapting a high-performance model for the analyses. In this particular study about the Ulus – Keçiören subway project the circumstances became especially challenging.

In the first phase of the study, main concepts of the consolidation settlement were covered by a thorough literature review. This survey study provided the vision and ability to evaluate the terms of the project and relate them to the widely recognized consolidation concept. Since the availability of noticeable levels of consolidation settlement was limited, the investigation works done at the project area had to be searched for locations with the potential to sustain settlement. The scarcity of such investigational works and the difficulty of acquiring them took a great deal of time and effort. After the determination of a region that possesses settlement treat, the area was assessed so that a specific system of analysis based on analytical methods could be carried out. These well-known methods were executed efficiently and settlement estimation was conducted. The results of estimated consolidation settlement due to groundwater drainage according to analytical calculations are: 13.13 cm for TA-1, 7.68 cm for TA-2, 11.47 cm for TA-3, 17.62 cm for TA-4, 21.34

cm for TA-5, 13.61 cm for TA-6, 11.16 cm for TA-7, 0.71 cm for TA-8, 0.12 cm for TA-9, 5.53 cm for TA-21, 19.21 cm for TA-23, 12.26 cm for TA-24, 5.85 cm for TA-25, 8.01 cm for UK-7, 10.99 cm for UK-8, 3.29 cm for UK-12A and 6.37 cm for UK-18A1. Consequently the results of these analytical methods suggested that the project area may confront remarkable amount of settlements due to groundwater drainage and precautions must be taken. The location where predicted settlements reached a serious level is near and around the Atatürk Cultural Center (A.K.M.) with almost 20 centimeters of ground subsidence.

Following the first phase of analyses a few locations (namely TA-3, TA-5, TA-9 and TA-23) from the same region were investigated once again with a detailed numerical model prepared by Plaxis. The settlement amounts predicted via the numerical model were: 9.46 cm for Section-1 (TA-3), 22.31 cm for Section-2 (TA-23), 1.63 cm for Section-3 (TA-9) and 20.96 cm for Section-4 (TA-5). Moreover, the results put forward by this approach once again suggested that the magnitude of settlements could endanger the course of the project if left unattended. Depending on all of the facts above, the project area must be carefully handled and the groundwater drainage must be controlled to prevent settlements.

The issue of ground deformations must be dealt with as the project works move on. Every section should be evaluated according to the tunnel cross-section to be excavated, the properties of soil material, the depth where excavation is done and the groundwater conditions. Extra care must be taken at places where a critical tunnel cross-section is to be excavated or the geological conditions get complicated.

The estimated values of consolidation settlements through analytical and numerical calculations need to be verified by careful monitoring of ground deformations during and after construction works. It must be emphasized that consolidation settlement is a very slow process and hence the monitoring period must be kept long. Even though the estimated values of settlement have low chance of occurrence, the project works must be carried out in caution.

Toward the end of this study some drastic changes have been made regarding the subway alignment as well as the method of excavation. The original alignment between Keçiören and Ulus has been changed as Keçiören –Tandoğan and the excavation is decided to be made by TBM. Along the new subway alignment Ankara Clay will be the dominant lithology. Adoption of TBM will also greatly eliminate the consolidation problems along the alluvial foundation crossings. This study, however, may be regarded as a case study for future subway tunnel constructions which may traverse through compressible and water saturated layers.

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APPENDICES





Figure A.1 Geological Cross-section of Subway Alignment



Figure A.1 Continuation of Geological Cross-section of Subway Alignment



Figure A.1 Continuation of Geological Cross-section of Subway Alignment



APPENDIX B

Table B.1 Coordinates, Elevation and Depth of Boreholes

Borehole	Depth	Y	X	Z		
	(m)					
TA-1	28.45	486 946.64	4 423 615.32	846.63		
TA-2	28.95	486 842.96	4 423 465.54	846.42		
TA-3	29.45	486 792.19	4 423 424.50	846.12		
TA-4	29.45	486 720.03	4 423 339.90	845.73		
TA-5	27.95	486 681.71	4 423 288.36	845.39		
TA-6	28.45	486 632.88	4 423 230.79	845.27		
TA-7	27.45	486 560.86	4 423 163.91	845.19		
TA-8	25.95	486 475.57	4 423 075.66	843.90		
TA-9	25.95	486 384.68	4 422 980.59	844.84		
TA-10	25.45	486 349.02	4 422 941.35	844.31		
TA-11	25.95	486 324.94	4 422 873.95	844.86		
TA-12	30.45	486 255.82	4 422 810.39	849.81		
TA-13	31.95	486 203.94	4 422 759.80	853.44		
TA-14	29.95	486 181.16	4 422 714.86	852.41		
TA-15	28.95	486 166.51	4 422 646.03	853.21		
TA-16	30.45	486 154.31	4 422 531.11	856.31		
TA-17	30.45	486 150.38	4 422 447.27	857.97		
TA-18	32.45	486 143.30	4 422 366.38	859.57		
TA-19	34.45	486 132.36	4 422 293.46	861.45		
TA-20	39.45	486 147.34	4 422 137.28	866.20		
TA-21	30.45	487 121.48	4 423 783.03	848.53		
TA-22	30.45	487 028.80	4 423 659.74	847.43		
TA-23	30.45	486 892.21	4 423 619.04	846.68		
TA-24	30.45	486 775.97	4 423 610.23	845.49		
TA-25	25.45	486 577.77	4 423 707.25	844.73		
UK-6	28.95	487 065.54	4 423 680.11	847.718		
UK-7	28.95	487 225.14	4 423 908.30	849.298		
UK-8	25.95	487 298.94	4 423 994.20	849.967		
UK-9	28.95	487 417.13	4 424 132.08	851.435		
UK-10	21.45	487 568.92	4 424 267.93	852.243		
UK-11	21.94	487 731.81	4 424 500.52	858.119		
UK-12	19.95	487 956.65	4 424 729.96	859.214		
UK-12A	27.45	487 951.62	4 424 794.22	859.59		
UK-13	19.92	488 022.82	4 424 845.69	857.741		
UK-14	19.95	488 065.82	4 424 901.67	856.654		
UK-15	21.95	488 135.00	4 425 125.50	858.744		
UK-15A	27.45	488 168.20	4 425 117.19	856.74		

Continuation of Table B.1

Borehole	Depth	Y	X	Z		
	(m)					
UK-15B	30.45	488 205.18	4 425 296.19	847.73		
UK-15C	25.95	488 240.98	4 425 420.61	847.43		
UK-16	27.45	488 226.17	4 425 508.53	849.4		
UK-16A	27.19	488 285.36	4 425 576.53	848.4		
UK-16B	30.00	488 316.10	4 425 686.24	848.88		
UK-17	30.00	488 388.17	4 425 845.01	850.312		
UK-18	30.00	488 363.53	4 425 927.76	850.6		
UK-18A	27.00	488 392.85	4 425 943.52	850.3		
UK-18A1	31.00	488 401.07	4 426 018.26	851.42		
UK-18B	30.00	488 432.85	4 426 079.11	850.71		
UK-18C	30.00	488 445.14	4 426 167.91	850.86		
UK-19	25.50	488 461.12	4 426 235.52	851.593		
UK-19A	32.20	488 444.36	4 426 321.41	852.56		
UK-20	28.50	488 496.06	4 426 465.05	852.957		
UK-20B	62.00	488 360.35	4 426 749.48	888.76		
UK-21A	61.00	488 345.18	4 426 978.31	896.12		
UK-21B	52.00	488 414.10	4 427 111.96	882.57		
UK-22A	39.50	488 567.83	4 427 220.63	870.66		
UK-22B	24.00	488 657.95	4 427 267.69	855.82		
UK-22C	28.50	488 699.41	4 427 267.93	855.76		
UK-22D	23.00	488 751.92	4 427 268.25	855.87		
UK-22E	23.00	488 807.17	4 427 280.02	856.01		
UK-23	19.50	488 854.51	4 427 315.19	856.2		
UK-23A	27.00	488 934.88	4 427 336.52	85659		
UK-24	21.00	489 035.09	4 427 357.74	857.847		
UK-24A	12.00	489 098.70	4 429 396.73	858.78		
UK-25	19.00	489 162.15	4 427 490.23	861.988		
UK-25A	15.00	489 246.58	4 427 571.42	865.75		
UK-26	17.00	489 381.78	4 427 814.10	877.186		
UK-27	24.00	489 381.78	4 427 900.51	880.737		

APPENDIX C

BOREHOLE LOGS

Source: Yüksel Proje Uluslararası A.Ş., Ankara (2005)

C.1. Borehole TA-1

۲	וטי	(5)	El	.	PF	20	DJE										
YÜ	KSEL PR	OJE ULU	SLAR/	ARASI 41	A.Ş.						SONDA						_
TE	L: (312) 4	95 70 00 P	FAX: (S	312) 49	5 70 24	1	SONDAJ LOG	U/BC	RING LOG		Borehoid		NO :		TA	-1	
w	www.yukselproje.com.tr										SAYFA Page	,	No :		1/	4	
PROJE	ADI / Pr	oject Na	ame			: U	LUS-KEÇİÖREN METRO HATTI	DELÍK	API / Hole Diameter	: H\	N (114	mm)				
SONDA	J YERI	/ Boring	Loca	tion		: A	SKI-TANDOGAN ARASI	YERAL	TI SUYU / Groundwater	: 6.	40 m. (2	1.04	.200	5)			_
KILOM	TRE/(Chainage / Resince	Pant			: 04	1905	MUH.BO	TAP / Start Einigh Da	to : 25	03 200	VV),	24.0	200	5	()	_
SONDA	U KOTI	/ Boning	tion	u1		: 20	46.63 m	KOORE	NAT / Coordinate (N-S	it : 4.	423 615	32	7.03	.200	<u> </u>		-
SONDA	J MAK	SYÖNT.	/D.Ri	a & N	let.	: M	obile Drill B-53 / Rotary	KOORD	NAT / Coordinate (E-V	V) y : 48	6 946.6	4					-
				ST	AND	ART	PENETRASYON DENEY!					£		-	αż		
5					s	tanda	art Penetration Test					Bua	ering	E S	Co Co		
Ĩ	NSI		DA	RBE	SAYI	SI	GRAFIK		JEOTEKNÍK TANIMI	AMA.		KVSti	eath	5	Ę		
pth (Type C	≴ 5	Nu	mb. c	of Blov	vs	Graph		Geotechnical Descri	ption			13	actu	Ē		
NDAJ I	MUNI	ANEVF	- 15 cm	-30 cm	-45 cm	N	÷				coFIL ofile	NINAY	RISMA	RIK / Fr	ROT%	% Q	GEON
sc Bo	N S	хă	0	12	ň		10 20 30 40 50 60				9 9	6	₹.	2	3	ĕ	3
0 - 1 - 2 - 3 - 4 - 4 - 5	SPT-1 SPT-2 SPT-3	1.50 1.95 3.00 3.45 4.50 4.95	6 5	4 5 10	6	7 11 19		YAPAY Çakıllı k kili sili- kurnlu. Gri reni- t yuvarla yarı yuv	DOLGU MALZEMESİ umlu killi SİLT. 	ert, kumlu teli; %20-25 uvarlak sert, yarı e-orta, sert, la ince							
								malzerr	malzemeli 5.70 m								
•								(Tanımı Sayaf 2/4 ' dedir.)							Ļ		
	DAYANI		Stre	Stron	a	1	TAZE Fresh	N :	0-2 COK YUMUSAK	V.Soft		ANE 4		GEL	SE G	VI	BC 005
R	ORTA D	AYANIMI	L	M.St	ong	ii.	AZ AYRIŞMIŞ Slightly W.	N :	3-4 YUMUŞAK	Soft	N : 5-	10	GE	/ŞEK		Loo	se
III IV	ORTA ZAYIF M.Weak		III	ORTA D. AYR. Mod. Weath.	N:	5-8 ORTA KATI	M.Stiff	N: 11	-30	ORI	A SI	KI	M.C	en			
v	IV ZAYIF Weak V COK ZAYIF V.Weak		V	TÜMÜYLE A, Comp.Weat.	N:	16-30 ÇOK KATI	V.Stiff	N : >5	-50	ÇO	K SIK	1	V.D	en			
								N :	>30 SERT	Hard							
KA1	COK	TESI TA		I - RC	10		SEVREK Wide (M)	QL E	PEK A7 S	NLAR - Prop	ortions		PER	A7		Stic	nti
% 25-50	ZAYI	F	Po	bor		1-	2 ORTA Moderate (M)	% 5-	15 AZ L	ttie	% 5-	20	AZ			Littl	
% 50-75	ORT	A	Fa	uir (2-	10 SIK Close (CI)	% 15	-35 ÇOK V	ery	% 20	-50	ÇO	<		Ver	y
% 75-90 % 90-10	0 COK	iyi	Go	ood ccellen	,	10	20 COK SIKI Intense (I) 20 PARCALI Crushed (Cr)	% 35	VE A	na							
SPT	Standart	Penetras	syon T	esti		к	Karot Numunesi		LOGU YAPAN		K	DNT	ROL				\neg
0	Standart	Penetrat	ion Te	est			Core Sample	ISIN	Logged By	Dr. Erbo	TiMUE	heci	ked	Marr	at Ci	1.544	
5	Disturbe	d sample				٣	Pressuremeter Test	Name	Jeoloji Müh.	Jeotekn	ik Müh.			Je	oloji I	Mūh.	
υD	Örselenn	nemiş Nu	mune			V	S Veyn Deneyi	İMZA Siac									
Undisturbed Sample							Vane Shear Test	aign		1			1				

