

ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

ENGINEERING GEOLOGICAL STUDY OF
THE PROPOSED ENTOTO TUNNEL

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JUNE 1999

**ENGINEERING GEOLOGICAL STUDY OF
THE PROPOSED ENTOTO TUNNEL**

**A Thesis Submitted to The School of
Graduate Studies, Addis Ababa University**

**In Partial Fulfilment of The Requirements
for The Degree of Master of Science in Geology & Geophysics**

By

Girma Wolderufael

June 1999

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Approved by: Board of Examiners

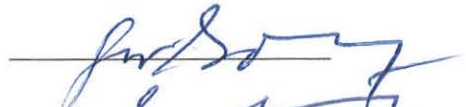
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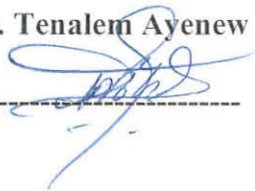


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June 1999.

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Abstract

For the developmant of Addis Ababa city's water supply, there needs to design a project of constructing two dams and a 4.8 km transmission tunnel through the Entoto ridge, by the Addis Ababa Water and Sewrage Authority (AAWSA). On the basis of the demand to construct a detailed engineering geological and geotechnical investigation for the proposed Entoto tunnel, the study was carried out after defining a tunnel corridor of about 10 km². The study focuses on the engineering geological investigation of this tunnel corridor.

The study area lies in the Shoa plateau that is formed of Tertiary rocks belonging to the Trap series. It is composed mainly of tuffs and trachytic lava flows which are called the Entoto silicics of Addis Ababa area. There are also basaltic lava flows outcropping on the top of the Entoto silicics as patches which area products of Tarmaber basalt. Undifferentiated Quaternary sediments which include residual, colluvial and alluvial deposits occur in various parts of the area.

Aerial photo interpretation, field traverses, insitu tests of rocks, as well as joint analysis from core orientation and surface observation and subsequent laboratory analysis were conducted. A number of exploratory boreholes drilled along the tunnel line indicated that a significant number of trachyte / tuff interfaces occur in the defined tunnel zone. From the borehole drilling and the geophysical investigation data, the area of the tunnel corridor appeared to be highly faulted and weathered. The position of the water table, which is above the tunnel zone, and the permeability of the rock masses were determined.

Engineering geological map at the scale of 1:10,000 was prepared. The rock mass strength was determined using Schmidt hammer and point load tester performed during field work and laboratory investigations. In addition to this in the field, rock material strength was done with field identification by using geological hammer and scratching by pocket knife.

The borehole data along the tunnel line, which is combined with surface mapping, confirmed the general geological assessment and allowed the choice of construction methods. The borehole data was used as a basis for detailed design of the tunnel. Using all the available information on the rock mass and its hydrogeological characteristics, rock mass classification schemes are developed. From all the existing methods of rock mass classification, the Rock Mass Rating (RMR) and the Rock Mass Quality Index (Q - system) are selected. Considering all the rock mass properties, using the above classification systems the most economic and suitable excavation method would be by drill and blast. The respective supporting method would also be by bolts and shotcretes.

It is hoped that this study will provide a sound geological basis during the design, and excavation stages of the tunneling. However, it should be emphasized that detailed geotechnical investigation on stress conditions of the rock mass and stability analysis, and further works at the portal areas and fault zones are very important.

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1. INTRODUCTION

1.1 GENERAL BACK GROUND

Addis Ababa, the capital city of Ethiopia, is expanding in all directions. Its population is also increasing in an alarming rate. The rapid increase in population has resulted in an increased demand and serious shortage of water in the city. The present water supply of the city from Gefersa and Lege Dadi reservoirs could not meet this growing demand. To alleviate the water shortage problem, the Addis Ababa Water and Sewerage Authority (AAWSA) which is responsible for the city's water supply has embarked on a large project named, "The Water Supply Project Stage III A".

The main task of this project is the construction of dams on Sibilu and Gerbi rivers, which are located 40 kms and 20 kms away respectively north of Addis Ababa. Since Entoto ridge, which is 3200 m.a.s.l., situated between the proposed dams and the city, it would be hard and costly to transmit the water from these dams through pipes by pumping. Hence, it needs to construct a transmission tunnel of 4.8 kms under the Entoto ridge.

The surface route generally follows the profile of the ground, thus requiring gradients in alignment. But absence of such gradients in the tunnel, for it follows short and direct route, may reduce operational and total cost depending on the length of project life.

Nevertheless, without careful geotechnical investigations prior to design and construction, and equally important, intensive continuous monitoring by geologists during construction, tunneling can result in completely uncontrolled and, at times, disastrous experience. Hence, if good geological and geotechnical investigation is carried out, tunneling is cheaper and safer than other means. Therefore, the desirability and necessity of constructing a transmission tunnel through the Entoto ridge is unquestionable.

The proposed tunnel axis was chosen in 1991 based on topographic conditions and shortest length of the tunnel during pre-feasibility study (unpubl. report, AAWSA, 1995). The proposed treatment plant site is located on the northern side of Entoto mountain range, hence treated water will be transmitted through the tunnel.

The current geological and geotechnical investigation work was started by first defining a tunnel corridor which covers an area of 10 km². Construction of the dams and the tunnel is scheduled to be finished around the year 2004.

1.2. LOCATION

The Entoto ridge is the southern margin of the Shoa plateau and a water divide of Awash and the Blue Nile river basin. The proposed tunnel axis is located across Entoto ridge just less than a kilo meter from Addis - Debre Markos road. It extends in the north - south direction for about 4.8 km. The out-let and inlet portals are situated at a distance of approximately 7 and 12 km north and northwest of the capital. The intake portal lies at a place where a small seasonal river emanates.

The study area, which covers a 10 km² tunnel corridor, is bounded between latitudes 9° 30'48"- 9° 06'44" N and longitudes 38° 41' 52"- 38° 44" E (fig.1.1). It is situated between altitudes 2560 and 2904 m.a.s.l.

1.3. CLIMATE AND VEGETATION

The climatic condition of Addis Ababa region is variable ranging from sub-humid to dry. In most cases it ranges from moderate to humid. The rainy season extends from July to late September. These wet periods, especially July and August, are the periods at which 80% of the total rainfall. The mean annual precipitation of Addis Ababa is around 1000 mm .

