GIS BASED ASSESSMENT OF EXCAVATION DIFFICULTY BY TBM - EPB ALONG MECİDİYE - TANDOĞAN SEGMENT OF THE TANDOĞAN - KEÇİÖREN METRO TUNNEL

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

BY

BENGİ ÖZBAŞ

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

SEPTEMBER 2007

Approval of the thesis:

GIS BASED ASSESSMENT OF EXCAVATION DIFFICULTY BY TBM- EPB ALONG MECİDİYE - TANDOĞAN SEGMENT OF THE TANDOĞAN - KEÇİÖREN METRO TUNNEL

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ABSTRACT

GIS BASED ASSESSMENT OF EXCAVATION DIFFICULTY BY TBM-EPB ALONG MECİDİYE - TANDOĞAN SEGMENT OF THE TANDOĞAN - KEÇİÖREN METRO TUNNEL

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September 2007, 121 pages

Tunnel structures are important investments especially for urban areas. Keçiören - Tandoğan metro alignment is one of those investments executed by Ankara Metropolitan Municipality. The purpose of this research is to evaluate Tunnel Boring Machine (Earth Pressure Balance type) (TBM-EPB) performance within different lithological units encountered along Tandoğan - Mecidiye segment of the Keçiören - Tandoğan metro tunnel. The evaluation is based on the data obtained from traditional site investigation methods, statistical approaches and Geographic Information Systems (GIS). Complex geological and hydrogeological conditions are found to be effective in the advancement of a TBM implemented in tunnel boring works. A good understanding of the geology is essential in such cases. Available field (in-situ) and laboratory tests have been used in order to determine geological, hydrogeological and geotechnical properties of the metro tunnel alignment. Advancements in the tunnel boring process also proved that hydrogeological conditions are effective on the performance of TBM so related data are considered carefully while preparing cross-section layers and calculating weights in order to display the distribution of excavation difficulty classes through the tunnel alignment.

Keywords: Keçiören - Tandoğan Metro, MCDA, GIS, TBM, excavation difficulty

ÖZ

TANDOĞAN-KEÇİÖREN METRO TÜNELİNİN MECİDİYE-TANDOĞAN KISMINDA TBM-EPB İLE KAZILABİLME GÜÇLÜĞÜNÜN CBS TABANINDA BELİRLENMESİ

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Tez Yöneticisi : Prof. Dr. Vedat DOYURAN
Eylül 2007, 121 sayfa

Kentsel yerleşim alanları için tünel yapıları önemli yatırımlardır. Tandoğan - Keçiören metro hattı Ankara Büyükşehir Belediyesi tarafından yürürlüğe koyulan bu tür yatırımlardan biridir. Bu araştırmanın amacı Keçiören-Tandoğan metro güzergahının Tandoğan - Mecidiye kesiminde geçilecek değişik litolojik birimler içinde Yer basınç Dengeli Tünel Açma Makinesinin (TBM-EPB) performansını belirlemektir. Değerlendirmeler, güzergah boyunca yapılan jeolojik etütler, sondajlar, yerinde ve laboratuar deneyleri, Coğrafi Bilgi Sistemleri (CBS) ve istatistik yöntemleri kullanılarak gerçekleştirilmiştir. Tünel kaçma işlemi sırasında karşılaşılan karmaşık jeolojik ve hidrojeolojik koşullar TBM performansını etkileyen önemli parametrelerdir. Bu durumda yerel jeolojinin çok iyi anlaşılması önemlidir. Bu nedenle tünel güzergahı boyunca jeolojik, hidrojeolojik ve jeoteknik özellikleri belirlemek amacıyla çok sayıda yerinde ve laboratuar deneyleri yapılmıştır. Tünel kazma çalışmaları sırasında hidrojeolojik koşulların da TBM performansı üzerinde önemli etkileri olduğu saptanmış bu nedenle tünel boyunca kazılabilme güçlüğü olabilecek kesimlerin gösterileceği kesitler hazırlanırken ve veri setlerinin ağırlıkları hesaplanırken bu durum dikkatle göz önünde bulundurulmuştur.

Anahtar kelimeler: Keçiören – Tandoğan Metrosu, ÇÖKV, CBS, kazılabilme güçlüğü

In the loving memory of my dear friend,

Nilay Aydın

ACKNOWLEDGEMENTS

I would like to sincerely thank my advisor, Prof. Vedat Doyuran for his patience, guidance and support in writing of this thesis. He has been inspiring me since the beginning of my studies in METU and has brightened my mind by means of geological engineering as well as view of life.

Very special thanks to my colleague Ms. Başak Şener for her kindest helps in applying statistical methods and GIS applications by sharing her experience and talent.

Many thanks to jury members for their valuable comments on improving this study and I also would like to thank to METU Geological Engineering Department staff for their interests.

Thanks are also due to Yüksel Proje International Co. for providing the data that this study is based on.

A heartfelt gratitude is conveyed to Assist. Prof. Pelin Bayındır for her continuous moral support and guidance. I also would like to thank to Mr. Mustafa Baydoğan for his ideas and help.

Finally I am deeply grateful to my family for providing me the chance to attend a graduate program and giving constant love and support. Their place is irreplaceable.

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INTRODUCTION

1.1 Purpose and Scope

Metro tunnels are important transportation structures especially for metropols. Tandoğan – Keçiören metro alignment is the extension of existing railway alignments (Ankara Metro and Ankaray) in Ankara and its construction has started in 2003 (see *Figure 1.1.*).

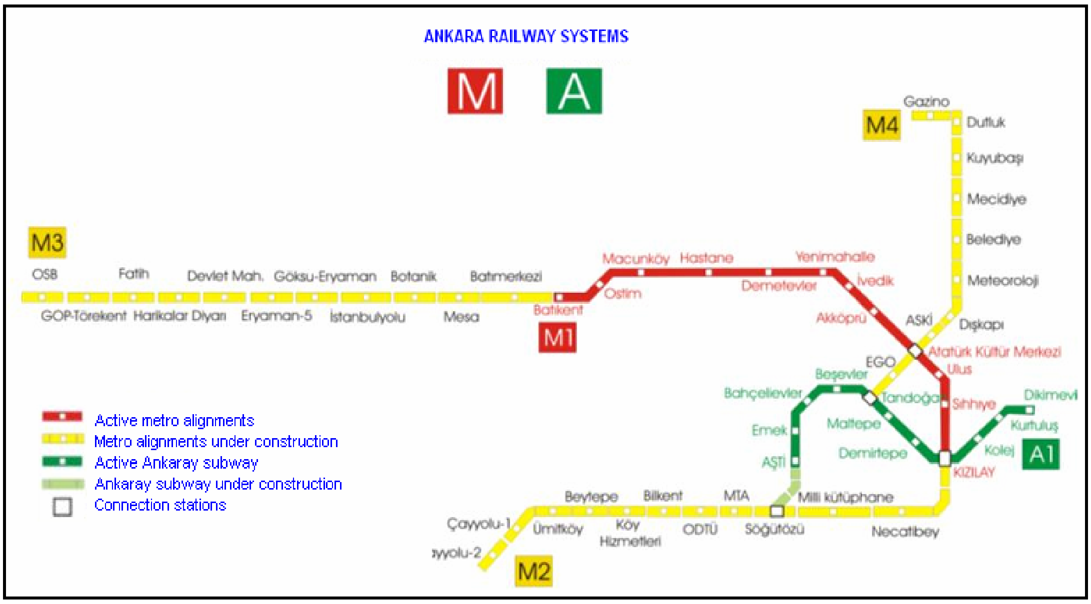

Figure 1.1. *Railway systems of Ankara under construction and in operation (www.ego.gov.tr)*

Tandoğan – Keçiören metro alignment, which is planned by the Ankara Metropolitan Municipality EGO General Directorate under the scope of the "Transportation Master Plan", has a total length of 10582 meters.

Tandoğan – Keçiören metro alignment had been studied by Yüksel Proje International Co., which involved geological-geotechnical evaluations, borings and laboratory studies.

In rock foundations tunnels were opened by drill and blast technique and a TBM was implemented to open tunnels in softer ground material. This thesis mainly focuses on the parts of the metro tunnel between Tandoğan and Mecidiye Stations where TBM is used. The purpose of this study is to determine parts of the tunnel with excavation difficulties where effect the performance of the TBM. The main issue that affects the performance of a TBM is the cutterhead of the machine. In the design phase of the TBM the site investigation results are considered and an appropriate cutterhead is implemented but the initial studies are usually not sufficient to operate the TBM with full performance. Different characteristics of the construction area may challenge the engineers while advancing in the tunnel. This study aims to find out those parts by evaluating borehole data, in-situ test results and laboratory studies' outputs and Geographic Information Systems (GIS) tools have been used in order to analyze, interpret and visualize the results. The base to create the output map in GIS environment is an AutoCAD file which contains the cross-section of the study area with details of borehole locations, lithologies, groundwater levels etc. that illustrates the locations of the data to be used in analysis.

For analyzing the complex data set in order to create an output map of the study area Multicriteria Decision Analysis (MCDA) were used. MCDA are used to deal with the difficulties that decision-makers encounter in handling large amounts of complex information. The principle of the method is to divide the decision problems into smaller more understandable parts, analyze each part separately and then integrate the parts in a logical manner (Malczewski, 1997). After analyzing the data according to MCDA the results are assigned to the related parts of the tunnel alignment to obtain the final excavation difficulty map by using TNT Mips which is a GIS program. This output map can be considered as an engineering geological map because it reflects engineering geological features of a geo-engineering construction.

Engineering geological mapping began to be developed with the first steps towards co-operation between geologists and engineers in the building of the larger engineering works such as tunnels, dams and railways. With the aid of these maps engineering geological problems can be considered much more easily and inspired from these maps different helpful sources can be prepared like it has been done in this thesis. Instead of working on an ordinary map to produce the final map a cross-section of the study area is used because a tunnel structure is practically assumed as linear.

1.2 General Information about the Metro Project

The study area is situated in the Centrum of Ankara. The topographical map quadrangle of the area is i29b1 on a 1:25000 scale map. In *Figure 1.2.* the location of the study area and route of the metro alignment can be seen.

It is planned to construct additional metro lines by the Ankara Metropolitan Municipality and some are still under construction. In *Figure 1.3* the position of the metro alignment under the scope of this study with the existing metro alignments and with the ones in planning and construction stage are illustrated. The solid lines refer to the existing metro alignments and dashed lines refer to the forthcoming lines. The dashed line in blue is the Keçiören – Tandoğan metro alignment.

Figure 1.2. *Location maps of the study area (Yüksel Proje, 2003)*

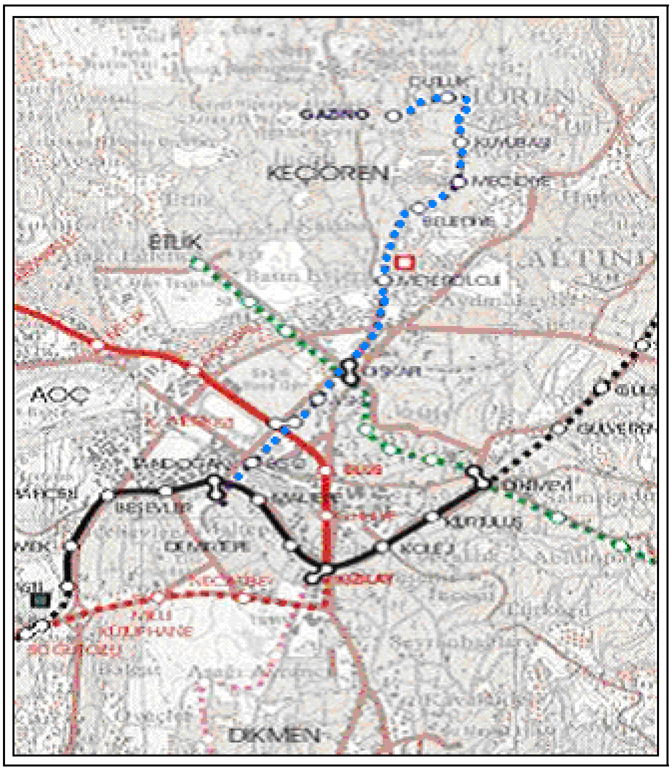

Figure 1.3. *Existing and forthcoming metro lines in Ankara and their position with the Keçiören – Tandoğan metro alignment (www.ego.gov.tr)*

Tandoğan – Keçiören metro alignment starts from Gençlik Street in Tandoğan and by intersecting the existing Ankaray and metro alignments goes through Anit Street to Kazım Karabekir Street. Then metro alignment follows Kazım Karabekir Street, Fatih Street, Kızlar Pınarı Street and leaves Kızlar Pınarı Street and passes through Gökçek Park starting from Kuyubaşı and reaches Dutluk Junction. From this point the alignment following the Nuri Pamir Street reaches the Gazino Junction and comes to an end on Aksaray Street. *Figure 1.4* illustrates the satellite image of the study area with the route of the metro alignment.

Figure 1.4. *Satellite image of the Keçiören – Tandoğan metro route*

Tandoğan-Keçiören metro alignment is a transportation corridor with a high potential of passengers. The upper parts of Keçiören have large residential areas so the demand of transportation increases day by day. The metro project is planned as to meet the demand of 46000 passengers per hour. The connections with the existing subway systems also provide the passengers to reach many other destinations. The properties of the metro alignment are given in the Tables 1.1., 1.2., 1.3 .

Table 1.1. *General features of the Keçiören – Tandoğan Metro Tunnel (www.ego.gov.tr)*

Properties of the alignment	Length
Alignment	Tandoğan - Keçiören
TBM tunnels	7232 m
Drill & blast tunnels	3358 m
Stations	1260 m
Total length of the alignment	10582 m

Table 1.2. *Applied construction techniques for each station (www.ego.gov.tr)*

Station properties	
TANDOĞAN Station	TBM
EGO Station	TBM
AKM Connecting Station	TBM
ASKİ Station	TBM
DIŞKAPI Station	TBM
METEOROLOJİ Station	TBM
BELEDİYE Station	TBM
MECİDİYE Station	TBM
KUYUBAŞI Station	Drill & Blast
DUTLUK Station	Drill & Blast
GAZİNO Station	Drill & Blast
Average station interval	1075 m
Minimum station interval	595 m
Maximum station interval	1537 m
Platform length	140 m
Platform widths	
Mid platform	11.5 m
Side platform	7 m

Table 1.3. *System properties*

System properties	
Length of the alignment	10582 m
Number of stations	11
Average station interval	1075 m
Commercial speed	42,17 km/hour
Maximum speed	80 km/hour
Daily working time	18 hours
Minimum track interval	90 seconds
Peak hours (Morning 7-9,30 Evening 16,30-19,30)	Morning 2 ,5 hours - Evening 3 hours
The ratio of passenger carried in peak hours per passenger carried daily	% 50,62

1.3 Geology of the Area

1.3.1 General Geology

The main formations observed in and around Ankara are; Dikmen formation, Alacaatlı formation, Hancılı formation, Volcanic Sequence, Akhöyük formation, Etimesgut formation and alluvial deposits. (*Figure 1.5.*)

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 Alacaatlı, Balıkuyumcu, Dereköy and Deveci villages. It also contains marl, claystone, sandstone and occasional sand – gravel layers. Along the subway route, Hancılı formation is represented by sandstone, siltstone and tuff alternations. This formation is closely associated with the volcanites of the same age.

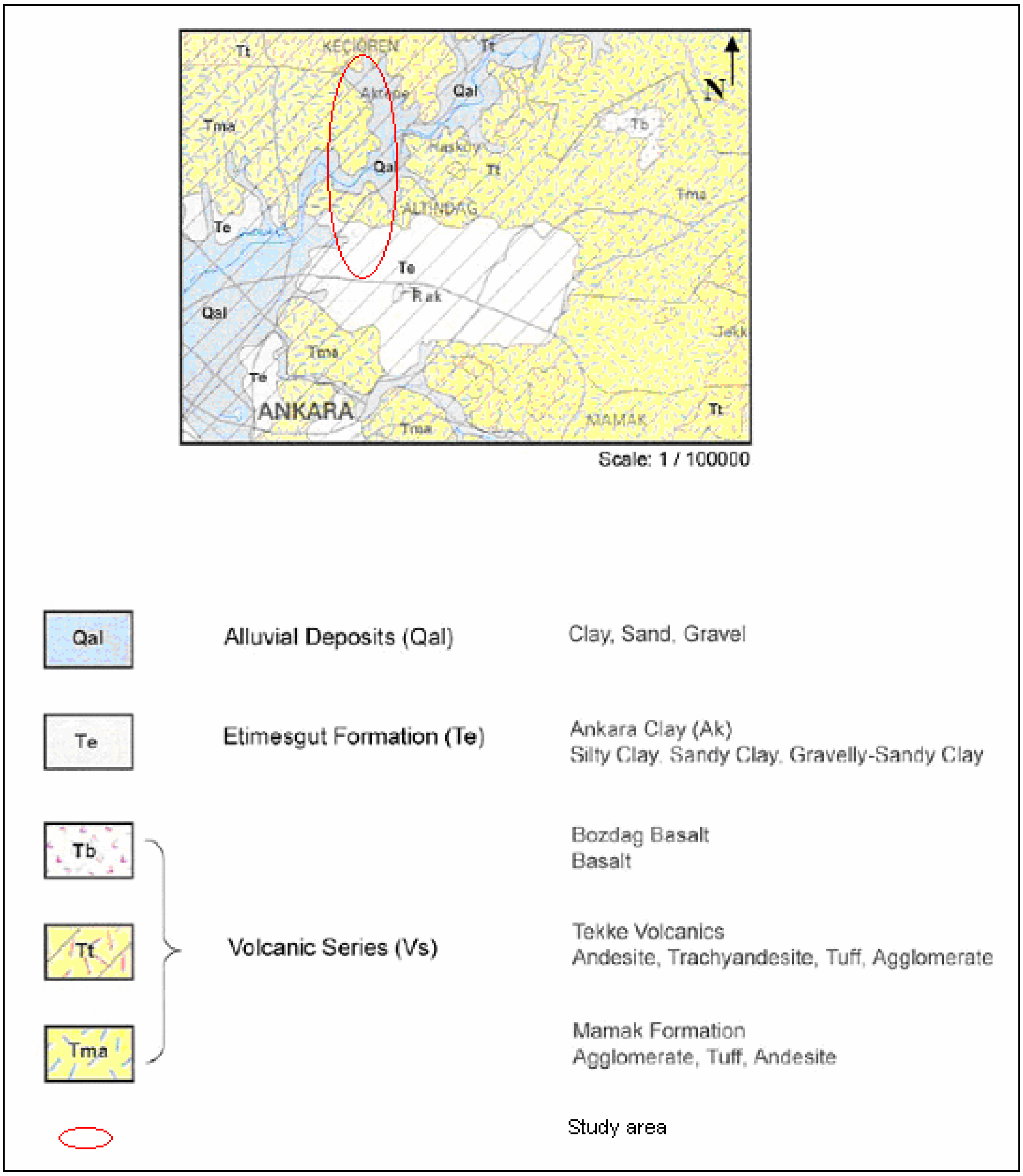

Figure 1.5. *Geological map of Ankara, Turkey (Akyürek et. al., 1997)*

Inside the city, near Ankara Castle, 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 (Altınbilek, 2006).

1.3.1.1 Hancılı Formation

It consists of clayey limestone, marl, siltstone, sandstone, and conglomerate and tuffite sequence and contains patches of gypsum and bituminous shale. Locally andesite sills are observed in the formation. Clayey limestones and marls are white and beige, thin to medium bedded, and intercalated with siltstones and sandstones. Siltstones are gray, weakly cemented, thinly bedded and laminated. Conglomerates and sandstones are yellowish gray, weakly cemented and do not show obvious layering. The Hancılı formation is deposited in streams and lakes in a terrestrial environment in which alluvial fans are developed at the basin margins (Project Report, 2005). This unit has no outcrop over the tunnel alignment so that it cannot be seen on the geological map of Ankara. It can only be seen in some boreholes.

1.3.1.2 Volcanic Sequence

The volcanic sequence on the Tandoğan-Keçiören metro alignment comprises andesite, dacite, basalt, tuff and agglomerate formed in a chaotic manner. Agglomerates are composed of different sized andesite, dacite, basalt blocks embedded within white, gray and red tuffaceous matrix. Andesites and dacites form steep topography. They are generally reddish, pinkish or grayish. Basalts are considered as the first products of the volcanism in the region. They are black and dark brown, have vesicles and show flow structures (Project Report, 2005).

1.3.1.3 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 it is 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 (Altınbilek, 2006).

1.3.1.4 Alluvium

The alluvial deposits are seen along the courses of rivers and their branches that intersect Tandoğan - Keçiören metro alignment. They are usually greenish gray and brown, have medium to high plasticity and consist of a mixture of sandy silty clay and clayey sand, clayey sandy gravel lenses and levels.

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 the Ankara Clay (Altınbilek, 2006).

1.3.1.5 Artificial Fill (Yd)

The artificial fill covering the alignment consists of excavated and road filling material. The maximum thickness of the artificial fill measured on the cross-section of the alignment is 16,13 m on Hatip Stream near Dışkapı Station and on the rest of the alignment it occurs mostly as a thin layer and has no importance.

1.3.2 Geology of the Tunnel Route

The main units encountered on the Tandoğan – Keçiören metro alignment are the Hancılı formation, Volcanic Sequence, Ankara Clay and Alluvium. These units are covered by artificial fill at the surface. The Original AutoCAD Cross-section of the study area is supplied by Yüksel Proje International Co. on which the lithological setting of the site was drawn according to the borehole data and the information it provides is used for the determination of the distribution and location of the lithological units where necessary.

1.3.3 Hydrogeology

The volcanic sequence is usually represented by agglomerate, tuff, andesite, and dacite. Agglomerate units are usually low permeable. Andesite and dacite formations allow the groundwater circulation due to the quantity and nature of the cracks they include. The tuff/tuffite lenses and levels that are included in those rocks are generally impermeable.

Ankara clay with impervious properties carries perched water tables in sand-gravel lenses. It is possible to come across with water related to the dimensions and the positions of the lenses although there is no occurrence of groundwater table in this unit. During the excavation water can be seen in some regions and other parts can be completely dry because there is no connection between sand-gravel lenses in Ankara clay.

The fluvial deposits followed on the valley bottoms of the rivers and their branches intersecting the alignment are represented by sandy clay, clayey sand and clayey sandy gravel levels. It is highly probable that these units, which usually contain groundwater and having permeable - high permeable properties, may negatively affect the excavations.

"Groundwater Depth Measurements" are recorded periodically by the contractor from monitoring wells. Both "Constant Head Permeability Test" and "Lugeon Test" are performed in these boreholes to determine the permeability of the soil and rock materials seen through the metro alignment.

Estimation of groundwater inflow into the tunnels is a difficult art, even if done carefully. The difficulties arise from several sources. The geology of the site may not be adequately understood. This is generally the case for metro tunnels in urban areas where the surface may entirely be covered by buildings and paved roads. The equations governing groundwater flow may not adequately represent the conditions. Particularly in fractured rock aquifers the uncertainties are more than those of porous media. The collected hydrogeological data may have limitations that are not accounted for. Due to dense settlement and heavy traffic of the urban environment subsurface investigations, both geotechnical and hydrogeological, are rather limited. In areas with complex geology, widely spaced boreholes can only provide general information about subsurface conditions (Doyuran, 2005).

Due to dense settlement the geology of the tunnel alignment is entirely based on borehole data. Some local rock exposures are also studied for the evaluation of rock mass characteristics. This study mainly covers the parts of the metro alignment composed of soil but the rock formations are also taken into account (Doyuran, 2005).

1.3.3.1 Permeability Values

In boreholes penetrating soft sedimentary rocks (Ankara clay and alluvium) constant head permeability tests have been performed. In the volcanic series and Hancılı formation water pressure tests have been conducted. The boreholes are then equipped with perforated PVC pipes for groundwater level measurements. The results obtained from these investigations are given below (Doyuran, 2005):

Alluvium: the alluvial deposits are composed of clay, silty clay, gravelly clay, clayey silty sand and sandy gravel. 173 constant head permeability tests have been conducted by Yüksel Proje in these units. Distribution of permeability within alluvial deposits is given in *Figure 1.6.* and 3.28×10^{-6} m/sec - 3.85×10^{-7} m/sec is the permeability range for those deposits.

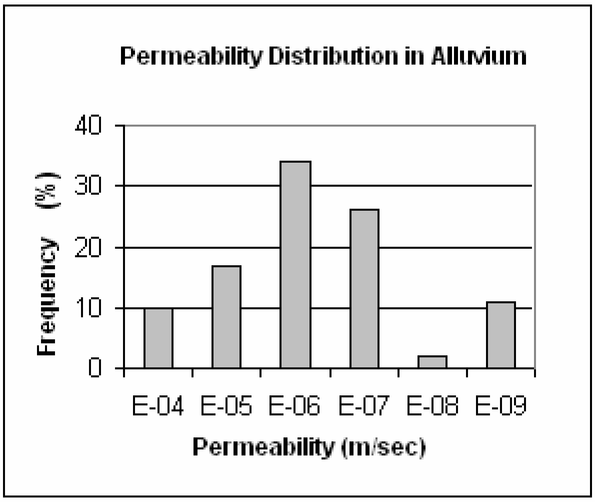

Figure 1.6. *Distribution of permeability values in alluvial units along Keçiören – Tandoğan metro tunnel (Doyuran, 2005)*

The permeability value is around 10^{-4} m/sec in boreholes UK-18A, UK-18A1 and UK-18B. As it can be seen in geological cross-section given in the second chapter this permeability value is caused by the early channel deposits in the alluvium where the clayey silty matrix is common.

The evaluations show that alluvial units have fairly good aquifer properties and in some places medium – weak aquifer properties but in total all alluvial units have aquifer properties. It is expected to come across with serious water problems especially in the regions where the boreholes mentioned above are drilled.

Ankara Clay: 135 constant head permeability tests have been conducted by Yüksel Proje in Ankara clay. The distribution of permeability values is given in *Figure 1.7.* The sand and gravel lenses observed in Ankara clay have permeability values between 10^{-6} m/sec and 10^{-7} m/sec but in general impervious levels are dominant in Ankara clay as expected. Due to this it is not possible to define an aquifer in this unit. The water observed in the boreholes drilled in Ankara clay should be accepted as leakage water.

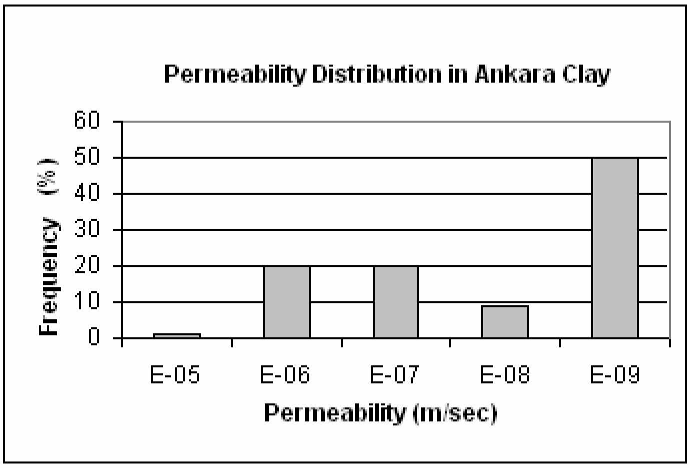

Figure 1.7. *Distribution of* permeability values in Ankara clay *along Keçiören – Tandoğan metro tunnel (Doyuran, 2005)*

Volcanic Units: the results of 39 water pressure tests performed in andesite, dacite, agglomerate and tuff units, also called volcanic series have yielded a wide range of Lugeon values. The tests results suggested that the permeability ranges between 1.17 – 10,25 and ≥25 Lugeon. *Figure 1.8.* depicts Rock Quality Designation (RQD) – Lugeon relationships. As it is seen there is no relationship between these two parameters. This may be attributed to the heterogeneity and anisotropy of the fractured rocks and also to the limitation of RQD concept. Although RQD indicates the degree of fracturing of the rock mass, it does not, however, take aperture, infillings, persistence, etc into consideration. Therefore the aperture of discontinuities is more important than frequency (Doyuran, 2005).