Continuation of Borehole TA-1

YÜKSEL PROJE																	
Yi Bi	YÜKSEL PROJE ULUSLARARASI A.Ş. Birlik Mahalesi B. Cadde No:41 06610 ÇANKAYA-ANKARA SONDA LLOGU / BORING LOG SONDA L																
TEL: (312) 495 70 00 FAX: (312) 495 70 24 SONDAJ LOC							SONDAJ LOO	GU / BORING LOG			SONDA	ຟ e	No: TA-1				
											SAYFA Page		No :		2/	4	
_				S	TAND	DART	PENETRASYON DENEY					뮲	2	Ê	GeR.		Γ
m)	NSI			DARBE SAYISI GRAFIK JEOTEKNIK TANIMLAM/					AMA		/Stren	atheri	300	1.Co			
DER epth (Type	An M	Nu	Numb. of Blows			Graph	4	Geotechnical Description			FILK	V/ We	racture	(TCR		
SONDA. Boring D	NUMUN Samp	MANEV	0 - 15 cr	15-30 cr	30-45 cr	N	10 20 30 40 50 50				ROFIL	AYANIA	YRISM	IRIK / F	AROT%	0D %	UGEON
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-		6.45	2	3	1	1		kumlu %10-2	killi SİLT. Nemli, düşük	plastisiteli;	<u> </u>						İ.
- 7								yuvarla	ik kumlu; %5-10 ince tar	neli, sert,	0						
<i>'</i>								yan yu	•anak çakılır. 6.40 m —		0						
-	SPT-5	7.50	3			,		Griren Islak, ir	kli, gevşek, killi siltli KUI nce-orta taneli-sert, varı	M. Nemli- vuvarlak:	.0.						
- 8	011-0	7.95	ľ		⁻	ľ		%20-2	5 düşük plastisiteli, ince	/	<u> </u>						
L		8.50							7.65 m	/	۷						
	UD-1	0.00															
- 9	SPT-6	9.00	4	5	6			1									
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- 10								1			°		ŀ				
ŀ		10.50						Koyu g	ri renkli, gevşek-orta sık	ı, çakıllı	· — ·						
- 11	SP1-/	10.95	3	4	5	9		%20-2	5 çok ince-ince taneli, da	asusiteii; iğilgan-orta	·						
							and long transferrations and long transferrati	taneli, s	ırı yuvarlak kumlu; eser sert, yarı yuvarlak çakıllı	oranda ince	· _ ·						
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- 13								┝──	13.10 m —								
ł	<u> </u>	13.50									<u> </u>						
- 14	SPT-9	13.95	5	8	11	19					. <u> </u>						
								Kahver	engimsi açık gri renkli, k killi SİLT, Nemli, dünük d	atı, çakıllı	°						
	110.2	14.50						plastisi	teli; %10-20 ince-orta ta	neli,	. <u>.</u> .						
- 15	00-2	15.00			ľ.			10 ince	yer yer iri taneli, sert, ya	arı yuvarlak-	. — ·						
	SPT-10	15 45	5	6	10	16	16	yuvarla	k çakıllı.								
16																	
									LOGU YAPAN		KC	ONTR	LOL				Ц
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								Name IMZA	Jeoloji Müh.	Jeotekn	iik Müh.			Jec	iloji M	üh.	\neg
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Continuation of Borehole TA-1


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26								Kızılım SİLT /	sı kahverengi, sert, çakı sittli KİL. Nemli, düşük-o	llı kumlu killi Irta							
								taneli,	dağılgan-orta sert, yarı y	ze-oπa ruvarlak	ē						
- 27	SPT-18	27.00	18	28	35	63		kumlu; çakıllı.	eser oranda ince taneli,	orta-sert							
-		27.45					63	Birim k	ireç konkresyonları içeri	r. İ	<u>+</u>						
- 20	- P2 -	27.90						25.40-	26.20 m. arası killi siltli ç	akıllı KUM.	<u>F</u> ≓⊇						
- 20	SPT-19	28.00	18	29	37	66		(ANKA	ARA KILI)		a-a						
- 29 - 30 - 31 - 32		-							KUYU SONU : 28.4	5 mt.							
- 33 -																	
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36																	
			_						LOGU YAPAN		K	ONT	ROL	L			
Not : Ku	uyuya ye	raltisuyu	göz	emle	ri için	28.0) m, Φ50 mm.	ISIM	Baris HASANCEB	Dr. Erha	n TIMUR	neck	led	Mura	at ÇİL	SAN	
pe	nore bo		p, 40	X9UX	10 CM	i. Kuy	u agzı betonu yapılmıştır.	Mame IMZA	Jeoloji Müh.	Jeotekr	lik Müh.			Jec	loji M	lüh.	
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C.2. Borehole TA-2

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PROJE	ADI / Pr	oject N	ame			: UL	US-KEÇIÖREN I	METRO HATTI	DELİK	ÇAPI	/ Hole Diamete	er	: N	W (89 r	mm)				
SONDA	WYER!	Boring	Loca	tion		: A5	KI-TANDOĞAN	ARASI	YERAL	TI SU	YU / Groundw	ater	: 6,	50 m. (2	21.0	4.200	5)			
KILOM	ETRE / C	Chainag	e			: 0+	726		MUH.B	OR.D	ER. / Casing D	epth	: 22	2.50m (NW)		-		
SONDA	U DER.	/ Boring	Dept	h		: 28	.95 m		BAŞ.BI		R. / Start Finish	Date	: 02	402.465	15/0	3.04	.200	5		
SONDA	UMAK			n & N	let	: 04 : SN	0.42 m		KOORL		/ Coordinate (F-W)	v 45	423 400 86 842 0	2.24 36				_	
50110/			10.74	SI	AND	ART	PENETRASYON	DENEY		2111211	/ ooorainate (<u> </u>	1		Ĩ£		-	œ		
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IV	III ORTA ZAYIF M.Wei IV ZAYIF Weak						ÇOK AYR.	Highly W.	N :	9-15	KATI	s	tiff	N: 3	1-50	SIK			Der	se
v	ÇOK ZAY	ſIF		V.We	ak	v	TUMUYLE A.	Comp.Weat.	N:	16-30	ÇOK KATI SERT	v H	Stiff	N: >	50	ÇO	C SIK	I	V.D	en
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- 7 - 8	SPT-5	7.50 7.58	<u>50</u> 8	_	-	R											
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- 10 - 11	SPT-7	10.50 10.70	42	<u>50</u> 5	-	R											
- 12	SPT-8	12.00 12.20	44	<u>50</u> 5	•	R	n.										
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- 14	SPT-9	- 13.95	8	9	12	21		Gri reni Islak, in yuvarla yuvarla	di, orta sıkı, killi siltli kun ce-iri taneli, sert, yuvarlı k, %20-25 ince-iri taneli, k kumlu, eser-%5 ince n	nlu ÇAKIL. ak-yarı . sert, yarı nalzemeli.	00000						
- 15	SPT-10	15.00 15.45	4	5	6	11		Yeşilim SİLT. N çok inci	14.60 m — si gri renkli, katı, kumlu emli, düşük-örta plastisi e-ince taneli, kumlu.	siltli KİL / killi iteli, %10-20	· _ · _ ·						
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C.3. Borehole TA-3

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C.4. Borehole TA-4

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ill	ORTA Z	AYIF		M.W	ak	ш	ORTA D. AYR. Mod. Weath.	N :	5-8 ORTA KATI	M.Stiff	N: 11	-30	ORT	A SI	a	M.C)en
v	III ORTAZAYIF M.Wea IV ZAYIF Weak V ÇOKZAYIF V.Wea						ÇOK AYR. Highly W. TÜMÜYLE A. Comp. Weat.	N: N:	9-15 KATI 16-30 COK KATI	Suff V.Stiff	N: 31 N: >5	-50 0	ÇOK	SIKI		Der V.D	ise ien
								N :	>30 SERT	Hard							
KA % 0-25	COK	ZAYIF		Poor	D	K	SEYREK Wide (W)	% 5	PEKAZ	Slightly	portions % 5		PEK	AZ		Slig	hti
% 25-50	ZAYI	F	Pe	nor		1-3	2 ORTA Moderate (M)	% 5-	15 AZ -	Little	% 5-3	20	AZ			Litt	e
% 50-75 % 75-90	ORT/	A	Fa	ur bod		2-	10 SIK Close (Cl) -20 COK SIKI Intense (1)	% 15 % 35	-35 ÇOK VE	And	% 20	-50	ÇOK			Ver	У
% 90-10	O ÇOK	iyi	E	cellen		>2	0 PARCALI Crushed (Cr)		1000		L		0				
SPT	Standart	Penetras	iyon Te	est ist		к	Core Sample		LOGU YAPAN Logged By		KC C	heck	ed				1
D	Örselenn	niş Numu	ne			P	Pressiyometre Deneyi	ISIM	Barış HASANÇEB	Dr. Erha	In TIMUR			Mura	at Çil	SAI	4
UD	Orselenmiş Numune Disturbed sample Örselenmemiş Numune						Pressuremeter Test Veyn Deneyi	IMZA	Jeoloji Müh.	Jeotek	nik Müh.			Jec	noji N	Nüh.	_
	Undisturt	ed Sam	ple		_		Vane Shear Test	Sign									

Continuation	of	Borehole	TA-4
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	YÜ	KS	E	L	Pl	R	DJE										
У В О Т	ÜKSEL P irlik Mahaii 6610 ÇAN EL: (312) 4	ROJE ULI Iesi 9. Cai KAYA-AN 195 70 00	JSLAF Ide No KARA FAX: ((312) 4	I A.Ş. 95 70 2	24	SONDAJ LOO	SU / B	ORING LOG		SONDA	U I	No :		т/	4-4	
w	ww.yuksel	proje.com	.tr								Borehol SAYFA	e	No :	_		14	
		1	T	S	TAND	ART	PENETRASYON DENEY				Page	5			le:	T	
n.Iči	S.		F		5	Stand	art Penetration Test		IFOTEWHILE TAKING			trengt	hering	30cm	Core		
DERIN oth (m	pe	≤ =	N	umb. (of Blo	ws	Graph		Geotechnical Descr	iption		ILIK/S	Weat	ture (CRJ		
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6	SPT-4	6.00	2	5	6	11								\vdash	\square		t
·		6.45	1					1			<u> </u>						
- 7											L						
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i i	SPT-5	7.50	3	3	5	8		C	his ante beto actuit human		. <u>.</u> .						
- 8		7.95			-	-		killi SİL	T. Nemli, düşük-orta pla	astisiteli;	· ·						
			1					%10-1	5 cok ince-ince taneli, da	ağılgan	·						
								yuvarla	k çakıllı.	sert, yan	<u>. </u>						
- 9		9.00						9.15-9.	20 m. arası killi siltli kun	nlu CAKIL	-0.0			ļ			
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		9.45									·						
- 10		10.00									· _ ·						
	UD-2									•	· ·						
	SPT-7	10.50	4	6	10	16					<u>°</u>						
- 11		10.95	1		10	10		Kahver	engimsi gri renkli, gevşe	ek, çakıllı	_ · _						
	ì							killi silti tapeli d	KUM. Islak, cok ince-in	ce-orta	. <u> </u>						
								plastisit	teli, ince malzemeli %3-	5 ince,	·						
- 12		12.00						sert, kö	şeli çakıllı.		[- <u>-</u> -						
	SPT-8		5	5	7	12			12.25 m —		<u>:0:0:</u>						
		12.45						÷.			<u></u>						
- 13								Kahver	engimsi gri renkli, katı-ç	ok katı,							
								çakıllı k olastisit	umlu siltli KİL. Nemli, di eli: %15-20 cok ince-inc	üşük-orta xe taneli	Fig						
	SPT-0	13.50	4	7		16	• 16	kumlu;	%5-10 ince yer yer orta	taneli, yarı	ē						
- 14		13.95		'		10		yuvana	k-yuvanak çakıllı.								
									14.30 m		<u> </u>						
	10.2	14.50						Grimsi	kahverengi, sert, çakıllı	kumlu killi							
15	00-3	15.00						SILT. N	lemli, düşük plastisiteli;	%20-25 ince-	2						
	SPT-10		16	17	17	34	34 7	%15-20	ince-orta-iri taneli, sert,	yuvarlak	· ·						
		15.45						çakıllı.				ĺ					
16								(Tanım	15.80 m 11 Sayfa 3/4 ' dedir.)		000						
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								ISIM Name	Banş HASANÇEBI Jeoloji Müh	Dr. Erhan	h TIMUR ik Müh			Mura	It ÇIL	SAN	
								IMZA Sign		UCONDAIN		+		500	yr mil	an.	