The summer has warm sunny days and cool nights. The highest temperatures are during the months of March, April and May and the lowest are in the months of July and August. November and December also have low temperatures with low rainfall. The average annual temperature is 16.45°C.

The entire part of the Entoto ridge is covered by different types of vegetation. The proposed tunnel alignment is densely vegetated with eucalyptus, juniper, etc. This dense forest makes the area somewhat inaccessible to investigation.

1.4. PREVIOUS WORKS

The general geological information concerning the present area of investigation has been furnished by many researchers. Mohr (1971), Kazmin, (1975), Morton, et al., (1979), Haile Sillassie Girmay and Getaneh Assefa (1989)

are some of them. The engineering geological mapping of Addis Ababa, at a regional scale, is also done by Kebede Tsehayu and Tadesse Haile Mariam in 1990. However, the various rock types and structures are not indicated at that scale.

The general tunnel alignment was initially chosen by Seureca (Consultants) in 1991 based on topographic conditions and length of tunnel. There was no geological investigation conducted during the feasibility study by Seureca.

A more recent work on the geology and geotechnics of the area is that done by Addis Ababa Water Supply Project Stage III A (1991). It was a feasibility investigation consisting of drilling and geotechnical studies. During this preliminary investigation, two alternative dam sites (upstream and down stream) at Sibilu and a dam site at Gerbi rivers have been studied. However, the investigations carried out were not sufficient to make a final choice of the dams and the tunnel route. Hence supplementary geophysical investigation and test drillings were recommended.

The surface as well as subsurface investigation of the area was conducted with the help of geophysical investigation by the Ethiopian Institute of Geological Surveys (EIGS, 1995). Drilling investigation consisting of 28 borehole along the tunnel axis is also performed by AAWSA in two phases in 1995. The investigation conducted by AAWSA for the project also includes a brief study of surface structures at the proposed dam and tunnel sites. The geophysical survey consisted of Seismic Refraction, Vertical Electrical Sounding (VES) and Transient Electromagnetic Surveys (TEM).

1.5 OBJECTIVE

This study is aimed at determining geological, hydrogeological and geotechnical properties of rocks of the surface and subsurface underlying the proposed tunnel route leading to a preparation of an engineering geological map of the area at the scale of 1: 10,000.

Available information about the geotechnical condition of the tunnel zones anticipated helps a great deal during designing and construction. The classification of rock mass for tunneling according to the rock mass rating (RMR) and rock mass quality (Q) systems is another objective. This will help during excavation and in designing supporting systems for different rock types.

Suggesting the appropriate methods for tunneling and to give design parameters for different lithologies and supporting mechanisms is also another focus of this study.

1.6. METHODOLOGY

Different investigation methods which are useful to provide information regarding the engineering geological, geotechnical and hydrogeological conditions have been conducted. These include: aerial photo interpretation, field geotechnical investigation and various data compilation.

Aerial photo interpretation was made using black and white photographs at the scale of 1:8,000 and 1:33,000. The mapping was conducted by taking additional traverses in the field. Some measurements of discontinuities and rock mass strength tests using Schmidt hammer were conducted.

Joint analysis from core orientation and surface observation was an important step in the evaluation of fracture flow and directional strength. Interpretation was carried out after fieldwork by introducing the field observation and laboratory results. Point load strength test, slake durability test and water chemistry tests are the laboratory tests used in the study. Mineralogical analysis for some rock samples is also performed. The final engineering geological map was made on topo map at the scale of 1:10,000. By using measurements of ground- water levels from piezometric installations, the groundwater contour map showing the form and elevation of the water table is constructed.

Geophysical investigation, borehole drilling, packer permeability test, piezometric data were obtained from the Addis Ababa Water Supply Project Stage III A. Based on these data rock masses in the study area was classified using Rock Mass Rating and Rock Quality Index (Q) classification systems. The design parameters, available methods for tunneling and supporting mechanism, are recommended by using the rock mass classification systems.

2. PHYSIOGRAPHY AND GEOMORPHOLOGY

2.1 GENERAL

Morphology of Addis Ababa and its surrounding areas is quite complex. Starting from relatively flat and undulating central part with an increase in elevation towards the north and the west and of gradual decrease in topography to the south.

There are many volcanic topographic features, which vary in elevation from about 3200 m in the north to 2500 m in the south. Among these volcanic features, the most important ones are Entoto ridge (3200 m), Mt.Yerer (3100 m), Mt. Furi (2839 m) and Mt. Wachacha (3385 m) (fig.2-1).

The steep slopes of Entoto ridge are located to the north of the city whereas to the south, though there are some hilly features, quite gentle slopes which extend to the Ethiopian Rift. Deep gorges, frequent rapids and waterfalls which clearly show evidences of a young morphology are also the morphological features present in Addis Ababa and its surroundings.

2.2. TOPOGRAPHIC FEATURES OF ENTOTO RIDGE

The steep slope of Entoto ridge, is the water divide between the Awash and the Blue Nile basins. Almost all the main rivers (like Kotebe, Akaki, and Kebena) originate mainly from Entoto ridge.