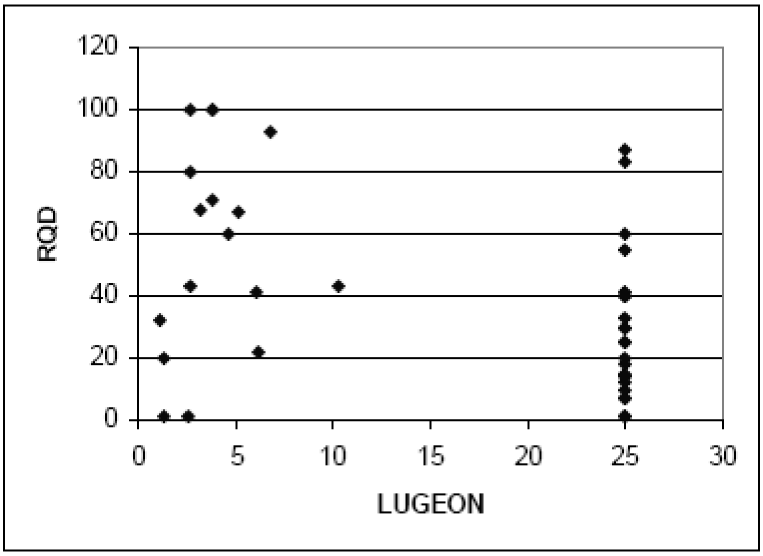

Figure 1.8. *RQD – Lugeon relationship of the volcanic series (Doyuran, 2006)*

As seen in Figure 1.8. Lugeon values are grouped between 1,17 and 10,25 Lugeons and at ≥25 Lugeons. Thus, a value of $4x10^{-7}$ m/sec is assigned as an average permeability of the jointed rocks (for 1,17-10,25 Lugeon interval) and a value of 10^{-5} m/sec (for Lugeon values ≥25) for the highly jointed and/or sheared zones.

1.3.3.2 Groundwater Inflows into the Tunnel

Based on the in-situ permeability test results the average permeability of the alluvium is estimated as $3{,}3 \times 10^{-6}$ m/sec. In this section, the hydraulic heads from the groundwater table to the invert of the tunnel range between 2 - 20 m. The radius of influence (L) is not known thus different L values ranging between 50 m and 150 m are adopted (doyuran, 2005). The results are given in *Figure 1.9*.

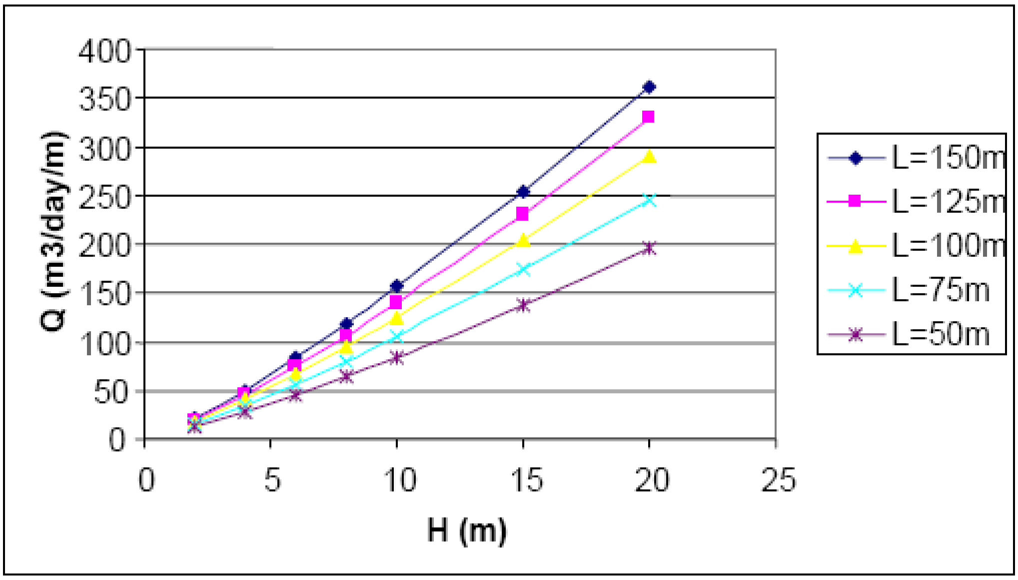

Figure 1.9. *Groundwater inflow into excavations under different hydraulic heads and radius of influence (Doyuran, 2005)*

From the *Figure 1.9* it is seen that even under most unfavorable conditions the maximum groundwater seepage into excavation is about 350 m^3 / day /m. This rate will gradually decrease as hydraulic head decreases as seen in *Figure 1.10.*

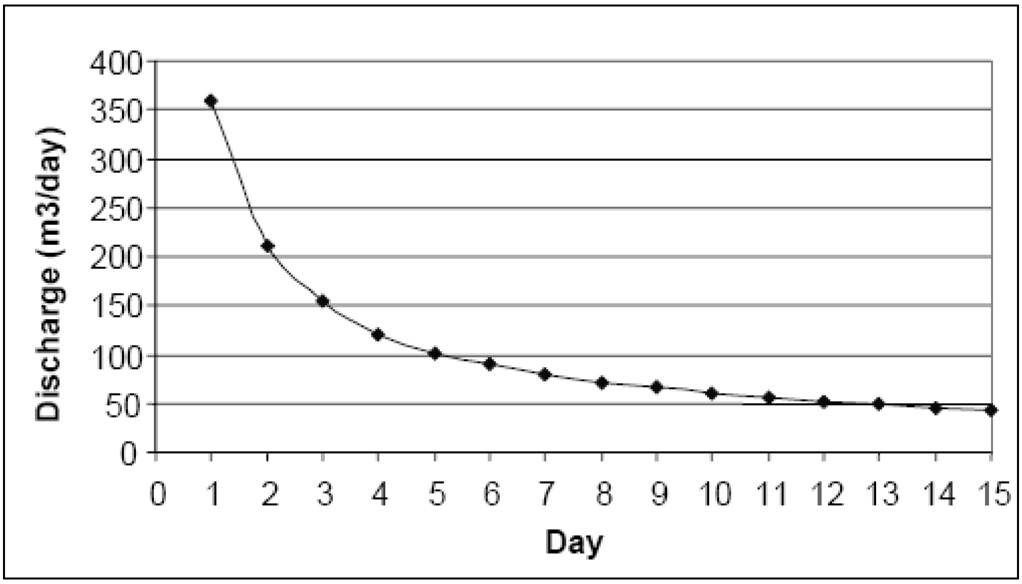

Figure 1.10. *Groundwater inflow vs. time (Doyuran, 2006)*

Uniform inflows are expected through the porous media of the alluvium. Sand and gravel dominant layers and/or lenses are encountered within the alluvium and significant inflows should be expected in those units.

The volcanic series from a fractured rock aquifer is characterized by high heterogeneity and anisotropy. Thus the permeability of the fractured rock may not be adequately characterized. The range of permeability of the rock mass may be even higher than that determined from the water pressure tests. Normally the longer and more open fractures will capture most of the flow and canalize it toward the tunnel (Raymer, 2001). This will result in non-radial flow paths.

1.4. General Information about Tunnel Boring Machine (TBM)

This section provides information about the tunnel boring machine (TBM) operation principals, construction method and main properties. Each project has unique study site that has different characteristics thus same TBM cannot be used in every project. Different design rules are applied according to the project in which TBM is going to be used. Main producers of TBMs are Germany and Japan in the world. The boring machine used in this project is made in Germany and further information about it is given below.

1.4.1. Tunnel Boring Machine (TBM)

A TBM is a complex set of equipment assembled to excavate a tunnel. The TBM includes the cutterhead, with cutting tools and muck buckets; systems to supply power, cutterhead rotation, and thrust; a bracing system for the TBM during mining; equipment for ground support installation; shielding to protect workers; and a steering system. Back-up equipment systems provide muck transport, personnel and material conveyance, ventilation, and utilities.

List of main constitutive items are given below:

i. Front face where the soil is excavated with special tools (shield or cutting wheel/cutterhead)
ii. Steering mechanism part with drive engines for forward movement.
iii. Control mechanism for deviation and inclination
iv. Removal installation for transporting excavated material through the machine to a separator or directly onto an independent transport system
v. Installations behind the working chamber permitting either further soil improvements (i.e. with rock bolts, shotcrete or injections) or are used for preliminary investigations
vi. Support installations within the protection of the shield tail
vii. Eventually grouting the void at the shielded tail created between the lining and the subsoil (AFTES, 1999).

The TBM implemented in Keçiören – Tandoğan metro poject is an Earth Pressure Balanced type with the following properties;

Model	: HERRENKNECHT S-324
Excavation Diameter	: 5,89 m nominal
Tunnel Liner Inner Diameter	: 5,24 m
Cutter Head Rotational Speed	: 6,1 rpm
Torque	: 4500 KNm
Hydraulic Thrust Cylinders	: 24 pieces x 125 tons
Total Force Resistance	: max 29255KN, 350 bars
Spiral Conveyor	: 1200 mm x 15m
Belt Conveyor	: 1200 mm x 50m
Total Installed Power	: 1393 kW, 1600 kVA transformer
Articulation	: Push forward 150 mm, Operation pressure 250 bars

In the first gantry of the TBM there are controllers and direct monitoring equipments that the operator will use during the excavation.

There are three main parts of TBM:

a. Front part and cutter head
b. Middle part
c. Gantries

Behind the cutter part of the TBM the following systems are loaded:

a. Ring segment mounter and segment installation crane
b. Gantries
c. Transport trolleys consist of segment carrier, excavated material carrier, slurry carrier and a diesel engine working between cutter head and equipment trucks
d. A transformer as a main power supply

e. Tunnel ventilation and shafts

In the entrance of the tunnel following items are seen:

a. A pit for the excavated material trolleys. There is an automatic inclined dumping platform for the evacuation trucks.
b. Ventilation fans.
c. Portable crane (Türkerler, 2006).

In the following Figures 1.11, 1.12, 1.13, a schematic view of an EPB – TBM and the TBM implemented in Keçiören – Tandoğan metro project can be seen.

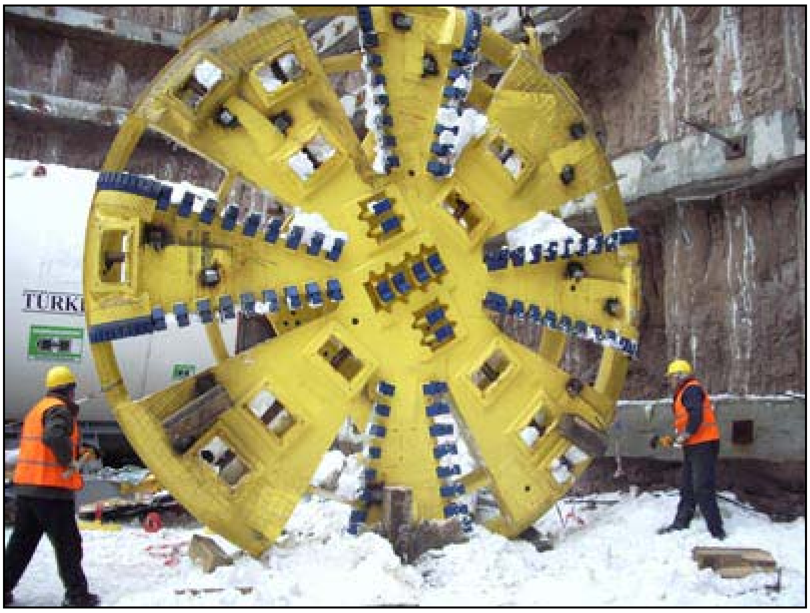

Figure 1.11. *The cutterhead of the EPB – TBM implemented in Keçiören – Tandoğan Metro Project*

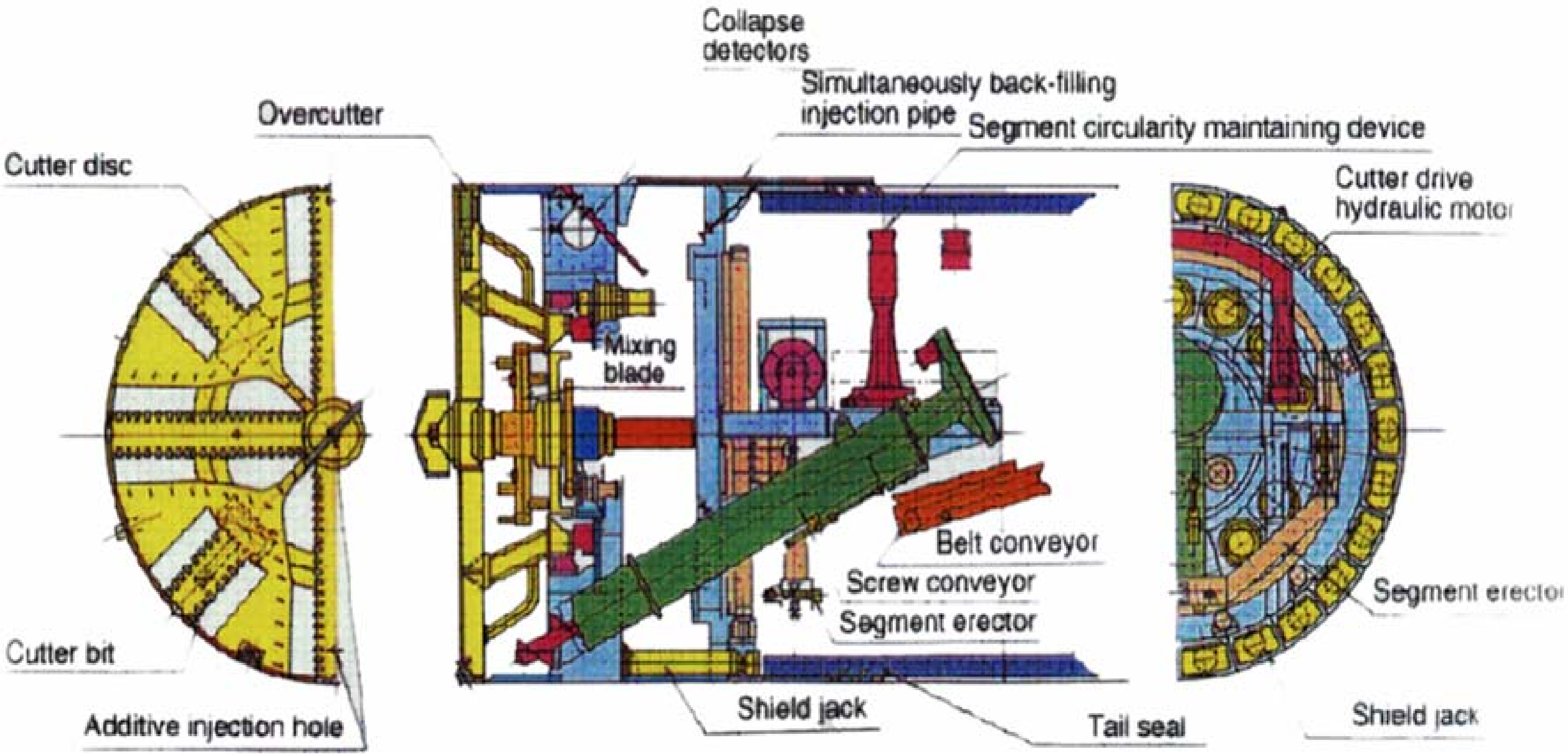

Figure 1.12. *Schematic illustration of the parts of an EPB - TBM*

Figure 1.13. *TBM implementation and portal site of the Keçiören – Tandoğan metro alignment*

1.4.2. TBM Application

TBM works by repeating the following steps in turn:

a. Tunnel boring part pushes itself to the tunnel face: For this operation, TBM presses the 24 hydraulic cylinders spread around the vertical circle containing shoes plated with teflon that apply 125 tons of pushing force on to the final tunnel segments’ side faces. Forward pushing operation continues during the whole of the excavation.

b. Cutter head turns: Various excavators fixed on the cutter head do the excavation; the material coming out during the excavation is carried with screwed conveyors to the transportation band. Later this material is carried to the wagons of the train waiting under the gantries by the conveyor band.

In the first step of excavation 1,60 meters and normally in the following steps 1,40 meters of tunnel line is excavated. This distance is the width of the tunnel segments mounted in each step.

During the excavation the guidance system directs the TBM. The tunnel alignment is corrected by adjusting the pressure of some pistons by the operator.

c. Guidance: The final situation of the tunnel face is measured by laser equipments. The new direction that will be followed during the next step is calculated by computer and TBM is adjusted according to this calculation. TBM is directed by using, laser equipments, pistons and pilot head.

One step of tunnel lining consists of a circular ring built up with six prefabricated tunnel segments. While the position of he segments in the ring is constant, the ring is mounted in the plane created by the tunnel section with a specific angle that has been described and calculated before. The length of the ring is 1,40 meters and one side of the ring is 4 cm shorter than the other side.

Changing the direction of the tunnel lining is done by giving a value to the angle described in the tunnel section above. This angle shows how the new group of segments will be mounted relative to the previous segments and provides the correct advancement of the tunnel alignment.

Those corrections during the implementation provide the ring to match the tunnel alignment direction defined by the TBM and the exact direction correction can only be done with TBM directed by laser beams.

d. The mounting of a group of tunnel segment is done: The erector vacuum which can move both on the tunnel axis and tunnel cross-section takes each segment from the wagons and positions them. Mounting process is done by bolting and Dötweyler P 19-820 joints are tightened.

e. Injection: In order to protect the TBM from injection grease is applied around it. This application provides TBM to slide easily on its route. Injection is applied around the outer part of the ring. Injection material is taken from the injection wagon. The empty space between the tunnel wall and the segments is filled with this application.

f. Train goes out from the tunnel: Train composed of 6 full excavation wagons, 1 empty injection wagon, 2 empty precast segment wagons, 1 staff wagon for 8 persons and 2 locomotives comes out of the tunnel and stops in the portal site.

g. Excavated material is thrown into the excavation pit: Wagons are designed to dump the material from their sides. A loader in the pit fills the trucks and material is transported to a disposal site.

h. Precast segments are loaded: A mobile crane carries the segments from the temporary storage place to the wagons.

i. Train enters the tunnel: Train consist of 7 empty excavation wagons, 1 full injection wagon and injection pump, 2 full precast segment wagons and 1 locomotive stops near the TBM. After all steps explained above are repeated as a cycle.

j. Construction speed: The tunnel construction speed is planned as 24 meters per day.

- Average speed; 1 ring/hour, (1 ring=1,40 meters)
- Multiple shift operation; 20/24 hour (Türkerler, 2006)

1.4.3. Measurement Methods

1.4.3.1. Measurements Prior to the Tunnel Boring

Application is done according to the country coordinate and elevation points. Primary applications are done in order to adjust the exact tunnel direction as planned earlier. When each control point is defined their data is loaded to the TBM computer and these data are protected under a set of passwords and security checks. This information can only be changed by authorized persons if needed.

1.4.3.2. Measurements After Tunnel Boring and Ring Mounting Processes

These measurements are done daily in order to control that the TBM computer is following the direction of the tunnel correctly. Laser equipment sends its beam on to the defined targets to check whether the tunnel alignment is going exactly as planned or not. These measurements are also important in defining the position of the next ring to be mounted.
Another part of these measurements is deformation control points. After each 40 meter period measurements are done in order to check the deformations on the side walls and bottom of the tunnel.

It is also important to define the deformation that may occur on top of the tunnel. The important structures on the top of the tunnel are defined before and while the TBM is advancing their elevations are checked by geometric leveling method and those control points on the surface are checked regularly.

1.4.4. Tunnel Lining- Prefabricated Segments

The lining of bored tunnels are done with prefabricated segments in different lengths and geometries with a width of approximately 1,40 meters. The inner diameter of the tunnel is 5240 mm and outer diameter is 5760 mm where the thickness of the tunnel wall is 260 mm. the segment configuration is five standard segment elements and one locking segment element. The total weight of these six segments is 14,400 tons and mounting is done by a crane with the capacity of 12 tons. Those segments are produced with suitable concrete material and neoprene joints makes them water proof.

1.4.5. Ring Mounting

D, B, C, K, E, A letters are assigned to the segments in order to define their correct place on the ring. Two wagons carry them inside the tunnel and each of them has a predefined place on the wagons. Segments are produced in a production site near the tunnel and while segments come to the daily storage site in the portal of the tunnel four hard, and wooden plates saturated with bitumen that have 3,2 mm width are glued and also a sponge material is glued on to the side of the segment where the wooden plate is applied and on to the shorter side of it.

1.4.6. Injection

To protect the rings during the excavation voids are filled with injection. The important point is to fill the voids and at this point the strength of the injection material is not so important. The water/concrete ration is adjusted according to the state of the soil and hydraulic conditions during the excavation.

1.4.7. General Considerations about TBM Performance

TBM excavation represents a big investment in an inflexible but potentially very fast method of excavating and supporting a tunnel (Barton, 1996). When unfavorable conditions are encountered without warning, time schedule and practical consequences are often far greater in a TBM driven tunnel than in a drill and blast tunnel.

The unfavorable conditions can be produced by either a rock mass of very poor quality causing instability of the tunnel or a rock mass of very good quality (i.e. strong and massive rock mass) determining very low penetration rates. However, it is to be observed that when using the full face mechanized excavation method, the influence of the rock mass quality on the machine performance has not an absolute value: the influence is in fact to be referred to both the TBM type used and the tunnel diameter.

Given that a TBM capable of advancing under whatever the geological condition is, it is also true that the overall result of a project depends on:

a. the type of the TBM used, and
b. the design and special construction characteristics of the TBM adopted.

In fact, it is not sufficient to just order from a qualified manufacturer a particular type of TBM; instead, a continuous collaboration and control of all design and construction details are essential by its intended user, the contractor. This is particularly true as far as there are still no "Accepted Standards" for the design and construction of a type of TBM, and each TBM to be constructed is to be considered as a prototype, one different from another one, given that:

a. the design and manufacturing of TBM's is a continuous, technologically innovative process,
b. each tunneling project has its own characteristics, and each specialized contractor has his own traditions and opinions (Barla&Pelizza, 2000).

1.4.7.1 Limiting Geological Conditions for TBM's Application

A limiting condition for the use of TBM excavation can be defined where the geological conditions are such that the same TBM cannot work in the execution modes for which it was designed and manufactured. For this reason the advance of the TBM is significantly slowed down or even obstructed. A geological condition is intended to be a limiting one only in relation to the type of TBM used, its design and special characteristics, and eventually any operating errors. A particular geological condition becomes a limiting one only when it is beyond a certain importance, or when the associated problems are beyond a certain level of severity, or else due to combination of events each being by itself not critical.

1.4.7.2. The Importance of Geological and Geotechnical Investigations

Despite the excellent performance of TBM's in favorable ground conditions in many cases the actual advancement rates have been below expectations and certainly less than claimed by TBM manufacturers. It would therefore be legitimate to think that, besides the unforeseen events, such as breakdown or failure of the TBM components, the rock mechanics problems are often under-evaluated or neglected. It should be noted that the purpose of construction is to achieve the objective of the design and that the work must be manlike (as defined in design), according to the specified safety factors and the expected time and cost (Pelizza&Barla, 2000).

The design has always been carried out by using a deterministic approach. Reality of construction, however, has never been so. This is due to the large number of uncertainties that cannot be avoided at the design stage: geological, geotechnical, hydrogeological uncertainties, different types of machines available, and different construction techniques (Pelizza, 1998).

Hence, at the design stage, it is impossible to know every aspect of the geological profile. It is, therefore, necessary to decide whether to optimize the choice of the construction method or the selection of the machine for a given tunnel, on the basis of the understanding of site geology and geotechnical conditions or of the level of prediction about these conditions.

The fundamental problem is always determined by the physical and geotechnical heterogeneity of the rock mass in which the tunnel is to be excavated. For a full face mechanized excavation, which is a rather rigid system, the strength heterogeneity of the material to be excavated is even more important, be it a rock or soil (Pelizza&Barla, 2000).

CHAPTER 2

DATA ACQUISITION AND PREPERATION

Site investigation is the process by which geological, geotechnical, and other relevant information which might affect the construction or performance of a civil engineering or building project is acquired. (Clayton et. al., 1995) Site investigation results are obtained from in-situ studies by means of borings and field tests. Geographical or spatial data are defined as undigested, unorganized, and unevaluated material that can be associated with a location. When data are organized, presented, analyzed, interpreted, and considered useful for a particular decision problem, they become information. Accordingly, geographical information is defined as georeferenced data that have been processed into a form that is meaningful and of real perceived value to decision makers (Malczewski, 1999). In order to create a basis for the preparation of cross-sectional layer for determining excavation difficulty, four different data sets have been used that are gained from those studies. In this chapter, the methods of acquiring data and the way data were used to create the cross-section layers will be described.

2.1 Investigation Studies

2.1.1 Geological Cross-section

By considering the foundation boring results the geological cross-section of Tandogan – Mecidiye segment of the metro alignment was drawn with 1/500 vertical and 1/5000 horizontal scales with AutoCAD. On this cross-section basically the location of the boreholes, lithological units, initial groundwater levels, the metro tunnel route, important structures and planned metro station locations can be seen. This cross – section provides a basis for this thesis by means of being the basic layer for the future analyses that will be discussed in the following chapters and from now on this cross-section will be called as the “Original AutoCAD Cross-section” (see *Figure 2.1- 2.6*)

As mentioned earlier in this thesis the area between Tandoğan and Mecidiye stations have been studied. This area refers to the region investigated by boreholes TA-20 – TA-1 and UK-6 – UK-27. TA-20 is the first borehole situated in Tandoğan and it is at 0+858 km of the alignment. UK-27 is the final borehole at Mecidiye station and it is at 6+264 km of the alignment.