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· •	ww.yuksel	proje.com.	lt It	312) 4	10 /0 2	•					Borehol	e	NO :			4-4	
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75	1			s	I AND S	Stand	PENETRASYON DENEYI art Penetration Test					ngth	jug	Ē	oreR.		
(ii) (iii)	INSI		DA	ARBE	SAY	ISI	GRAFIK	1	JEOTEKNÍK TANIMI	AMA		VStre	eathe	re (30	D.T.C		
epth	LType	RA III	Nu E	mb.α Γε	of Blo E	ws	Graph	{	Geotechnical Descri	ption		F	A/W	ractu	(TCF		
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26									1		<u> </u>	-		-	-	-	F
-											- <u>-</u>						
07		27.00			ł			Kahver	rengi sert çakıllı kumlu	killi SİLT/	<u>e</u>						
-21	SPT-18	27.00	16	25	29	L. Nemli, orta-yüksek pla	stisiteli;										
-		27.45			л çok ince-ince taneli ku neli, dağılgan-orta sert ç	mlu; %5-10 akıllı.											
- 28					irec konkresvonları iceri		- <u>-</u>										
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- 29		29.00					 Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria Antonio Maria	9			÷						
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Not : K	uyuya ve	raltisuv	u göz	lemie	ri icin	29.0	0 m. Φ50 mm.	ISIM	Logged By	Dr Erte	C	heck	ed	M		RÁN	
p	erfore bo	ru indiri	lip, 40	x40x	15 cm	1. kuy	u ağzı betonu yapılmıştır.	Name	Jeoloji Müh.	Jeoteki	nik Müh.			Jec	at Çil Dioji N	.SAN Iüh.	
								MZA Sign.									

C.5. Borehole TA-5

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YÜ Birl 066	KSEL PRO	OJE ULUS si 9. Cadd AYA-ANK	SLARA Ie No:4	ARASI	A.Ş.		SONDALLOG		PINCLOC		SONDA.					-	
TE	L: (312) 48 w.yukselpi	5 70 00 F roje.com.t	AX: (3	12) 49	5 70 24	•	SUNDAJ LUG	0/80	RING LOG		Borehole		10 :		TA	-5	
											Page	N	lo :		1/	4	
PROJE	ADI / Pr	oject Na	ime		_	: UL	US-KEÇİÖREN METRO HATTI	DELIK Ç	API / Hole Diameter	: H\	N (114	nm))				
SONDA	J YERI /	Boring	Loca	tion		: AS	SKI-TANDOGAN ARASI	YERALT	I SUYU / Groundwat	er : 4.	90 m. (2	1.04	.200	5)	A/ \		
KILOME	TRE/C	hainage	Dont	b		: 0+	485 m	BAS BIT	TAR / Start Finish	ate : 09	03 200/	5/10	0.03	200	5		
SONDA	I KOTU	/ Eleval	Lion	<u>n</u>		: 84	5 39 m	KOORD	NAT / Coordinate (N	S)x : 4	423 288	36	0.00	.200	<u> </u>		
SONDA	J MAK.8	YÖNT	/D.Rie	a & M	let.	: Fo	premost Mobile / Rotary	KOORD	NAT / Coordinate (E	-W)y : 48	6 681.7	1					
T				ST	AND	ART	PENETRASYON DENEY					£	_	2	ц,		
ō	_	l			S	tanda	art Penetration Test	,				reng	erin	30cm	Con		
Ξ£	S INS		DA	RBE	SAYI	SI	GRAFIK	•	JEOTEKNÍK TANIN	ILAMA		KS	feath	ire (P.		
pt	U P	5 5	Nu	mb. c	of Blov	vs	Graph		Geotechnical Desc	ription		Ę	A/W	ractu	6(TC		1_
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	SPT-3	4.95	7	8	10	18	10	Nemli, o	düşük-orta plastisiteli;	%5-10 çok	- <u>-</u>						
- 5								ince-inc	e tanea kunitu.		<u></u>						
		5 50						4									
	UD-1	0.00							5.60 m		·						
6	00-1	6.00					BULLY ON FALL FILL LIGH TO B	(Tanın	n Sayta 2/4 dedir.)								L.
D	AYANIN	ALILIK /	Stre	ngth			AYRIŞMA / Weathering		NCE DANELI / Fine	Grained	IRI D	ANE		oan	se G	Frair	ed
11	ORTA D	MLI AYANIMI		M.Str	rong	i	AZ AYRIŞMIŞ Slightly W.	N:	3-4 YUMUŞAK	Soft	N : 5-	10	GEN	/ŞEK	ŞER	Loc	se
111	II ORTA DAYANIMLI M.Str III ORTA ZAYIF M.W						ORTA D. AYR. Mod. Weath.	N :	5-8 ORTA KATI	M.Stiff	N: 11	-30	ORI	TA SI	KI	M.I)en
	V ÇOK ZAYIF Weal						ÇOKAYR. Highly W. TÜMÜYLEA. Comp.Weat.	N:	9-15 KATI 16-30 COK KATI	Stiff V.Stiff	N: 31	-50	ÇO	K SIK	1	V.C	len
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% 75-90 % 90-10	D COK	iYİ	G	ood cceilen	nt	10	20 COK SIKI Intense (I) 20 PARCALI Crushed (Cr)	% 35	VE	And							
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	Standart	Penetrat	ion Te	st		P	Core Sample	ISIM	Logged By	Dr. Erba		Check	ked	Mire	at C	I.SA	N
°	Örselenmiş Numune Disturbed sample						Pressuremeter Test	Name	Jeoloji Müh.	Jeotek	nik Müh.			Je	oloji	Müh.	
UD	Disturbed sample D Örselenmemiş Numune					V	S Veyn Deneyi	IMZA Sion									
	Örselenmemiş Numune Undisturbed Sample						Vane Shear Test	algn					1				

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Y B O T	ÜKSEL PF irlik Mahali 6610 ÇANI EL: (312) 4	ROJE ULU esi 9. Cad KAYA-AN 195 70 00	JSLAR Ide No KARA FAX: ()	ARASI :41 312) 49	A.Ş. 95 70 2	4	SONDAJ LC	GU / B	ORING LOG		SONDA	5	No :		TA		
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ō					S	Stand	art Penetration Test					sngth	gi	(m)	oreR		
E RIVE	SINS a		D/	ARBE	SAY	ISI	GRAFIK	7	JEOTEKNİK TANIM	LAMA		K/Sth	eathe	re (3	۲. ۲		
C DE Depth	Typ I	VRA /Run	E	E S	E	ws	Graph	┥	Geotechnical Descr	iption	1_	MLIL	A/W	ractu	%(TCI		
SONDA Boring I	NUMU Samp.	MANE	0-150	15-30 0	30-45 c	N	10 20 30 40 50 6				PROFII Profile	DAYANI	AYRIŞM	KIRIK / 8	KAROT9	RQD %	LUGEOI
6	SPT-4	6.00	4	6	7	13		-				-		-	-	-	H
		6.45		ľ	Ĺ			Kahve	rengimsi gri renkli, katı,	kumlu siltli	<u></u>						
~								ince-in	ice taneli kumlu.	%5-10 ÇOK	<u></u>						
- '								<u> </u>	7.20 m		<u> = -</u>						
·		7.50					And Annual Annua	2			<u>⊷</u>						
	SPT-5	7 95	8	6	9	15	15	Açık k	ahverengi, katı, çakıllı k	umlu killi	<u>.</u> 						
- 0	(1.55						plastisi	sıltli KIL. Nemli, düşük- iteli, %20-25 çok ince-in	orta ce taneli,	ē						
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		0.00						çakıllı,	nen, dagigan-ona sert,	yan yuvanak	1						
- 9	SPT-6	9.00	3	5	7	12		<u> </u>			È÷						
		9.45						Kahve	rengimsi gri renkli, killi s	iltli kumlu							
								ÇAKIL	. Nemli, ince-orta taneli, varlak-vuvarlak, %20-30	orta-sert, ince-orta	E-È						
- 10								taneli,	orta sert kumlu; %15-20	düşük	<u>+-</u>						
		10.50						plastisi	iteli, ince malzemeli.	-	000						
	SPT-7	10.05	7	7	7	14		Açık ka	ahverengi, katı, kumlu si	iltli KİL.							
- 11		10.95						Nemli,	düşük-orta plastisiteli; 9	610-15	<u> </u>						
								çok inc	10.85 m	· /	· _ · _						
				<i></i>					10.85 m -		· ·						
- 12	SPT-8	12.00	5	7		16					·						
		12.45	Ĩ	Ċ	Ĩ		16										
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- 13	110-2	13.00									م						
		13.50						Açık ka KİL / ki	ahverengi, çok katı, çakı illi SİLT, Nemli, düsük-o	llı kumlu siltli da	<u> </u>						
	SPT-9	12.05	5	7	9	16	• 16	plastisi	teli; %15-20 çok ince-inc	be yer yer	· ·						
- 14		13.95						kumlu;	neli, dağılgan-orta sert, y %5-10 ince taneli, dağıl	gan yer yer							
								orta se	rt, yarı yuvarlak çakıllı.		 ,						
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- 15	SPT-10	15.00		7	。	16		1			Ľ.						
	SP 1-10	15.45	Ĭ	1	1		1.6	-	·		. <u> </u>						
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								ISIM	Banş HASANÇEBI	Dr. Erha	C n TIMUR	heck	ed	Mura	it ÇIL	SAN	-
								Name IMZA	Jeoloji Müh,	Jeotekr	ik Müh.	_		Jeo	loji M	üh.	_
								Sign	I.			- 1					

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YÚ Bir 06	ÜKSEL PR flik Mahalik i610 ÇANI	OJE ULU Isi 9. Cad (AYA-AN)	SLAR/ de No: (ARA	ARASI. 41	A.Ş.		SONDALLOG		PINGLOG		SONDA	1 .					
TE	L: (312) 4 ww.yukselp	95 70 00 l roje.com.	=AX: (3	12) 49	5 70 2	4	SONDAJ LOG	0/80	RING LOG		Borehol		No :		ТА	-5	
											SAYFA Page	'	No :		3/	4	
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nLiG N	NSI		DA	RBE	SAYI	SI	GRAFIK		JEOTEKNÍK TANIML	AMA		UStre	ather	e (30	D.U.C		
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16		40.00		÷				Açık kal KİL / kil	hverengi, çok katı, çakılı li SİLT. Nemli, düşük-orl	ı kumlu siltli a	· _ ·						
	SPT-11	16.50	8	11	14	25	25	plastisit	eli; %15-20 çok ince-ince	e yer yer	·				1.		
- 17		16.95						kumlu;	%5-10 ince taneli, dağılg	an yer yer	· ·						
		17 50						orta ser	t, yarı yuvarlak çakıllı.								
	UD-3	. 17.50							17.60 m								
- 18	SPT-12	18.00	10	14	17	31		Kahver	engimsi gri, sıkı, killi sittli	KUM.							
-	SF 1-12	18.45		17			X1	Nemli-Is yuvarlal	slak, ince-orta taneli, ser <; eser oranda düşük pla	t, yarı ıstisiteli,							
								ince ma	Izemeli.		2					ĺ	
- 19									19.20 m								
-		19.50						Kabuar	, naimai an ook aku killi	cilti	0.00			1			
	SPT-13	10.05	29	32	25	57	57	kumlu Ç	AKIL. Nemli-Islak, ince	orta-iri	000						
- 20		13.55						taneli, s 20 ince-	ert, yarı yuvarlak-yuvarla orta taneli, sert, yarı yuv	ak; %15- /arlak	000						
-								kumlu;	%5-10 düşük plastisiteli,	ince	<u> </u>						
- 21		21.00						maizen	20.40 m		- <u>-</u> a						
21	SPT-14	21.00	13	20	21	41					F:-;;						
ł.		21.45															
- 22								Kahven	engi, sert, cakıllı kumlu s	ittli KIL.	Ĕ=Ť						
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-	SPT-15	22.50	23	24	22	46	46 •	ince tar	eli, dağılgan yer yer orta	i sert, yarı	000						
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pe	arfore bo	ransuyu uru indiril	ip, 40	emiei ix40x	15 cm	27.50 n. kuy	/u ağzı betonu yapılmıştır.	SIM Name	Barış HASANÇEBI Jeoloji Müh.	Dr. Erhan Jeotekni	K Müh.			Mura Jeo	t ÇILS	SAN üh.	
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C.8. Borehole TA-8