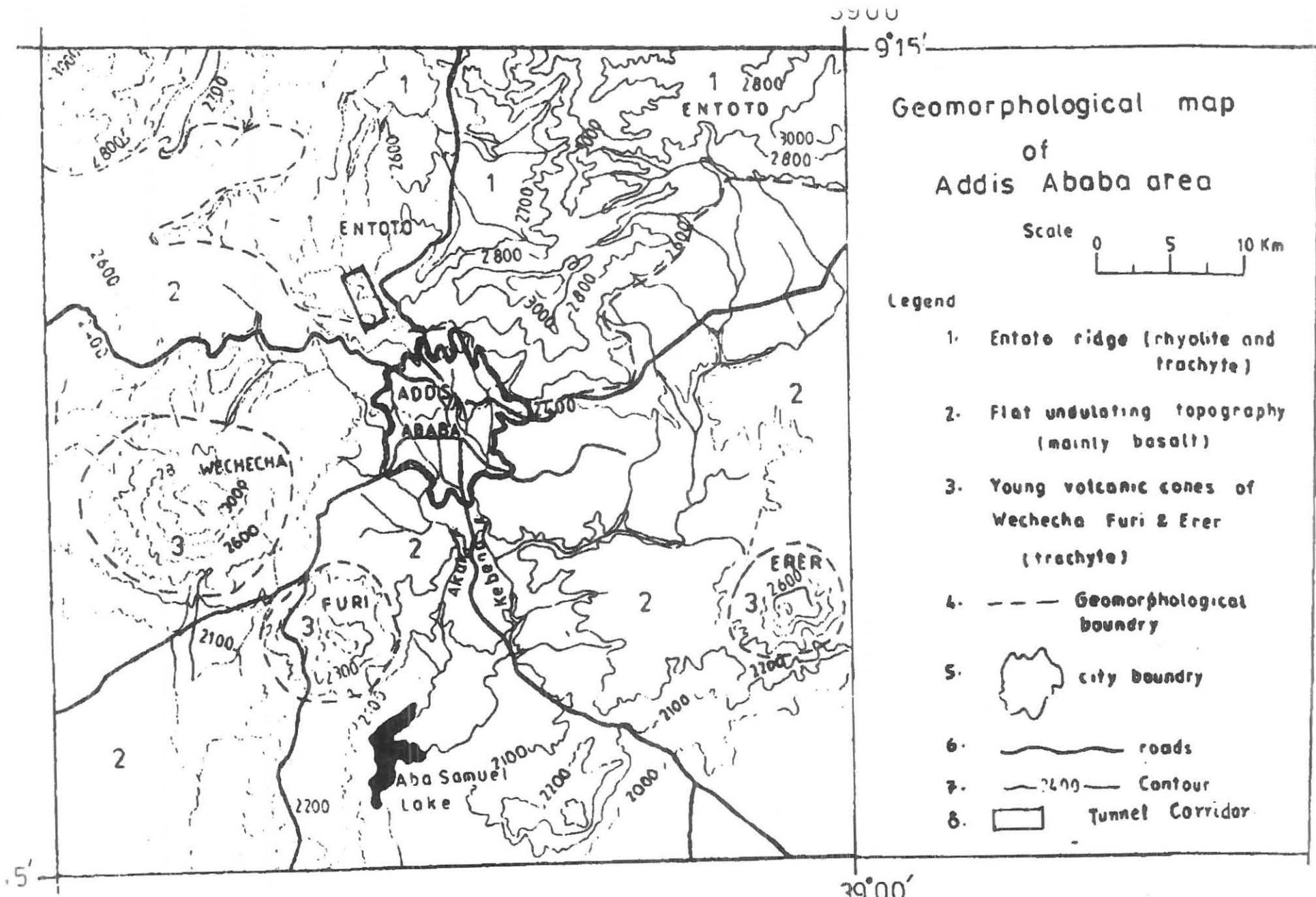


Fig.2.1 Geomorphological map of Addis Ababa area (Kebede and Tadesse, 1990)

The landscape of the whole area of Addis Ababa reflects a complex geological structure which has been further modified by the action of erosion.

1.3 DRAINAGE SYSTEM

The texture of the drainage system is influenced by rock type and structure, the nature of vegetation cover and the type of climate (Bell, 1993). Rock type has a strong influence on the drainage texture, that is a low drainage density tends to form on resistant or permeable rocks; whereas weak, highly erodible rocks are characterized by a high drainage density.

Except for small streams, all the main rivers originate in areas of highest elevation (mainly the Entoto ridge) and flow to the south and southeast direction to join the Awash river.

The steep slopes of the northern part of the city are characterized by a drainage pattern of dendritic behavior. Towards the south, due to the morphology of the area, the drainage pattern is constituted by the main rivers or big tributaries characterized by wide meandering type of flow. In the west, southeast and central parts of the city, they form terraces and alluvial deposits. In the tunnel corridor, there are only seasonal streams flowing to all directions from the top of the ridge.

2.4. EROSION

Since Addis Ababa is on the Shoa plateau where almost 80% of the rainfalls during the three months of the rainy season, erosion is intensive. The other factors which also give high erosive capacity, for the runoff of the city is the presence of steep slopes. As a matter of fact there are a number of rill and gully erosion on slopes of Entoto ridge and Wechacha (Kebede Tsehayu and Tadesse Haile Mariam, 1990)

Addis Ababa is built just on the foot of Entoto, where all the streams are emanating. Frequently, the velocity of the streams is higher to make the erosion high.

Entoto ridge and other hilly areas are densely covered with eucalyptus tree that can retard erosion, but due to expansion of the city settlements started on slopes and illegal deforestation, some part of these areas are becoming bare. In addition, the presence of weathered rocks like tuffs and residual soils facilitate erosion. That is why the development of rill and gully erosion is so fast and common in the area.

The tunnel corridor shows highly rugged topography with some gullies. Particularly, the presence of extremely weathered trachytic and basaltic lava flows that are almost changed to residual soils, at the portal areas facilitate the intensity of erosion. That is why the portal areas are composed of thick soils of residual, colluvial and alluvial deposits.

3. SITE INVESTIGATION

After the proposal of the Entoto tunnel, geological and geotechnical investigations were conducted by different groups and organizations as part of Addis Ababa Water Supply Project Stage IIIA..

The different investigations conducted include: geophysical survey, borehole drilling, packer test, piezometer installation and laboratory tests.

3.1. GEOPHYSICAL SURVEY

Geophysical survey was carried out along Entoto corridor, which covers an area of 6 km.², by the Ethiopian Institute of Geological Survey (EIGS) in 1995, and this section relies heavily on the results of the survey.

The geophysical survey consisted of Seismic Refraction, Vertical Electrical Sounding (VES), Transient Electro Magnetics (TEM) and Magnetic Surveys. These investigations were aimed at obtaining subsurface geological information for the design of the tunnel route.

The geophysical data gained from this survey is used for determining the hydrogeological and physico-mechanical properties of rocks of the subsurface underlying the proposed tunnel route. The geological structure prevailing in the study area are also obtained from the survey. The geophysical investigation was done after leveling geophysical observation points along the following profiles (see Fig. 3-1).