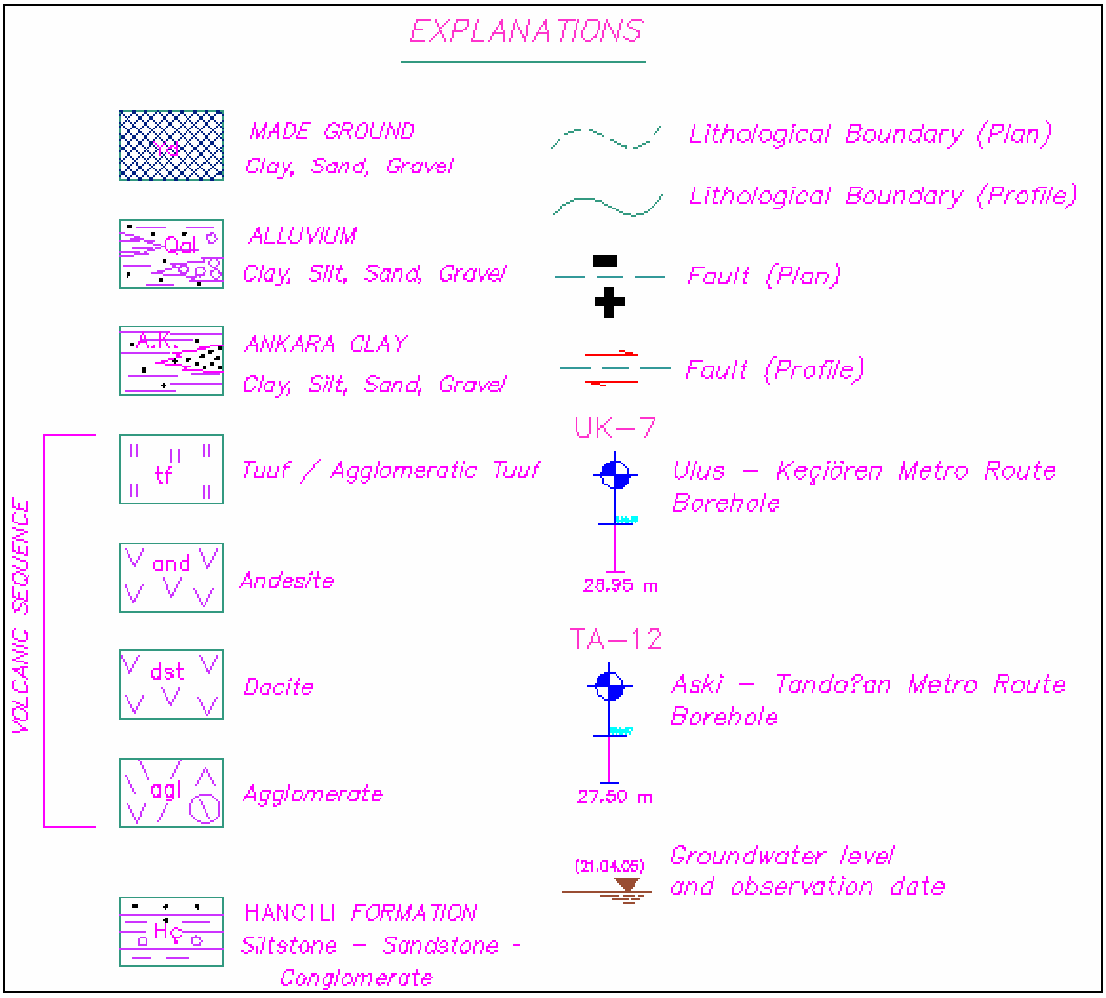

Figure 2.1. *Legend of the AutoCAD cross-section*

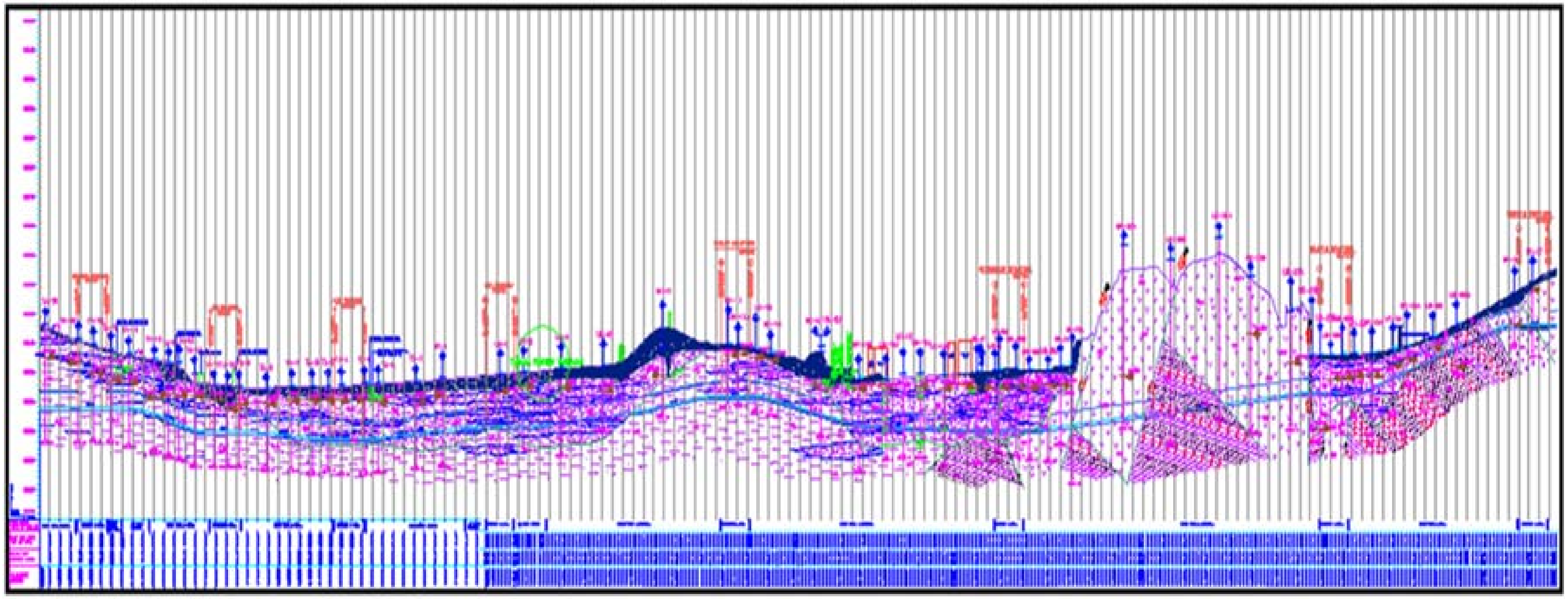

Figure 2.2. *The original AutoCAD cross – section of the study area*

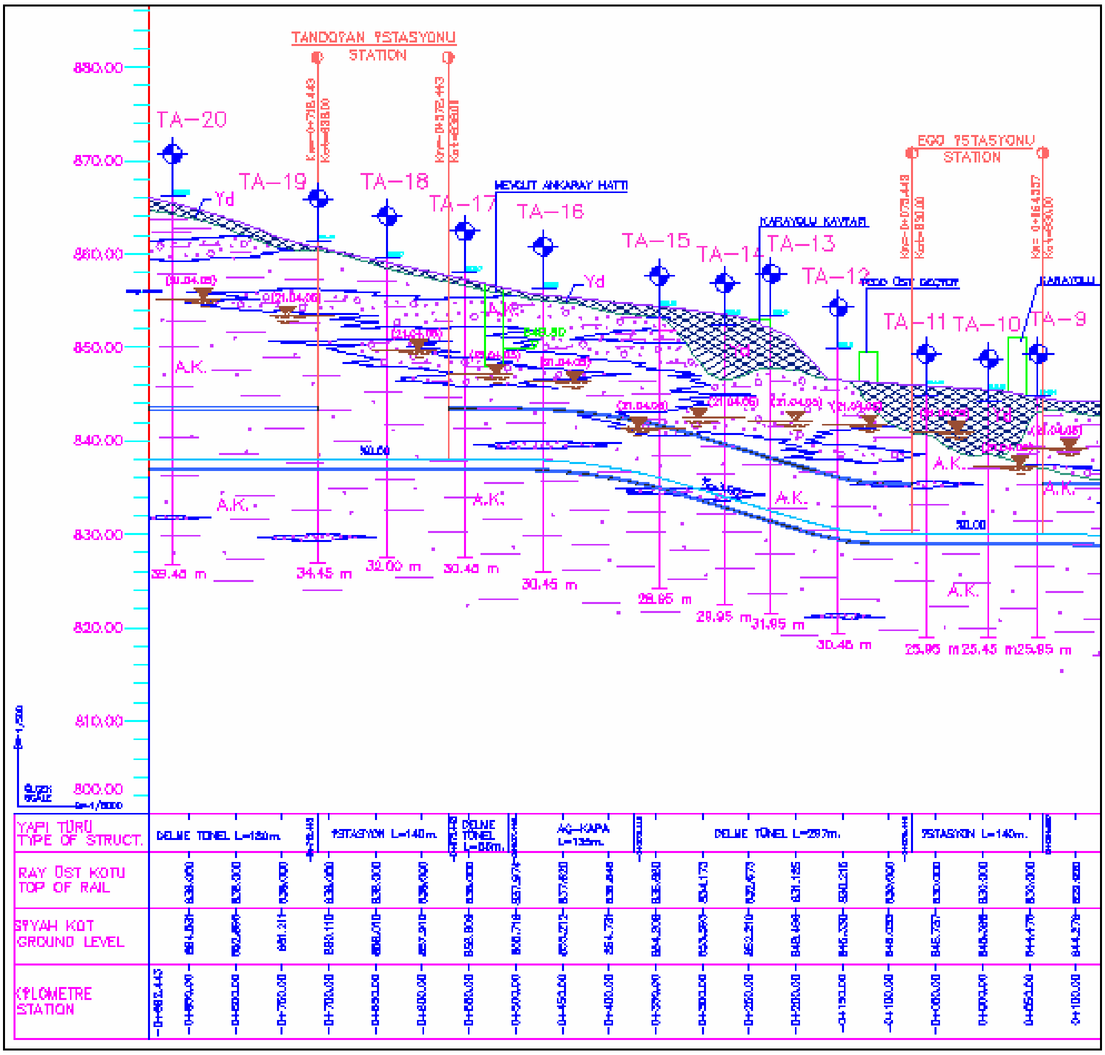

Figure 2.3. *Segment of the metro alignment between Tandoğan and EGO stations*

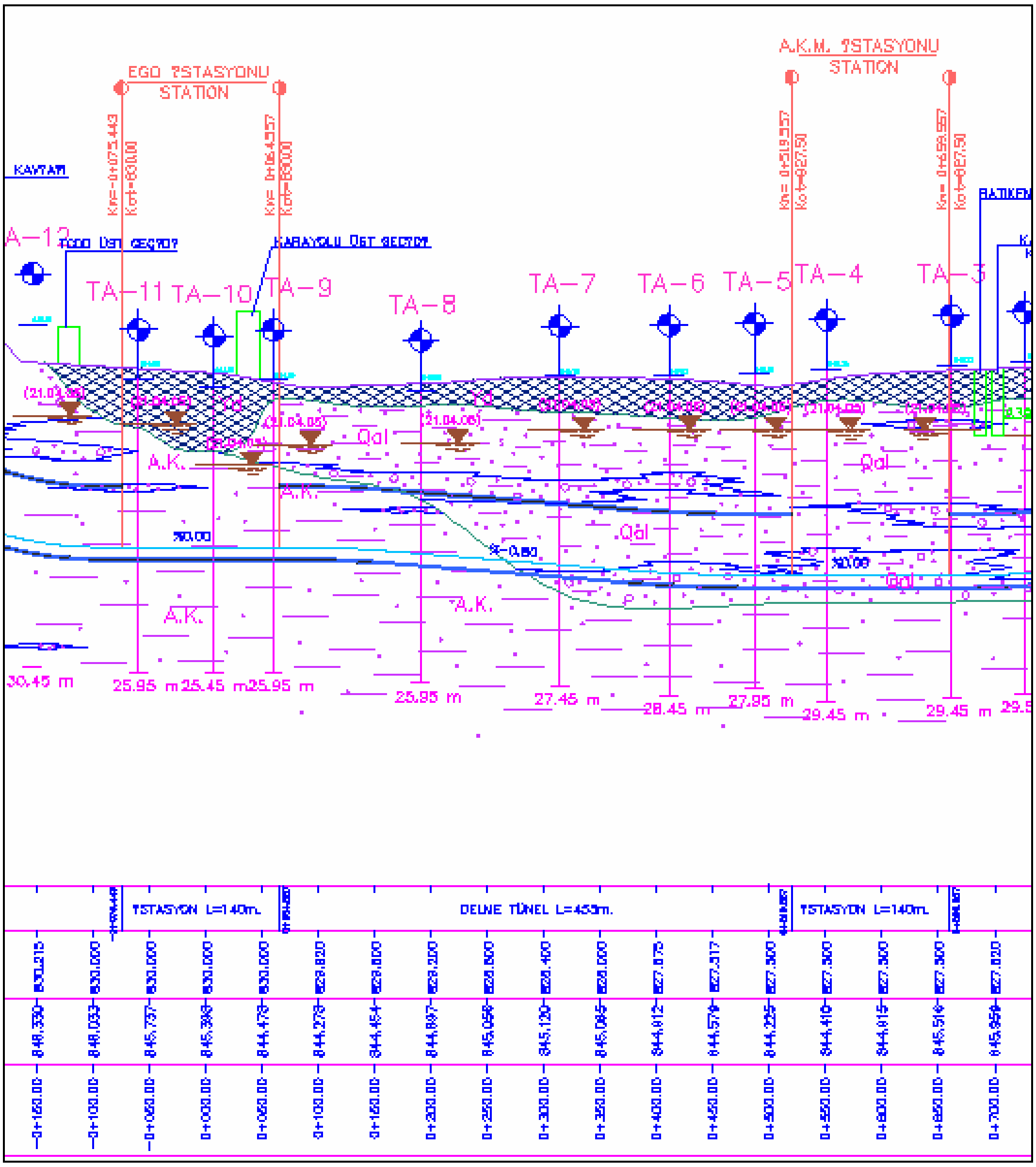

Figure 2.4. *Segment of the metro alignment between EGO and AKM stations*

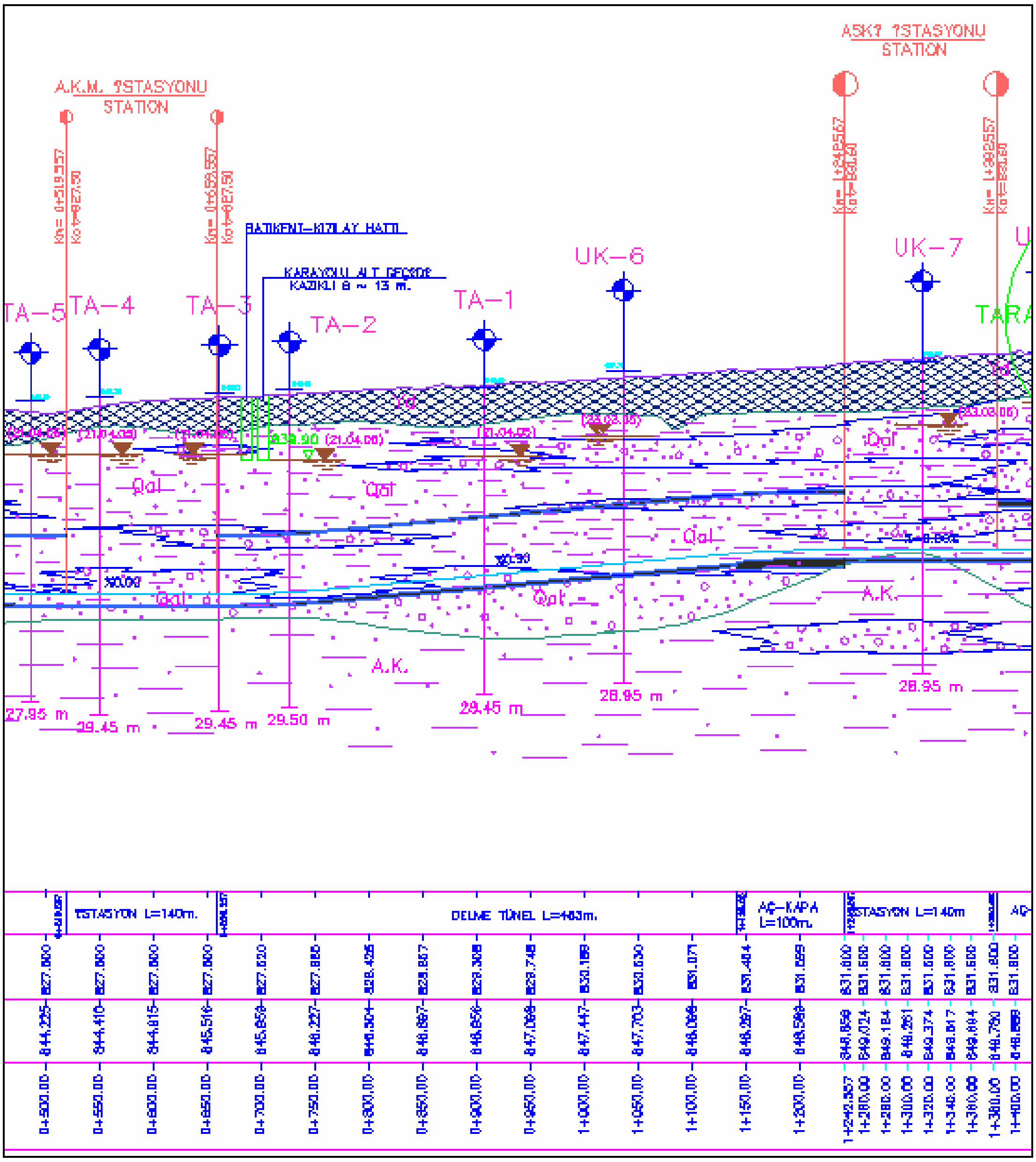

Figure 2.5. *Segment of the metro alignment between AKM and ASKI stations*

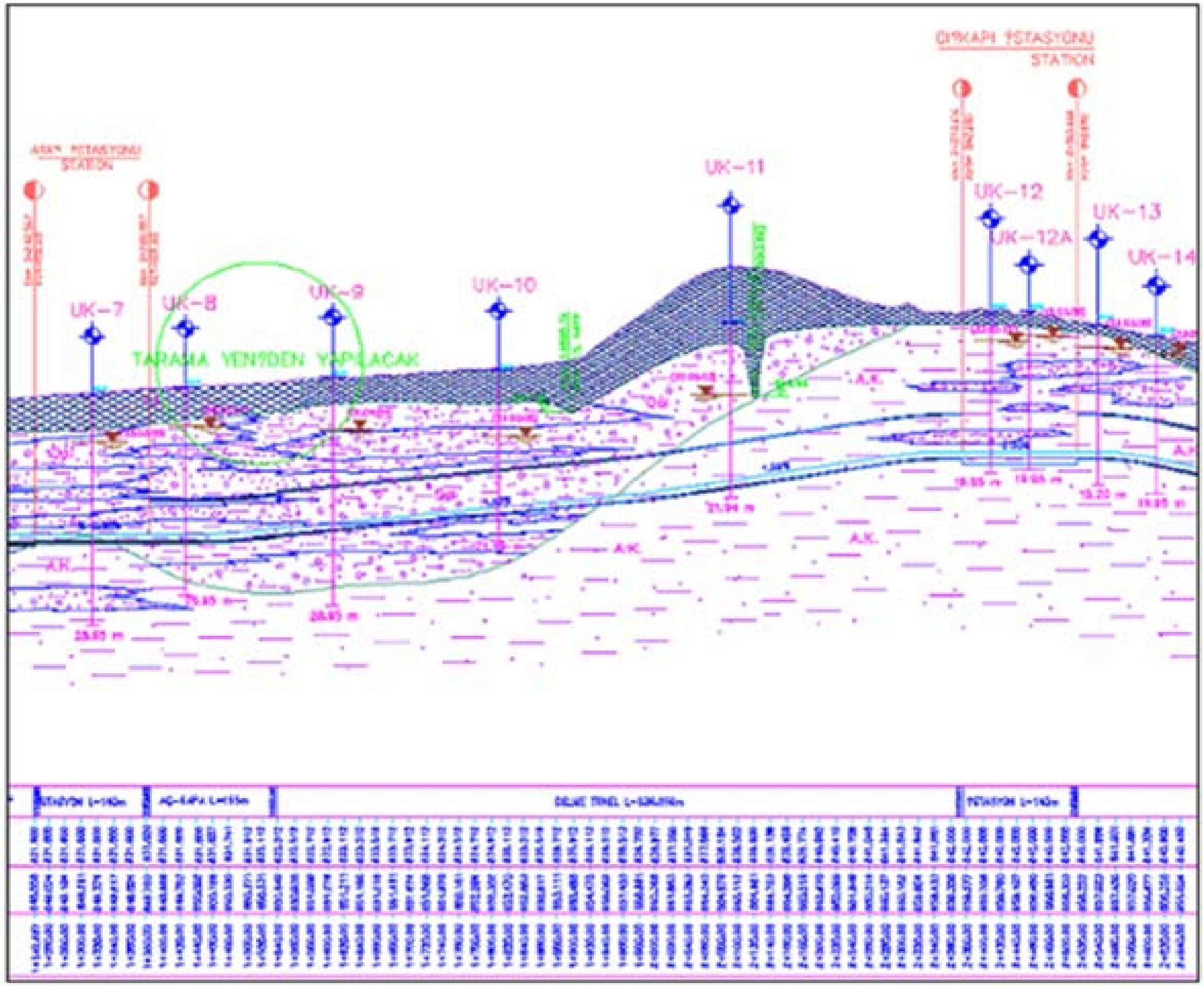

Figure 2.6. *Segment of the metro alignment between ASKİ and Dışkapı stations*

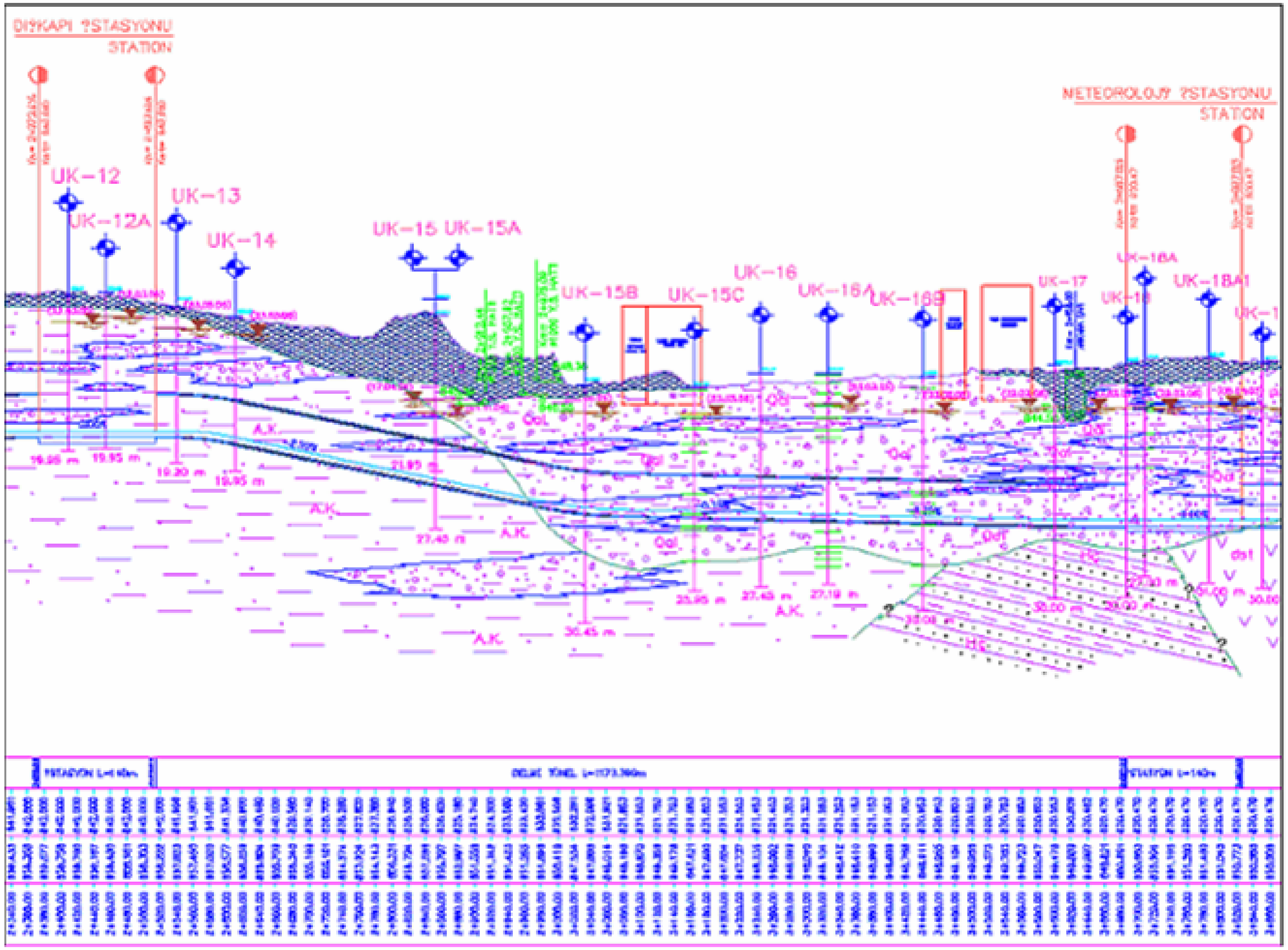

Figure 2.7. Segment of the metro alignment between Dışkapı and Meteoroloji stations

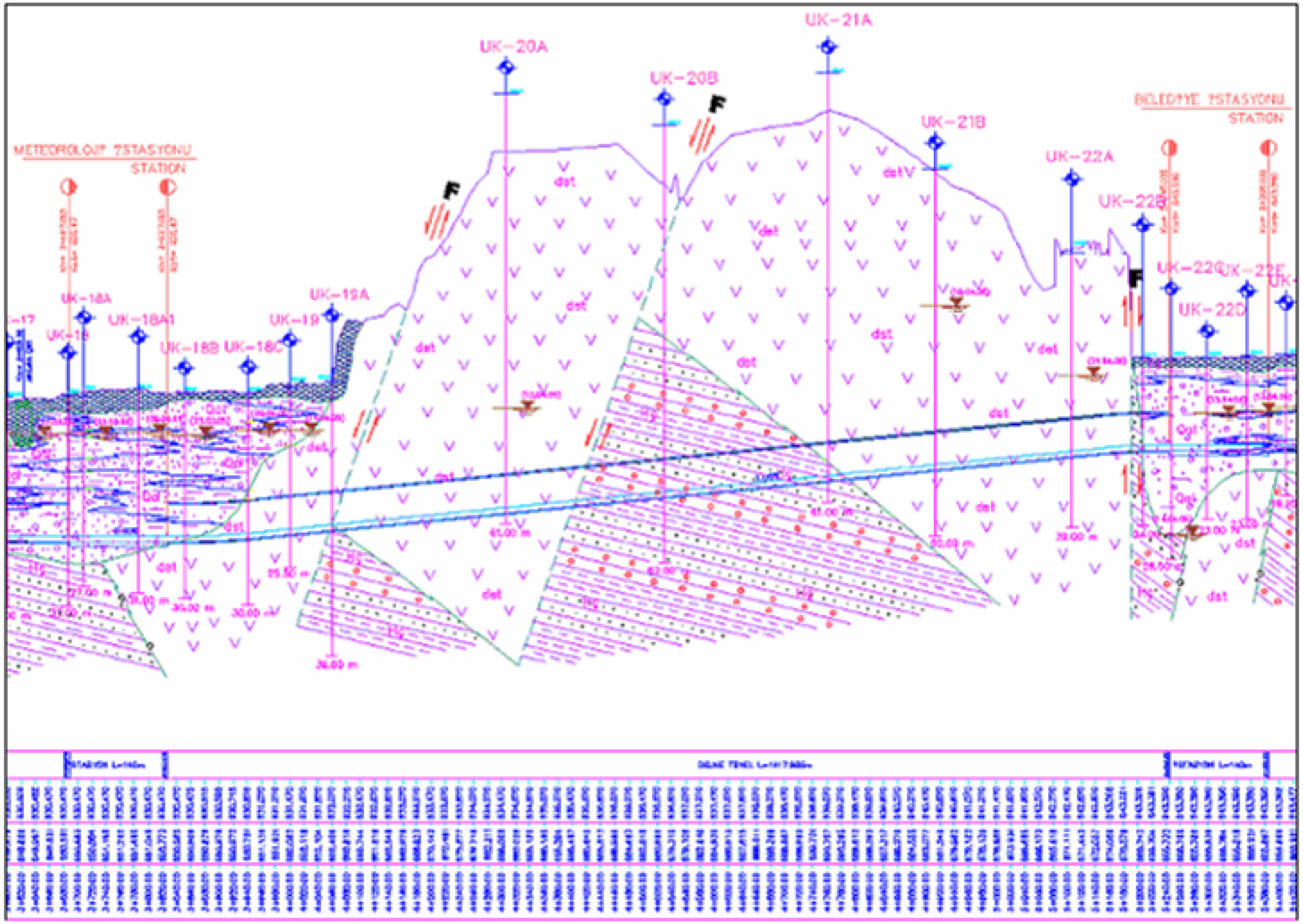

Figure 2.8. *Segment of the metro alignment between Meteoroloji and Belediye stations*

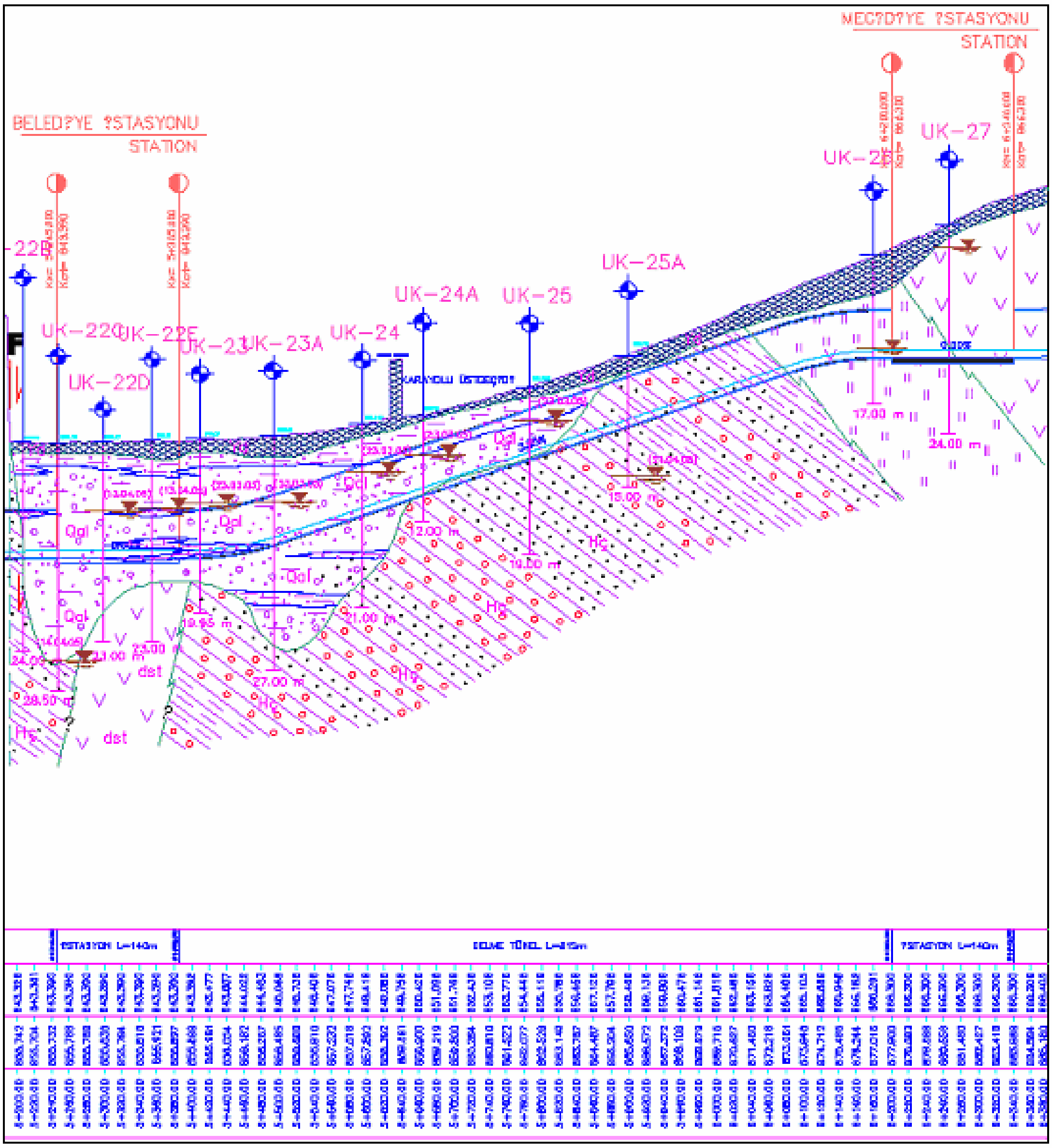

Figure 2.9. *Segment of the metro alignment between Belediye and Mecidiye stations*

2.1.2 Foundation Borings

To determine the soil types, thicknesses, contact relations, geological and geotechnical properties a total of 82 boreholes were opened between Tandoğan-Gençlik Street and Keçiören-Aksaray Street. In this thesis 62 borehole data are used with the total length of approximately 1760 meters.