ſ	נטו	(5	EL	. 1	PF	20	JE												
YU Bir 06	iksel PR 1ik Mahalle 610 ÇANK	OJE ULU Isi 9. Cade (AYA-ANK	SLARA ie No:4 (ARA	RASI	A.Ş.		SONDA	JLOG	U / BC	RIN	GLOG		SONDA	J,	No ·		ТА	-8	
TE	L: (312) 4 w.yukselp	95 70 00 F roje.com.t	FAX: (3 r	12) 49	5 70 24	•							Borehole SAYFA	e .			1/	3	_
PROJE	ADI / Pr	niect Na	me	-		: UI	US-KECIÓREN METRO H	HATTI	DELIK		Hole Diameter	: H	Page W (114	mm)	_			
SONDA	U YERI	Boring	Loca	tion		: AS	KI-TANDOĞAN ARASI		YERAL	TI SUY	/U / Groundwa	er : 4.	60 m. (2	1.04	.200)5)			
KILOM	ETRE / C	Chainage	e			: 0+	189		MUH.BO	DR.DE	R. / Casing De	pth : 3.	00m HV	V, 12	.00n	n (N	W)		
SONDA	U DER.	/ Boring	Dept	h		: 25	.95 m		BAŞ.Bİ	T.TAR	/ Start Finish	Date : 2	2.03.200	5/2	4.03	.200	5		
SONDA	U KOTU	/ Eleva	tion			: 84	3.90 m		KOORD	INAT	/ Coordinate (N	-S)x : 4	423 075	.66					
SONDA	J MAK.	SYÖNT.	/D.Rig	3 & M	et.	: D-	500 / ROTARY		KOORE	INAT.	/ Coordinate (E	-W)y :4	B6 475.5	7		_		_	_
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Eln (m)	N B		DA	RBE mb.o	SAYI f Blow	SI	GRAFIK			Geot	echnical Des	rintion		Š.	Wea	Ę.	S.		
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											1.30 m								
-		1.50						1111					. 9						
	SPT-1		4	5	6	11		1111	Kahvere	engims	si gri renkli, orti	ı sıkı, killi siltli	0.						
- 2	<u> </u>	1.95						1111	kumlu ς	AKIL	/ çakıllı KUM.	Vemli-Islak,							
									ince-ort	a tane	li, sert, yarı yuv	arlak; %20-25	0						
-		2.50						i iiii	%5-10 (a-in ta Iŭsŭk	nell, sert, yari y olastisiteli ince	uvanak çakılır; maizemeli	0						
	UD-1							11111		ayan	p		1						
- 3		3.00						1 1111											
	SPT-2		6	7	6	13					3.20 m								
-		3.45						T IIII	Kabuer	andime	si ori renkli kat	cakili kumlu	· ·						
									killi SİL	r. Nen	nli, düsük plast	siteli: %15-20	<u>– ، ما</u>						
- 4	•								ince-ort	a tane	li, orta-sert, ya	yuvarlak	L		1				
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	SPT-3		7	9	7	16	+ 16		Crime's		and hat ask	lla kurenta killi							
- 5		4.95							SiLT / s	iltli Kİ	engi, kati, çak L. Nemli, düsü	a kumu kili -orta	H	·					
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		5.40							dağılga	n kum	lu; eser oranda	ince taneli,							
6									sert, kö	șeli ça	kılı.		HE-						
0	DAYANIMLILIK / Strength						AYRIŞMA / Weathering	9		NCE	DANELÍ / Fine	Grained	· IRI D	AN	ELI/C	Coan	se G	rain	ed
1	DAYANIMLI Stronger ORTA DAYANIMLI M.St					1	TAZE Fresh		N :	0-2 (ÇOK YUMUŞAK	V.Soft	N: 0-	4	ÇOP	GE	/\$EK	V.L	005
11	ORTA DAYANIMLI M.Str ORTA ZAYIF M.We						ORTA D. AYR. Mod. V	ly W. Weath.	N: N:	3-4 5-8 (YUMUŞAK ORTA KATI	M.Stiff	N : 5-	1-30	OR	VŞEK TA SI	ĸ	M.C	se)en
IV	ZAYIF			Weak		īV	ÇOK AYR. Highly	W.	N :	9-15	KATI	Stiff	N: 31	1-50	SIK	1		Der	ise
v	ÇOK ZA'	YIF		V.We	ak	v	TUMUYLE A. Comp.	.Weat	N :	16-30	ÇOK KATI SERT	V.Stiff Hard	N: >5	50	ço	K SIK	1	V.D	en
KA	A KALI	TESİ TA	NIM	- RC	D	۲	IRIKLAR - 30 cm / Fractu	ures			0	RANLAR - Pro	portions	5					
% 0-25	ÇOK	ZAYIF	V.	Poor		1	SEYREK Wide (W)		% 5	1	PEKAZ	Slightly	% 5		PER	(AZ		Slig	hti
% 25-50 % 50-75	ZAYI ORT	F A	Po	or ir		1-	Z URTA Moderate 10 SIK Close (Cl)	(M)	% 5-1 % 15	-35	COK	Very	% 5-	20	AZ COI	ĸ		Ver	e v
% 75-90	75-90 IYI Good					10	-20 ÇOK SIKI Intense (I)		% 35		VE	And			4.51	-			1
% 90-10	90-100 ÇOK İYİ Excellent T Standart Penetrasyon Testi					>2	0 PARÇALI Crushed (Cr)			OCIL VADAN		L	0.117	001				
SPI	PT Standart Penetrasyon Testi Standart Penetration Test					ĸ	Core Sample				Logged By		K (Chec	ked				
D	Örselenmiş Numune					P	Pressiyometre Deneyi		ISIM	Ba	INŞ HASANÇEBİ	Dr. Erha	n TİMUR		Ĺ	Mur	at Çi	LSAN	4
UD	Disturber	d sample	mune				Pressuremeter Test		Name		Jeoloji Müh.	Jeotek	nik Müh.			Je	oloji I	Mūh.	
30	Örselenmemiş Numune					V3	Vane Shear Test		Sign										

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YI Bi Of	ÜKSEL PR riik Mahali 3610 ÇANI	ROJE ULU esi 9. Cad KAYA-AN	de No: KARA	ARASI 41	A.Ş.		SONDALLO				SONDA						
TE	EL: (312) 4 ww.yuksel;	195 70 00 proje.com.	FAX: (3 tr	312) 49	95 70 2	4	SONDAJ LOU	50 / B	ORING LOG		Boreho	e	No :		T/	4-8	
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6	SPT-4	6.00	6	6	8	14		Grimsi	kahverengi, katı, çakıllı	kumlu killi	<u>-</u>						
		0.45						plastisi	siltli KIL. Nemli, düşük-c teli; %15-20 çok ince-ini	rta ce taneli,							
- 7								dağılga köseli o	ın kumlu; eser oranda ir zakıllı.	ice, sert,		1					
		7.50		50					7.30 m —		5 5						
	SPT-5	7.74	48	9	-	R	Ry				<u> </u>						
- 8								Kahver siltli ku	engimsi gri renkli, sıkı-ç mlu ÇAKIL. İslak, ince-c	ok sıkı, killi Irta-iri taneli,	9. Å 0					1	
								sert, ya	rı yuvarlak-köşeli; %20-	30 ince-iri	0.00						
								oranda	düşük plastisiteli, ince r	nalzemeli.	200						
- 9	SPT.6	9.00	22	18	22	40		(ALUV	YON)		000						
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								Kizilim Sit T / I	si kahverengi, sert, çakı sitti Kil Nemli düsük-c	llı kumlu killi rtə vər vər	<u></u> - <u>-</u>						
- 12	SPT-8	12.00	15	20	25	45		yüksek	plastisiteli; %10-15 cok	ince-ince yer							
		12.45						yer orta yuvarla	k kumlu; %5-10 ince tar	ert, yarı neli, orta sert,	<u>+</u> -						
- 13								yarı yuv	varlak çakıllı.								
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		13.50						Konkres	syonları içenr.		<u> </u>						
- 14	SPT-9	13.95	14	21	25	46	46	10.40-1 seviveli	0.80 m. arası killi siltli k (ANKARA KİLİ)	umlu ÇAKIL	<u></u>						
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	SPT-10		14	20	22	42	142		,								
		15.45									E-Ť						
16											<u>–α</u>						
									LOGU YAPAN Logged By		K0	heck	ed				
								ISIM Name	Barış HASANÇEBİ Jeoloji Müh.	Dr. Erha Jeotekr	n TİMUR nik Müh.			Mur	at Çil bioji N	.SAN	1
								MZA Sign									



C.9. Borehole TA-9

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YI Bi	ÜKSEL PR	OJE ULU esi 9. Cad	ISLAR	ARASI	A.Ş.															
OE TE	610 ÇANI EL: (312) 4	64YA-AN	KARA FAX: (312) 4	95 70 2	4	5	SONDAJ LOO	GU / B	ORI	NG LOG			SON	DAJ	No	:	Т	A-9	_
w	ww.yukselp	proje.com.	tr											SAY	FA	No	:		1/3	
PROJE	ADI / PI	roject N	ame			: U	LUS-KECIÖREN	METRO HATTI	DELÍK	CAP	I / Hole Diame	ter	· .	Page IW/ / 1) 14 m	m)				_
SOND	J YERI	/ Boring	Loca	ation		: A	SKI-TANDOĞAN	ARASI	YERA	LTIS	UYU / Ground	water	: 5	.70 m	. (21	04.2	005)			_
KILOM	ETRE / (Chainag	e			: 01	-058		MUH.E	BOR.	DER. / Casing	Dept	1 : 9	.00m	(HW)				
SOND/	AJ DER.	/ Boring	Dep	th		: 25	5.95 m		BAŞ.B	IT.TA	R. / Start Finis	sh Da	te : 2	3.03.2	2005	/ 25.0	3.20	05		
SONDA	J MAK	&YÖNT.	/D.R	ig & N	Net.	: M	obile Drill B-53 / F	Rotary	KOOR		T / Coordinate	(N-S)x : 4	422 9	4 68	9				
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-		3.45							Kabua	Anaia	nei eri mekli k	aht a	k kat	Ĕ-						
- 4	– P1 –	3.90					Kahverengimsi gri renkli, kati-çok katı,								희					
1	UD-1	4.00					yüksek plastisiteli; %20-25 çok ince-orta							5	-					
- 1		4.50							kumlu;	0agiig %15-	20 ince-ver ve	/an yu er orta	taneli, sert	<u> </u>	_					
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6															-					
			Stra				AVRIEMA	in na m		in or					Ť.					
1	DAYANI	ILI	000	Strong	,	1	TAZE	Fresh	N :	0-2	COK YUMUSA	ne Gra K	V.Soft	IRI N :	0-4	IELI/	K GE	VSEK	VL	ed DOS
11	ORTA DA ORTA ZA	YANIML	I	M.Stn M.We	ong	11	AZ AYRIŞMIŞ ORTA D. AYR	Slightly W.	N:	3-4	YUMUŞAK		Soft	N :	5-10	GE	VŞE	<	Loo	se
IV	ZAYIF Weak					IV	ÇOK AYR.	Highly W.	N :	9-15	KATI		Stiff	N :	31-50	SI	a	IKI	Der	se
·						v	TUMUYLE A.	Comp.Weat,	N : N :	16-30 >30	ÇOK KATI SERT		V.Stiff Hard	N :	>50	çç	K SI	¢I.	V.D	en
KAY % 0-25	COK	TESI TA	NIM	- RQ	D	K	RIKLAR - 30 cm	/ Fractures				ORA	ILAR - Prop	ortio	ns					_
% 25-50	0-25 ÇOK ZAYIF V.Poor 25-50 ZAYIF Poor 50-75 ORTA Sair					1-2	ORTA	Moderate (M)	% 5-	15	AZ .	Litt	e e	%	5 5-20	PE AZ	KAZ		Slig	nti e
% 50-75 % 75-90	50-75 ORTA Fair 75-90 IYi Good					2-1 10-	0 SIK (20 COK SIKI =	Close (CI)	% 15	-35	ÇOK VE	Ve	Ŷ	%	20-50	ço	к		Ven	1
% 90-10	75-90 IYI Good 90-100 COK IYI Excellent					>20	PARÇALI	Crushed (Cr)				~0								
	Standart I	Penetrati	on Te	st		к	Core Sample	es:			LOGU YAPAN Logged By				KON Che	ROL				
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JD	Örselenm	emiş Nur	nune			VS	Veyn Deneyi	ar i 650	IMZA		Jeoloji Müh.	-	Jeotekn	ik Müh		+-	Je	leoloji Müh.		
UD Örselenmemiş Numune Undişturbed Sample							Vane Shear T	est	Sign											