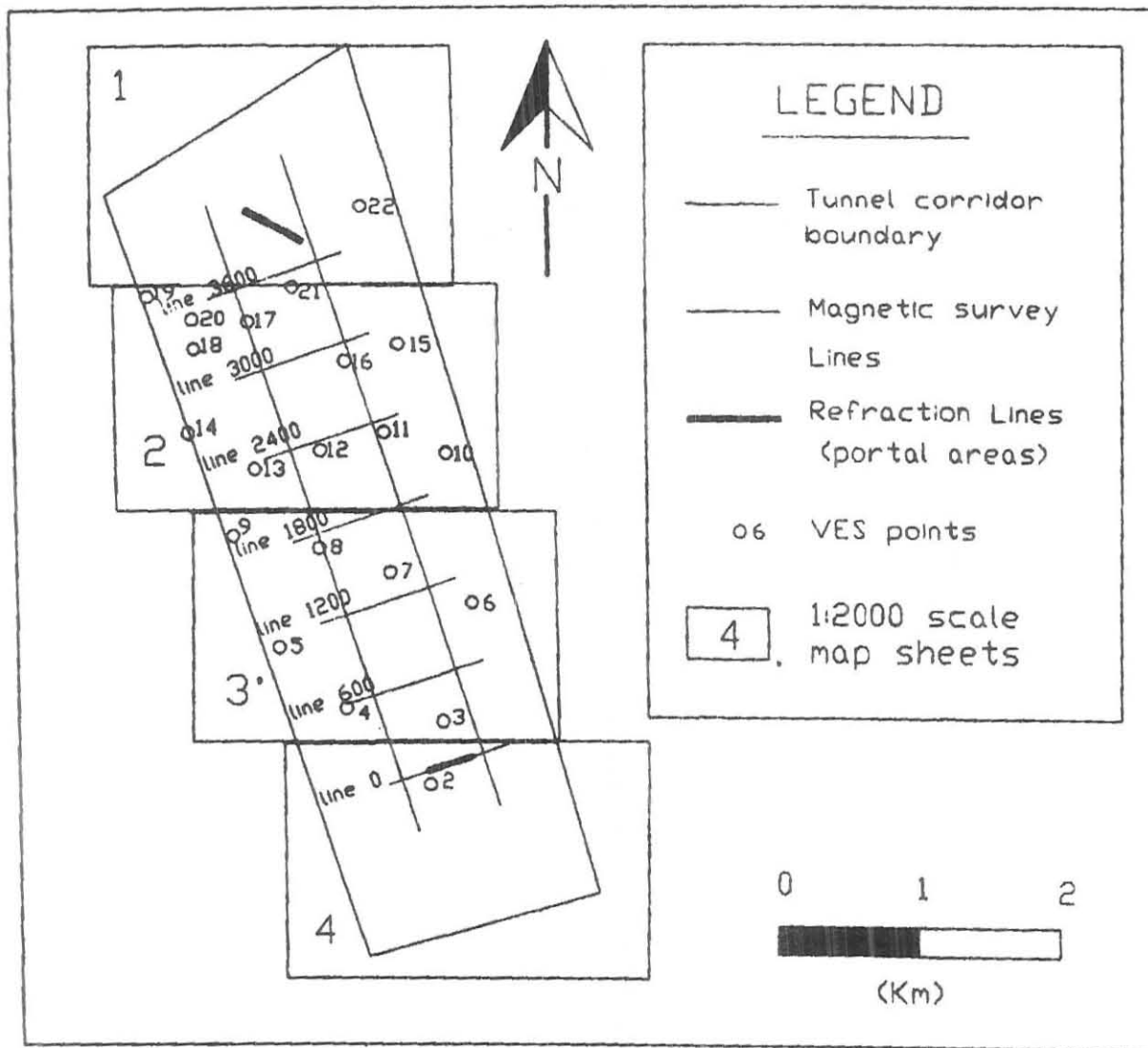


Figure 3-1 Geophysical survey layout of Entoto Tunnel Site (source: EIGS, 1995)

Table 3-1:- Survey lines along the proposed Entoto tunnel site.

Main Profiles /Tie Lines	Cross Profiles	Total length	Station
Tie line 200 E (4700 m long) and tie line 800E (4400 m long) both parallel to the base line	Seven parallel profiles with 1000 m length each and one with 940 m length	16040 m	10 m

3.1.1. Seismic Refraction

The refraction seismic survey conducted at the tunnel portals in line profiling system (i.e. shot points and geophones in the same line) with 5 m geophone spacing was used during the survey.

The seismic data sampled at a record time of 200 m for direct, central and reverse shots and 500 m for the far shots were recorded in the Terraloc's hardware and then analyzed. The depth of investigation in the portal areas was 35 m and the geophone spacing used was 5 m.

A. Intake Portal Line

The line extends in NNW-SSE direction with a length of 400 m and passes just over the proposed intake portal.

According to the report compiled by EIGS, the intake portal depicts three

layers, that are characterized by different velocities (fig. 3-2). The first layer is characterized by velocities between 630 m/s and 1000 m/s and thickness reaches up to 11 m. The layer indicates higher values (900-1000 m/s) in the extreme NNW part of the area and in the thickest SSE extreme zone. It thins out towards the center and gradually pinches out between pickets (reference points) 90E and 180E (fig.3-2a). The observed velocities may be attributed to highly decomposed rock and clay.

The second layer exhibits widely varying velocities (1360 m/s - 2800 m/s). The maximum thickness of this layer is 21 m and is found near picket 115, while the maximum velocity (2800 m/s) corresponds to the zone of rock exposure. To the east of this zone the longitudinal wave velocity decreases and the layer thins out and completely pinches out near picket 320E.

The observed wide velocity variations indicate the variation in the degree of weathering of the upper part of the bedrock. The intermediate velocities (1950 - 1960 m/s) may indicate that the associated weathered rock is partially saturated.