During boring two truck mounted Mobile Drill B – 53, one truck mounted Crealius D – 750, one truck mounted D – 58 and one Soil Mec SM – 103 brand palette boring machines were used.

Borings had been conducted used by Yüksel Proje International Co. according to the “Technical Specifications for Ground Investigation Studies”(1995) prepared by The General Directorate of Highways, Department of Technical Affairs and “Tandogan – Kecioren Metro Alignment’s Technical Specifications”. To produce the foundation profile disturbed (SPT), undisturbed (UD) and core samples were obtained from the bore holes. To determine the in-situ resistance “Standard Penetration Tests” and “Pressiometer Tests” were conducted in highly and/or completely weathered levels of the rock units and in the soil units within 1, 5 m intervals. Besides “Constant Head Permeability” and “Lugeon” tests were conducted to determine the permeability of the soil and the rock units. The disturbed samples (SPT) were protected in polyethylene bags and Shelby tubes (undisturbed/UD samples) were sealed by applying paraffin on to the caps. The core samples were preserved in the core boxes. All samples were taken to Yuksel Proje International Co. Soil Investigation Laboratory for testings.

In order to determine the groundwater levels boreholes were equipped with perforated PVC pipes set through the end of the boreholes. When static levels are maintained the monitoring program involving monthly measurements of static water levels had been conducted.

All the data like the soil/rock descriptions of the units encountered through out the boreholes, SPT N values, SPT N graphics, disturbed (SPT) and undisturbed (UD) sample depths, maneuver lengths, percent core recovery, constant head permeability test, Lugeon test, pressiometer test depths and groundwater levels were indicated on the boring logs. The results of the pressiometer tests, constant head permeability tests and Lugeon test were also given in the report format.

2.2 Evaluation of Data Sets and Their Relation with TBM Performance

Four different data sets are used in the preparation of layers which are:

1. Lithology
2. Hydrostatic Pressure
3. Permeability
4. Standard Penetration Test Values & Unified Soil Classification System (USCS)

These data sets are mostly related to the geological features of the ground and hydrogeological aspects that are determined according to the field and laboratory tests. Approximately %75 of the alignment consists of soft ground and generally, in soft ground, major concerns are opening stability and control of displacement field. Soft ground tunneling is likely dominated by failure and admissible displacement criteria. Ground conditioning (improvement and reinforcement) might play an important role. In consolidated clay, the optimization of values and quantities of the slurry pressure and grouting pressure is required for TBM technology.

In urban environment, major concerns are related to: shallow overburden, existence of nearby structures, foreign objects inside the ground, constraints for alignment, restrictions for auxiliary works, and high visibility of damage. Ground conditions are normally challenging, characterized by recent weak geological formations near the ground surface, by frequently changing conditions due to the occurrence of lenses, layers, boulders, etc., and by presence of ground water above the tunnel or crossing the tunnel profile. These features permitting safe and economic tunneling in soft ground under urban conditions using TBM´s with slurry or Earth Pressure Balance (EPB) type of face support can be summarized as follows:

1. Efficient TBM technology
2. Reliable design procedure
3. Improved methods of conditioning
4. Advanced grouting technology
5. Reliable risk management (Kovari, Ramoni, 2004)

It is well known, that tunneling is not a risk-free technology. Tunnels are regarded as so called “heavy risks”, because each tunnel is a specific unique project on its own in a unique combination of ground / soil. The “right” construction methods with the right experience parties involved are crucial for the success. The main most important factor however, the geology, is only known to a limited extent. Any accident during construction as well as in use provokes a substantial interruption and often a standstill till the problems are solved (Andreossi, 2001). The purpose of the thesis is to determine the parts of the alignment that may affect the TBM performance and because of this reason the main aim while choosing the data sets is to define the geological aspects of the study area as clearly as it can be. Lithology, SPT and USCS results are useful in defining the ground material types, their resistance against TBM and lithology of the ground which directly influences the behavior of groundwater. It is also important to determine the hydrostatic pressure values on the top of the rail elevation because EPB TBM’s, which is also used in this project, acts against the pressures caused by the removal of the material until the supporting linings are implemented. It also acts against water flowing into the tunnel thus an over inflow might negatively affect the performance of the TBM. Strength, in-situ stress, bloc fall may also affect the operation of TBM in rocks but excavations in this study were held in alluvium and Ankara clay dominantly so that these aspects were not taken into account. *Table 2.1.* shows major geotechnical conditions that affect the TBM performance:

Table 2.1*. Impacts of geotechnical conditions on TBM operations (www.usace.army.mil/publications, TBM Performance Concepts and Performance Prediction)*

Major Geotechnical Conditions	**Consequences/Requirements**
Loosening loads, blocky/slabby rock, overbreak, cave-ins	At the face: cutterhead jams, disc impact loading, cutter disc and mount damage possible, additional loss on available torque for cutting, entry to the face may be required with impact on equipment selection, recessed cutters may be recommended for face ground control. In the tunnel: short stand-up time, delays for immediate and additional support (perhaps grouting, hand-mining), special equipment (perhaps machine modifications), gripper anchorage and steering difficulty, shut-down in extreme cases of face and crown instability. Extent of zones (perhaps with verification by advance sensing/probe hole drilling) may dictate shield required, and potential impact on lining type selection (as expanded segmental linings may not be reasonable), grouting, and backpacking time and costs may be high.
Groundwater inflow	Low flow/low pressure - operating nuisance, slow-down, adequate pumping capability high flow and/or high pressure - construction safety concerns, progress slow or shutdown, special procedures for support and water/wet muck handling, may require advance sensing/probe hole drilling. Corrosive or high-salt water - treatment may be required before disposal, equipment damage, concrete reactivity, problems during facility operation. Equipment modifications (as water-proofing) may be required if inflow is unanticipated - significant delays.

Table 2.1. continuing

Squeezing ground	Shield stalling, must determine how extensive and how fast squeeze can develop, delays for immediate support, equipment modifications may be needed, if invert heave and train mucking - track repair and derail downtime.
Ground gas/hazardous fluids/wastes	Construction safety concerns, safe equipment more expensive, need increased ventilation capacity, delays for advance sensing/probing and perhaps project shut-down, special equipment modifications with great delays if unanticipated, muck management and disposal problems.
Overstress, spans, bursts	Delays for immediate support, perhaps progress shut-down, construction safety concerns, special procedures may be required.
Hard, abrasive rock	Reduced PRev and increased Fn - TBM needs adequate installed capacities to achieve reasonable advance rates, delays for high cutter wear and cutterhead damage (especially if jointed/fractured), cutterhead fatigue, and potential bearing problems
Mixed-strength rock	Impact disc loading may increase failure rates, concern for side wall gripping problems with open shields, possible steering problems.
Variable weathering, soil-like zones, faults	Slowed progress, if sidewall grippers not usable may need shield, immediate and additional support, potential for groundwater inflow, muck transport (handling and derails) problems, steering difficulty, weathering particularly important in argillaceous rock.
Weak rock at invert	Reduced utilization from poor trafficability, grade, and alignment - steering problems.

2.2.1 Lithology

Based on borehole logs the Original AutoCAD Cross-section is drawn by Yüksel Proje International Co. which clearly illustrates the lithological distribution of all units. In the borehole logs four lithological units have been distinguished. These are Hancılı formation, Ankara Clay, volcanic sequence and alluvium. The detailed look out to the lithological units in the area with their abbreviations on the Original AutoCAD cross-section was given in the previous chapter. This cross-section consists of all necessary drawings for assigning the excavation difficulty ranking values after converting it into .rvc format of TNT Mips which is the GIS program used in further analysis.

2.2.2 Hydrostatic Pressure

After completion of the boreholes systematic measurements of groundwater levels were carried out and this provides basis for the calculation of hydrostatic pressure (HP) values on the top of the rail elevation.

The hydrostatic pressure values are calculated by considering groundwater thickness on the top of the rail elevation. For every 10 m of the water column 1 atm pressure is assumed to be applied on the top of the rail. The groundwater levels are determined by systematic measurements carried out in each borehole. These measurements display how the existing aquifers are affected by natural and artificial hydrogeological factors. Through these measurements the seasonal fluctuations of the groundwater levels in the aquifers can be observed. Additionally the decrease in the groundwater levels by drainage during the excavations and increase by implementing the tunnel linings can also be observed. Such kinds of measurements are important in understanding the behavior of the aquifers against natural factors, artificial factors and their hydraulic properties. The following hydrographs (Figure 2.10. – 2.17.) display the systematic measurement results in the related part of the metro alignment.

It is expected to have maximum groundwater level in April - May and minimum in August. The part of the alignment investigated by the initial boreholes of the TA series is coherent to the natural seasonal changes of groundwater level.

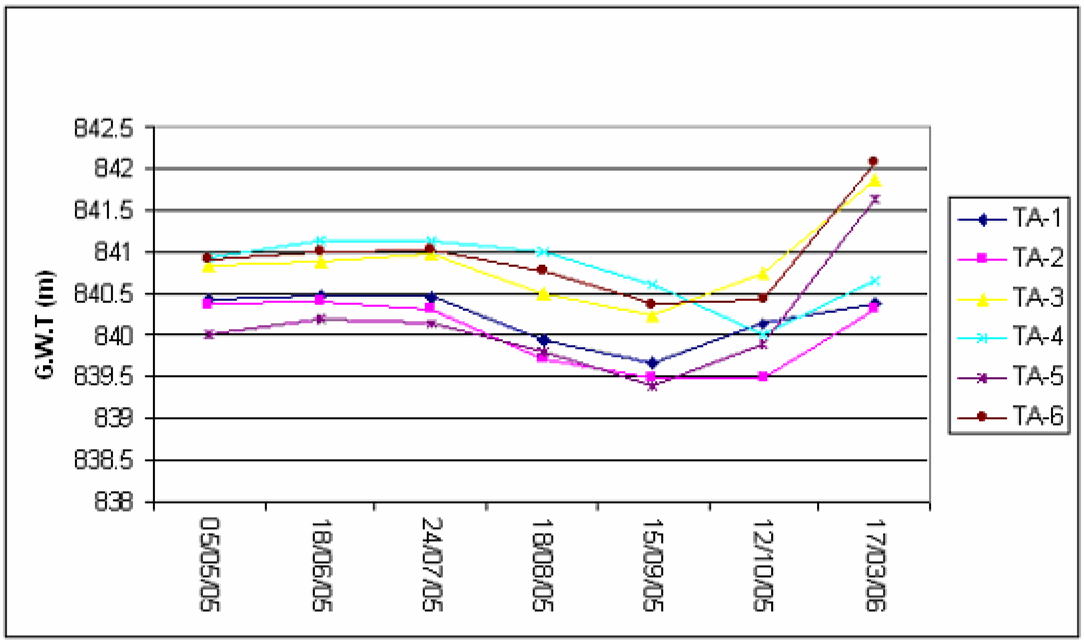

Figure 2.10. *Well hydrograph of TA-1 – TA-6 (Doyuran, 2006)*

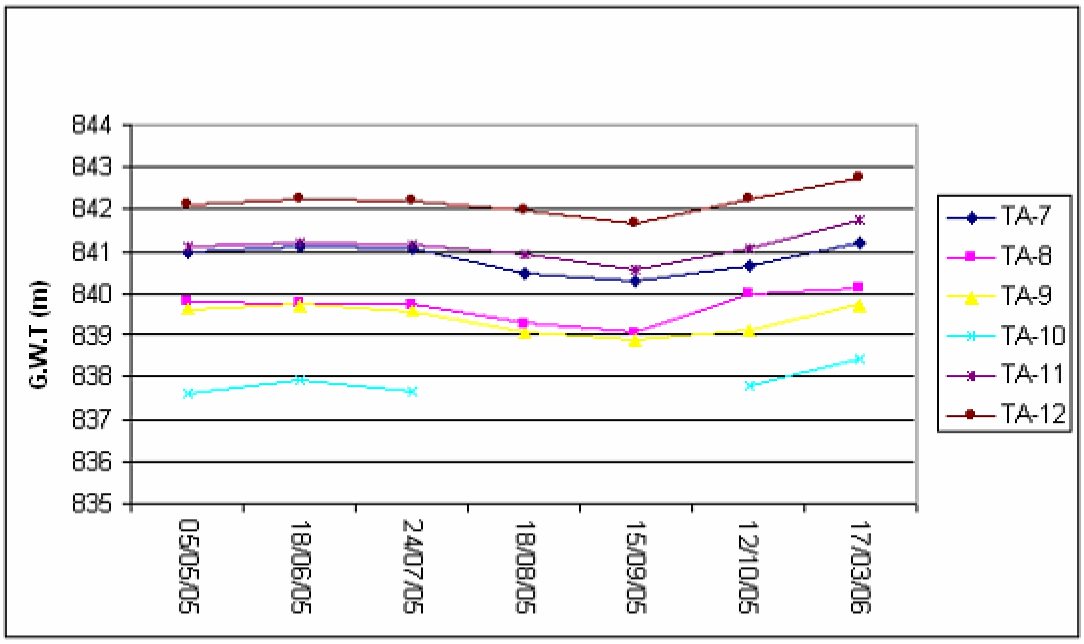

Figure 2.11. *Well hydrograph of TA-7 – TA-12 (Doyuran, 2006)*

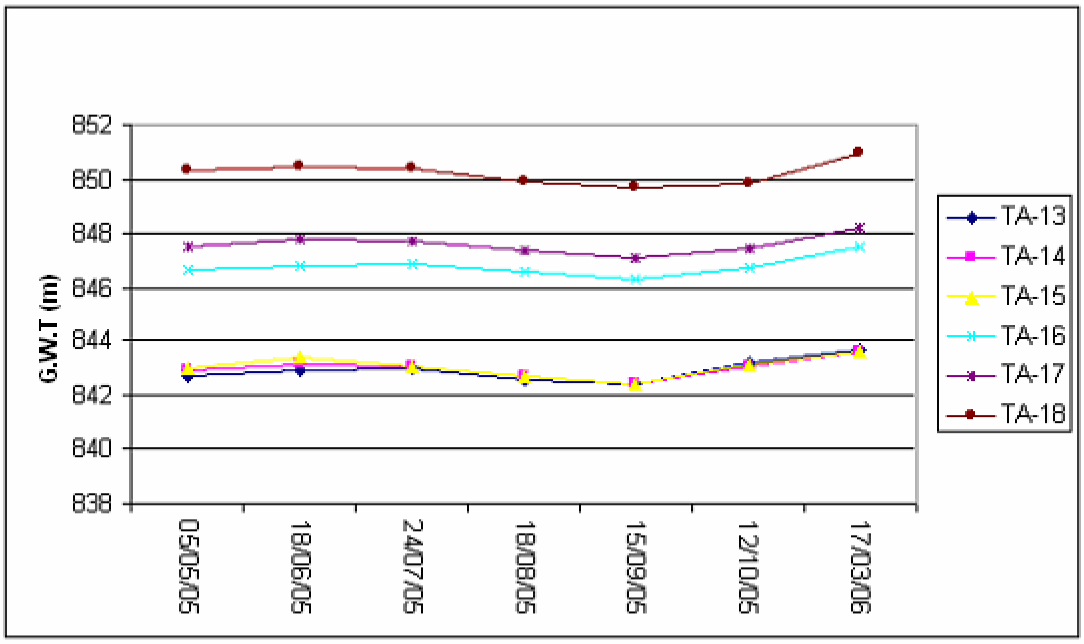

Figure 2.12. *Well hydrograph of TA-13 – TA-18 (Doyuran, 2006)*

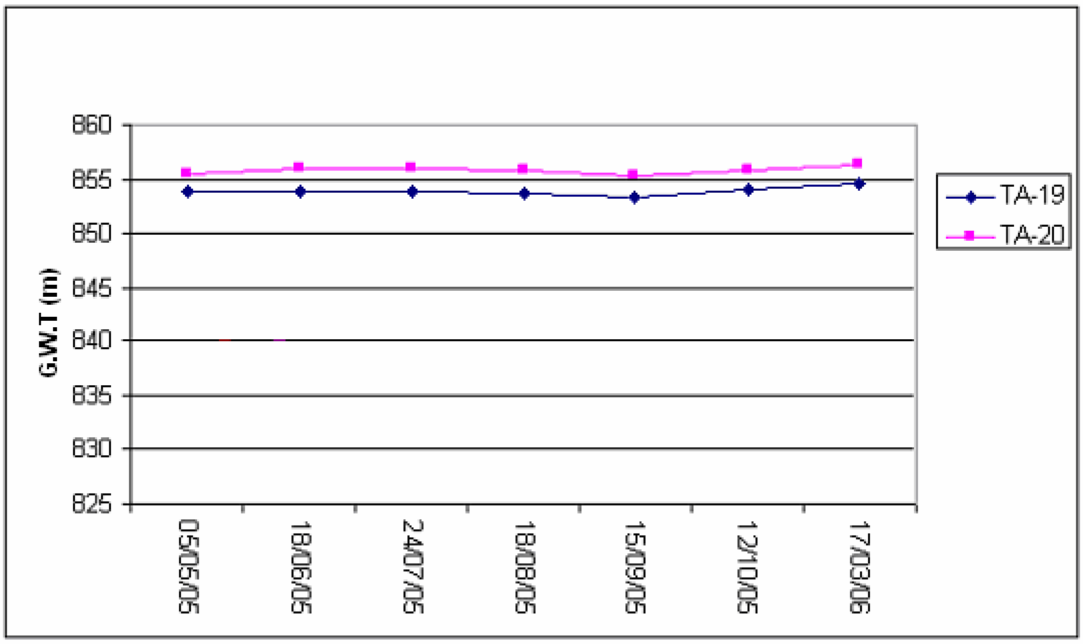

Figure 2.13. *Well hydrograph of TA-19 – TA-20 (Doyuran, 2006)*

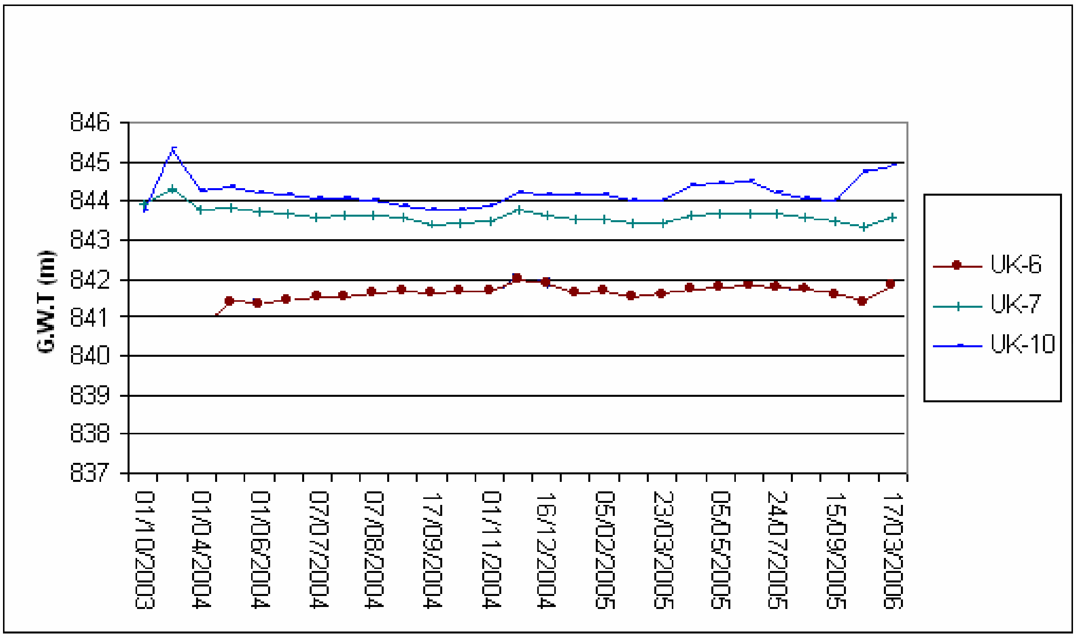

Figure 2.14. *Well hydrograph of UK-6 – UK-10 (Doyuran, 2006)*

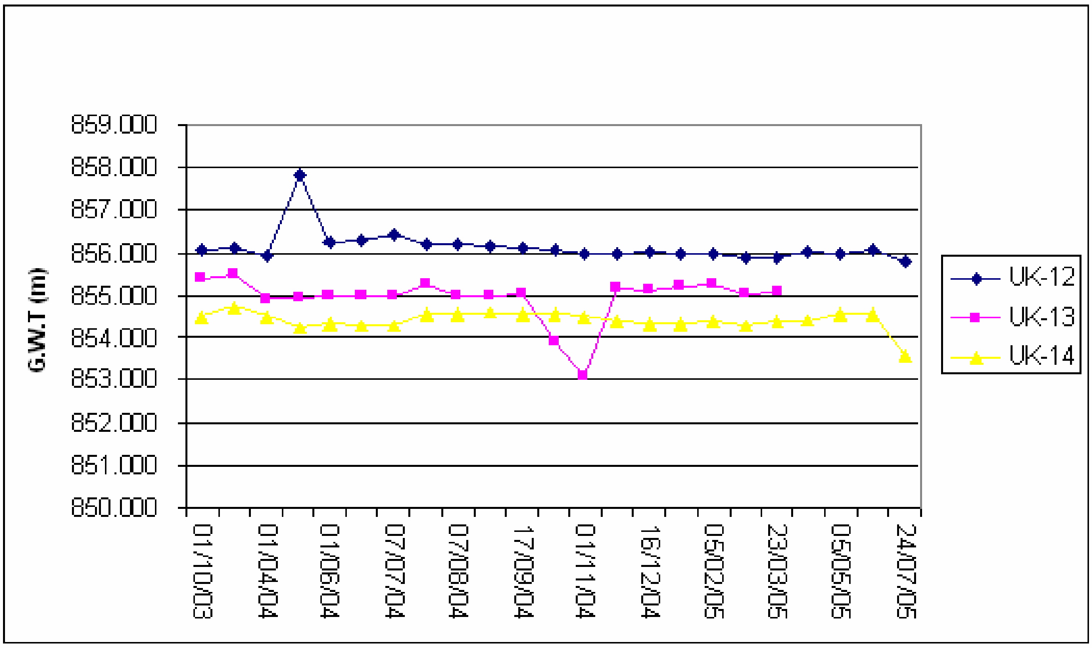

Figure 2.15. *Well hydrograph of UK-12 – UK-14 (Doyuran, 2006)*

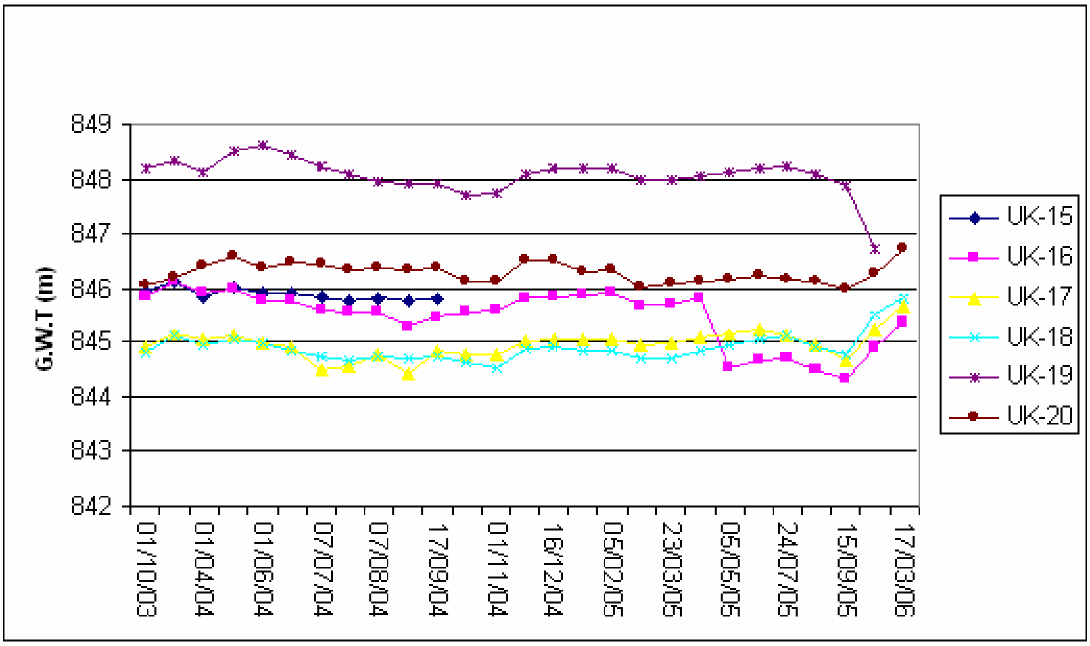

Figure 2.16. *Well hydrograph of UK-15 – UK-20 (Doyuran, 2006)*

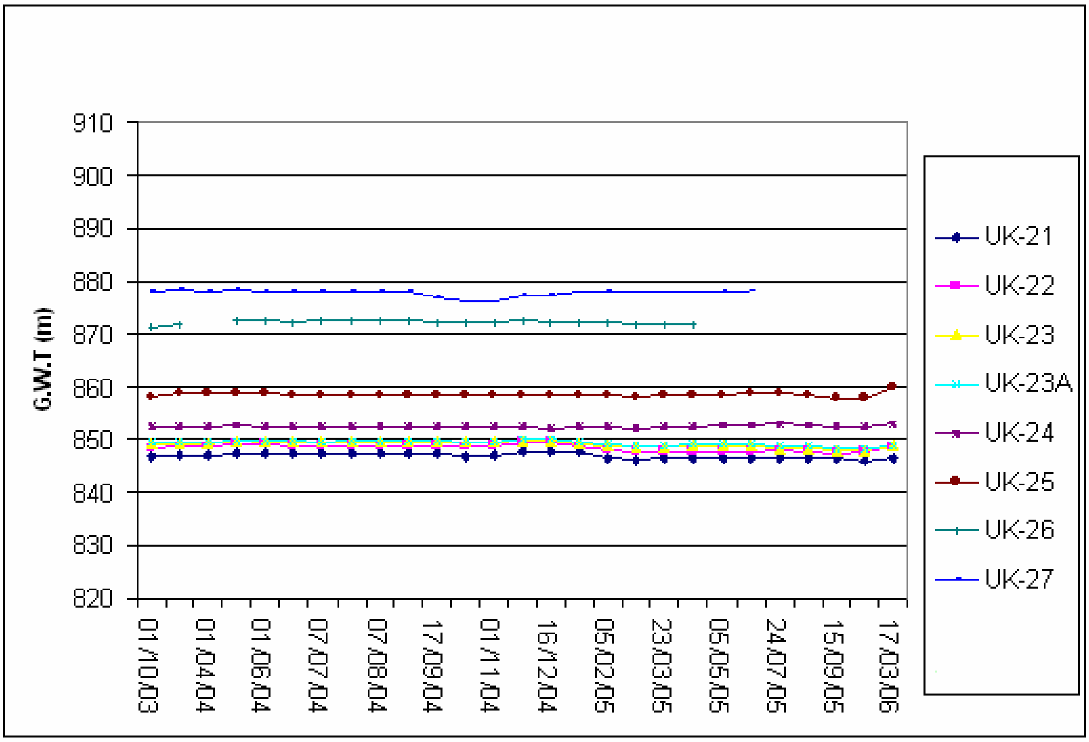

Figure 2.17. *Well hydrograph of UK-21 – UK-27(Doyuran, 2006)*

The portion of the alignment between UK-12 and UK-14 mainly refers to Ankara clay and fluctuations are less notable than in granular materials investigated by TA series.

The part of the alignment between UK-15 – UK-20 again refers to a part with granular material. Groundwater maintains the same level till summer and starts to fall in mid-summer.

The portion between UK-21 – UK-27 contains granular material with bedrock underlying. Seasonal fluctuations are not noticeable in these boreholes.

The water level measurements are taken on a monthly basis from each observation well. In general all groundwater levels have the maximum height in April – May and minimum in August. In March 2006, the groundwater level increased more than in the other years. The water table roughly follows the topography and it fluctuates within 2 m to 10 m below the surface. Within the volcanic series the hydrostatic pressure at the invert level of the tunnel ranges between 3, 5 bars and 1 bar. In the alluvium, however, the hydrostatic pressure is generally less than 2 bars. The calculations of hydrostatic pressure values are carried out according to the average of groundwater levels measured. The changes in vertical scale are considered while interpreting the results of hydrographs (Doyuran, 2006).

Table 2.1. displays the hydrostatic pressure values for UK-6 as an example.