C.10. Borehole TA-21

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Y B	ÜKSEL PF	ROJE ULL esi 9. Cad	JSLAR	ARASI	A.Ş.			• •			k.									
T	EL: (312) 4	95 70 00	FAX: (312) 4	95 70 2	4	s	ONDAJ LOO	GU / B	ORI	NG LOG			SONDA	U le	No :		TA	-21	
Ň	ww.yuksel;	proje.com	. u											SAYFA		No ·		1	14	-
PROI		roject N	ame			• 11	US.KECIÓREN		IDE! IK	CAD				Page						_
SOND	AJ YERI	/ Boring	Loca	ation		: A	SKI-TANDOĞAN	ARASI	YERA	LTIS	UYU / Groun	dwate	r : 6	70 m. (2	nm 21.0) 4.200	35)			-
KILON	ETRE / (Chainag	e			: 1+	-149		MUH.E	BOR.	DER. / Casin	g Depi	th : 3	0.00m (NW)	,,,			-
SOND	AJ DER.	/ Boring	Dep	th		: 30).45 m		BAŞ.B	IT.TA	R. / Start Fir	nish Da	ate : 2	9.03.200)5/3	30.03	3.200)5		
SOND	AJ KOTU	L/ Eleva	tion /D Pi	0.8.1	Int	: 84	8.53 m	1001	KOOR	DINA	T / Coordina	te (N-S	S) x : 4	423 783	3.03					_
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- 3	SPT-2	3.00	5	5	5	10											ļ			
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									Nemli,	ince-	orta taneli, se	i yuvarlak;	-							
	CDT 2	4.50			.				7630-40	Jauş	uk plastisiteli	ince r	naizemeli.							
- 5	SP1-3	4.95	12	10	15	25	1 26 6		Grimm		4.70	m	-1.	0.0						
Ť.									ÇAKIL	Nen	li-islak, ince	-iri tan	eli, sert, varı	<u>•</u> ;•						
-						ł			yuvarla	k-yuv	varlak; %20-3	30 ince	-orta taneli,	0						
6						- 1			sert, ya malzen	n yuv neli.	varlak kumlu;	eser	oranda ince	o.oo				÷.,		
	AYANIM	LILIK /	Stree	ngth			AYRISMA / We	athering		INCE	DANEL / F	ine Gr	bank					0.6	ain	_
I.	DAYANIN	ALI		Strong	,	1	TAZE	Fresh	N :	0-2	ÇOK YUMUŞ	AK	V.Soft	N : 0-4		ÇOK	GEV	ŞEK	V.Lo	os
IR	ORTA ZA	YIF	.1	M.Stre	ak	Ш	AZ AYRIŞMIŞ ORTA D. AYR.	Slightly W. Mod. Weath.	N: N:	3-4 5-8	YUMUŞAK ORTA KATI		Soft M Stiff	N: 5-1	0	GEV	SEK		Loos	ie In
IV V	ZAYIF Weak ÇOK ZAYIF V.Weak						ÇOK AYR.	Highly W.	N :	9-15	KATI		Stiff	N : 31-	-50	SIKI	~ 91		Den	se
· _	ÇOK 2AI	H-		v.vve.	<u> </u>	V TÜMÜYLE A. Comp.Weat.					ÇOK KATI SERT		V.Stiff Hard	N : >5	0	ÇOK	SIKI		V.De	'n
KA)	COK	TESI TA	NIMI	- RQ	₽	KIRIKLAR - 30 cm / Fractures					0514.0	ORA	NLAR - Prop	ortions						
% 25-50	25 COK ZAYIF V.Poor -50 ZAYIF Poor						ORTA N	% 5- % 5-	15	AZ /	Lit	ightly tie	% 5 % 5-2	20	PEK AZ	AZ		Sligh	.6	
% 50-75 % 75-90	0-75 ORTA Fair 5-90 IYI Good						0 SIK C	Close (CI)	% 15	-35	ÇOK	Ve	ry	% 20-	50	ÇOK			Very	
% 90-10	O ÇOK	Y	Ex	cellent		>20	PARÇALI C	rushed (Cr)	<i>7</i> 0 35		*E	~								
591	Standart I	Penetrati	yon Te on Te:	esti st		к	Karot Nurnune Core Sample	-	LOGU YAPAN KONTROL								_			
Þ	Örselenm	iş Numur	ne	-		P	Pressiyometre	Deneyi	i ISIM Bariş HASANÇEBI Dr. Erhan TIMUR					Murat ÇILSA						
ar	Örselenm	emiş Nur	mune			vs	Pressuremete Veyn Deneyi	r Test	Name IMZA		Jeoloji Müh.		Jeotekn	ik Müh.			Jeo	oji M	üh.	_
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Ŋ	וטי	KS	El	_	PF	20	JE										
YI Bi	ÜKSEL PR	COJE ULU esi 9. Cad	de No:	ARASI 41	A.Ş.										`		
	E10 (CAN L: (312) 4	95 70 00	KARA FAX: (; Ir	312) 49	5 70 2	4	SONDAJ LOO	SU / BC	DRING LOG		SONDA Borehol	ر لہ e	NO :		ТА	-21	
			_								SAYFA Page	1	NO :		2/	4	
-				SI	AND	ART	PENETRASYON DENEY					ngth	ring	(m	oreR.		
(JNLIG	INSI		DA	RBE	SAY	ISI	GRAFIK	1	JEOTEKNIK TANIML	AMA		K/Stre	feather	ire (30	R)T.C		
U DER	NE C Type	VRA /Run	Nu E	mb. c	f Blov	ws	Graph	ł	Geotechnical Descrip	otion	_	IMLIL	MA / M	Fractu	%(TC		z
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6	SPT-4	6.00	7	15	10	25	25	Killi silti	i kumlu ÇAKIL.		<u>000</u>						
-		6.45							6.70 m		0.00						
- 7								Gri ren	kli, katı, çakıllı kumlu killi	SILT / siltli							
_		7.50						ince-inc	e taneli, dağılgan-orta s	ert, yarı	Ë-11						
	SPT-5		8	12	14	26	26 111 11- 26	yuvarla yuvarla	k kumlu; %5-10 ince-init k çakıllı 7.90 m	aneli, sert,	<u> </u>						
- 8		7.95						1			0						
F								Gri ren	kli, orta sıkı, killi siltli çak	ills	0						
- 9		9.00						yuvaria	k; %10-15 ince-orta tanell, ser	eli, sert,	Ë <u>S</u> E						
, in the second s	SPT-6		9	13	15	28	128	yuvarla malzen	k çakıllı, eser oranda inc neli.	3	0.0						
r -		9.45						8.80-9. seviyeli	30 m. arası çakıllı kumlu	killi SILT	0						
- 10	– P2 –	9.90						1	40.00		0						
		10.50															
	SPT-7		5	3	8	11		düşük p	astisiteli; %10-20 çok ir	nce-ince							
- 11		10.95						taneli, c	lağılgan kumlu.		<u></u>						
-		11.50							11.30 m		0.0	1					
- 12	UD-1	12.00						Gri ren	di, sıkı, killi siltli kumlu Ç	AKIL /	0	- 2					
	SPT-8		5	22	20	42	42	yuvarla	k; %20-25 ince-orta yer	, sert, yarı yer iri taneli,	-				2		
-		12.45						sert, ya oranda	rı yuvarlak-yuvarlak çak ince malzemeli.	illi; eser			[
- 13											0						
		13.50							renkli, kati-cok kati, kur	nlu killi							
	SPT-9		6	12	14	26	• 26	SILT. N	lemli, düşük plastisiteli;	%20-30 ince	<u> </u>						
- 14		13.95						oranda	ince-orta taneli, sert, ya	n yuvarlak	·-0.						
-	- P3 -	14.40						Çakılır.	enginsi gi renkli oda s	kı killi citti	P						
- 15		15.00						çakıllı k	(UM. Islak, çok ince-orta	taneli, sert,	0						
	SPT-10		12	6	8	14		yarı yu yarı yu	varlak çakıllı; %5-10 düş	ük	<u>۽ ٻ</u>						
- 16		15.45						plastisi	eli, ince malzemeli. 15.25 m —	/	<u> </u>						
							and a second sec	(Tanım	LOGU YAPAN		<u>і — </u>	I DNTI	ROL			1	Ц
								ISIM	Logged By Bariş HASANÇEBİ	Dr. Erha	an TÍMUR	heck	red	Mur	at Çil	LSAN	-
								Name IMZA	Jeoloji Müh.	Jeotek	nik Müh.		\vdash	Je	oloji N	lüh.	
								LSIGD.									

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Y B O	ÜKSEL Pf irlik Mahali 6610 CAN	ROJE ULL esi 9. Cad KAYA-AN	JSLAR Ide No IKARA	ARASI	A.Ş.							_					
T W	EL: (312) 4 ww.yuksel	95 70 00 proje.com	FAX: (312) 49	95 70 2	24	SONDAJ LOG	SU/B	ORING LOG		SONDA Borehoi	e i	No :		TA	-21	
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- 10	SPT-12	10.00	9	16	19	35	35				- · -						
·		18.45															
- 19																	
								çakıllı l	kumlu killi SİLT. Nemli, ç	ok katı-sert, Jüşük							
ŀ	SPT-13	19.50	7	14	18	32	32	plastisi taneli,c	teli; %10-20 çok ince-ind lağılgan-orta sert, yarı yı	ce uvarlak	· ·						
- 20		19.95						kumlu; kumlu	%5-10 ince taneli, sert,	yarı yuvarlak	: <u>-</u> .:						
	- P4 -	20.40						Konno.			·_ ·						
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- 21	COT 14	21.00		1.0	14	24					·_ ·						
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- 23	SPT-15	22.95	14	46	44	90	90 •				0.0						
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- 24		24.00						Gri ren	kli, sıkı-çok sıkı, killi siltli	kumlu	<u> </u>						
	SPT-16	24.45	35	42	35	77	777	ÇAKIL. yuvarla	. Islak, ince-iri taneli, ser k-köşeli; %20-30 ince-iri	t, yarı i taneli, sert,	000						
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- 25											°. Ö.						
		25.50							· .		°.o.°						
26	SPT-17	25.95	28	26	15	41	41	17	25.80 m		ŏ٥،						
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							·	MZA Sian	Jeoloji Muh.	Jeotekn	ik Müh.			Jeo	ioji M	uh.	-

١	YÜKSEL PROJE YÜKSEL PROJE ULUSLARARASI A.S. Birlik Mahallesi 9. Cadde Nox1 06610 ÇANKAYAANIKARA SONDAJ LOGU / BORING LOG																
YÚ Bit Ođ	ÜKSEL PR fiik Mahalie 610 ÇANIE	OJE ULU Isi 9. Cad (AYA-AN)	SLAR/ de No: KARA	ARASI 41	AŞ.		SONDALLOG	U/80	BING LOG		SONDA	J.					
TE	L: (312) 4 ww.yukseip	95 70 00 l roje.com.	FAX: (3 tr	12) 49	95 70 2	4	0010200 200				Borehol	e	NO :			-21	
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nLlG n	NSI		DA	RBE	SAY	ISI	GRAFIK		JEOTEKNİK TANIML	AMA		UStre	ather	e (30	D'L		
DER pth (ype ype	¥ ≣	Nu	mb. c	of Blo	ws	Graph		Geotechnical Descrip	otion			N/	ractur	L L		
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26											F						
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- 27		27.00									E						
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-		27.45						Kızılıma	a kahverengi, sert, çakıl	lı kumlu killi	F÷2						
- 28								SILT / s	siltli KİL. Nemli, düşük-or	ta a taneli	E						
								kumiu;	%5-10 ince-orta taneli, s	ert, yuvarlak	10:0						
- 1		28.50						çakılır.			Q						
- 29	SP1-19	28.95	14	19	27	46		Birim ki	reç konkresyonları içerir.		·						1
23								(ANKA	RA KILI)		E	1					
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		20.00									- <u>-</u> °						
- 30	SPT-20	30.00	12	18	29	47					<u> </u>					1	
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Not : K	uyuya ve	raltisuv	u göz	emle	ri icin	29.0	0 m, Ø50 mm.	ISIM	Logged By Baris HASANCEBI	Dr. Erh	n TIMUR	Check	ked	Mer	at Ci	LSAM	N
pe	erfore bo	ru indiri	lip, 40	x40x	15 cn	n. ku	u ağzı betonu yapılmıştır.	Name	Jeoloji Müh.	Jeotek	nik Müh.		_	Je	oloji I	Müh.	
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C.11. Borehole TA-23