The bottom velocity layer shows widely varying longitudinal wave velocity (2530 - 6900 m/s). This velocity variation could be attributed to possible change in lithology and /or degree of weathering of rocks. The probable fault associated with a weak zone at picket 190E (near borehole 2) seems to be continuous. Moreover, the lower velocities observed between pickets 160 and 290 could probably be indicative of groundwater percolation.

B. Outlet Portal Line

This profile extends in the WSW-ENE direction and passes the proposed outlet portal. The length of this line is 400 m, like in the intake portal. According to figure 3-2b, the seismic section corresponding to the outlet portal line discerns three main layers (refractors). The uppermost layer, covering the whole profile except between pickets 70 and 90, is characterized by lower velocities generally varying between 300 m/s and 700 m/s and thickness reaching a maximum of 6 m. The relatively high velocities (500 m/s - 700 m/s) correspond to the western zone while the thicker zone is found in the central part of the area near picket 320. The zone around picket 80 shows rock exposure which could probably be associated with a possible low flow probably, of basic composition through the indicated fault.

The second layer generally exhibits velocities ranging between 815 m/s and 1100 m/s to the east of picket 90. Towards the west, however, a 1770 m/s velocity is noted. The thickness of this layer varies from 7 to 15 m. The portion east of picket 320 may be considered as a four layered zone, the third horizon of which has a 1310 m/s velocity. The thickness of the later varies between 14 and 29 m. The bottom layer is characterized by widely varying velocities (1820 m/s and 4760 m/s).

The extreme western part of the profile, between pickets 0 and 120 exhibits the highest velocity (4760 m/s) while the western portion between pickets 120 and 400 shows lower velocity (1820 - 2700 m/s). The lowest velocity corresponds to the zone between pickets 120 and 160 (fig.3-2b).

The observed low bottom refractor velocities could probably be attributed to weathered volcanic rocks, while higher velocities reflect moderately to slightly weathered and fresh rocks. Moreover, on the basis of the

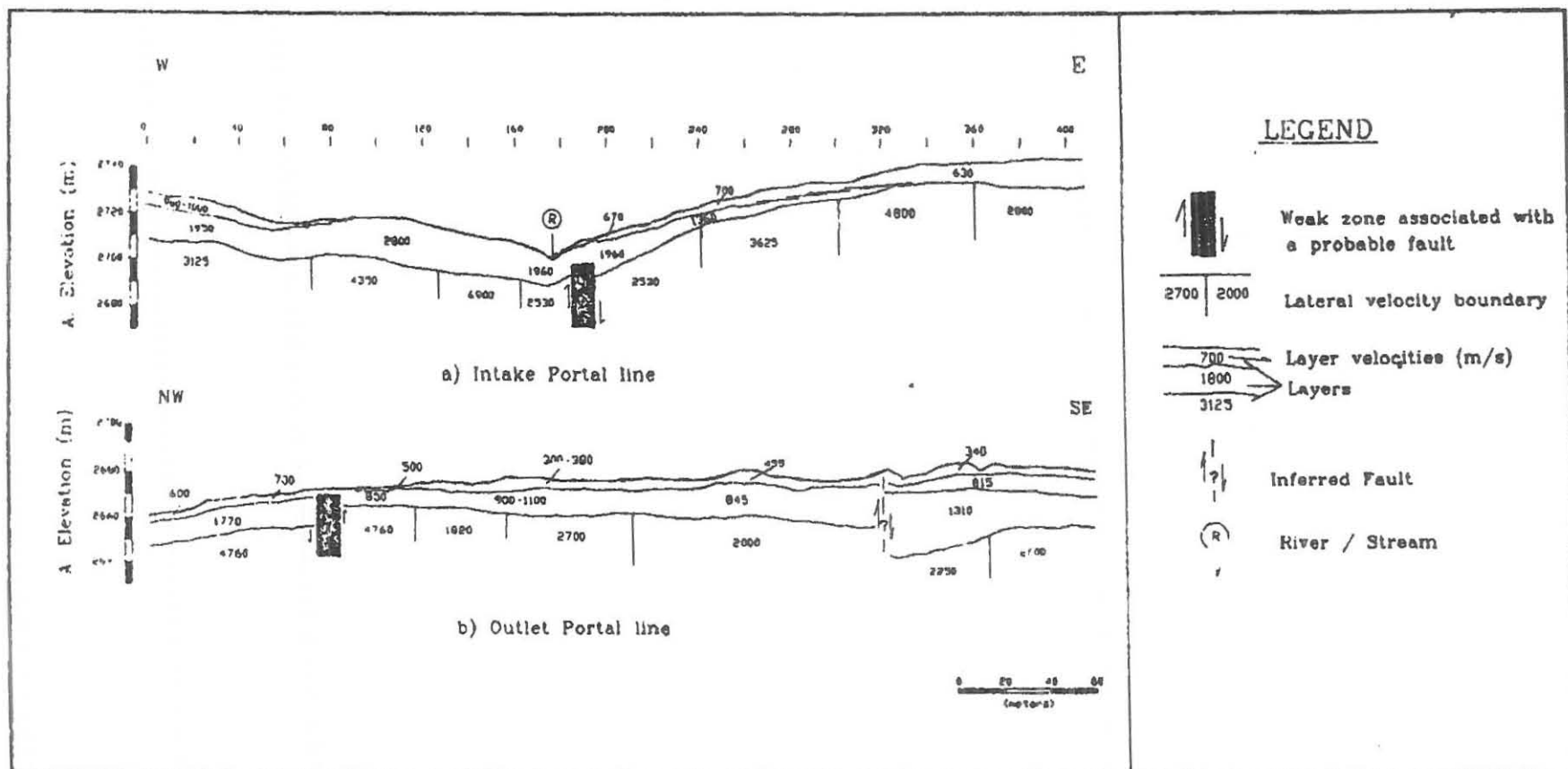


FIG.3-2. Refraction Seismic sections along the Portal areas,Entoto tunnel area(*Source: EIGS, 1995*)

refracted velocities weak zones associated with probable faults are identified.

1.1.2. Vertical Electrical Sounding (VES)

Data was collected from 24 VES points around the tunnel corridor which are far apart. Their locations were determined to be at critical points where subsurface information required. Low resistivity zones were observed in some geo-electrical sections. From the available resistivity data, interpretation was made.