Table 2.2. *Input values for calculating the hydrostatic pressure*

Bore Hole	Groundwater Level (m)	Top of the Rail Elevation (m)	Hydrostatic Pressure	
			(m)	(atm)
UK-6	841,7	830,5	11,2	1,1

After calculating hydrostatic pressures, the isobar lines with 0,5 atm interval is drawn in order to obtain the over all hydrostatic pressure distribution along the tunnel route. At this point all other unnecessary drawings on the Original AutoCAD Cross-section are erased to have a pure layer of hydrostatic pressure distribution.

2.2.3 Permeability

The amount of ground water that will be encountered depends on the porosity and permeability of the units and the height of the water table above the tunnel level. These factors are determined from field and laboratory investigations.

As mentioned earlier the granular lenses within Ankara clay has permeability values ranging between 10^{-6} m/sec and 10^{-7} m/sec. However, impervious levels dominate the lithology as expected. On the contrary the alluvial units overlying the volcanic sequence and Ankara clay are permeable. Alluvium is considered as the part of the study area that causes difficulty against tunnel boring. “Constant Head Permeability” and “Lugeon” tests were executed to determine the permeability (k) values with changing depth intervals. The results range between 10^{-4} – 10^{-9} m^3/sec and there are also very small values considered as zero (0) referring to the impervious layers. The detailed information about the permeability was given in Chapter I.

To provide ease in classifying the permeability data the coefficients in front of the exponentiation with base 1/10 are omitted. Six classes of permeability have been created from 10^{-4} to 10^{-9} (like 10^{-5}, 10^{-6} etc.) and 0 values are taken into account under the sixth class of 10^{-9}.

After determining the permeability data, each value is assigned to the related depth of the considered borehole on the Original AutoCAD Cross-section and other irrelevant drawings have been erased to obtain a basis for the "Permeability Layer".

2.2.4 Standard Penetration Test Values and Unified Soil Classification System (USCS) Results

Standard Penetration Test (SPT) is an in-situ testing method which provides the relative density of especially granular material. It is carried out in a borehole, by driving standard split spoon sampler using repeated blows. In the study area SPT is carried out in each borehole and the results are illustrated typically by means of blow numbers and SPT-N vs. depth graphs on boring logs for every test length.

Besides; the laboratory tests were also performed to determine the percentage of different grain sizes in soils driven from the borings and the content of each unit is symbolized according to the "Unified Soil Classification System" (USCS). Soil classification systems are set up to allow the expected properties of the soil in a given situation to be conveyed in shorthand form (Clayton et. al., 1995). The USCS is a soil classification system used in engineering and geology disciplines to describe the texture and grain size of a soil. The classification system can be applied to most unconsolidated materials, and is represented by a two-letter symbol (ASTM, 1985). Each letter is described below in *Table 2.3* :

Table 2.3. *Unified Soil Classification System (USCS)(American Society for Testing and Materials, 1985, D 2487-83, Classification of Soils for Engineering Purposes: Annual Book of ASTM Standards.)*

First and/or second letters	**Definition**	**Second letter**	**Definition**
G	gravel	P	poorly graded (uniform particle sizes)
S	sand	W	well graded (diversified particle sizes)
M	silt	H	high plasticity
C	clay	L	low plasticity
O	organic		

Major divisions			**Group symbol**	**Group name**
Coarse grained soils more than 50% retained on No.200 sieve	gravel > 50% of coarse fraction retained on No.4 sieve	clean gravel	GW	well graded gravel, fine to coarse gravel
			GP	poorly graded gravel
		gravel with >12% fines	GM	silty gravel
			GC	clayey gravel
	sand ≥ 50% of coarse fraction passes No.4 sieve	clean sand	SW	well graded sand, fine to coarse sand
			SP	poorly-graded sand
		sand with >12% fines	SM	silty sand
			SC	clayey sand
Fine grained soils more than 50% passes No.200 sieve	silt and clay liquid limit < 50	inorganic	ML	silt
			CL	clay
		organic	OL	organic silt, organic clay
	silt and clay liquid limit ≥ 50	Inorganic	MH	silt of high plasticity, elastic silt
			CH	clay of high plasticity, fat clay
		Organic	OH	organic clay, organic silt
Highly organic soils			Pt	peat

Test results provided by Yüksel Proje International Co. are evaluated for determining the distribution of soils through the alignment. The distribution of soil types between the stations according to the USCS is given in the *Table 2.4.*

Table 2.4. *USCS results between stations*

Stations	Boreholes	USCS Distribution
TANDOĞAN – EGO	TA-20-9	53% silt of high plasticity, elastic silt(MH), 10% silt(ML), %10 silty gravel(GM), 10% silty sand(SM), 6% clay(CL), 4%poorly graded gravel and 7% SW,GW, CH, SP
EGO – AKM	TA-9-3	33% silt of high plasticity, elastic silt(MH), 22% clay (CL), 14% silty sand(SM), 10% silt(ML), %9 silty gravel(GM), 5% clay of high plasticity, fat clay(CH) and 7% SW,GW,SC
AKM – ASKİ	TA-3-1-UK-6-8	20%silt of high plasticity, elastic silt(MH), 19% clay(CL), 17% silty sand(SM), 12% silt(ML), 7% well graded sand, fine to coarse sand (SW), 6% silty gravel(GM), 6% well graded gravel, fine to coarse gravel (GW), 5% clay of high plasticity, fat clay(CH) and 8% GP,SP,SC
ASKİ – DIŞKAPI	UK-8-13	34% silt of high plasticity, elastic silt(MH), 22% silty sand(SM), 10% silt (ML), 9% clay of high plasticity, fat clay(CH), 7% clay (CL), 7% clayey sand (SC) and 11% GM,SW,GW,SP
DIŞKAPI –METEOROLOJİ	UK-13- 18B	27% silty sand(SM), 15 % clay (CL), 12% well graded sand, fine to coarse sand (SW), 12% silt of high plasticity, elastic silt(MH), 9% silty gravel(GM), 5% well graded gravel, fine to coarse gravel (GW) and 20% GP,ML,CH,GW,SC
METEOROLOJİ – BELEDİYE	UK-18B-23	27% silty sand(SM), 16% clay (CL), 14% silty gravel(GM), 14% well graded sand, fine to coarse sand (SW), 9% well graded gravel, fine to coarse gravel (GW), 6% clayey sand (SC), 6% poorly graded gravel and 8% MH,ML,SP,GC + volcanic units
BELEDİYE – MECİDİYE	UK-23-26	30% silty sand (SM), 18% clayey sand (SC), 14% clay (CL), 9% silty gravel(GM), 7% well graded sand, fine to coarse sand (SW), 7% well graded gravel, fine to coarse gravel (GW) and 15% GP, ML, CH, SP, GC + volcanic units and Hancılı formation

Soil types and SPT- N values are helpful parameters in defining the geological features of the study site thus combining both SPT-N results and USCS parameters can be helpful for determining the TBM performance. Correlations between SPT-N values and soil or weak rock properties are wholly empirical, and depend upon an international database of information. Because the SPT is not fully standardized these correlations cannot be considered particularly accurate in some cases, and it is therefore important that users of SPT and the data it produces have a good appreciation of those factors controlling the test, which are:

1. variations in the test apparatus
2. the disturbance created by boring the hole; and
3. the soil into which it is driven. (Clayton et. al.,1995)

In cohesive soils, especially those that are relatively weak and compressible, penetration tests provide only a guide for preliminary estimates. Instead of empirical methods, analytical methods are used for more critical situations (Hunt, 1986). As this study covers a wide construction area and aims to give a general idea about the performance of the TBM in operation, SPT correlation tables versus the cohesiveness of the soils are used.

By considering the USCS results, SPT-N values and the conversion tables given below, the units in the study area are divided into 11 main classes. *Table 2.5. (a)* is used to classify the units that are non – cohesive according to the USCS results. Blow number (SPT-N) is found from the table for each sand and the test depth is classified by means of compactness. Same procedure is followed for cohesive units by using the *Table 2.5. (b)*.

Table 2.5. *(a) Correlations between SPT-N Value and compactness for non-cohesive units (Hunt, 1986)*

Compactness	N(SPT)
Very Loose	<4
Loose	4-10
Medium Dense	10-30
Dense (compact)	30-50
Very Dense	>50

Table 2.5. *(b) Correlations between SPT – N and consistency for cohesive units (Hunt, 1986)*

Consistency	N(SPT)
Very Soft	2
Soft	2-4
Medium(firm)	4-8
Stiff	8-15
Very Stiff	15-30
Hard	30

An example table (*Table 2.6.*) below to illustrate the distribution of different units according to their SPT-N and USCS results. The geotechnical descriptions on borehole logs, SPT-N values and USCS results are both taken into account while preparing the SPT layer for further analysis. At points where geotechnical descriptions and USCS results are not in agreement, information coming from the laboratory tests were used. For example the 15,25m test level* of UK-7 is explained as yellowish brown, medium, silty sand. It contains 25-35 % fines and silty-clayey bands.

According to USCS this level is composed of fine grained soil between MH and CL thus it is considered as a cohesive material although described as non - cohesive.

Table 2.6. *Sample table for SPT-N correlations by considering USCS results*

Borehole **UK-7**	**Depth**	**SPT-N**	**Unit**
	1,75	16	AF(Clay)
	3,25	16	AF(Clay)
	4,75	33	Sand (SP-SM)
	6,25	27	Sand (SC)
	7,75	5	Clay(ML)
	9,25	4	Clay/SiltyClay (ML)
	10,75	9	Clay/SiltyClay (ML)
	12,25	39	Sand (SP)
	13,75	16	Clay (CL)
	15,25	14	Sand *
	16,75	10	Alluvium(Clay) (CL)
	18,25	28	AnkaraClay (MH)
	19,7	20	AnkaraClay (MH)
	21,25	21	AnkaraClay (MH)
	22,75	24	AnkaraClay (MH)
	24,3	R(52)	Sand (SP)
	25,75	45	Sand (SM)
	27,25	46	Clay/SiltyClay (MH)
	28,75	38	Clay/SiltyClay (MH)
BH End	**28,95**		

CHAPTER 3

MULTICRITERIA DECISION ANALYSIS (MCDA)

Decision analysis is a set of systematic procedures for analyzing complex decision problems. The basic strategy is to divide the decision problem into small, understandable parts; analyze each part; and integrate the parts in a logical manner to produce a meaningful solution. Decision making itself, however, is broadly defined to include any choice or selection of alternative courses of action, and is therefore of importance in many fields in both the social and environmental sciences including geographical information sciences (Malczewski, 1997). In this thesis the decision problem can be described as determining the parts of the study site that affects the TBM performance. The site has been evaluated by means of geology, permeability, hydrostatic pressure and the results of standard penetration test with the combination of USCS as the parts of the problem.

MCDA have various aspects but in this chapter mainly the points that are related to this study are explained.

3.1. Elements and Classification of Multicriteria Decision Problems

In general, MCDA problems involve six components (Keeney and Raiffa, 1976; Pitz and McKillip, 1984):

I. A goal or a set of goals the decision maker wants to achieve,
II. The decision maker or a group of decision makers involved in the decision making process with their preferences with respect to the evaluation criteria,
III. A set of evaluation criteria (objectives and/or physical attributes)
IV. The set of decision alternatives,

V. The set of uncontrollable (independent) variables or states of nature (decision environment)

VI. The set of outcomes or consequences associated with each Multicriteria decision making (MCDM) problem can be classified on the basis of the major components of multicriteria decision analysis;

a. Multiobjective decision making (MODM) versus multiattribute decision making (MADM)
b. Individual versus group decision maker problems, and
c. Decisions under certainty versus decisions under uncertainty (*Figure 3.1.*)

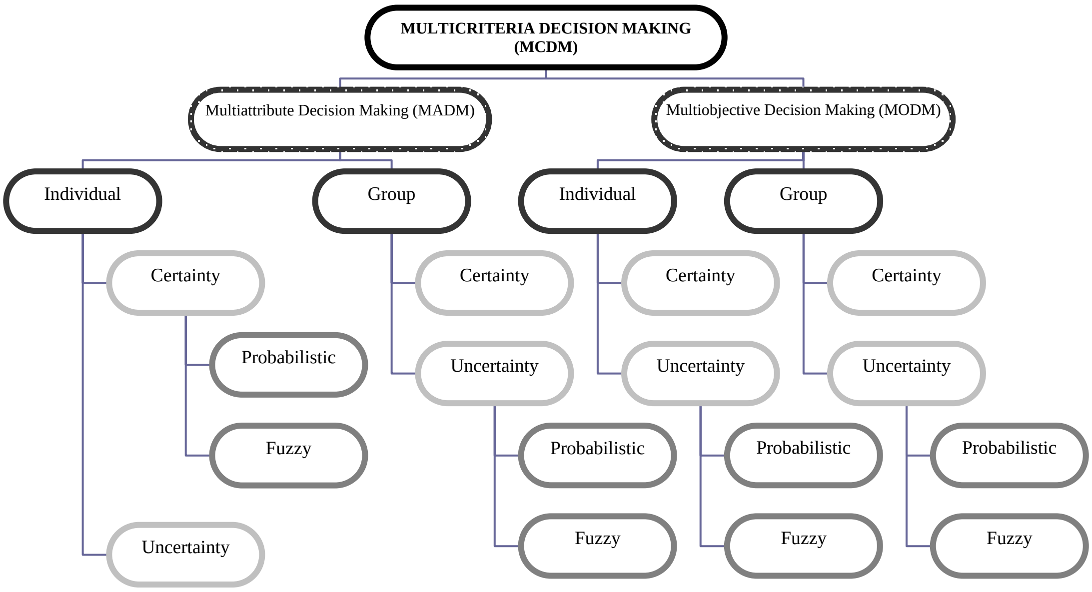

Figure 3.1. *Classification of multicriteria decision problems (Malczewski,1999)*

The distinction between MADM and MODM is based on the evaluation criteria which are the standards of judgments or rules on which the alternatives are ranked according to their desirability. Criterion is a general term and includes both the concepts of attributes and objectives.

Attributes are the properties of elements of a real-world geographical system. More specifically, an attribute is a measurable quantity or quality of a geographical entity or a relationship between geographical entities. In the context of a decision-making problem, the entities and the relationships are referred to as the objects of decision. An attribute is used to measure performance in relation to an objective. It can be thought of as the means or information sources available to the decision maker for formulating and achieving the decision maker's objectives (Starr and Zeleny, 1977).

An objective is a statement about the desired state of the system under consideration. It indicates the directions of improvement of one or more attributes. Objectives are functionally related to, or derived from, a set of attributes. For any given objective, several different attributes might be necessary to provide complete assessment of the degree to which the objective might be achieved (Malczewski, 1999).The following table provides the use and comparison of MODM and MADM approaches briefly;

Table 3.1. *Comparison of MODM and MADM Approaches (Hwang and Yoon, 1981; Starr and Zeleny, 1977)*

	MODM	**MADM**
Criteria defined by:	Objectives	Attributes
Objectives defined:	Explicitly	Implicitly
Attributes defined:	Implicitly	Explicitly
Constraints defined:	Explicitly	Implicitly
Alternatives defined:	Implicitly	Explicitly
Number of alternatives:	Infinite (large)	Finite (small)
Decision maker's control:	Significant	Limited
Decision modeling paradigm:	Process-oriented	Outcome-oriented
Relevant to:	Design/search	Evaluation/choice
Relevance of geographical data structure:	Vector-based GIS	Raster-based GIS

Both MADM and MODM problems can be further classified as individual and group decision making depending on the goal-preference structure of the decision maker(s). If there is a single goal preference, the problems is considered as individual decision making regardless of the number of decision makers involved in the process. However, if the individual or interest groups are characterized by different goal preferences, the problem becomes the group decision making (Malczewski, 1997).

Finally, MCDM problems can be categorized into decision under certainty and decisions under uncertainty depending on the amount of information about the decision situation that is available to the decision maker(s) and analyst(s) (Keeney and Raiffa, 1976; Keeney, 1980). If the decision maker has perfect knowledge of the decision environment, then the decision is made under conditions of certainty. Most real-world decisions involve some aspects that are unknowable or very difficult to predict. This type of decision making is referred to as decisions under conditions of uncertainty. The decision under uncertainty may be further subdivided into two categories; probabilistic and fuzzy decision making (Leung, 1988; Eastman et. al., 1993). The probabilistic decisions are handled by probability theory and statistics. And the outcome of a stochastic event is either true or false. However, if the situation is ambiguous, the problem is structured as the degree of how much an event belongs to a class. This type of problems is handled by fuzzy set theory (Zadeh, 1965).

3.2. Steps of Multicriteria Decision Analysis

Decision making involves a sequence of activities that starts with decision problem recognition and ends with recommendations. The main steps of MCDA can be summarized as:

1. Problem definition
2. Determining evaluation criteria (attributes and objectives)
3. Generating alternatives
4. Weighting the criteria
5. Determining the proper decision rules
6. Sensitivity analysis
7. Recommendation

The *Figure 3.2.* illustrates these steps in a framework that is organized in terms of the sequence of activities involved in a spatial multicriteria decision making analysis. There are number of alternative ways to organize the sequence of activities in the decision-making process but the following framework mainly focuses on the "Value-Focused Approach" that uses the evaluation criteria as the fundamental element of the decision analysis (Malczewski, 1999).

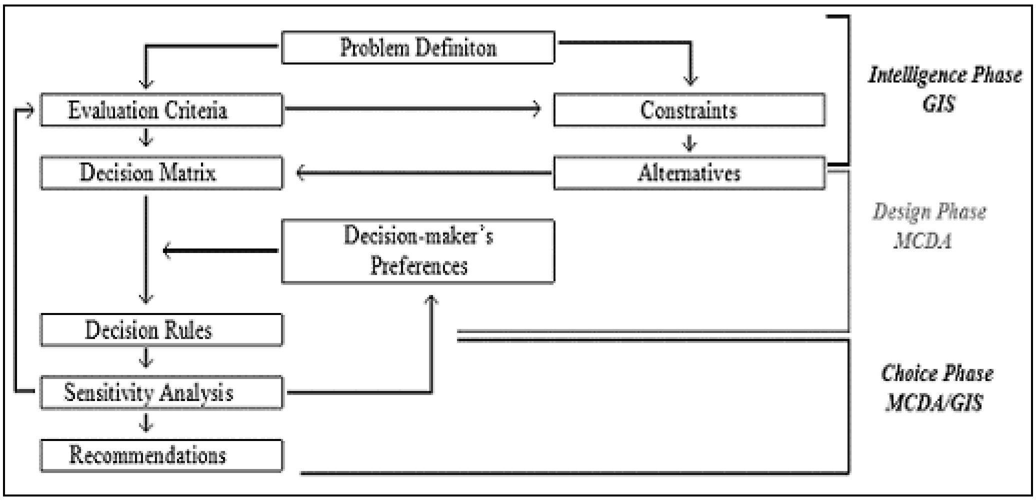

Figure 3.2. *Framework for spatial multicriteria decision analysis (Malczewski, 1999)*

3.2.1. Problem Definition

Any decision-making process begins with the recognition of the decision problem. The decision problem is perceived difference between the desired and existing states of a system. It is a gap between the desired and existing states of a system. The problem definition overlaps the intelligence phase (see *Figure 3.2.*) of decision making (Malczewski, 1999).

3.2.2. Evaluation Criteria

Once the decision problem is defined, the spatial multicriteria analysis focuses on the set of evaluation criteria (objectives and attributes). This step involves specifying;

1. A comprehensive set of objectives that reflects all concerns relevant to the decision problem, and
2. Measures for achieving those objectives that are called attributes.

The evaluation criteria are associated with geographical entities and relationships between entities and therefore can be represented in the form of maps (Malczewski, 1999). In this thesis for handling the data in GIS environment cross-section layers have been created standing for those maps that are also called evaluation criterion maps.

3.2.3. Alternatives

The process of generating alternatives should be based on the value structure and be related to the set of evaluation criteria. To each alternative there is assigned a decision variable. Variables are used by the decision maker to measure the performance of attributes. Depending on the problem situation, the decision variables may be deterministic, probabilistic or linguistic.

In a real-world situation, very few spatial decision problems can be considered unconstrained. Constraints represent restrictions imposed on the decision space. They determine the set of feasible alternatives. In terms of GIS, the constraints are used to eliminate points, lines, polygons and/or rasters characterized by certain attributes and/or certain values of attributes from consideration (Malczewski, 1999).

3.2.4. Criterion Weights

At this stage, the decision maker's preferences with respect to the evaluation criteria are incorporated into the decision model. The preferences are typically expressed in terms of the weights of relative importance assigned to the evaluation criteria under consideration. The purpose of criterion weights is to express the importance of each criterion relative to other criteria. Given the set of alternatives, attributes, and associated weights, the input data can be organized in the form of a decision matrix or table (Malczewski, 1999). Assigning weights of importance to evaluation criteria accounts for;

1. the changes in the range of variation for each evaluation criterion, and
2. the different degrees of importance being attached to these ranges of variation (Kirkwood, 1997).

A number of criterion-weighting procedures based on the judgments of the decision makers have been proposed in the multicriteria decision literature (Pitz and McKillip, 1984; Nijkamp and Rietveld, 1986; Schoemaker and Waid, 1982; Kleindorfer et al., 1993). The procedures include ranking, rating, pairwise comparison and trade-off analysis. The comparison of their major features is shown in *Table 3.2*.

Table 3.2. *Summary of methods for assessing criterion weights (Pitz and McKillip, 1984; Nijkamp and Rietveld, 1986; Schoemaker and Waid, 1982; Kleindorfer et al., 1993)*

Methods Features	**Ranking**	**Rating**	**Pairwise Comparison**	**Trade-off Analysis**
Number of judgments	n	n	n(n-1)/2	< n
Response scale	Ordinal	Interval	Ratio	Internal
Hierarchical	Possible	Possible	Yes	Yes
Underlying theory	None	None	Statistical/heuristic	Axiomatic/deductive
Ease of use	Very easy	Very easy	Easy	Difficult
Trustworthiness	Low	High	High	Medium
Precision	Approximations	Not precise	Quite precise	Quite precise
Software availability	Spreadsheets	Spreadsheets	EXPERT CHOICE (EC)	LOGICAL DECISION (LD)
Use in GIS environment	Weights can be imported from a spreadsheet	Weights can be imported from a spreadsheet	Component of IDRISI	Weights can be imported from LD

3.2.4.1. Ranking Methods

The simplest method for assessing the importance of weights is to arrange them in rank order; that is, every criterion under consideration is ranked in the order of the decision maker's preference. The ranking methods are very attractive, due to their simplicity. However, practical usefulness of these methods is limited by the number of criteria to be ranked. In general, the larger the number of the criteria used, the less appropriate is the method (Voogd, 1983). The ranking method can also be criticized for a lack of theoretical foundation. In this thesis there are four data sets determined that are under the experts knowledge so simple ranking method is found to be practical and descriptive.

3.2.4.2. Rating Methods

The rating methods require the decision maker to estimate weights on the basis of a predetermined scale; for example, a scale of 0 to 100 can be used. One of the simplest rating methods is the point allocation approach. It is based on allocating points ranging from 0 to 100, where 0 indicates that the criterion can be ignored, and 100 represents the situation where only one criterion need to be considered in a given decision situation.

An alternative to the point allocation method is ratio estimation procedure, a modification of the point allocation method. It starts by assigning an arbitrary weight to the most important criterion, as identified by one of the ranking methods. A score of 100 is assigned to the most important criterion and proportionally smaller weights are then given to criteria lower in the order. Then the score assigned to the least important attribute is taken as anchor point for calculating the ratios. The rating method can be criticized for the lack of theoretical foundation or formal foundations and also the meaning of the weights assigned to the criteria might be difficult to justify (Malczewski, 1999).

3.2.4.3. Pairwise Comparison Method

The pairwise comparison method was developed by Saaty (1980). This method involves pairwise comparisons to create a ratio matrix. This method can be described in three steps:

1. Development of the pairwise comparison matrix: The method employs an underlying scale with values from 1 to 9 to rate the

alternative preferences for two criteria. Then the desired scores are written to the comparison matrix and the remaining entries are found accordingly. Scale of pairwise comparison can be seen in *Table 3.3.* .

Table 3.3. *Scale for pairwise comparison Saaty (1980)*

Intensity of importance	**Definition**
1	Equal importance
2	Equal to moderate importance
3	Moderate importance
4	Moderate to strong importance
5	Strong importance
6	Strong to very strong importance
7	Very strong importance
8	Very to extremely strong importance
9	Extreme importance

2. Computation of the criterion weights: This step involves the following operations:

   (a) Sum the values in each column of the pairwise comparison matrix;

   (b) Divide each element in the matrix by its column total (the resulting matrix is referred to as the normalized pairwise comparison matrix); and

   (c) Compute the average of the elements in each row of the normalized matrix, that is, divide the sum of normalized scores for each row by the number of criteria.

3. Estimation of the consistency ratio: In this step we determine if our comparisons are consistent. It involves the following operations;

(a) Determine the weighted sum vector by multiplying the weight of the first criterion times the first column of the original pairwise comparison matrix, then multiply the second weight times the second column, the third criterion times the third column of the original matrix, finally, sum these values over the rows; and
(b) Determine the consistency vector by dividing the weighted sum vector by the criterion weights determined previously. After this lambda (λ) and consistency index (CI) has to be found.

(c) The value for lambda is simply the average value of the consistency vector and the calculation of CI is based on the observation that λ is always greater than or equal to the number of the criteria under consideration (n) for positive, reciprocal matrixes, and λ=n if the pairwise comparison matrix is a consistent matrix. Accordingly, λ-n can be considered as a measure of the degree of inconsistency. This measure can be normalized as follows:

$$CI = (\lambda - n)/\ (n-1)$$

The CI term, referred to as consistency index, provides a measure of departure from consistency. Further, we can calculate the consistency ratio (CR), which is defines as follows:

$$CR = CI/RI$$

where RI is the random index, the consistency index of a randomly generated pairwise comparison matrix. (see *Table 3.4.*)

Table 3.4. *Random Inconsistency Indices (RI) for n = 1,2,3,...,15 (Saaty, 1980)*

n	*RI*	N	*RI*	n	*RI*
1	*0.00*	6	*1.24*	11	*1.51*
2	*0.00*	7	*1.32*	12	*1.48*
3	*0.58*	8	*1.41*	13	*1.56*
4	*1.90*	9	*1.45*	14	*1.57*
5	*1.12*	10	*1.49*	15	*1.59*

The advantage of pairwise comparison method is that only two criteria have to be considered at a time but if many criteria are being compared, the method may get very large and advantage turns into a disadvantage. The method can easily be implemented in a spreadsheet environment (Kirkwood, 1997) and the method has been incorporated into GIS-based decision-making procedures (Eastman et al. 1993)

3.2.4.4. Trade-off Analysis Method

This approach requires the decision maker to compare two alternatives with respect to two criteria at a time and assess which alternative is preferred. A critical assumption behind this method is that trade-offs the decision maker is willing to make between any two criteria do not depend on the levels of the other criteria (Malczewski, 1999). The weakness of this method is the decision maker is presumed to obey the axioms and can make fine grained in difference judgments but in addition, the method can be implemented within the spreadsheet environment (Kirkwood, 1997).

3.2.5. Decision Rules

The unidimensional measurements (geographic data layers) and judgments (preferences and uncertainty) must be integrated to provide an overall assessment of the alternatives. This is accomplishes by an appropriate decision rule or aggregation function. The set of decision consequences forms the decision outcome space. Since a decision rule provides an ordering of all alternatives according to their performance with respect to the set of evaluation criteria, the decision problem depends on the selection of the best outcome (or an ordered set of outcomes) and the identification of the decision alternative (or alternatives) yielding this outcome (or outcomes) (Malczewski, 1999) .

3.2.5.1. Applied Decision Rules under the Scope

When dealing with multiple objective or multiple attribute decisions, combination of methods is often more effective than a single technique. To facilitate comparisons among the various methods they are grouped into four main categories:

1. Weighting methods
2. Sequential elimination methods
3. Mathematical programming methods
4. Spatial proximity methods

The class of weighting methods has received the most attention and particular models within this class have been the most widely applied. Weighting methods can be classified as follows:

1. Inferred preferences
   a. Linear regression
   b. Analysis of variance
   c. Quasi-linear regression
2. Directly assessed preferences: general aggregation
   a. Trade-offs
   b. Simple Additive Weighting (SAW)
   c. Hieararchical Additive Weighting
   d. Quasi-additive weighting
3. Directly assessed preferences: specialized aggregation
   a. Maximin
   b. Maximax

Although these weighting methods seem very diverse, they all have the following characteristics:

- A set of available alternatives with specified attributes and attribute values;

- A process comparing attributes by obtaining numerical scaling of attribute values (intra-attribute preferences) and numerical weights across attributes (inter-attribute preferences);
- A well-specified objective function for aggregating the preference into a single number for each alternative;
- A rule for choosing the alternative (or rating the alternatives) on the basis of the highest weight (MacCrimmon, 1973).