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Bi Di	ÜKSEL PF irlik Mahali 8610 ÇANI EL: (312) 4	ROJE ULU esi 9. Cad KAYA-AN 195 70 00	ISLAR Ide No KARA FAX: (ARASI	A.Ş. 95 70 2	4	5	SONDAJ LOO	GU/B	ORI	ING LOG			SON	DAJ	No :		TA	-23	_
w	ww.yukselp	proje.com.	tr											Boreh	A	Nex				-
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SOND	ALI YER	/ Boring	loca	ation		: U	LUS-KEÇIOREN	METRO HATTI	DELIK	ÇAF	PI / Hole Diar	neter	: +	W (11	4 mn	n)	151	_		_
KILOM	ETRE /	Chainag	e			: 01	868		MUH.E	OR.	DER. / Casir	na Dept	h : 4	.50m F	W. 2	4.200) n (N	W)		-
SOND	AJ DER.	/ Boring	Dep	th		: 30).45 m		BAŞ.B	IT.T/	AR. / Start Fi	nish Da	ate : 1	7.03.2	005 /	18.03	3.200	5	-	-
SOND	AJ KOTL	/ Eleva	tion			: 84	6.68 m		KOOR	DIN/	AT / Coordina	ate (N-S	S)x :4	423 6	19.04		_			
SOND	AJ MAK. T	&YONT.	/D.R	ig & N	let.	: Fo	premost Mobile /	Rotary	KOOR	DİNA	AT / Coordina	ate (E-V	N)y : 4	86 892	.21	_	_			_
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pth (c	ype	≴ 5	NL	mb. d	of Blo	ws	Gra	aph	1	Ge	otechnical	Descri	ption	i	ILK.	Ve	ctrie	CR)		
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0 1 2 3 4 5 - 6	SPT-1 SPT-2 SPT-3	1.50 1.95 3.00 3.45 4.50 4.95	2	5	8	13 10 18			YAPAN 0.00-0. Killi sitt Kahver yan yu plastisi Kahver sittli ku %10-20 kumlu, (Tanga	r DO 25 B li kur varla teli ir varla teli ir nlu (n yu) inco eser	DLGU MALZE Jeton mlu ÇAKIL. 3.70 imsi gri renkli li, ince-orta ta k, eser orana ce malzeme 4.75 msi gri renkli ÇAKIL. Nemi varlak-yuvar e-iri taneli, se r oranda ince 5.70	D m	: bk, killi sitti rta-sert, ik ikı, killi ince-orta, yuvarlak meli,		10 0 0 04					
Ď	AYANIN	ILILIK /	Stre	ngth			AYRIŞMA / We	athering	(ranin	INC	E DANELI / F	II.) Fine Gr	ained	· iRi		ELI/C	oars	e G	aine	d
i U	DAYANIA ORTA DA		1	Strong M.Stro	9000	1	TAZE	Fresh	N :	0-2	ÇOK YUMU	ŞAK	V.Soft	N : ()-4	ÇOK	GEV	ŞEK	V.Lo	os
ш	ORTA DAYANIMLI M.Strong ORTA ZAYIF M.Weak						ORTA D. AYR	Mod. Weath.	N:	3-4 5-8	ORTA KATI		Soft M.Stiff	N: 1	5-10 11-30	GEV ORT	ŞEK A SIR	a	Loos M.D	e en
v	V ZAYIF Weak / ÇOK ZAYIF V.Weak						ÇOK AYR. TÜMÜYLE A	Highly W. Comp.Weat	N: N·	9-15	KATI		Stiff	N : :	31-50	SIKI	SIK		Den	se
		TEOLT	ALL ST	PC	_		DIKI AD		N :	>30	SERT		Hard	<u> </u>	- 30		on		v.D8	81
% 0-25	ÇOK.	ZAYIF	V.I	- RQ Poor	-	1	SEYREK	Wide (W)	% 5		PEKAZ	ORA	NLAR - Prop	portion %	s	PEK	A7		Sligh	
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% 75-90	iYi	`	Go	nod .		10-	20 ÇOK SIKI	lintense (I)	% 15	-35	ÇOK VE	Ve	ary nd	%	20-50	ÇOK			Very	
% 90-10 SPT	0 ÇOK Standart	Penetras	Ex	cellent esti		>2	PARÇALI	Crushed (Cr)			1001111	~~~~					_	_		
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P	Disturbed	is Numu sample	ne			Р	Pressiyometr	e Deneyi ar Test	ISIM Name	1	Bariş HASANÇ	EBI	Dr. Erhar	n TIMUI	2	T	Mura	t ÇİL	SAN	
UD	Örselenm	emiş Nu	nune			٧Ş	Veyn Deneyi	. (051	MZA	-	Jeoloji Müh		Jeotekn	IK MÜħ.		+	Jeo	юј: М	uh.	
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- 10		16.50							16.30 m -		E		l				
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- 18		18.00			ļ				17.80 m								
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								yuvarla	k kumlu; %5-10 düşük p alzemeli.	olastisiteli,	°,o°						
- 21	SPT-14	21.00	25	35	23	58					ိုလိ						
-		21.45					58;				0.0						
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	SPT-15	22.50	32	25	13	38	38 •		22.70 m		000						
- 23		22.95	-	_				Kahver	engi, sert, cakıllı kumlu	killi SİLT /	<u> </u>						
								siltli KİL	Nemli, düşük-orta plas	stisiteli;							
								kumlu;	eser oranda ince taneli,	yer yer orta-							
- 24	SPT-16	24.00	14	19	27	46	and products of the sector of	sen, ya	n yuvanak çakılır.								
-		24.45	14		~			Birim yı	oğun olarak kireç konkre	esyonu içerir.							
- 26							Anna Anna Anna Anna Anna Anna Anna Anna	25.50-2 seviyeli	5.65 m. arası killî siltli k	umlu ÇAKIL							
20								ANKA	RA KILL)		E						
-	SPT-17	25.50	14	18	23	41		1.111			E						
26		25.95		10	2.5				1001194844								
								ISIM	LOGU TAPAN Logged By	Dr. Etho	C	heck	ed	Marrie	- Cit	CAN	
								Name	Jeoloji Müh.	Jeotekn	ik Müh.	_		Jeo	ioji N	lüh.	
								Sign									



C.12. Borehole TA-24

TURES, PROJ. ULUB, ANARASIA, S. Bins Manuala, Cades Not 1 DBC PANOLY ADDXAD. SONDAJ LOGU / BORING LOG SONDAJ VERSI PROJ. NUMERALING, SONDAJ USAN DE NUMERALING, SONDAJ VERSI (10), 483 102,	١	(Ü)	KS	E		PF	30	JE													
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PROJE ADJ / Project Name : UUS-KEC/OREN METRO HATT DELK CAPI / Hole Diameter : WV (9 mm) SONDAJ VER // Exclusion (Consumption) : ASKI TANDOGAN ARASI YERULTI SUM (Gold Consumption) : ASKI TANDOGAN ARASI YERULTI SUM (Gold Consumption) : ASKI TANDOGAN ARASI SUNDAJ VER // Formg Depth : 30.05 / 27 03 2005 : ASKI TANDOGAN ARASI YERULTI SUM (Gold Consumption) : ASKI TANDOGAN ARASI : ASKI TANDOGAN ARASI : YERULTI SUM (Gold Consumption) : ASKI TANDOGAN ARASI : YERULTI SUM (Gold Consumption) : ASKI TANDOGAN ARASI : YERULTI SUM (Gold Consumption) : ASKI TANDOGAN ARASI		.yukuu	noje.com	•											SAYFA		No :		1	/4	
SONDAU VER/I / Boring Location : A 84/1 TANDOGAN ARASIS VERALT SLV/L (Snumwater : 6 4 4 no. (21 04 2005) SONDAU MAX ALDER / Joring Deph : 30 4 5 m BAS BIT TAR. / Stan Finish Date : 25 03 2005 (27 03 2005) SONDAU MAX AVONT /D Rig & Met. : B45 4 B m KOORDINA / Coordinate (N-5) x : 44 20 (23 04 20 04) SONDAU MAX AVONT /D Rig & Met. : Costalia D-750 / Rotary KOORDINA / Coordinate (N-5) x : 44 20 (23 04 20 04) SONDAU MAX AVONT /D Rig & Met. : Costalia D-750 / Rotary KOORDINA / Coordinate (N-5) x : 44 20 (23 04 04) SONDAU MAX AVONT /D Rig & Met. : Standart Prentation Test JEOTEXNIK TANIMLAMA Geotechnical Description If git git git git git git git git git git	PROJE	ADI / P	roject N	ame			: UI	US-KEÇİÖREN M	METRO HATTI	DELİK	ÇAPI	/ Hole Diam	eter	: 1	W (89	mm)			_	-
NLCMETER / Chainage : 0 / 781 MULH BOR DER. / Casing Depth : 3 0.0 m (WV) SONDAU DER. / Deing Depth : 30.45 m BAS BIT 7.AT. / Stan Finish David PMS) x: 4 423 610 23 SONDAU DER. / Stan Finish David PMS) x: 4 423 610 23 SONDAU KOTU / Elevation : 58.45 m KOORDINAT / Coordinate (N-5) x: 4 423 610 23 SONDAU MAK AVONT, D.Rig & Mut. Sondau MAK AVONT, D.Rig & Mut. Sondau MAK AVONT, D.Rig & Mut. Sondau MAK AVONT, D.Rig & Mut. Sondau MAK AVONT, D.Rig & Mut. Sondau MAK AVONT, D.Rig & Mut. Sondau MAK AVONT, D.Rig & Mut. Sondau PAR AVISI & GRAPIK Sondau PAR AVISI & GRAPIK Sondau PAR AVISI & GRAPIK Sondau PAR AVISI & GRAPIK Sondau PAR AVISI & GRAPIK Sondau PAR AVISI & GRAPIK JEOTEKNIK TANIMLAMA & Geotechnical Description Jeo	SONDA	J YERİ	/ Boring	Loca	ation		: A\$	SKI-TANDOĞAN	ARASI	YERAL	TIS	JYU / Groun	dwater	: 5	5.40 m. (21.0	4.20	05)	-		_
SCNDAU DER / Foring Depth : 30.45 m BAS_81T,TAR. / Start Fnish Date : 25.00.2006 / 27.03.2005 SCNDAU KOTU Levisition : 84.54 m KCORDINAT / Coordinate (E-W) y : 44.25 E0.3.2006 / 27.03.2005 SCNDAU KOTU Levisition : 84.54 m KCORDINAT / Coordinate (E-W) y : 44.25 E0.3.2006 / 27.03.2005 SCNDAU KOTU Levisition : 84.54 m KCORDINAT / Coordinate (E-W) y : 44.65 m/ SCNDAU KOTU Levisition : 84.54 m Coordinate (E-W) y : 44.65 m/ SCNDAU KOTU Levisition : 84.54 m/ Coordinate (E-W) y : 44.65 m/ SPT-1 : 50 : 57 23 24 10 20 30 60 : 50 - 2 : 1.50 : 27 23 24 47 : 11	KİLOM	ETRE / (Chainag	e			: 0+	781		MUH.E	BOR.D	ER. / Casing	g Depth	1 : 3	8.00m (N	W)					
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- 7								düşük p taneli kı	ilastisiteli, %15-20 ince-ç umlu.	ok ince	<u>°</u>				· .		
_		7.50							7.30 m —		0.00		1				
	SPT-5	1.00	8	11	17	28	28.	Gri renk Nemli-I	di, orta sıkı, killi siltli kum slak, ince-iri taneli, sert, y	ılu ÇAKIL. varı vuvarlak	000						
- 8		7.95						köşeli;	%20-30 ince-iri taneli, se	rt, yarı plastisiteli	000						
-								ince ma	Izemeli.	plustishen,	0.0	ł –					
- 9		9.00							8.80 m		0.0						
	SPT-6		14	5	10	15					÷ <u> </u>						
-		9.45									F±-						
- 10		10.00						Gri renk SILT / s	di, katı-çok katı, çakıllı k siltli KİL. Nemli, orta-yük:	umlu killi sek	a						
-	UD-2	10.50						plastisit kumlu:	eli; %10-20 çok ince-inc %5-10 ince taneli, sert, y	e taneli yarı yuvarlak							
	SPT-7	10.00	4	6	8	14	14	çakıllı.		,,							
- 11		10.95									5						
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- 12		12.00						<u> </u>	11.90 m —								
	SPT-8		24	24	17	41					000						
-		12.45						ÇAKIL.	di, sıkı-çok sıkı, kıllı sıltlı İslak, ince-iri tanelî, seri	kumlu t, yarı	20						
- 13								yuvarla sert, ya	k-yuvarlak; %20-30 ince n yuvarlak kumlu; eser o	-iri taneli, oranda	000						
		13,50				ĺ		düşük p	astisiteli, ince malzeme	eli.	<u> </u>						
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ŀ		14.50						Grimsi	kahverenkli, cok katu ca	kıllı kumlu	E-1						
- 15	00-3	15.00				1		siltli Kil	Nemli, düşük-orta plas	stisiteli;	⊡—́⊇						
	SPT-10	15.45	7	9	12	21	21 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	ince tar	neli, sert, yarı yuvarlak ç	akıllı.	E				Ľ		.
16		10.40															
					-				LOGU YAPAN Logged By		ĸ	ONTI Check	ROL	-		·	-
								ISIM Name	Barış HASANÇEBİ Jeoloji Müh.	Dr. Erha Jeotek	an TÌMUR nik Müh.		Γ	Mur Je	at Çi oloji)	LSAN Müh.	1
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- 18		18 00						ince ta	neli, sert, yarı yuvarlak ç	çakılır.	<u></u>						
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-		18.45						ļ	18.70 m								
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- 21		21.00						(ALŪV	YON)		000						
	SPT-14	21.45	28	26	27	43	43	1			<u> </u>						
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- 23	5-1-13	22.95	.,	13	19	34											
								SILT /	sı kahverenkli, sert, çak siltli KİL. Nemli, düşük-o	ıllı kumlu killi orta-yer yer							
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- 24	SPT-16	24.00	21	18	23	31		ince-ye çakıllı.	r yer orta-iri taneli, yarı y	yuvartak							
		24.45					31	23.80-2	4.70m, ve 29.70-30.18	m, arası killi	60						
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26	SPT-17	25.95	18	26	37	63	63										
									LOGU YAPAN		KC	NTR					-
								ISIM Name	Barış HASANÇEBİ Jeoloji Müh.	Dr. Erhar	TIMUR			Mura	t ÇİL bii M	SAN	-
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C.13. Borehole TA-25