Accordingly, some geoelectric sections (VES 11 and VES 16) show four resistivity layers, while some consists of three geoelectric layers (fig. 3-3 and 3-4). The surfacial layer having resistivity varying between 195 and 570 ohm-m corresponds to the top soil (also 140 - 180 ohm-m).

The underlying layer is characterized by lower resistivities ,ranging from 13 to 22 ohm - m; at some places it reaches up to 35 ohm-m. It is probably correlated to highly weathered volcanic rock. It has a wide variation in thickness ranging from 20 to 140 m. The observed resistivities as well as widely varying thickness are interpreted to be the response of a highly weathered rock. The large thickness of the highly weathered section under VES 6 may be the result of structural effect.

The fourth layer has resistivities of 54 to 68 ohm-m, which possibly indicate that the rocks at depth are moderately weathered and /or fractured. The resistivities in the eastern part are generally higher than those in the western portion.

LEGEND



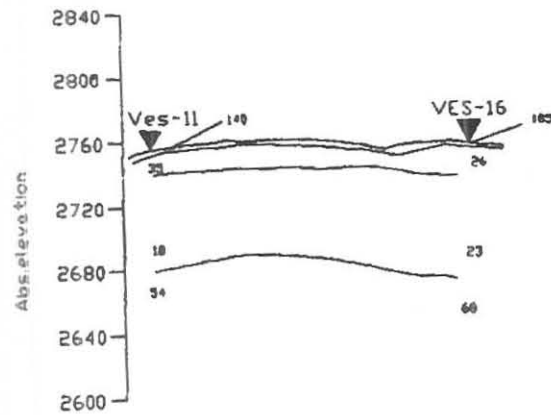
VES point

75

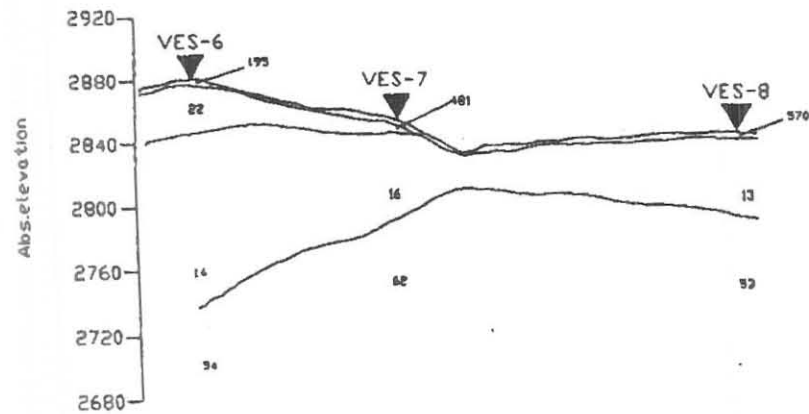
48

Lithologic boundry

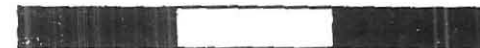
(a)



(b)



0 100 200 300



(meters)

FIG. 3-3. Geoelectric sections on different lines within the Entoto tunnel area.
(Source: EIGS, 1995)