Not to give unnecessary information that maybe out of the scope of this thesis only the weighting methods that have been used in analysis is going to be described instead of explaining all of the weighting methods one by one. As mentioned before applying more than one analysis method gives more satisfactory results in MCDA. To this end “Simple Additive Weighting” (SAW) and “Analytical Hierarchy Process” (AHP), which is proposed by Saaty after McCrimmon’s classification of weighting methods, are found to be appropriate for this study.

3.2.5.1.1. Simple Additive Weighting (SAW)

Simple additive weighting (SAW) methods are the most often used techniques for tackling spatial multi-attribute decision making. The techniques are also referred to as weighted linear combination (WLC) or scoring methods. They are based on the concept of weighted average (Malczewki, 1999).

In the direct assessment weighting method, simple additive weighting, to each of the attributes, the decision maker assigns importance weights which become the coefficient of the variables. To reflect decision maker’s marginal worth assessment within attributes, the decision maker also makes a numerical scaling of intra-attribute values. Decision maker then can obtain a total score for each alternative simply by multiplying the scale rating for each attribute value by the importance weight assigned to the attribute and then summing all these products over all attributes. After the total scores are computed for each alternative, the alternative with the highest score is the one prescribed to the decision maker (Mac Crimmon, 1973). The decision rule evaluates each alternative, Ai, by the following formula:

$$A_i = \Sigma_j \, w_j \, x_{ij}$$

where x_{ij} is the score of the *i* th alternative with respect to the *j* th attribute, and the weight w_j is a normalized weight, so that $\Sigma \, w_j = 1$. The weights represent the relative importance of the attributes. The most preferred alternative is selected by identifying the maximum value of A_i ($i = 1,2,\ldots,m$)

The GIS based Simple Additive Weighting method involves the following steps:

1. Definition of the set of evaluation criteria (map layers) and the set of feasible alternatives,
2. Standardization of each criterion map layer,
3. Definition of the criterion weights,
4. Construction of the weighted standardized map layers,
5. Generation of the overall score for each alternative using the overlay operation,
6. Ranking of the alternatives according to the overall performance score (Malczewski, 1999).

Although this technique is easy to apply, it runs the risk of ignoring interactions among the attributes (MacCrimmon, 1973). This approach can also be criticized for its ignorance of the definition of the units used for each attribute. Thus the greatest disadvantage of the SAW methods is that they tend to be ad hoc procedures with little theoretical foundation to support them. However, because they are easy to use, SAW methods are actually quite widely applied in real-world settings (Massam, 1988; Janssen, 1992; Eastman et al., 1993)

3.2.5.1.2. Analytical Hierarchy Process

The analytic hierarchy process (or AHP), as proposed by Saaty (Saaty, 1980, 1983, 1990, and 1994), is a later development after weighted sum and product models and it has recently become increasingly popular. The AHP is a decision analysis method that ranks alternatives based on a number of criteria. Its robust design enables the decision-maker to incorporate subjectivity, experience, and knowledge intuitively and naturally into the decision process. AHP considers both qualitative and quantitative information and combines them by decomposing unstructured problems into systematic hierarchies.

AHP uses subjective assessment followed by "simple" matrix algebra to establish the optimal rank (and weighted average score) for alternatives based on predetermined criteria. Given a set of criteria, the analyst repeatedly compares one criterion to another until all possible pair-wise comparisons are completed. If the criteria are quantitative, then deterministic mathematical relationships of each pair-wise comparison may be used. If the criteria are non-quantitative, the subjective scale shown in *Table 3.3.* (Saaty, 1980) is used.

The AHP principles can also be described in three steps;

1. Develop the AHP hierarchy: consists of decomposing the decision problem into a hierarchy that contains the most important elements of it. The hierarchical structure consists of four levels; goal, objectives, attributes and alternatives. The attribute concept links the AHP method to GIS-based procedures.

2. Compare the decision elements on a pair-wise base: it involves three steps; (a) development of a comparison matrix at each level of the hierarchy, beginning at the top and working down; (b)

computation of the weights for each element of the hierarchy and; (c) estimation of the consistency ratio.

3. Construct an overall priority rating: The final step is to combine the relative weights of the levels obtained in the above step to produce composite weights. This is done by means of a sequence of multiplications of the matrices of relative weights at each level of the hierarchy. First, the comparison matrix is squared and the row sums are calculated and normalized for each row in the comparison matrix. This process is continued when the difference between the normalized weights of the iterations become smaller than a prescribed value (Saaty, 1990).

Although AHP has flexibility, ease of use and been incorporated into GIS environment (Banai, 1993; Eastman et al.,1993; Jankowski, 1995; Siddiqui et al., 1996); ambiguity in the meaning of the relative importance of one element of the decision hierarchy when compared to another element, the number of comparisons for large problems the use of 1 to 9 scale and the argument that the type of the questions asked during the process of pairwise comparisons are meaningless (Belton, 1986) and can be considered as the disadvantages of AHP (Malczewski, 1999).

3.2.6. Sensitivity Analysis

Subsequent to obtaining a ranking of alternatives, sensitivity analysis should be performed to determine robustness. Sensitivity analysis is defined as a procedure for determining how the recommended course of action is affected by changes in the inputs of the analysis. To be more specific, it aims at identifying the effects of the changes in the inputs (geographical data and the preferences of the decision maker) on the outputs (ranking of alternatives) (Malczewski, 1999).

3.2.7. Recommendation

The end result of a decision-making process is a recommendation for future action. The recommendation should be based on the ranking of the alternatives and sensitivity analysis. It may include the description of the best alternative or a group of alternatives considered candidates for the implementations. The solutions to spatial multicriteria decision problems should be presented in both decision geographical space and outcome space (Malczewski, 1999).

MCDA are used widespread in different fields of different studies. Geographic decision making is one of those fields and there are many examples like site selection studies, land suitability analysis, risk assessment studies etc. All cited works are carried out on maps of the study sites but in this thesis the place subject to construction is linear when compared to other types of works and additionally the structure to be constructed is an underground project. This kind of study is not common in literature so GIS application on a vertical platform as a cross-section by using MCDA, which is a part of this thesis, is prone to be one of the first examples in its field.

CHAPTER IV

PRODUCTION OF THE LAYERS

The data in GIS systems are most commonly organized by separate thematic maps or set of data. Each of these thematic maps is referred to as a map layer, coverage, or level. A map layer is a set of data describing a single characteristic of each location within a bounded geographical area. Only one item of information is available for each location within a single layer (Malczewski, 1999). In this study cross-section layers have been created instead of map layers because the analyses have been carried out, on the vertical plane through a tunnel alignment. The cross-section layers can be seen on *Figure 4.1.*

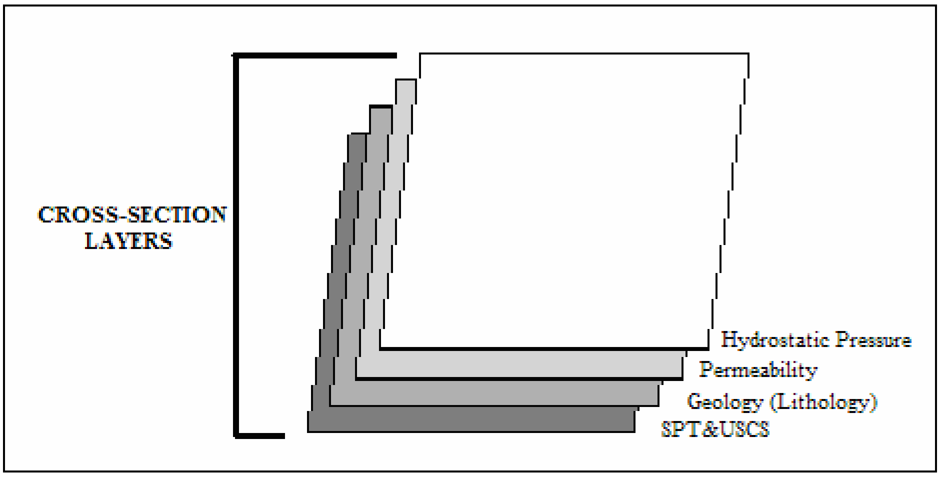

Figure 4.1. *The input cross-section data layers*

By using the Original AutoCAD Cross-section as a base four different cross-section layers have been produced in TNT Mips. The files coming from AutoCAD are in ".dxf" format which are not operable in TNT Mips. There are some cadastral data saved in DXF files, created by using AutoCAD software.

In order to manage all related data on the same platform, cross-section drawn by using AutoCAD software in DXF format was converted into a .rvc file of TNT Mips and further analysis were performed on GIS platform.

To ensure that all maps in a GIS database overlay accurately, the data set is georeferenced to a common coordinate system. In many countries the Universal Transverse Mercator (UTM) projection is commonly used to define coordinates in GIS (Malczewski, J., 1999). Due to the exaggeration of the scale of cross-section prepared in AutoCAD and because the processes of this study are not carried out on a horizontal plane assigning original UTM coordinates is not meaningful. To provide the exact overlap of cross-sectional layers an arbitrary user-defined coordinate system has been used as 1 unit for y-axis and 6 units for x-axis. In *Figure 4.2.* the coordinates for each side of the cross-section can be seen.

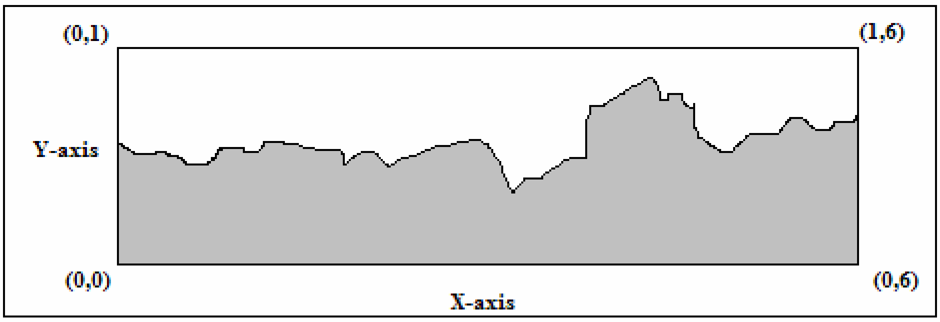

Figure 4.2.*The arbitrary user-defined coordinates of the study area*

4.1. Lithology Layer

The lithology layer base prepared in AutoCad illustrates all geological units in the study area. Polygons for each lithological unit are created on the cross-section where they are encountered and after that this .dxf file is converted into .rvc for further analysis in TNT Mips. The .dxf and .rvc formats of this layer can be seen in *Figure 4.3 and 4.4.* The colored lines belong to the .dxf format of the drawing and gray tones indicate the raster file prepared in TNT Mips. The lithological units drawn on the AutoCAD layer and their ranking values can be seen in *Table 4.1.* Ranking values are assigned by the expert decision makers who are familiar with the characteristics of the study site.

Table 4.1. *Lithological units and their ranking values*

Lithological Unit	Ranking
Artificial Fill	1
Alluvium	
clay	2
silt	6
sand	9
gravel	10
Volcanic Sequence	3
Hancili Formation	1
Ankara Clay	1

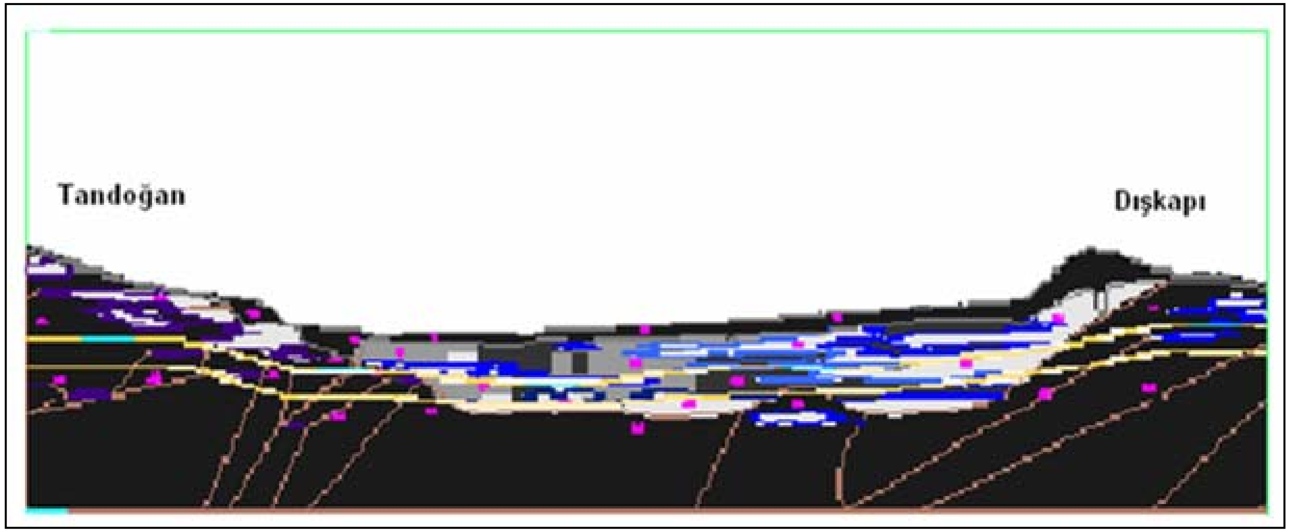

Figure 4.3. *The lithology layer between Tandoğan - Dışkapı stations in .dxf and .rvc formats*

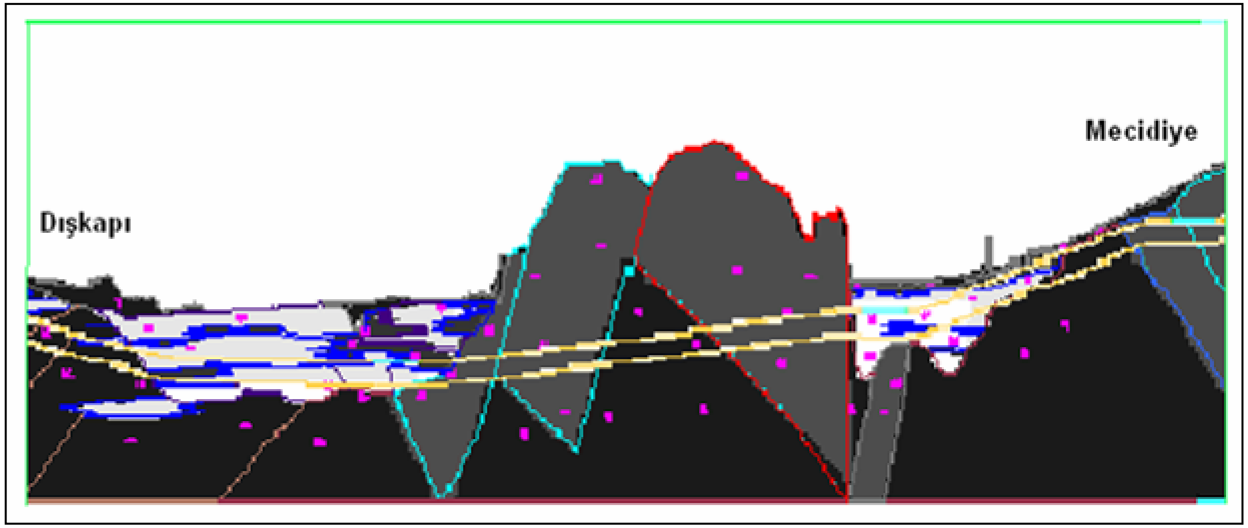

Figure 4.4. *The lithology layer between Dışkapı – Mecidiye stations in .dxf and .rvc formats*

Additional lines as a mesh are drawn in order to provide ease in assigning the correct ranking values to the polygons because TNT Mips realizes even a tiniest line break and this causes multiple selection of polygons with different properties. After converting the .dxf file into .rvc format a vector data file is obtained. The assignment of the polygon values is done on this file and following that the vector layer is converted into a raster layer to be used in further analysis. The distribution of the classes by means of pixel count on the raster layer of lithology is given in *Table 4.2.* .

Table 4.2. *Distribution of the lithology ranking values in the raster format of the layer*

Cell Value	Count
0	135228
1	81909
2	5268
3	22402
6	2433
9	12484
10	2420

The zero values refer to the areas that are above the ground level which means the empty space.

4.2. Hydrostatic Pressure Layer

The determination of hydrostatic pressure values was explained in the second chapter. The hydrostatic pressure values calculated for the top of the rail elevation in every borehole are assigned to their location and according to them the hydrostatic pressure intervals for every 0,5 atm is found. Those intervals are then correlated in AutoCAD by means of drawing the isobar lines. The class intervals and their ranks are given in *Table 4.3*.

Table 4.3. *Hydrostatic pressure intervals and their ranking values*

Hydrostatic pressure interval	Ranking
0,5-1	2
1-1,5	4
1,5-2	6
2-2,5	8
2,5-3	10

Figure 4.5. (a) illustrates the hydrostatic pressure distribution and *Figure 4.4. (b)* focuses on a certain part of the cross-section to provide a detailed overview from the .dxf file of hydrostatic pressure layer. After completing the correlation of isobar lines in AutoCAD it is seen that maximum pressure value applied on the top of the rail elevation is in 2-2,5 atm interval where has the rank value of 8. To provide a better consistency 2, 5-3 atm interval was also taken into account in the analysis and five hydrostatic pressure classes have been created.

In *Figure 4.5. (c)* both vector and raster forms of the hydrostatic pressure layer can be seen. After converting the .dxf file into .rvc format a vector data file is obtained. The assignment of the polygon values is done on this file and following the vector layer is converted into a raster layer for further analysis. The distribution of the classes by means of pixel count on the raster layer of hydrostatic pressure is given in *Table 4.4.* .

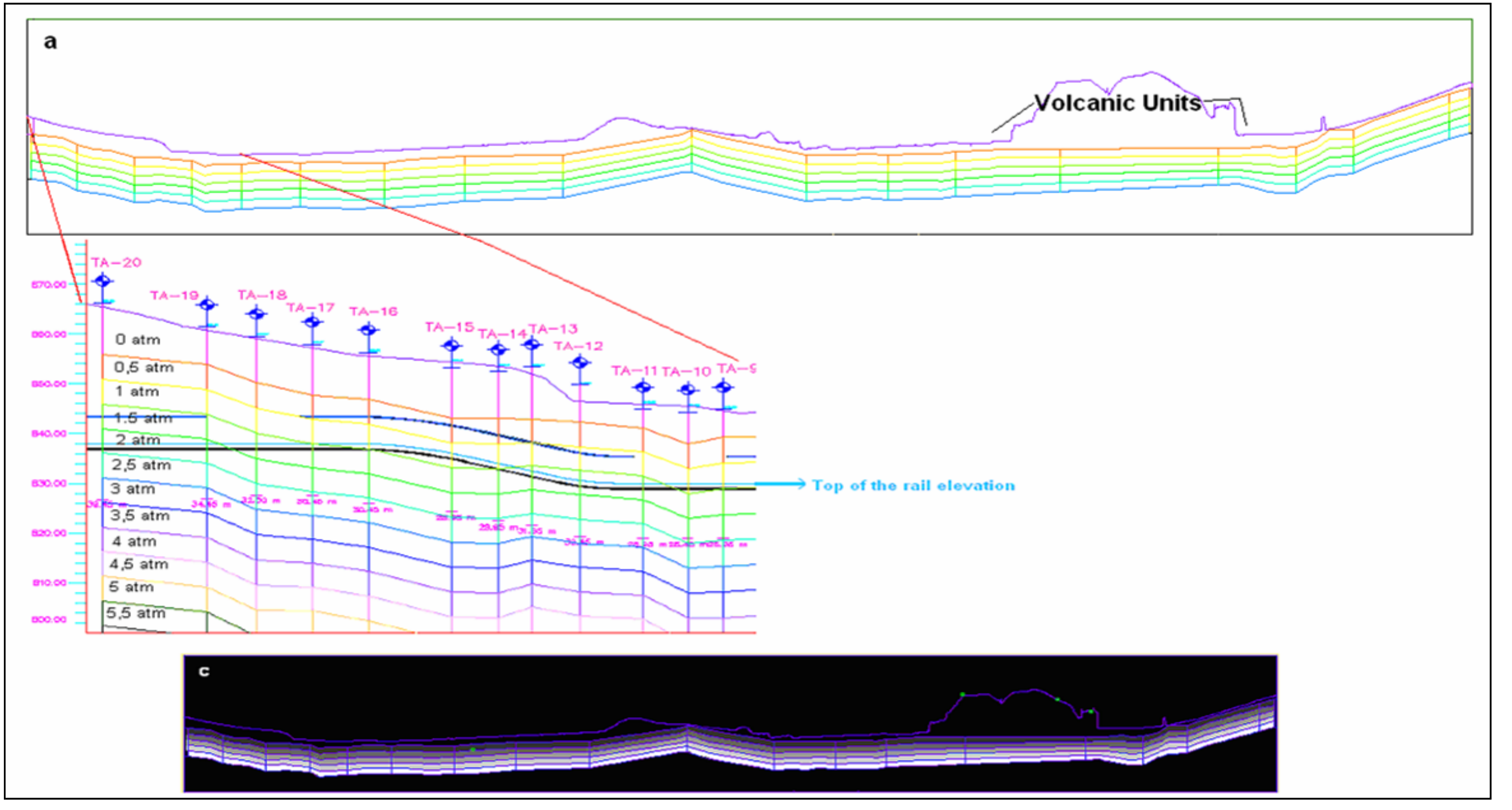

***Figure 4.5. (a)** The hydrostatic pressure layer base drawn in AutoCAD .dxf format **(b)** A detailed view from the hydrostatic layer base and **(c)** The hydrostatic pressure layer prepared in TNT Mips both in vector and raster formats*

Table 4.4. *Distribution of the hydrostatic pressure intervals' ranking values in the raster format of the layer*

Cell Value	Count
0	209746
2	10376
4	10792
6	10488
8	10587
10	10155

Zero values indicate that the region is an empty space or has a hydrostatic pressure value greater than 3 atm where is not in the consideration of the analysis. An exception, that has to be remembered, as explained in the second chapter is that the hydrostatic pressure values under the volcanic rocks are not taken as they are because the groundwater measurements in some of the boreholes yielded too high values that are not actually realistic. An adjustment is done according to the situation of the groundwater levels in the other boreholes.

4.3. Permeability Layer

Production of the permeability layer starts with preparation of an excel worksheet that consists of depth vs. coefficient of permeability values. A sample can be seen in *Table 4.5.*.

Table 4.5. *Permeability values for borehole TA-13*

Borehole #	casing length	test length	depth	k
TA-13				
	6	1,5	7,5	3,47E-06
	9	1,5	10,5	0,00E+00
	12	1,5	13,5	3,06E-06
	13,5	3	16,5	0,00E+00
	13,5	6	19,5	0,00E+00
	13,5	13,5	27	4,96E-08

When permeability values are clearly defined each of them are assigned to the related depth of the considered borehole in AutoCAD and boreholes are correlated all over the tunnel alignment (see *Figure 4.6. (a) and (b)*). As mentioned before the coefficients in front of the exponentiation with base 1/10 are omitted and as a result permeability classes are defined as seen in *Table 4.6.*.

Table 4.6. *Permeability and their ranking values*

Permeability (m/sec)	Ranking
E-04 $=10^{-4}$	10
E-05 $=10^{-5}$	9
E-06 $=10^{-6}$	7
E-07 $=10^{-7}$	5
E-08 $=10^{-8}$	3
E-09 $=10^{-9}$	1
E+00 = 0	1

After all the permeability file in .dxf format is converted to .rvc format for analysis in the GIS platform. At this point while converting a .dxf file into a .rvc file colors cannot be kept so while assigning the attribute values to polygons in TNT Mips, AutoCAD program has to be open for assigning the correct value to the correct polygon. In correlating the boreholes line duplication is also possible while working on AutoCAD and this also causes a problem such as having polygons that has no attribute value. This situation can mostly be realized after converting the vector into raster that is why such kind of files has to be processed very carefully.

Additional lines by means of a mesh are also drawn on the permeability layer in order to assign the attribute values easily while working on TNT Mips (see

Figure 4.6.(c)). The raster cell count for each class is given in *Table 4.7.* . The zero values again indicate that the region is an empty space or permeability value is not necessary for that area. The regions without permeability value are considered to cause no risk by means of hydraulic effects so the lack of data can be ignored at those places.

Table 4.7. *Distribution of the permeability ranking values in the raster format of the layer*

Cell Value	Count
0	231836
1	7351
3	3765
5	8424
7	6978
9	2728
10	1062

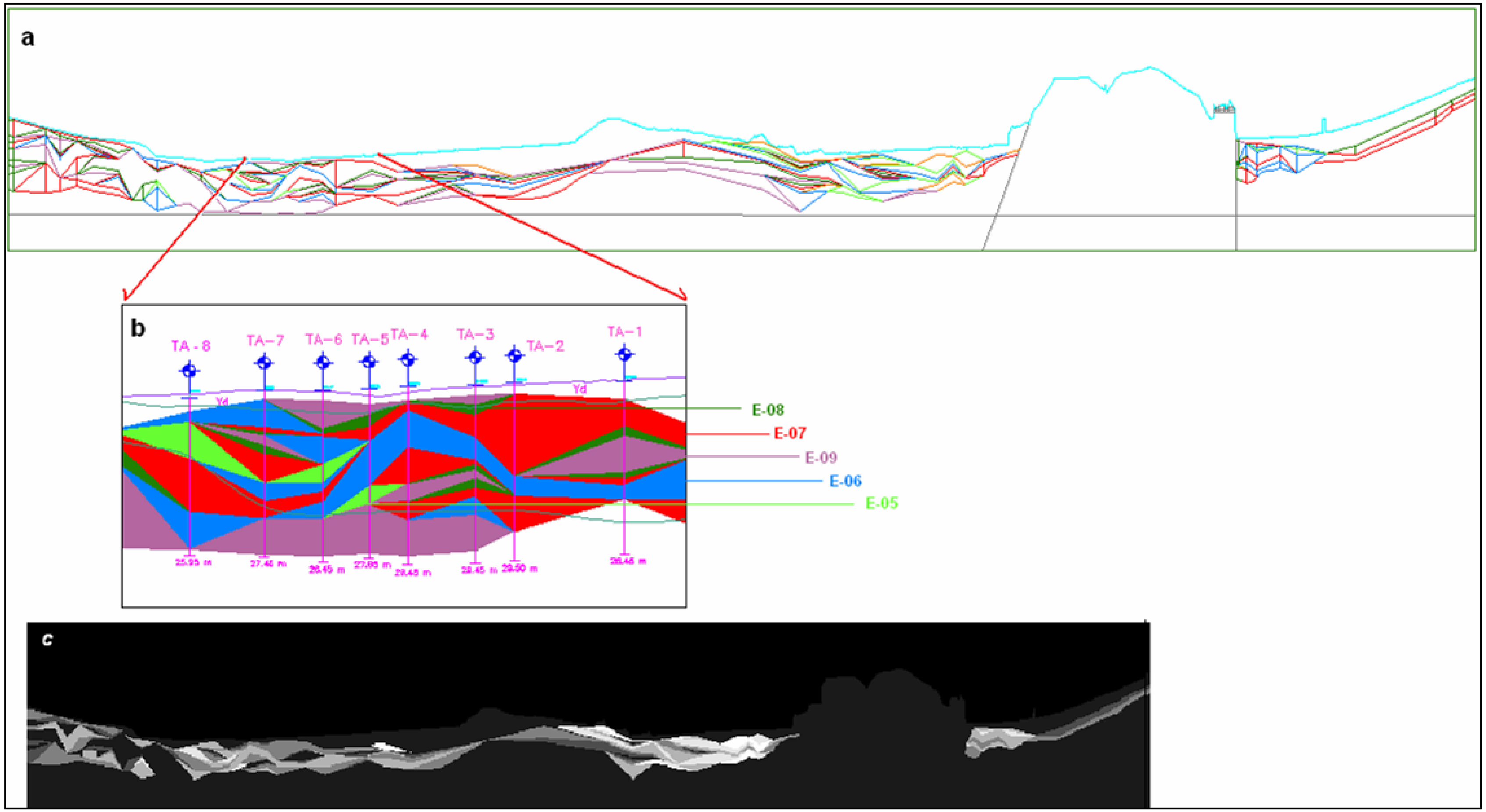

Figure 4.6. *(a) The permeability layer base drawn in AutoCAD .dxf format (b) A detailed view from the permeability layer base and (c) The permeability layer prepared in TNT Mips*

4.4. SPT Layer

Base on SPT test results and the USCS ranking values for each class is given in *Table 4.8.*.

Table 4.8. *Classification of units according to SPT-N blow numbers and cohesiveness and their ranking values*

Non-cohesive material	SPT-N #	Ranking
Compactness		
Very loose	<4	10
Loose	4-10	8
Medium dense	10-30	6
Dense(compact)	30-50	4
Very dense	>50	2
Cohesive material		
Consistency		
Very soft	<2	6
Soft	2-4	5
Medium(firm)	4-8	4
Stiff	8-15	3
Very Stiff	15-30	2
Hard	>30	1

The original geological cross-section is modified to produce the related layer to be used in TNT Mips. All the ranking values are given a specific color and for each level they are assigned to make a correlation between boreholes. This drawing and two detailed views of correlated boreholes can be seen in *Figure 4.7. (a,b,c)*.