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SOND	UKOTI	/ Boring	tion	th		25	45 m		BAŞ.B		AR. / Start Fin	ish Da	ate : (01.04.20	05/0	02.04	1.200)5		
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L H	DAYANIA			Strong	2	1	TAZE	Fresh	N :	0-2	ÇOK YUMUŞ	AK	V.Soft	N : 0-	4	ÇOK	GEV	ŞEK	V.Lo	os
н	ORTA ZA	YIF		M.Suc M.We	ak	141	ORTA D. AYR.	Slightly W. Mod. Weath.	• N :	3-4 5-8	YUMUŞAK ORTA KATI		Soft M.Stiff	N: 5-	10 -30	GEV	ŞEK	a	Loos	en
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10	Disturbed sample						Pressuremete	r Test	Name	Ľ	Jeoloji Müh.		Jeotekr	nik Müh.			Jeo	loji M	aAN üh.	
00	Örselenmemiş Numune Undisturbed Sample					VS	Veyn Deneyi Vane Sheer T	est	IMZA Sign						-					

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	62	9.90						yer yük taneli k	sek plastisiteli; %20-25 (umlu; eser oranda ince t	cok ince-ince taneli, sert,	<u></u>						
- 10	10.2	10.00						yan yuv	varlak-yuvarlak çakıllı.								
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- 18	SPT-12	18.00	7	9	11	20	 Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Materia Antonio Mate	taneli,	sert, yarı yuvarlak-yuvar	lak çakıllı.							
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- 24		24.00						yan yu yan yu	varlak kumlu; %3-5 ince varlak çakıllı.	taneli,sert,	<u>a a</u>						
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ŗ		24.45									<u> </u>						
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perfore boru indirilip, 40x40x15 cm. kuyu ağzı betonu yapılmıştır.								Name	Barış HASANÇEBI Jeoloji Müh.	Dr. Erhar Jeotekn	ik Müh.			Mura Jeo	t ÇİL I	SAN üh.	
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C.14. Borehole UK-7

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	m.yuxoop	operation in a											· ·	SAYFA Page	٨	io:		1/	4	
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SONDA	J DER.	Boring	Dept	h		: 28	3.95 m		BAŞ.BİT	TAR	. / Start Finish	Date	: 11	1.09.20	003	/ 13	.09	200	13	_
SONDA	JKOTU	/ Eleval	tion		- 1	: 84	19.298 m.	ABY	KOORD	INAT	/ Coordinate ((N-S))	: 40	122 00	145	M				-
ŞONDA	J MAK.8	SYONT.	D.R	9 & M 57	et.		DENETRASYON DENEY		KOORD	INAT	/ Coordinate ((E-VV)	×	423 90	0.30	Î.		αċ		
				0,	SI	anda	Int Penetration Test								engt	sring	(m)	Core		
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th (n	N C	∢ ⊆	Nu	mb. o	f Blov	vs	Graph			Geot	echnical De	scripti	on		Ē	Ň	actur	TCR		
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- 5		4.95							orta, se	ert; %	15-20 ince-	iri, se	rt-az sert,	· .						
									yarı yu	varla	k-yarı köşel	i çakı	llı; %5-10							
ŀ									ince ma	alzen	5.60 m			0.9	4					
6									(Tanin	nı Sa	yfa 2/4 dedir	r.)								
C	AYANIN	ILILIK /	Stre	ngth		_	AYRIŞMA / Weatheri	ng		NCE	DANELÍ / Fin	e Gra	ned	ÎRÎ (DAN	ELI/C	Coan	se G	rain	ed
1	DAYANI	MLI		Stron	9	1	TAZE Free	sh	N :	0-2	ÇOK YUMUŞAI	K V	Soft	N: 0	4	ÇOI	GE	SEK	V.L	oos
111	ORTA Z	1	M.Str M.We	ong sak	11	AZ AYRIŞMIŞ Slig ORTA D. AYR. Mos	htly W. d. Weath.	N: N:	3-4 5-8	YUMUŞAK ORTA KATI	N N	I.Stiff	N: 5	-10 1-30	OR	TA SI	ĸ	M.E	Se Den	
īV	ORTA ZAYIF M.we ZAYIF Weak					iv	ÇOKAYR. Higi	hly W.	N :	9-15	KATI	s	tiff	N: 3	1-50	SIK	1		Der	ise
v	ÇOK ZA	YIF		V.We	ak	v	TÜMÜYLE A. Con	mp.Weat.	N:	16-30 >30	ÇOK KATI SERT	v t	.Stiff lard	N: >	50	ço	K SIK	1	V.D	en
KA	A KALI	TESI T/	ANIM	- RC	D	۲	(IRIKLAR - 30 cm / Frac	ctures		- 00	(ORAN	LAR - Pro	portion	\$					_
% 0-25	ÇOK	ZAYIF	V.	Poor		1	SEYREK Wide (V	N)	% 5		PEK AZ	Sligh	ntiy	% 5	20	PER	(AZ		Siig	hti
% 25-50 % 50-75	ORT	A	Fa	or Úr		2-	2 OKTA Modera 10 SIK Close (Cl)	% 15	-35	ÇOK	Ven	,	96 2	0-50	ço	ĸ		Ver	y
% 75-90	lYi		G	bod		10	-20 ÇOK SIKI Intense	(1)	% 35		VE	And								
% 90-10 SPT	0 ÇOK	Penetras	E)	cellen esti	t	>2	0 PARÇALI Crusher Karot Numunesi	d (Cr)			LOGU YAPAN			L	ONT	ROL			-	_
3-1	Standart	Penetrat	ion Te	st		r.	Core Sample				Logged By				Chec	ked				
D	Örselenn	niş Numu	ne			Ρ	Pressiyometre Dene	iyi	ISIM		Özgür AVŞAR		Dr. Erha	n TIMUR	ł		Fatih	KAF	ACA	N
UD	Disturbed sample Örselenmemis Numune					V	Pressuremeter Test Veyn Deneyi		MZA		Jeoloji Mult.		Jeoleki	ak Muñ.		⊢	160	TIZIK	Muñ.	_
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epth	√E C	Rur	Nu	mb.o	f Blov	NS	Graph		Geotechnical Descrip	otion		MLIL	A/V	ractu	6(TCI		z
Soring D	NUMUR Samp.	MANEV 30YU/I	0 - 15 cr	15-30 cr	30-45 cr	Ν	10 20 30 40 50 60				PROFIL	JAYANI	AYRISM	KIRIK /	KAROT	800 %	LUGEO
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		6.45	Ĩ				/ 4	iri, ser	t, yuvarlak-yarı köşe	li; %20-30							
								ince-iri,	, sert çakıllı. 6,90 m		0.00						
- 7								Yeşilim	si gri renkli, kati k	umlu KİL	3-2						
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-9	SPT-6	9.00	7	1	3	4			9.10 m —		<u> </u>						
-		9.45											ŀ				
								Kovu	ori renkli vumusak-	kati cakili							
- 10								siltli K	dL / killi SILT. N	emli, orta-	<u> </u>				1		
-		10.50						yüksek vuvarla	c plastik; eser-%10 k-vari vuvarlak cakilli	ince, sert,							
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- 11		10.95									2-2						1
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		10.00															
- 12	SPT-8	12.00	8	15	24	39	N 89	Gri ren sert,	%15-25 ince-iri, e	ort çakıllı,	¢						
ŀ		12.45						%10-2	0 ince malzemeli.		°						
13							X				0. D						
- 13									13.10 m		<u>-</u>	1					
-		13.50						Kahve	renkli cok katı siltli	KIL. Orta	<u> </u>		1				
14	SPT-9	13.95	4	7	9	16	• 116	yüksek	plastisiteli; eser	%5 ince		ļ					1
- 14		10,00						kumlu.									
ŀ		14.50							14.50 m								
	UD-1	15.00						Sarime	a kahverenkli	orta suk							
- 15	SPT-10	10.00	6	7	7	14		siltli K	(UM. Ince, sert; %)	25-35 ince	, °						ļ
·		15.45						malzer siltli kil	neli, 3-5cm bantları icerir.	kalınlıkta	°				1		
16											••••						
									LOGU YAPAN Logged By		ĸ	ONT Chec	ROL				
								ISIM Name	Özgür AVŞAR Jeoloji Müh.	Dr. Erha Jeotek	n TIMUR nik Müh.			Fatih	KAR blizik l	ACA! Muh.	N
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YI Bi	ÜKSEL PR	OJE ULU esi 9. Cad	ISLAR	ARASI	A.Ş.						0.01/0						
TE	EL: (312) 4	195 70 00	FAX: (312) 49	95 70 2	4	SONDAJ LOO	SU / B	ORING LOG		SONDA Borehol	le l	No :		UK	(-7	
Ű	ww.yuksei	proje.com									SAYFA Page	1	No :		3/	4	
				STA	ANDA	ART P	ENETRASYON DENEYI					ngth	Bu	Ê	SreR		
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DERI apth (IE di Vpe	R P	Nu	mb. c	Blo	ws_	Graph		Geotechnical Descrip	otion		E.	A / WE	ra ctu	6(TCR		
SONDAJ Boring D	NUMUN Samp.	MANEV BOYU/	0 - 15 cr	15-30 cr	30-45 cr	2	10 20 30 40 50 60				PROFIL Profile	DAYANI	AYRIŞM.	KIRIK / F	KAROT%	ROD %	LUGEON
16								(Tanır	ni Sayfa 2/4 ' dedir.)		0.00						
ŀ	EDT 11	16.50			_	10				-ner seb	2						1.
- 17	5P1-11	16.95	4	4	Ď	10		Kahve Nemli,	orta-yüksek plastisit	eli %10-15							
	_ P1_	17.40						iri tane (Alüvo	li malzeme içermekte (on)	adir.							
									17.80 m								
- 18		18.00														1	
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- 19											2-2						
		19.50									- <u>-</u> 2						1
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- 20		19.95									<u> </u>	1.	-				
	- P2 -	20.40						Kırmız kumlu	ımsı kahverenkli, killi SİLT / siltli K	çok katı,	1-2						
- 21		21.00						yüksek	plastisiteli; %5-15	ince,sert	- <u>_</u> e						
	SPT-14		7	9	12	21		konkre	eser çakılıl. Yer ye syonları içerir.	er karbonat							
		21.45						(ANK/	ARA KILI)		1.1						
- 22											<u> </u>	1					
		22.60									1						
	SPT-15	22.50	8	10	14	24	24 0				°						
- 23		22.95									-:						
	- UD-2	23.50						Soum	23.30 m	T Nemli		1		1			
								orta-yi	iksek plastik.	LI. Nemi,	- <u>-</u>						
- 24	SPT-16	24.00	25	31	21	52			24.10 m		0	1					
ŀ		24.45															
- 25								Gri re	nkli, sıkı, çakıllı KU	M. Ince-iri,				1			
25								yarı kö	işeli, sert çakıllı; ese	r-%10 ince			1				
	SPT-17	25.50	15	10	26	45	45	malzer	neli.								
26		25.95	13	19	20						<u> </u>		RÓI				Ц
								ISIM	Logged By Ozour AVSAR	Dr Erbs		Chec	ked	Fatih	KAP	ACA	N -
								Name	Jeoloji Müh.	Jeotek	nik Müh.			Jec	fizik	Müh.	
								Sign					I				