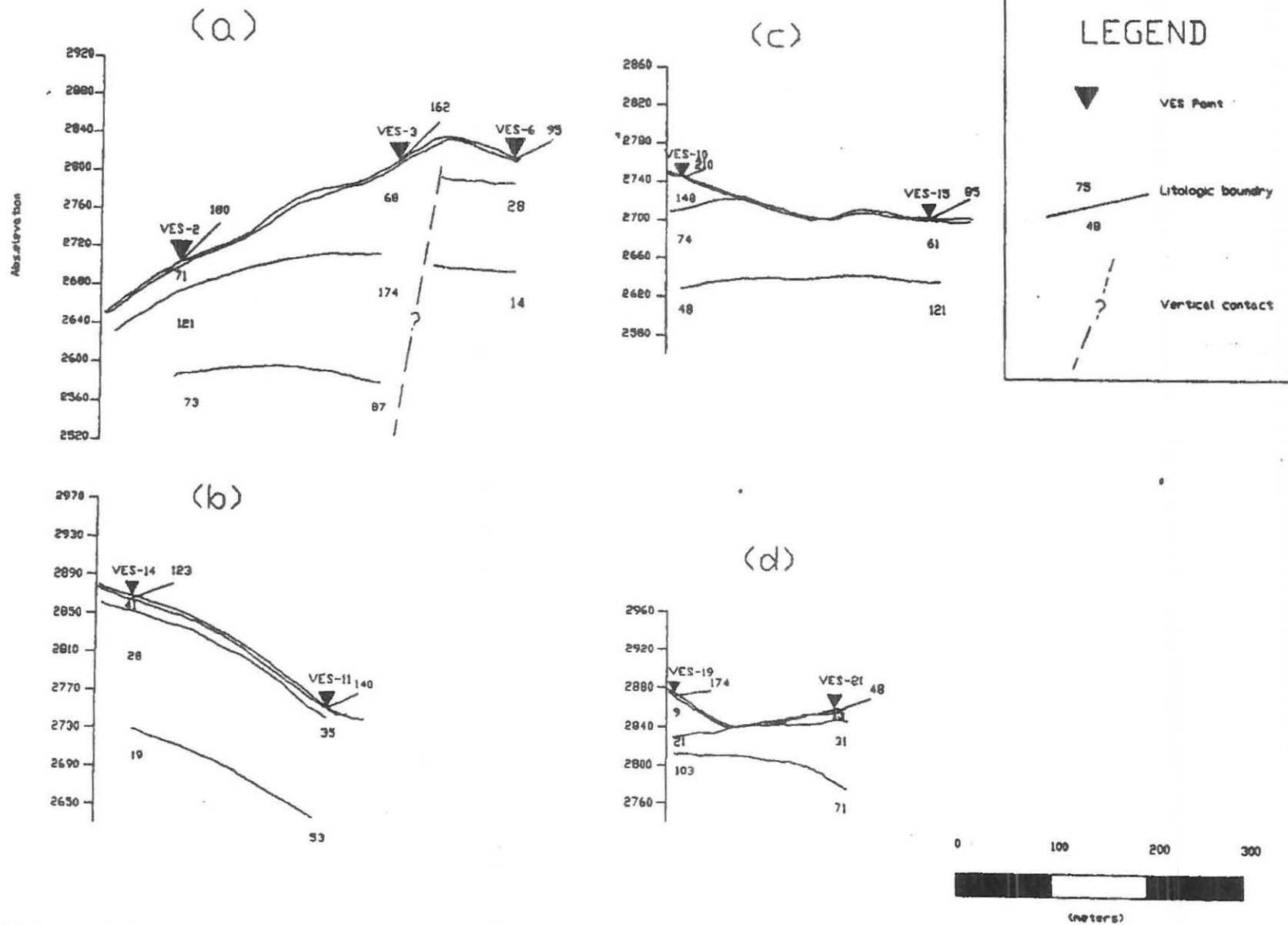


FIG. 3-4. Geoelectric sections on different sites within the Entoto tunnel area.
(Source: EIGS, 1995).

This high resistivity contrast might be an indication of a lithologic contact.

3.1.3. Transient Electromagnetic Survey (TEM)

Transient electromagnetic survey was conducted by EIGS. This method of investigation is selected for its large penetration depth. There are a total of 10 profiles, with a length of 1.5 km up to more than 2 km. The spacing between profiles at north and southern part is 400 m and 4 profiles the at central part each with 500 m spacing. TEM has helped much in showing the subsurface geological condition of the tunnel corridor. Faults are indirectly detected by this method. The location of first phase drilling was guided by TEM investigation.

3.1.4. Electromagnetic Survey

Electromagnetic measurement was made uniformly at every 5 m interval all over the survey sites. It covers about 3369 observation points. From the magnetic survey some results are observed.

An abrupt lateral shift linearly trending magnetic anomaly is observed just north of line 3600 (fig.3.1). This lateral shifting indicates the lithologies having different anomalies which might be the contact between the trachyte and the basalt at the northern NE-SW fault. Another remarkable structure is also indicated. These two magnetically high regions distinguished by a thin and sharp fall in magnitude may indicate a fault.

Generally, the magnetic data indicate the existence and approximate position of major and minor geological structures. Faults and shear zones as well as contacts have been outlined by the magnetic survey.

3.2. CORE DRILLING

Drilling is the most important and expensive method of investigation. A total of 28 boreholes with varying depths were drilled at Entoto proposed tunnel corridor. In all the holes, double core drilling was conducted upto the tunnel zone, which is about 30mts. or 20mts. above and 10mts. below the center line (table 3-2).

Drilling methods utilized to drill the holes along the tunnel corridor were tripping (at the beginning of bore holes), double core barrel (with HQ and NQ sizes for non tunnel zone), triple core barrel drilling (with HQ and NQ size with in tunnel zone) and mud drilling.

Most drillings were in HQ size and when there was problem in the HQ hole size, it was reduced to NQ size. Both vertical and inclined drillings were conducted. The depth of boreholes drilled range from 20 m up to 290 m. The inclinations of the boreholes range from 50 to 75 degrees. A careful logging of cores was carried out by AAWSA. Hence, lithologies, Total Core Recovery (TCR), estimation of strength and fracture frequency, Rock Quality Designation (RQD), and angles of discontinuities with core axis were described from the borehole logs. From all the boreholes drilled in the area, only some located at the tunnel zone and some others are selected for interpretation in this work.

Table 3-2 Details of boreholes drilled in the tunnel corridor(source : AAWSA).

No.	Borehole	Collar elevation (masl)	Length (m)	Angle (°)	Azimuth
1	ENT95-1	2727.92	90	90	0
2	ENT95-2	2737.52	160	60	115
3	ENT95-3	2755.87	140	90	0
4	ENT95-4	2783.44	180	60	280
5	ENT95-5	2808.5	220	90	0
6	ENT95-6	2776.41	170	60	280
7	ENT95-7	2862.22	255	70	140
8	ENT95-8	2795.32	190	60	90
9	ENT95-9	2888.64	290	65	290
10	ENT95-10	2822.5	230	65	120
11	ENT95-11	2675.39	89.8	50	0
12	ENT95-12	2656.26	50	55	120
13	ENT95-13	2673.51	65	55	300
14	ENT95-14	2685.27	80	65	120
15	ENT95-15	2760	160	60	300
16	ENT95-16	2642	25	90	0
17	ENT95-17	2644	31	75	0
18	ENT95-18	2637	20	90	0
19	ENT95-19	2655	30	75	90
20	ENT95-20	2667	47	75	90
21	ENT95-21	2639	15	90	0
22	ENT95-22	2713	95	75	270
23	ENT95-23	This borehole was not drilled. Second phase starts from here.			
24	ENT95-24	2768	143	90	0
25	ENT95-25	2807	182	90	0
26	ENT95-26	2874	250	90	0
27	ENT95-27	2732	105	90	0
28	ENT95-28	2632	12	90	0
29	ENT95-29	2630	15	90	0

Table 3-3 Double and single Packer Test results(Source : AAWSA)