After all the .dxf file of SPT is converted into .rvc and ranking values by means of attributes are assigned to each polygon and this vector file is converted into raster. Both the vector and raster views in TNT Mips are illustrated in *Figure 4.7. (d)*. The distribution of the classes through out the cross-section is given in *Table 4.9.* where zero values indicate that the region is an empty space or a part of the area without a SPT result. There is no low blow count in non-cohesive materials less than 4 thus, in no region ranking value of 10 is assigned and also ranking values of 6 and 5 are not available for cohesive materials due to high SPT-N.

Table 4.9.*Distribution of the permeability ranking values in the raster format of the layer*

Cell Value	Count
0	223205
1	12379
2	9335
3	4248
4	6402
6	4965
8	1610

The production of layers can be explained briefly as a step by step procedure as follows:

1. Acquisition of related data
2. Creating a .dxf file in AutoCAD by using the Original AutoCAD Cross-section as a base
3. Converting the .dxf file into .rvc format for to be used in TNT Mips which is in vector format
4. Preparing the attribute tables by assigning the data to the related polygons
5. Converting the vector format into raster in order to use it in further analysis

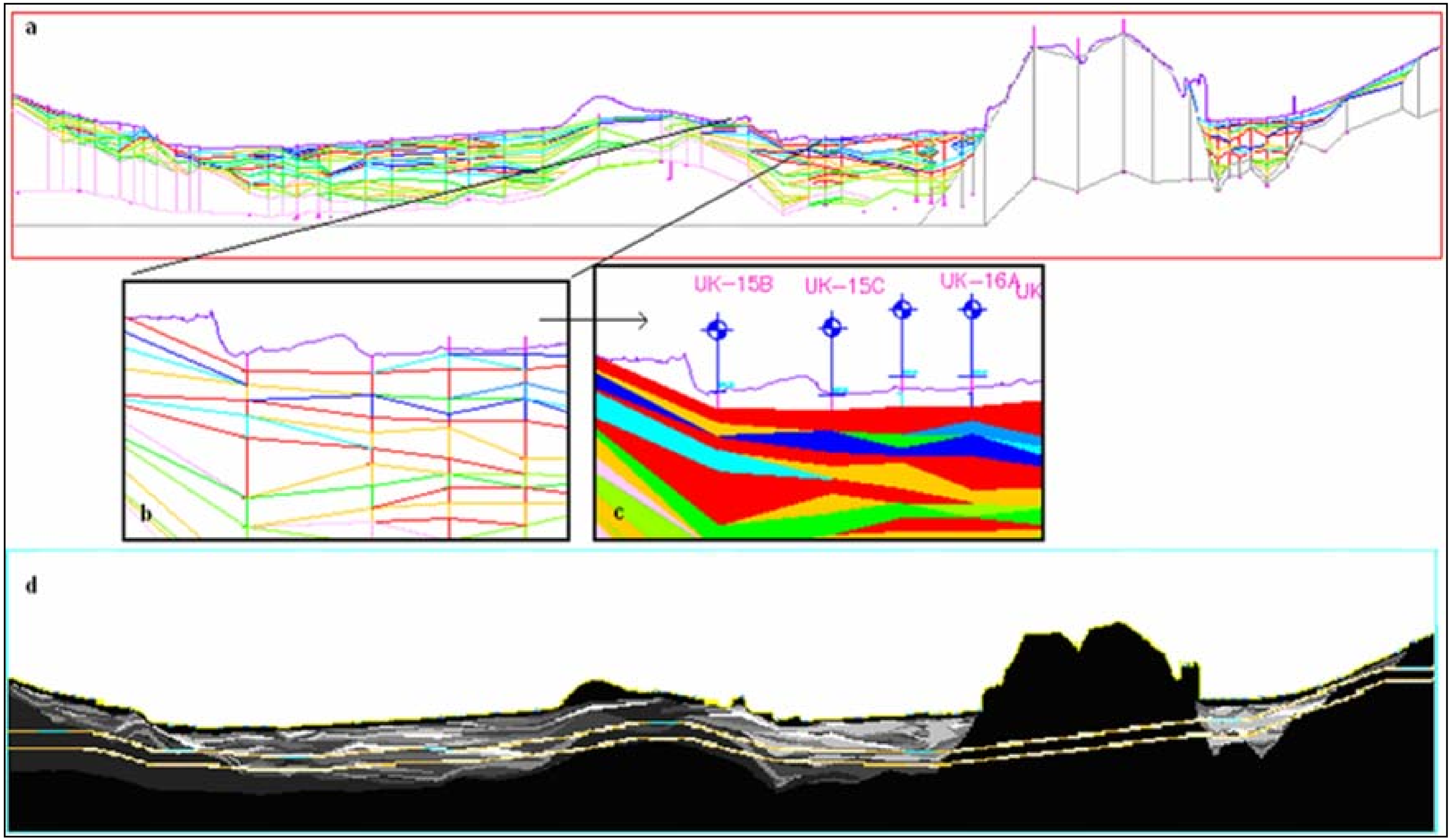

Figure 4.7. (a) *The SPT & USCS layer base drawn in AutoCAD .dxf format,* ***(b,c)*** *Two detailed views from the SPT & USCS layer base and* ***(d)*** *The SPT & USCS layer prepared in TNT Mips with the extents of the tunnel*

CHAPTER V

ANALYSES

For the determination of excavation difficulties along metro tunnel alignment two different methods are used for calculating the weights that are “Simple Additive Weighting” and “Analytical Hierarchy Process” as explained in the third chapter. The ranking values for each input value is determined by an expert decision maker and afterwards weights are calculated in two ways in order to be more certain about the results.

The Original AutoCAD Cross-section illustrates the upper part of the ground as well as the whole of the tunnel alignment. The only part to be excluded in the analyses is this empty space but as there is no attribute value that can be assigned to here this part automatically gets zero value in every layer so that there is no need to prepare a mask.

The input layers in raster form and their ranking values as a legend are given in *Figure 5.1.* Colored lines indicate the extents of the tunnel and the profile of the cross-section. On those layers further analyses are done, weights are given with multiplication operations by using geoformulas and final cross-sections are obtained consequently.

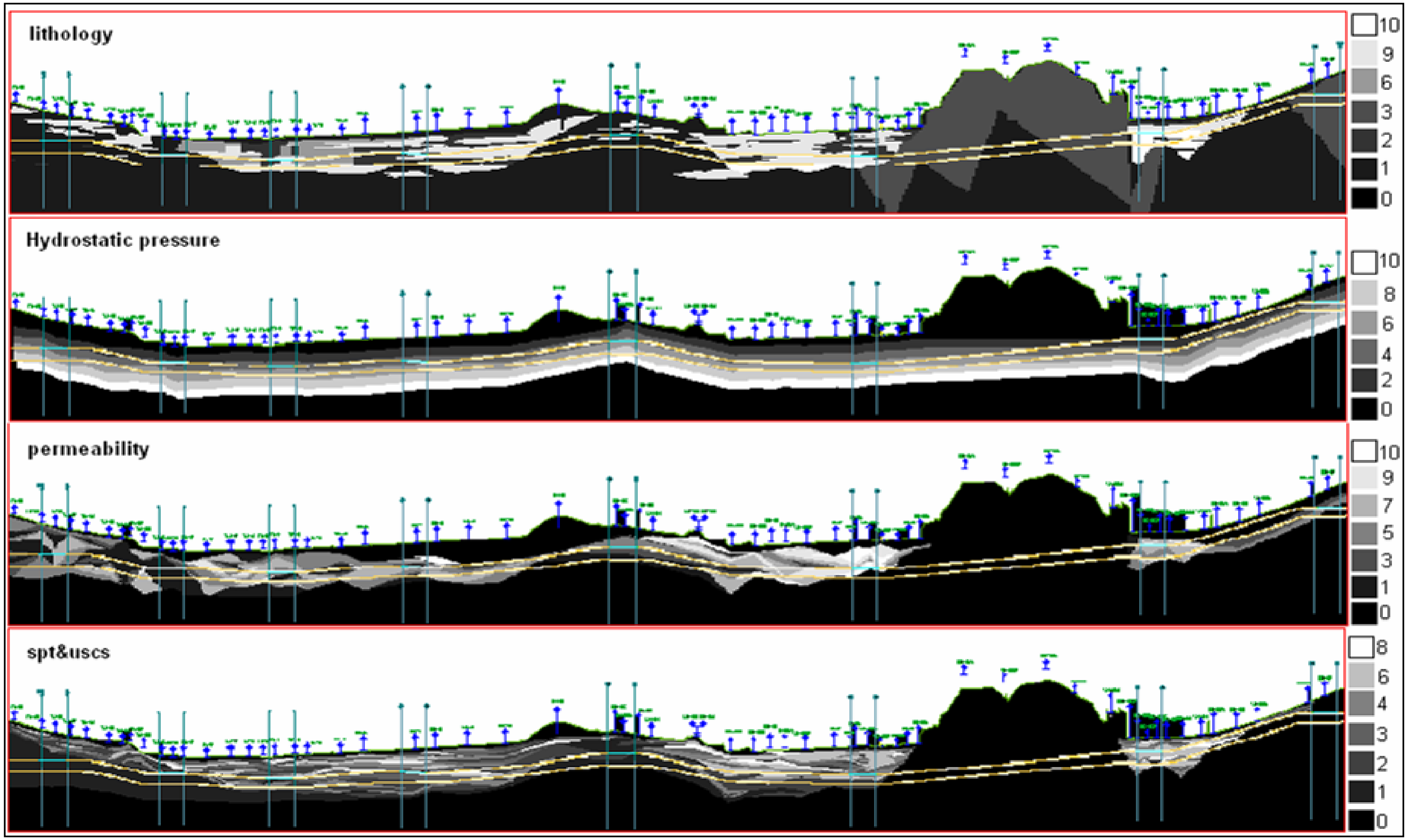

Figure 5.1. *Cross- section layers and the ranking values' legend in raster format*

5.1. Calculation of Weights

5.1.1. Simple Additive Weighting (SAW)

As mentioned before two methods are used for calculating weights. In the first one, Simple Additive Weighting (SAW), weights are assigned to the criterion classes by a decision maker who is an expert about the subject that is considered. The procedure basically consists of directly giving weight values to the attributes according to their importance. *Table 5.1.* gives the criterion weights and their normalized values determined by SAW method. The normalization is done in order to have an overall score for each class by dividing each weight by sum of the weights.

Table 5.1. *Criterion Weights Defined by SAW Method*

Data Layer	Weight	Normalized Weight
Lithology	3	0,2143
Hydrostatic Pressure	5	0,3571
Permeability	4	0,2857
SPT&USCS	2	0,1429

When the weights are calculated each layer is multiplied with the defined weight by using a geoformula and following four layers are added to obtain a result cross-section of excavation difficulty. Later this result cross-section is reclassified by using a geoformula. There are five classes determined by studying the raster histogram of the cross-section edited by SAW method. The classification scale and distribution of classes through the tunnel alignment in percentage is given in *Figure 5.2.*.

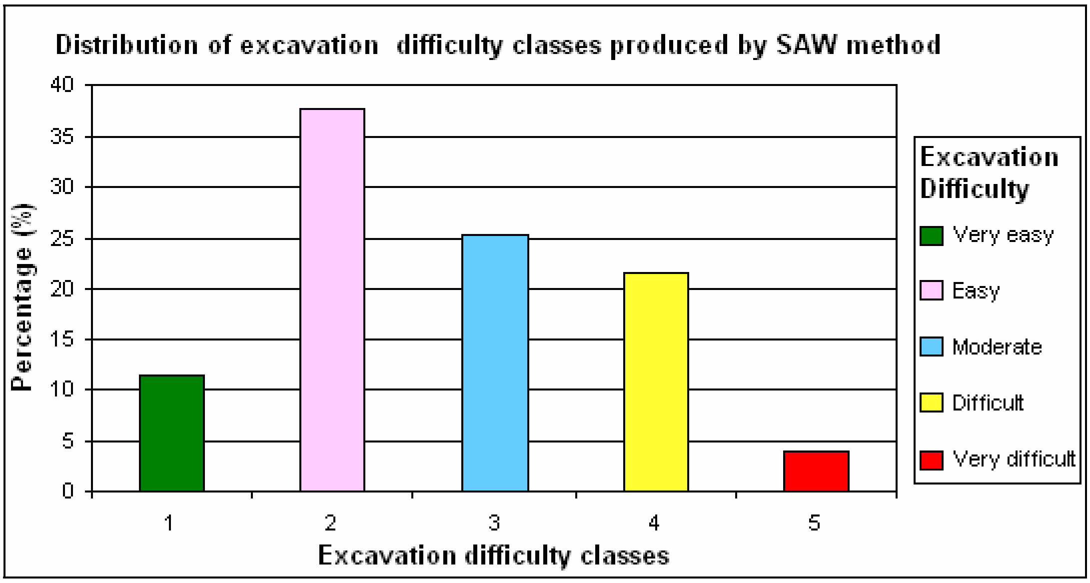

Figure 5.2. *The distribution of excavation difficulty classes produced by SAW method through the tunnel alignment*

It can be seen in the *Figure 5.2.* that 11 % of the alignment has no excavation difficulty, 38 % can be constructed with minimum difficulty, in 25% of the alignment some excavation difficulties may occur, 22 % is difficult to construct and 4% surely affects the TBM performance.

Figure 5.3. is the legend for the reclassified layer of both SAW and AHP. The summation of SAW applied input layers in an unclassified form and the distribution of these classes all over the alignment is shown in *Figure 5.4.* (a,b) and in (c) the distribution of excavation difficulty classes only through the tunnel alignment can be seen.

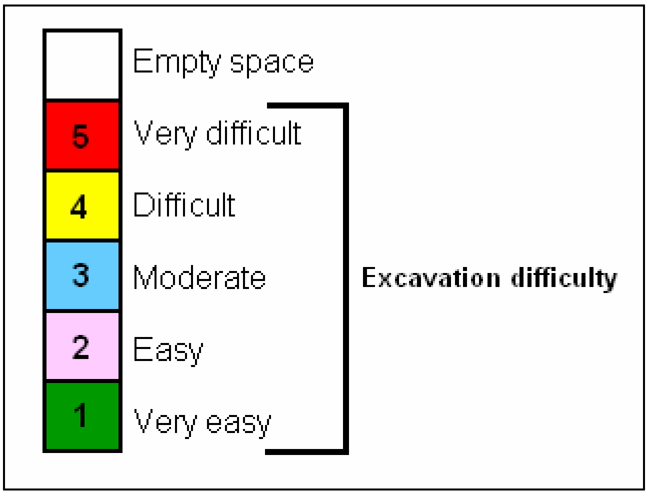

Figure 5.3. *Legend for the reclassified SAW and AHP layers*

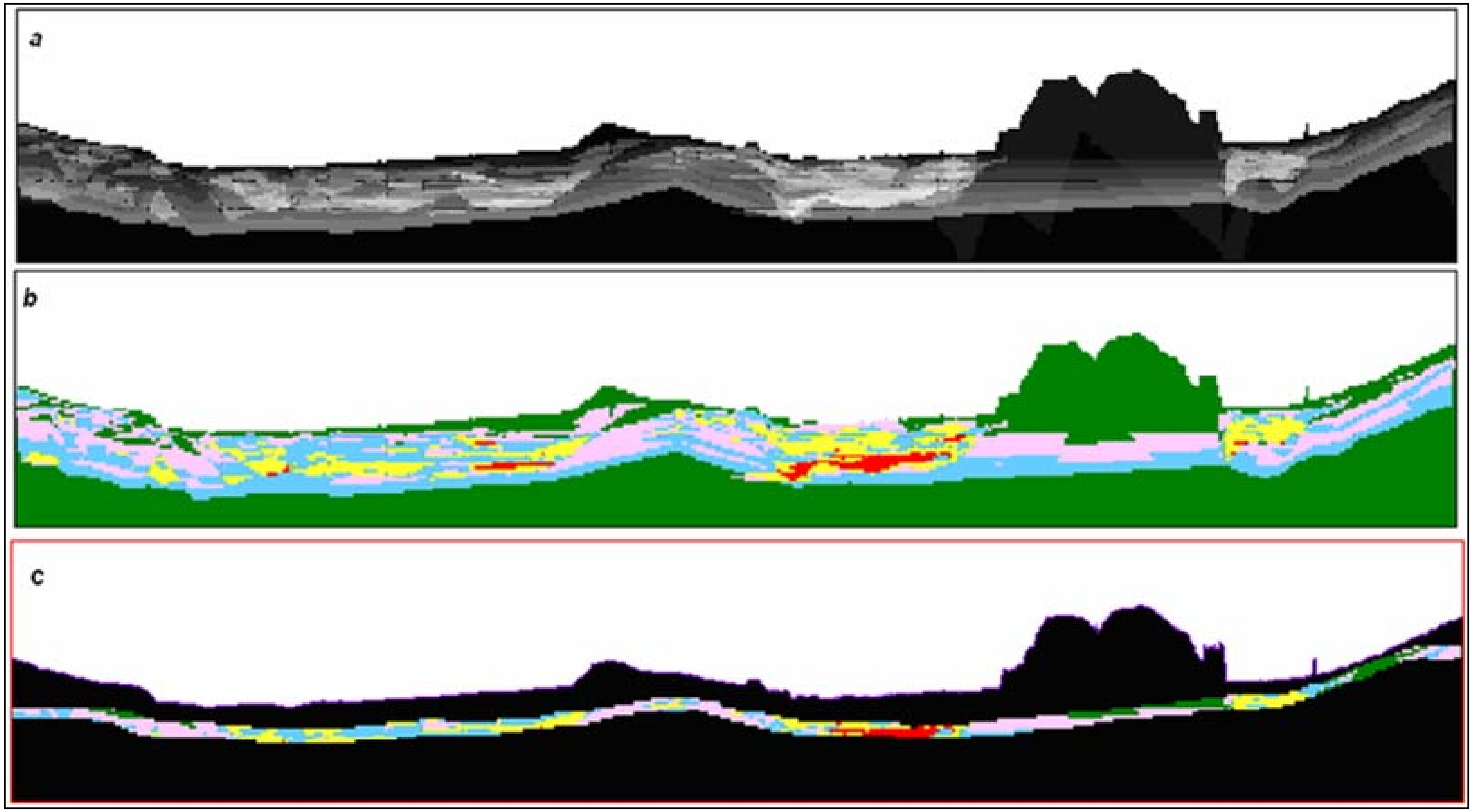

***Figure 5.4. (a)** The summation of SAW applied layers before reclassification, **(b)** The reclassified form of SAW layer and **(c)** The distribution of excavation difficulty classes through the tunnel alignment obtained by SAW method*

5.1.2. Analytical Hierarchy Process (AHP)

AHP involves decomposing the decision problem into smaller, more understandable parts. The goal is to determine the units that have the highest resistance against excavation and the criteria for this process are lithology, hydrostatic pressure, permeability and SPT&USCS results as mentioned before.

To establish the optimal weights, first of all one criterion is compared to one another repeatedly until all possible pair-wise comparisons are completed. The pair-wise comparison values are determined according to the scale of importance proposed by Saaty which was given in Cahpter – III. This pair-wise comparison is again a process which is expert dependent. Afterwards a decision matrix is prepared, its square is taken and the row sums are calculated and normalized to obtain weights. This procedure is repeatedly done until the difference between the weights reach very small value. Because of having small number of criteria decomposition of the problem thus calculating the weights has not been a long process. The decision tree of the problem is given in *Figure 5.5.* and the decision matrix used in the analysis is given in *Table 5.2.* .

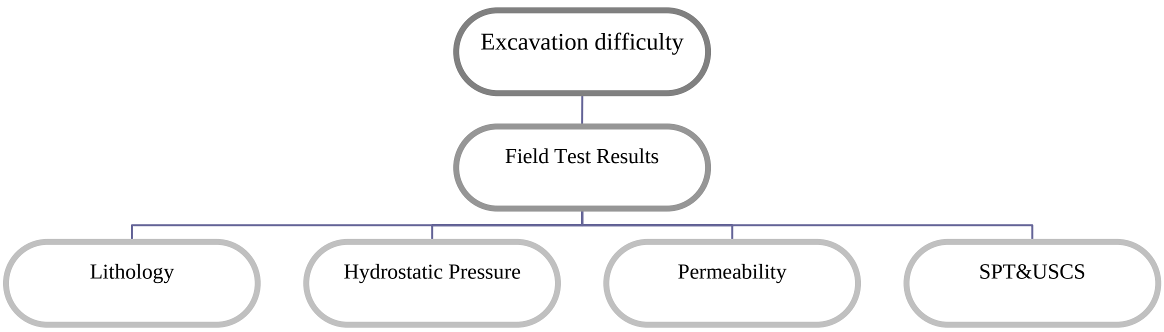

Figure 5.5. *The decision tree of the AHP method*

Table 5.2. *The pair-wise comparison matrix*

	Permeability	**Lithology**	**SPT&USCS**	**Hydrostatic Pressure**
Permeability	1	5	7	1/2
Lithology	1/5	1	2	3
SPT&USCS	1/7	1/2	1	1/7
Hydrostatic Pressure	2	1/3	7	1

The normalized weights calculated by using AHP method are given in *Table 5.3.* .

Table 5.3. *Criterion Weights Defined by AHP Method*

Data Layer	**Normalized Weight**
Lithology	0,2474
Hydrostatic Pressure	0,2936
Permeability	0,4069
SPT&USCS	0,0521

After the calculation of weights according to the AHP a geoformula is written and layers are multiplied with the weights by using this geoformula and they are added in order to obtain an AHP result cross-section. The percentages of the classes and their classification scale are given in *Figure 5.6.*. The AHP applied cross-section before reclassification can be seen in *Figure 5.7. (a)*). This cross-section is then reclassified into five classes that have been determined according to the raster histogram of the AHP cross-section by using another geoformula (see *Figure 5.7. (b,c)*).

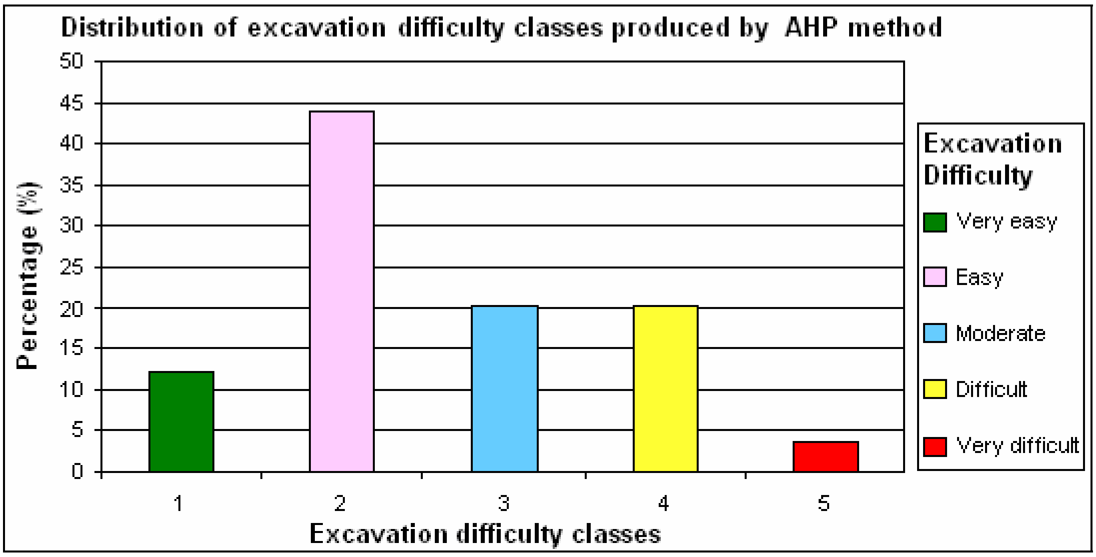

Figure 5.6. *The distribution of excavation difficulty classes produced by AHP method through the tunnel alignment*

This figure shows that 12 % of the alignment can easily be constructed, 44 % is not prone to cause any engineering difficulty, 20 % causes moderate excavation difficulty, 20 % causes high excavation difficulty and 3 % absolutely has the capacity to affect TBM performance.

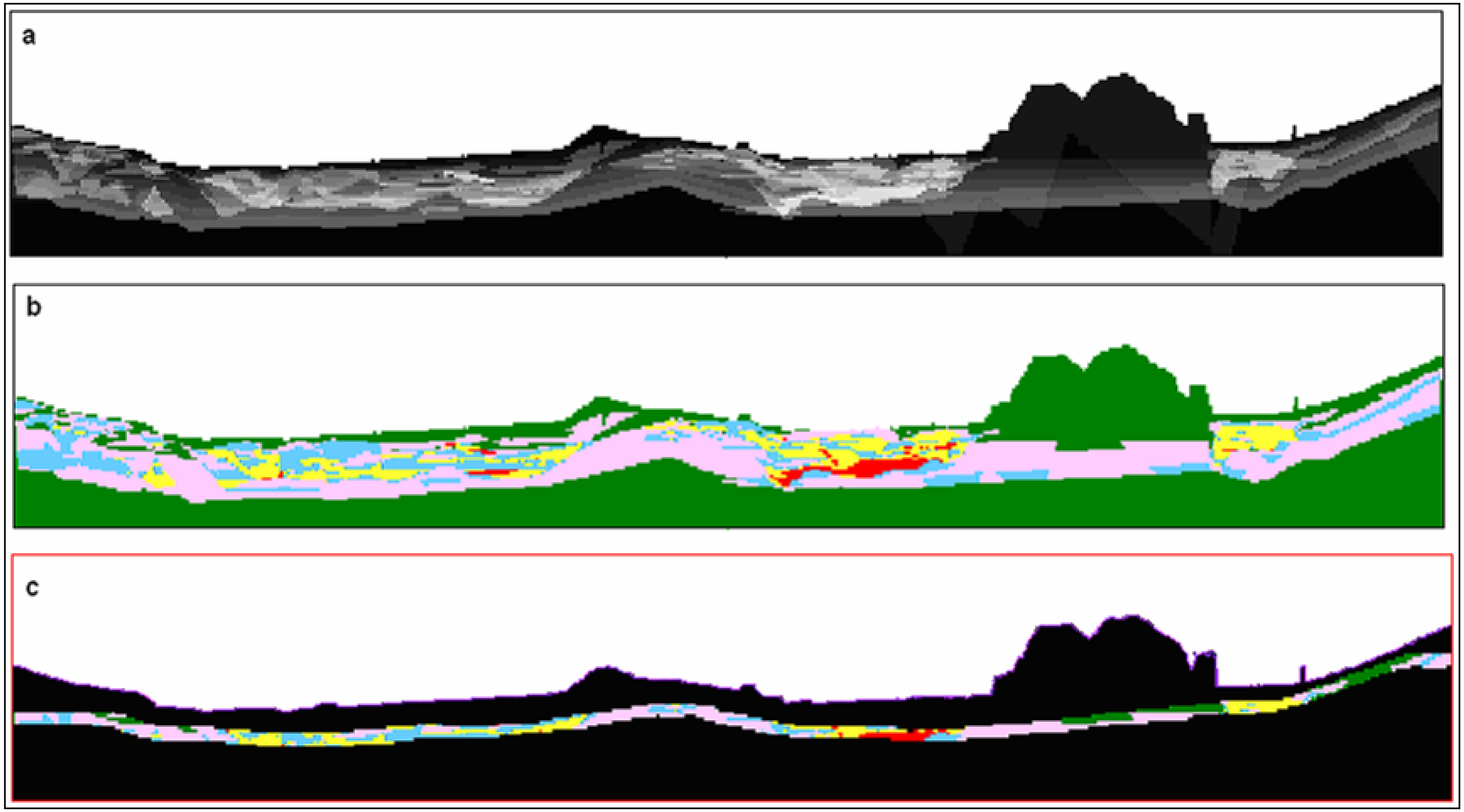

Figure 5.7. (a) *The summation of AHP applied cross-section before reclassification,* ***(b)*** *The reclassified form of AHP cross-section and,* ***(c)*** *The distribution of excavation difficulty classes obtained by AHP through the tunnel alignment*

5.2. The Comparison of Applied Methods

The methodology for comparing the two applied methods has been adopted from Süzen, 2002. The aim is to find out the change of difficulty score within a pixel for AHP and SAW layers. A geoformula is written in order to compare the two result cross-sections produced by SAW and AHP. As a first step for each class of SAW layer values ranging from 1 to 5 are assigned which means SAW class values are kept as they are. Secondly AHP class values are multiplied by 10 and those values ranging from 10 to 50 are assigned as new attributes like 10 for pixels that have value 1 in AHP layer and 2 for 20 etc. After the multiplication, two maps are added and the results are shown in *Table 5.4.*

Table 5.4. *The comparison matrix of two applied methods*

SAW / AHP	1	2	3	4	5
10	58,217%	0,639%	0	0	0
20	0,052%	14,040%	9,778%	0	0
30	0	0,445%	8,102%	1,269%	0
40	0	0	0,473%	5,597%	0,109%
50	0	0	0	0,109%	1,170%

	Correct Class		Acceptable Class		Mismatched Class

If a pixel has the value of 11, 22, 33, 44 or 55 this means in both SAW and AHP layer this pixel is in the same class of excavation difficulty and these pixels are accepted as correct classes. Values 12, 21, 23, 32, 34, 43, 45 and 54 indicates that there is a difference of one class of excavation difficulty and this situation can be accepted in comparison of these two different layers. To be clearer if a pixel has the value of 45 in the comparison layer of these two methods this means when AHP layer considers that this pixel matches the part of the alignment with difficult level of excavation, SAW shows that this pixel is on the part of the alignment with very difficult level of excavation. These matches are called as acceptable classes.