Continuation	of	Borehole	UK-7
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Y Bi Of	ÜKSEL Pr nik Mahal 5610 ÇAN	ROJE ULL esi 9. Cad KAYA-AN	ISLAR Ide No KARA	ARAS	A.Ş.					SON		1100	SU / B			SONDA	2.1					
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epth e	VE C	RA	Nu	IMD. (of Bio	ws			G	aph			4	Geotechnical De	escription		רו	N N	ractur	TCR		
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26							4.02	1111	111	1		1111	1			· · ·		Ē	1	1	-	Ħ
-							1410	1111		111		111.	(Tan	mi Sayfa 3/4 ' dec	lir.)	·						
- 27		27.00						1111					-	26.80 r	n							
~	SPT-18	27.00	15	21	25	46	1111	1115	1111			46	1			· - ·						
-		27.45					1111	1112	(11)	111		1111	Kırmı	imsi kahverenk	li-bordo, sert,	·;						
- 28	- P3 -	27.90							1111		+		vükse	sitti KIL / Killi Si k plastik: %5-1	T. Nemli, orta- ince-iri, ser	·						
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-		28.50						111-		111		iiii	KONK	esyonları içerir.								
- 20	SPT-19	28.95	13	17	21	38					38					· _ ·	{					
- 29							1111		1111		1113	111		KUYU SONU : 2	8.95 mt.							
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- 30							1 1 1	1111	111	1111			7 50	•								
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and the second sec										Name	Jeoloji Müh,	Jeotekr	ik Müh.		,	Jeof	izik N	lüh.				
													IMZA Sign									

C.15. Borehole UK-8

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YŪ Bir 06	IKSEL PR lik Mahalle 610 ÇANK	OJE ULU Isi 9. Cadi	SLAR de No: KARA	ARASI 41	A.Ş.			SONDAJ LOG	iU / BC	RIN	IG LOG		1	SONDA	J.,	No :		UK	(-8	_
w	w.yukselp	roje.com.	tr tr	512) 48	5 /0 2	•								Borehol SAYFA	e I	No :		1/	3	_
PROJE		niect Ni	me			: 11	US-KECIÓREN	METRO HATTI			/ Hole Diamet	er	: 20	00 mm	-					-
SONDA	J YERI	/ Boring	Loca	tion		: K/	ZIM KARABEK	R CADDESI	YERAL	TI SU	JYU / Groundw	ater	: 4.	80 m ((16	.02.	200	4)		-
KILOM	ETRE / C	Chainag	e			: 1-	+405		MUH.B	OR.D	ER. / Casing D	Depth	: 25	5.50 m	1			,		
SONDA	J DER.	/ Boring	Dept	th		: 25	5.95 m		BAŞ.Bİ	T.TA	R. / Start Finish	n Dat	e : 09	9.09.20	003	/11	1.09	.20	3	
SONDA	и коти	/ Eleva	tion			: 84	19.967 m.		KOOR	DINA	T / Coordinate	(N-S)	y :48	37 298.	945		_			
SONDA	J MAK.	SYÖNT.	/D.Ri	g & N	let.	: M	D B53-AUGER	/ ROTARY	KOORD	DINA'	T / Coordinate	(E-W)x :4	423 99	4.20	7				
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in the last	JA C	¥5	Nu	mb. a	of Blov	VS .	Gr	aph	4	Geo	technical De	scrip	tion		E	5	acti	Ę		
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1	DAYANI			Stron	g	1	TAZE	Fresh Sliphthy M	N:	0-2	ÇOK YUMUŞA	ĸ	V.Soft	N: 0-	4	COL	GE	/ŞEK	V.L	2005
	ORTA Z	AYIF		M.Str	eak	IB	ORTA D. AY	R. Mod. Weath.	N:	5-8	ORTA KATI		M.Stiff	N: 11	1-30	OR	TA SI	ĸ	M.C	se)en
IV	ZAYIF			Weal	ĸ	IV	ÇOK AYR.	Highly W.	N:	9-15	KATI		Stiff	N : 31	1-50	SIK	1		Der	se
v	ÇOK ZA	YIF		V.We	ak	· ·	TUMUYLE A	. Comp.Weat.	N: N·	16-30 ≥30	ÇOK KATI SERT		V.Stiff Hard	N: >	50	çoi	K SIK	3	V.D	en
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% 0-25	ÇOK	ZAYIF	V.	Poor		1	SEYREK	Wide (W)	% 5		PEK AZ	Sk	ahtly	% 5		PEN	(AZ		Slig	hti
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% 75-90	iYi		G	bod		10	-20 ÇOK SIKI	Intense (I)	% 35		VE	An	d			,				
% 90-10 SPT	0 ÇOK Standart	1Yi Penetras		cellen esti	t	>2	0 PARÇALI Karot Numu	Crushed (Cr)			LOGU YAPAN				ONT	ROI				
or i	Standart	Penetrat	ion Te	st		n.	Core Sampl	e			Logged By				Check	ked				
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						Ì		orta yer	yer yüksek plastisiteli;	%10-20 çok							
								sert, ya	ri yuvarlak, yer yer orta,	dağılgan-							
- 21	SPT-14	21.00	24	30	32	62		orta ser olarak i	r, yarı yuvarlak çakılır; E tireç konkresyonları içer	ir.							
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- 22											0-0						
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- 23		22.95									0						
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								Name	Jeoloji Müh.	Jeotekr	nik Müh.			Jeo	loji M	üh.	_
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Continuation of Borehole UK-12A

Continuation of Borehole UK-12A

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w	ww.yukselp	xroje.com.	tr													SAYFA Page	1	No :		4/	4	
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ph (r	E CIN	¥ 5	Nu	mb. (of Blo	ws			Gr	aph				Geotechnical Descri	ption		LLK.	/ Me	actur	(TCR)		
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	boru	indirilip,	kuyu	ağzı	betor	nu yap	oilmiş	tır.					ISIM Name	Barış HASANÇEBİ Jeoloji Müh.	Dr. Erh: Jeotek	an TIMUR mik Müh.			Mura	at ÇIL Noji M	SAN	÷
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C.17. Borehole UK-18A-1

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	YÜKSEL PR Birlik Mahali	ROJE ULL esi 9. Cad	ISLAR	ARASI	A.Ş.															
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	www.yukseij	proje.com	.tr											SAYFA		No :		1	/4	
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-		4.50							sert, ya	ri yuvi	arlak; %20-3 ari yuwarlak	30 ince	-orta-iri							
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	ORTA D	AYANIML AYIF	.1	M.Str M.We	ong Jak	81 211	AZ AYRIŞMIŞ ORTA D. AYR	Slightly W.	N:	3-4	YUMUŞAK		Soft	N : 5-	10	GEV	ŞEK		Loos	e
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~	ÇOK ZA	YIF		V.We	ak	v	TUMUYLE A.	Comp.Weat.	N: N:	16-30 >30	ÇOK KATI SERT		V.Stiff Hard	N : >5	0	ÇOK	SIK		V.De	n
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UD	Örselenm	i sample nemis Nu	mune			VS	Pressuremet Veyn Denevi	er Test	Name		Jeoloji Müh.		Jeotek	nik Müh.		-	Jeo	loji M	üh.	_
	Undisturt	ed Samp	ie			-9	Vane Shear	Test	Sign											

Continuation of Borehole UK-18A-1

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	ww.yukseij	proje.com	.e								SAYFA	N N	No :		2	/4	
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m In	NSI		DA	ARBE	SAY	ISI	GRAFIK	1	JEOTEKNÍK TANIM	LAMA		Stren	atheri	e (30c	01.Co		
J DER	Type Type	RA Run	Nu	mb. c	of Blo	ws	Graph	ł	Geotechnical Descri	iption		VLIK	A/ We	ractur	TCR.		
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6	SPT-4	6.00	6	8	7	15	. 15	Açık gi killi SİL	rimsi kahverengi, orta ka T. Nemli, düsük plastisi	iti, kumlu teli:	<u> </u>	-					
r i		6.40						%20-2	5, çok ince-ince taneli ku	imiu.							
- 7								Grimsi	kahverengi, orta sıkı, ki	lli siltli sert vari	0						
-		7.50						yuvarla	ak; eser oranda düşük pl	astisiteli,	1	1					
	SPT-5	7.05	4	2	4	6			7.20 m		<u> </u> °	1					
- 8		7.95						Gri ren	kli, gevşek-orta sıkı, çak	ullı killi siltli	0						
-								yarı yu	slak, çok ince-ince-orta varlak; %20-25 düşük pl	taneli, sert, astisiteli,	0						
- 9		9.00						ince m sert, vi	alzemeli; eser oranda in Ivarlak cakıllı.	ce taneli,	0						
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		9.45						-			000						
- 10								Kahver	rengimsi gri renkli, orta s	akı, killi siltli	00						
		10.50						kumlu yuvarla	ÇAKIL. Islak, ince-orta-i R çakıllı; %20-25 ince-o	ri taneli, sert, rta-iri taneli,	60						
	SPT-7		4	15	14	29	29 .	sert, ya	in yuvarlak kumlu; eser	oranda	· · · O					1	
- 11		10.95									000			1			
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12		12.00						}	11.70 m		÷						
- 12	SPT-8	12.00	2	4	4	8		1									
ŀ		12.45						Gri ren	kli, gevşek, çakıllı killi sil	tli KUM.	0						
- 13								plastisi	teli, ince malzemeli; ese	r oranda ince	00						
								taneli, s	sert, yuvarlak çakıllı.								
	SPT-9	13.50	5	7	14	21	21				0						
- 14		13.95						Kahver	engi, orta sıkı, killi siltli k	(UM. Islak,	- 0						
								ince-or oranda	ta taneli, sert, yarı yuvar düşük plastisiteli, voğur	lak; eser ica ince	<u> </u>						
							1	malzen	neli 14.80 m		- 0						
- 15	SPT-10	15.00	4	5	6	11		Gri reni Nemli-i	kli, orta sıkı, çakıllı killi s slak, çok ince-ince tane	illi KUM. II: %30-40	-						
ŀ		15.45						düşük j	plastisiteli, ince malzeme	eli; eser	0		1				
16								çakıllı.	and taries, sett, yett yu	anan-Auşeli							
									LOGU YAPAN Logged By		K	ONTR	ROL				
								ISIM Name	Barış HASANÇEBİ Jeoloji Müh.	Dr. Erhan Jeotekn	n TÍMUR ik Müh.			Mura Jec	at ÇİL Moji M	.SAN Iüh.	1
								IMZA									

YÜKSEL PROJE YÜKSEL PROJE ULUSLARARASI A.Ş. Birlik Mahallesi 9. Cadde No:41 06610 ÇANKAYA-ANKARA TEL: (312) 495 70 00 FAX: (312) 495 70 24 SONDAJ LOGU / BORING LOG SONDAJ No : UK-18A-1 Borehole SAYFA w.yukseloroje.com.tr No 3/4 age STANDART PENETRASYON DENEYI DAYANIMLILIK/Strength CoreR. AYRIŞMA / Weathering KIRIK / Fracture (30cm) Standart Penetration Test SONDAJ DERİNLİĞİ Boring Depth (m) NUMUNE CINSI Samp. Type MANEVRA BOYU/Run JEOTEKNÍK TANIMLAMA DARBE SAYISI GRAFIK KAROT%(TCR)/T. Numb, of Blows Graph Geotechnical Description Ę LUGEON 0 - 15 cm 5 PROFIL Profile RQD % Ν 15-30 30-45 10 20 30 40 50 60 11</ 10 20 30 40 50 60 16.00 0 16 UD-1 16.50 |...|0||.|. SPT-11 4 5 8 13 16.95 17 0 °0 D 18 18.00 SPT-12 7 8 12 20 18.45 - 19 0 19.50 <u>50</u> 9 Image: Second lv SPT-13 -V -R 19.59 v dst V 20 v V 9 0 K-1 v ۷ IV IV lν v v v ~ 21 21.00 V dst SPT-14 9 18 11 29 ٧ v 21.45 v ٧ 22 K-2 14 0 v dst V v 22.50 W SPT-15 20 36 48 84 v v 22.95 23 v v =/= v V dst Cr ٧ 84 19 K-3 ν v - 24 v П ĥ. CI v v 24.50 ٧ v v =/= v K-4 • 25 Cr 51 10 dst V v 25.50 W v v K-5 II П CI 77 9 26 V LOGU YAPAN KONTROL Dr. Erhan TİMUR Logged By Banş HASANÇEBİ ISIM Murat CILSAN Jeoloji Müh. Jeoteknik Müh Jeoloji Müh. IMZA

Continuation of Borehole UK-18A-1

Continuation of Borehole UK-18A-1

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th (n	e C	≤ ⊑	Nu	mb. c	f Blo	ws	Graph		Geotechnical Descri	otion		Ę	Š	cture	CR)		
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- 28								27.00-2	27.40 m, 27.90-28.30 m,		V . V	11	11	CI			
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pe	erfore bo	oru indiri	ip, 40	x40x	15 cn	n. kuy	ru ağzı betonu yapılmıştır.	Name	Jeoloji Müh.	Jeoteki	nik Müh.			Jec	loji N	lüh.	
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