In *Figure 5.8.* the bar chart of correct and acceptable classes is illustrated. It can be seen that there is a high percentage of matching classes through out the cross-section. The total percentage of pink bars is 87 % and the total percentage of green bars is 13 %. There are no mismatched classes.

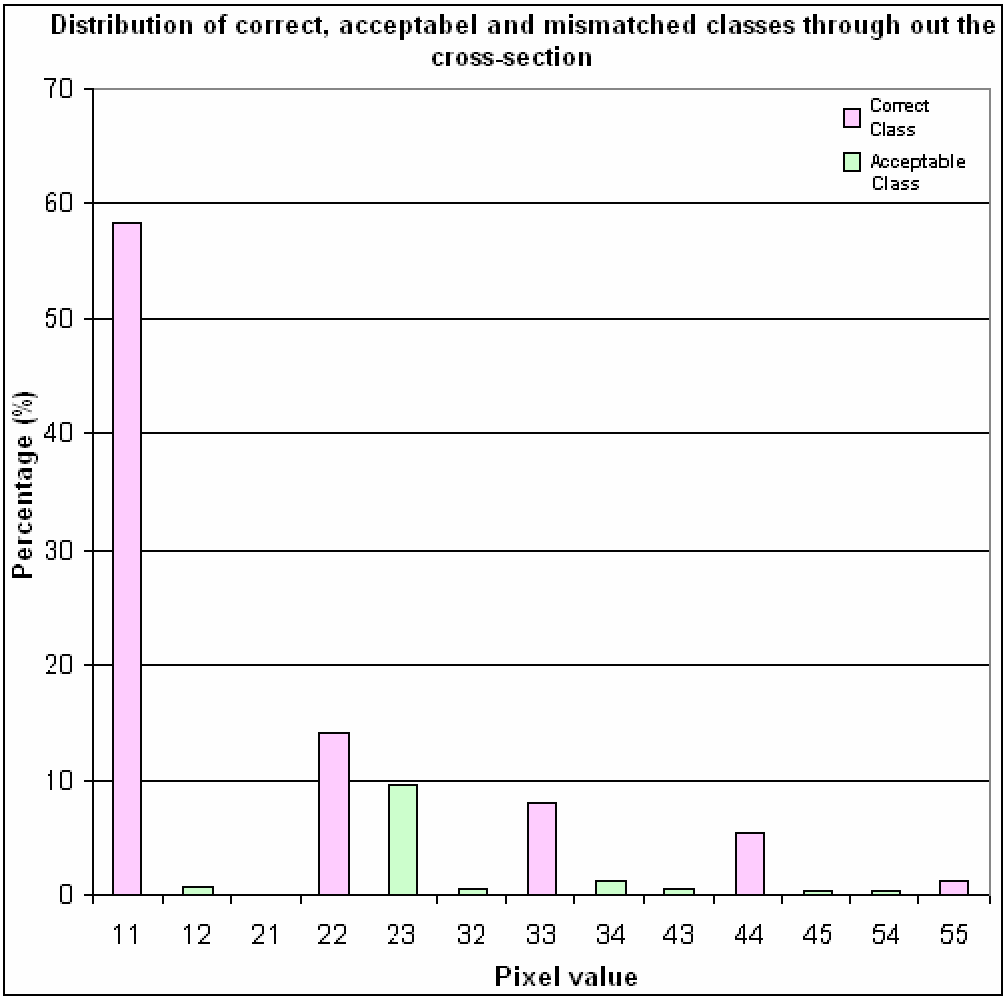

Figure 5.8. *The bar chart of correct, acceptable and mismatched classes through out the cross-section*

Although *Figure 5.8* illustrates the distribution of correct and acceptable classes through out the cross-section the important area to be considered is the tunnel alignment. *Figure 5.9* shows the distribution of correct and acceptable classes through the metro tunnel. The percentage of correct classes increases to 88% when this part is considered. A detailed look out to the results proves that all over the cross-section the "easy" and "very easy" excavation difficulty levels are dominant but when it comes to the tunnel alignment approximately 59 % of very easy excavation level decrease to 12% and relatively easy, moderate, and difficult level of excavation classes shows an increasing trend. Finally when the percentage of very difficult level of excavation is around 1,3% when whole of the cross-section is considered, it increases to 3,5 % in the tunnel alignment.

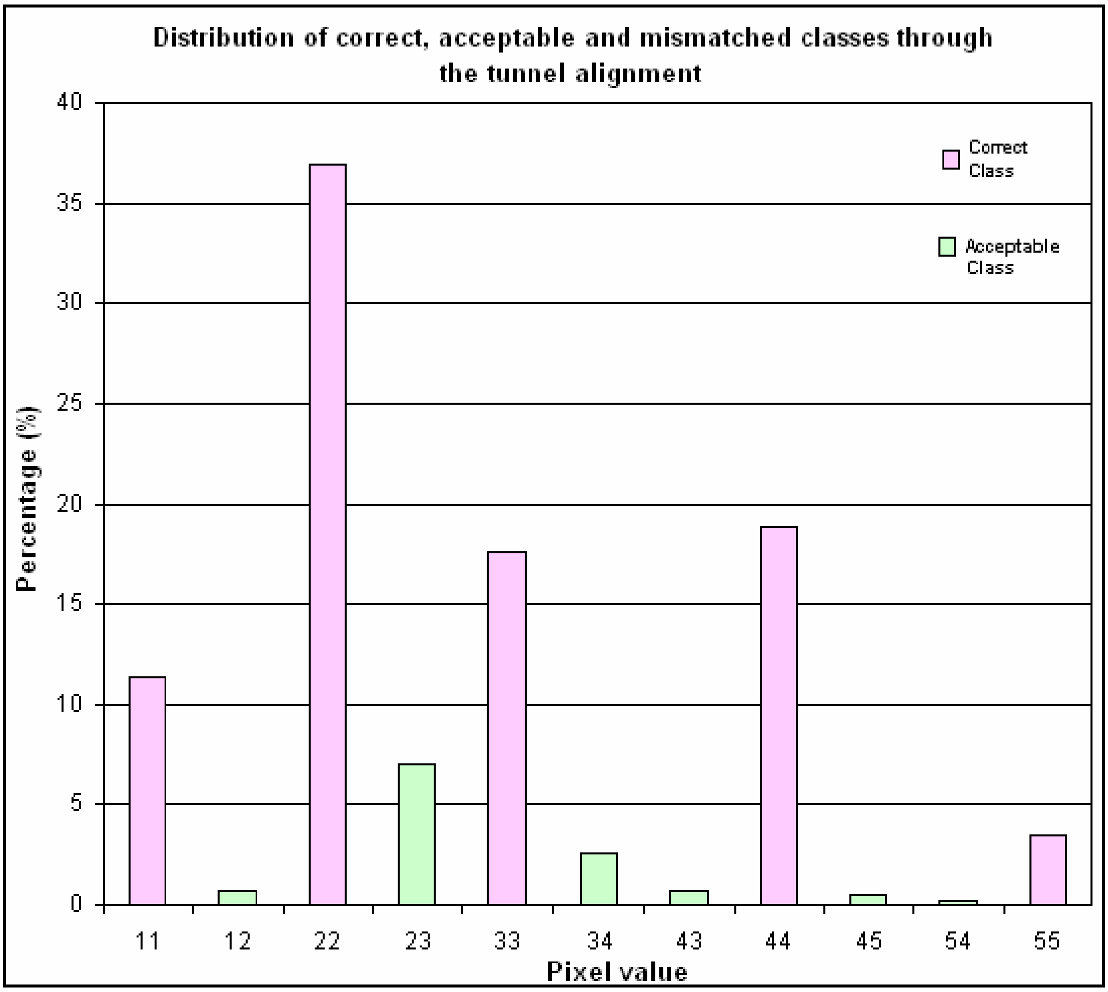

Figure 5.9. *The bar chart of correct, acceptable and mismatched classes through the tunnel alignment*

In *Figure 5.10. (a,b)* the distribution of correct and acceptable classes can be observed. As there is no mismatched class it is not possible to see such regions.

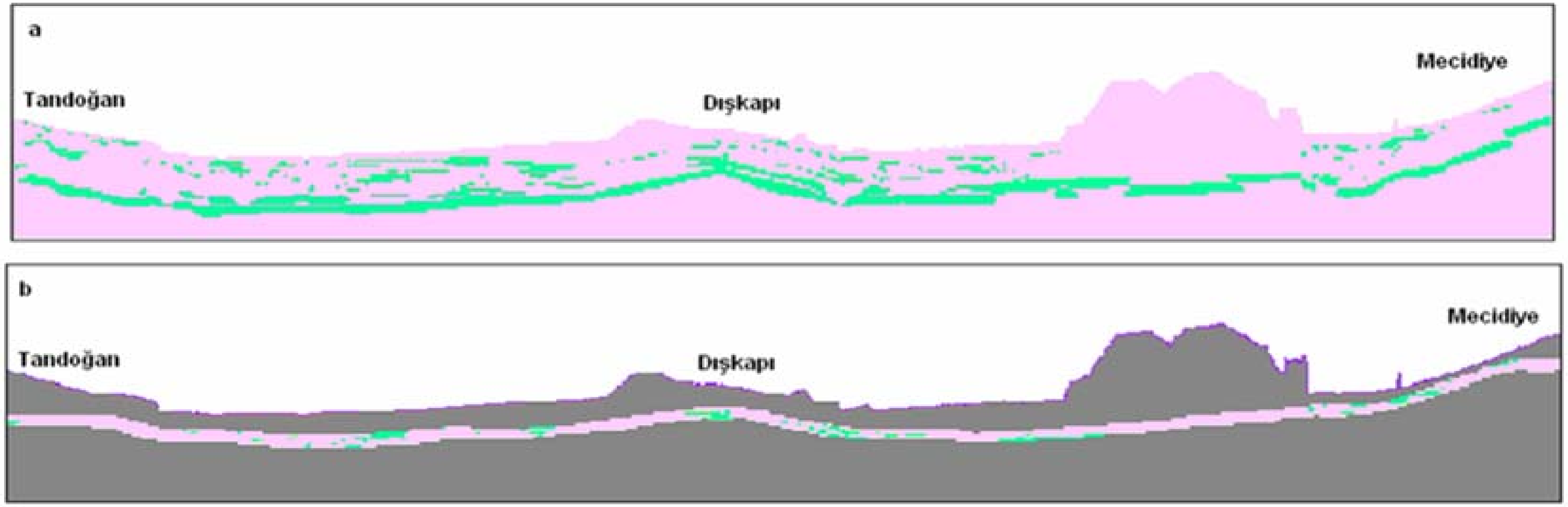

Figure 5.10. (a) *The distribution of correct and acceptable classes through out the cross – section*
(b) *The distribution of correct and acceptable classes through the tunnel alignment*

CHAPTER VI

GENERAL EVALUATION AND DISCUSSION

This thesis has been prepared to demonstrate the parts of a construction site where engineering difficulties might occur by integrating data obtained from traditional site investigation methods with Geographic Information Systems and statistical approaches. The geological cross-section of the area prepared by considering the borehole data and in-situ observations has created a basis for analysis in determining parts with excavation difficulties.

Complex geological and hydrogeological conditions are found to be effective in the advancement of a TBM implemented in tunnel boring works. A good understanding of the geology is essential in such cases so a reasonable number of field and laboratory tests had been conducted in order to determine geological, hydrogeological and geotechnical properties of the study site. Advancements in the tunnel boring process also proved that hydrogeological conditions are effective on the performance of TBM so related data is considered carefully while preparing cross-section layers and calculating weights.

As a first step factors that may affect the performance of a TBM was determined and related to them statistical methods had been applied in order to calculate the importance of the factors according to each other. After that a GIS program TNT Mips had been used to create cross-section layers by assigning attribute values for each feature of the considered aspect of the study site that are hydrostatic pressure, permeability, lithology and STP results. Final step was to overlay each layer in order to display the distribution of excavation difficulty classes. The distribution of excavation difficulty classes between the Tandoğan – Mecidiye stations of Keçiören – Tandoğan metro alignment are given below.

6.1. Results between Stations

6.1.1. Tandoğan and EGO Stations

The over all geological composition between Tandoğan – EGO stations is 53% silt of high plasticity, elastic silt(MH), 10% silt(ML), %10 silty gravel(GM), 10% silty sand(SM), 6% clay(CL), 4%poorly graded gravel and 7% SW,GW, CH, SP according to USCS. Excavation difficulties were not expected in Ankara clay. The green and pink parts of the figures are mainly composed of this unit where the excavation difficulty level is minor. Units with coarser grain sizes and high permeability combination causes difficulty in some parts of this section where are indicated with blue and yellow.

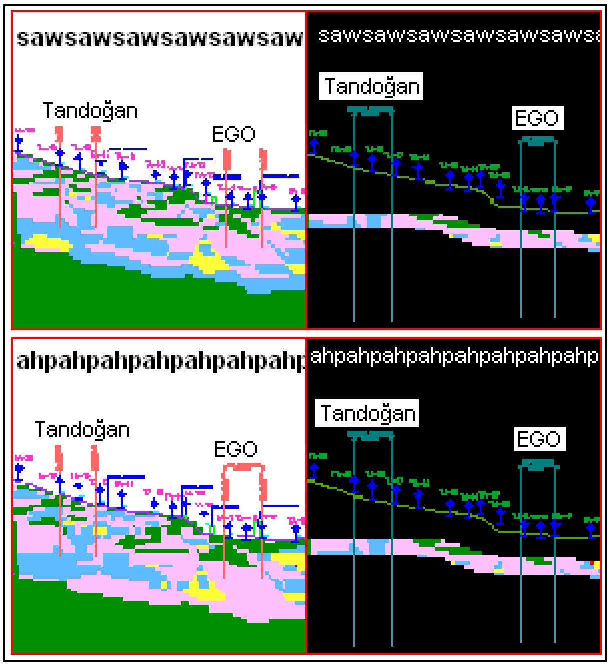

Figure 6.1. *Distribution of excavation difficulty classes between Tandoğan and EGO stations*

6.1.2. EGO and AKM Stations

33% silt of high plasticity, elastic silt(MH), 22% clay (CL), 14% silty sand(SM), 10% silt(ML), %9 silty gravel(GM), 5% clay of high plasticity, fat clay(CH) and 7% SW,GW,SC is the geological combination of this section. In the middle of these two stations serious excavation difficulties are expected in the parts indicated with red and yellow colors. Paying special attention to these parts may be essential in tunnel boring process.

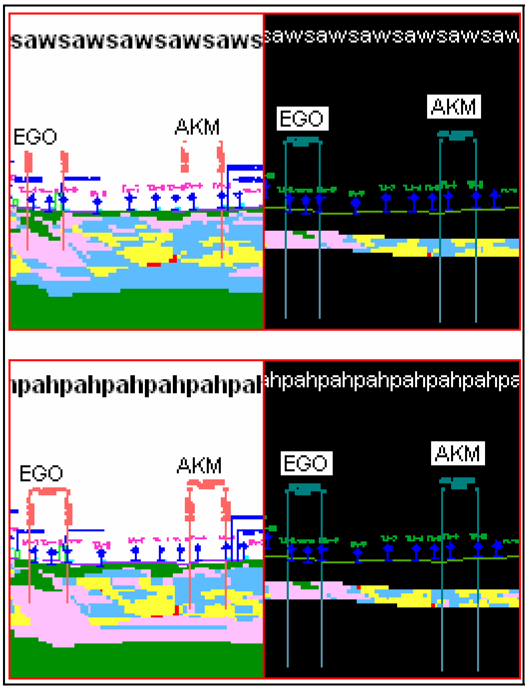

Figure 6.2. *Distribution of excavation difficulty classes between EGO and AKM stations*

6.1.3 AKM and ASKİ Stations

The USCS distribution between these stations is 20%silt of high plasticity, elastic silt(MH), 19% clay(CL), 17% silty sand(SM), 12% silt(ML), 7% well graded sand, fine to coarse sand (SW), 6% silty gravel(GM), 6% well graded gravel, fine to coarse gravel (GW), 5% clay of high plasticity, fat clay(CH) and 8% GP,SP,SC. As it can be observed grain size is getting larger as the alignment goes through Mecidiye station. While the section between AKM and ASKİ stations displays yellow parts indicating difficulty in excavation, after ASKİ station the excavation difficulty level gets higher as red parts are observed in this place.

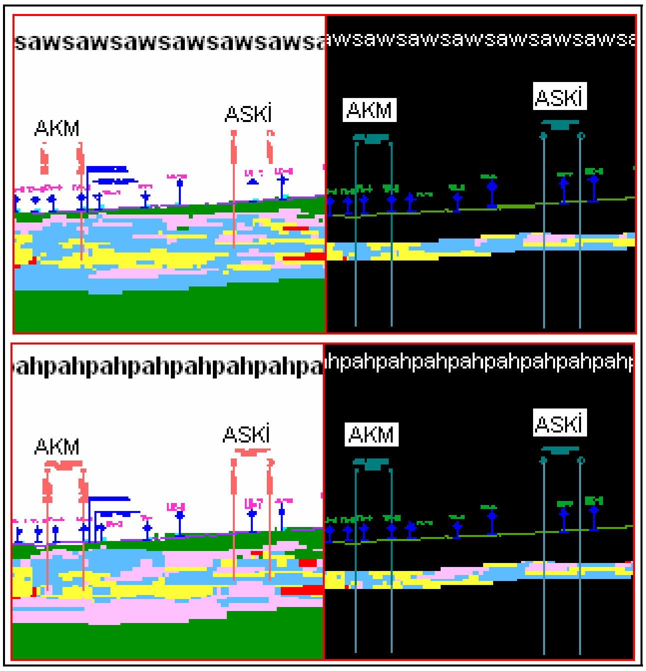

Figure 6.3. *Distribution of excavation difficulty classes between AKM and ASKİ stations*

6.1.4. ASKİ and Dışkapı Stations

The geological combination of this section is 34% silt of high plasticity, elastic silt(MH), 22% silty sand(SM), 10% silt (ML), 9% clay of high plasticity, fat clay(CH), 7% clay (CL), 7% clayey sand (SC) and 11% GM,SW,GW,SP. As the parts with very difficult level of excavation are passed lower difficulty levels are again reached in this part because finer grains again become dominant and hydrogeological conditions are not so aggressive.

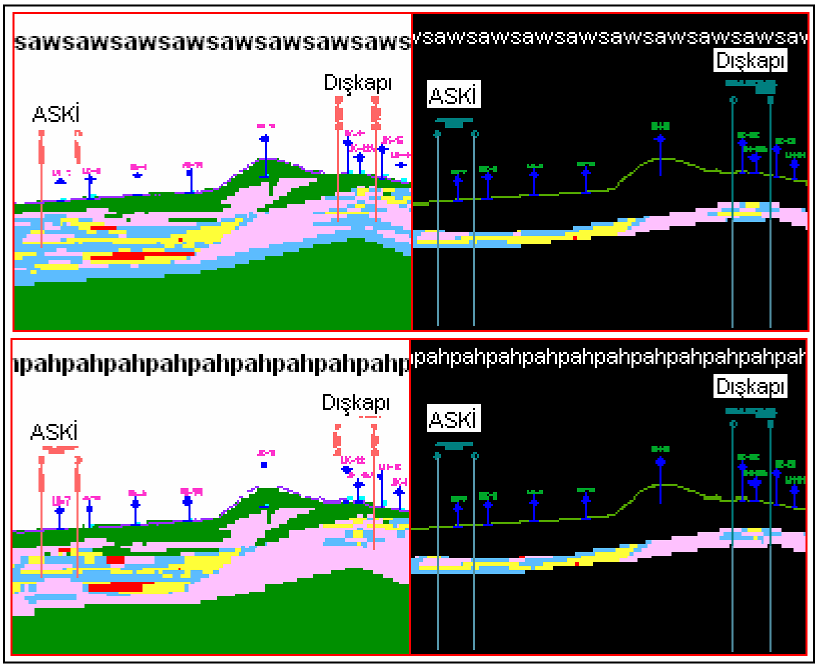

Figure 6.4. *Distribution of excavation difficulty classes between ASKİ and Dışkapı stations*

6.1.5. Dışkapı and Meteoroloji Stations

The most serious excavation problems are prone to be seen between Dışkapı and Meteoroloji stations of the alignment according to the results. This part is an old river channel with the geological composition of 27% silty sand(SM), 15 % clay (CL), 12% well graded sand, fine to coarse sand (SW), 12% silt of high plasticity, elastic silt(MH), 9% silty gravel(GM), 5% well graded gravel, fine to coarse gravel (GW) and 20% GP,ML,CH,GW,SC where coarse grains are dominant and hydrogeological constraints are available. Taking necessary precautions would be useful while drilling this part in order not to decrease the TBM performance.

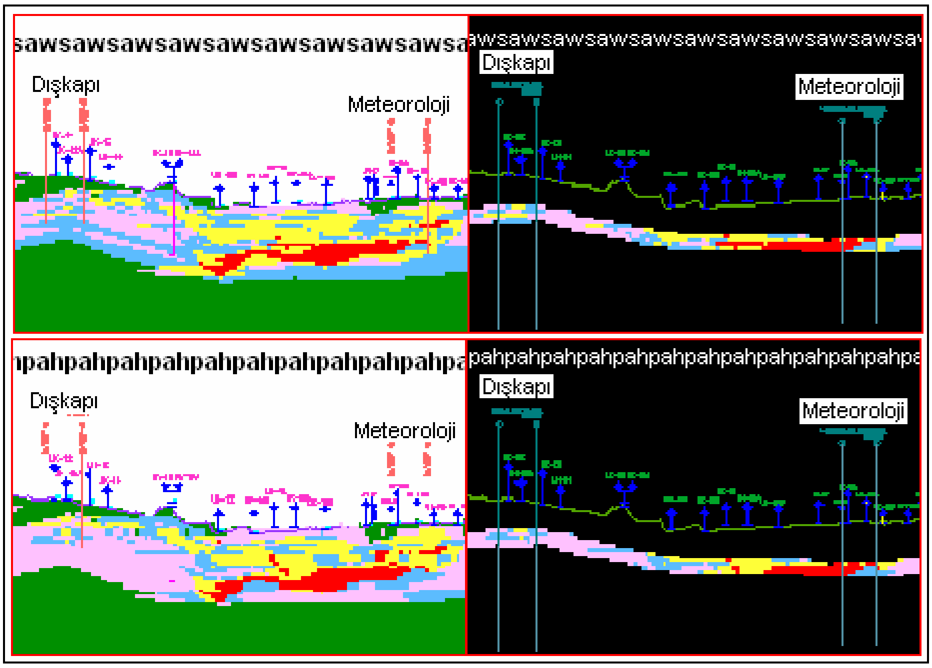

Figure 6.5. *Distribution of excavation difficulty classes between Dışkapı and Meteoroloji stations*

6.1.6. Meteoroloji and Belediye Stations

This part mainly corresponds to the area composed of volcanic sequence explained before. The only part containing finer ground material is where stations are built and the composition of these points is 27% silty sand(SM), 16% clay (CL), 14% silty gravel (GM), 14% well graded sand, fine to coarse sand (SW), 9% well graded gravel, fine to coarse gravel (GW), 6% clayey sand (SC), 6% poorly graded gravel and 8% MH, ML, SP, GC. The tunnel follows a route where passes just from the top of the pink and blue parts in the middle of the cross-section. Low level of difficulty is dominant between the stations but as mentioned earlier Meteoroloji station must be considered carefully.

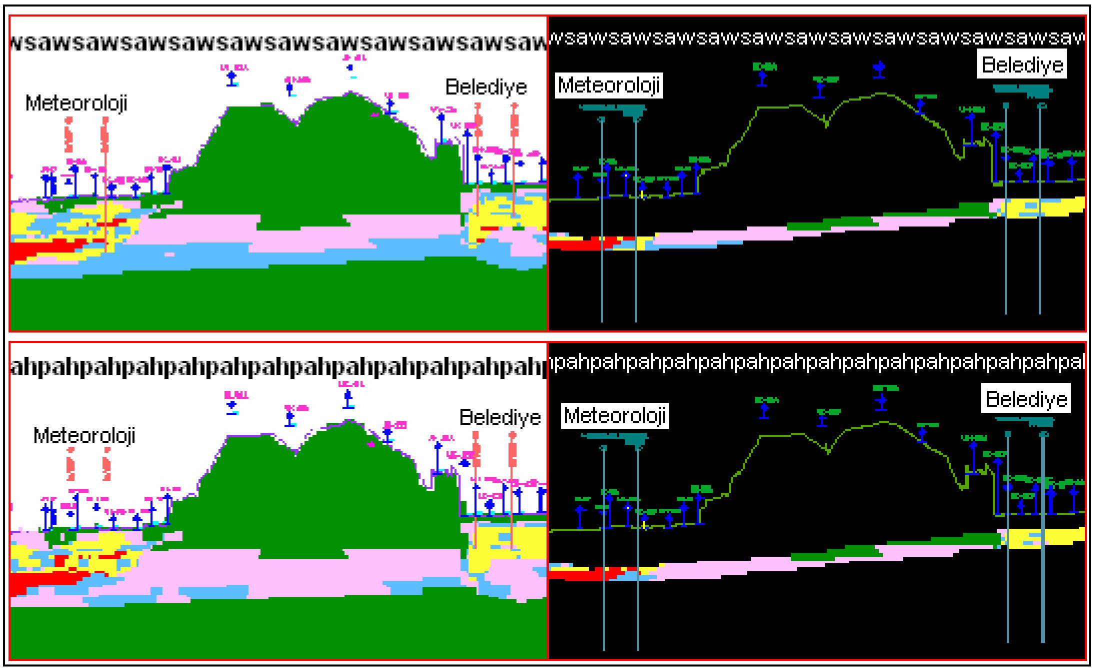

Figure 6.6. *Distribution of excavation difficulty classes between Meteoroloji and Belediye stations*

6.1.7. Belediye and Mecidiye Stations

According to SAW result very difficult level of excavation can be seen in Belediye station but when considering the overall results for both SAW and AHP cross-section this station is on difficult level of excavation. As tunnel advances to Mecidiye station maximum moderate level of excavation difficulty is seen. The geological composition of this section according to USCS is 30% silty sand (SM), 18% clayey sand (SC), 14% clay (CL), 9% silty gravel(GM), 7% well graded sand, fine to coarse sand (SW), 7% well graded gravel, fine to coarse gravel (GW) and 15% GP, ML, CH, SP, GC and volcanic units and Hancılı formation are seen as bedrock. Volcanic units are observed in Mecidiye station close to the surface.

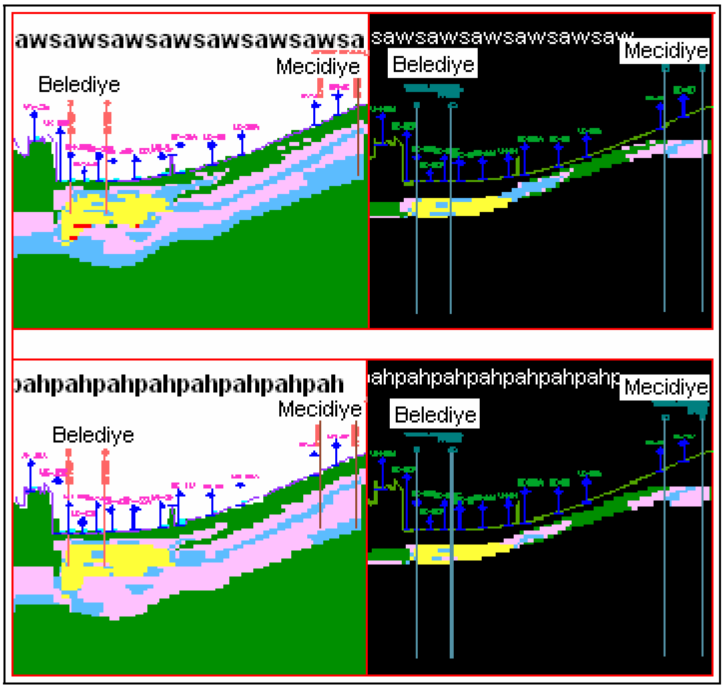

Figure 6.7. *Distribution of excavation difficulty classes between Belediye and Mecidiye stations*

When the results are considered generally SAW method displays more conservative results than AHP. The contractors are free to choose whether to be on the safe side or vice versa but as the comparison of the two methods proves AHP and SAW results are mainly in agreement. Changes between two methods are slight.

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