NO	BOREHOLE	SINGLE PACKER TEST	DOUBLE PACKER TEST	MINIMUM AND MAXIMUM HYDRAULIC CONDUCTIVITY RANGE (CM/SEC)
1	ENT - 95 - 1	12	3	$K=1.14 \times 10^{-7} - 1.47 \times 10^{-4}$
2	ENT - 95 - 2	10	2	$K=3.59 \times 10^{-7} - 2.79 \times 10^{-4}$
3	ENT - 95 - 3	14	3	$K=6.59 \times 10^{-7} - 1.14 \times 10^{-3}$
4	ENT - 95 - 4	9	1	$K=1 \times 10^{-6} - 1 \times 10^{-3}$
5	ENT - 95 - 5	10	-	$K=1 \times 10^{-7} - 1 \times 10^{-4}$
6	ENT - 95 - 6	6	-	$K=3.07 \times 10^{-7} - 5.87 \times 10^{-4}$
7	ENT - 95 - 7	5	-	$K=2.44 \times 10^{-6} - 1.56 \times 10^{-4}$
8	ENT - 95 - 8	5	1	$K=0.53 \times 10^{-6} - 2.18 \times 10^{-4}$
9	ENT - 95 - 9	3	1	$K=2.34 \times 10^{-7} - 4.14 \times 10^{-3}$
10	ENT - 95 - 10	5	-	$K=1.94 \times 10^{-7} - 3.45 \times 10^{-5}$
11	ENT - 95 - 11	4	-	$K=(3.46 - 10) \times 10^{-4}$
12	ENT - 95 - 12	4	-	$K=2.89 \times 10^{-4} - 8.54 \times 10^{-5}$
13	ENT - 95 - 13	5	-	$K=1.80 \times 10^{-6} - 3.35 \times 10^{-4}$
14	ENT - 95 - 14	5	-	$K=1.5 \times 10^{-6} - 6.8 \times 10^{-4}$
15	ENT - 95 - 15	6	-	$K=1.49 \times 10^{-9} - 1.22 \times 10^{-4}$
16	ENT - 95 - 16	-	-	Not Conducted
17	ENT - 95 - 17	5	-	$K=4.94 \times 10^{-6} - 3.75 \times 10^{-4}$
18	ENT - 95 - 18	-	-	Not Conducted
19	ENT - 95 - 19	3	-	$K=1 \times 10^{-4}$
20	ENT - 95 - 20	3	-	$K=3.97 - 7.77 \times 10^{-4}$
21	ENT - 96 - 21	-	-	Not Conducted
22	ENT - 96 - 22	6	-	$K=6.34 \times 10^{-6} - 1.90 \times 10^{-5}$
23	ENT - 96 - 24	5	-	$K=1.25 \times 10^{-4} - 1.01 \times 10^{-1}$
24	ENT - 96 - 25	5	-	$K=1.14 \times 10^{-4} - 6.10 \times 10^{-5}$
25	ENT - 96 - 26	5	-	$K=1.14 \times 10^{-6} - 1.19 \times 10^{-5}$
26	ENT - 95 - 27	5	-	$K=0 - 5.88 \times 10^{-5}$
	TOTAL	141	11	

3.3. PACKER TEST

Packer tests have been carried out by adding pressurized water to the bore-hole sections under pressure. The rate at which water flows into the surrounding ground through a response section sealed off by casing or packers was measured. Such tests were carried out in almost all Entoto tunnel boreholes, except some portal boreholes.

Both single and double packer tests are performed. In double packer testing the variation in hydraulic conductivity throughout the test hole is determined. The zone of rock to be tested in a drill hole is isolated by using two packers which seal off the drill hole, the water being pumped in to the space between the packers. The tests are performed from the base of the hole upwards. In single packer test, the zone between the bottom of the packer and the base of the drill hole has been tested. The response section was from 3 m to 6 m and length of zone interest varied from 20 m to 60 m. The permeability is then calculated from a flow fracture curve.

A single packer test was carried out following each interval of drilling process for fixed response section, which varied from 3 m to 6 m. A double packer test was carried out after the completion of drilling where a section was not tested by using single packers due to a technical reason or where required to test a certain doubtful section after the drilling. This was conducted after choosing the response section sealing it off by double packers at both sides.

The total number of single packer and double packer tests along the Entoto tunnel corridor was 141 and 11, respectively. The detail is provided in table 3-3.

Table 3-4 Peizometric Installation(AAWSA)

NO.	BORE-HOLE	ANGLE (o)	PIEZOMETER ZONE	
			DEPTH (m)	VERTICAL DISTANCE (m)
1	ENT - 95 -1	90	P1-63 - 66 p2- 78-81	63 - 66 78 - 81
2	ENT - 95 -2	60	91.30 -94.30	79.07 - 87.67
3	ENT - 95 -3	90	116 - 119	116 - 119
4	ENT - 95 -4	60	199.60 - 202.60	199.60 - 202.60
5	ENT - 95 -5	90	234.70 - 238.70	220.55 - 224.60
6	ENT - 95 -6	60	154.06 - 157.30	133.42 - 136.23
7	ENT - 95 -7	70	276.75 - 282.75	250.82 - 256.26
8	ENT - 95 -8	60	168 - 191	170.39 - 173.10
9	ENT - 95 -9	65	67.0 - 70.0	51.32 - 53.62
10	ENT - 95 -10	65	36.0 - 39.0	32.63 - 35.35
11	ENT - 95 -11	50	59.00 - 62.00	53.47 - 56.19
12	ENT - 95 -12	65	135.50 - 138.50	53.93 - 56.64
13	ENT - 95 -13	65	12.85 - 15.85	12.85 - 15.85
14	ENT - 95 -14	65	17.10 - 20.10	16.52 - 19.42
15	ENT - 95 -15	60	17.00 - 20.00	17.00 - 20.00
16	ENT - 95 -16	90	13.70 - 16.70	13.23 - 16.13
17	ENT - 95 -17	75	33.33 - 35.33	32.19 - 35.09
18	ENT - 95 -18	90	12.20 - 15.20	12.20 - 15.20
19	ENT - 95 -19	75	87.95 - 90.95	84.95 - 87.85
20	ENT - 95 -20	75	139.30 - 142.30	139.30 - 142.30
21	ENT - 95 -21	90	168.60 - 171.60	168.60 - 171.60
22	ENT - 95 -22	75	247.0 - 250.0	247.0 - 250.0
23	ENT - 95 -23	-	-	-
24	ENT - 95 -24	90	139.3-142.3	
25	ENT - 95 -25	90	168.6-171.6	168.6-171.6
26	ENT - 95 -26	90	247.0-250.0	247.0-250.0
27	ENT - 95 -27	90	95.80-98.80	95.80 - 98.80
28	ENT - 95 -28	90		
29	ENT - 95 -29	90		

3.4. PIEZOMETER INSTALLATION

In order to monitor seasonal ground water fluctuation, piezometers were installed in all the boreholes drilled by AAWSA. The piezometers installed with their tips located in the tunnel zone. Single piezometers were put in each of the boreholes drilled except in borehole 1, where two piezometers were installed at different levels.

The total length of 2935.96 m dia. 3/4 " PVC piezometers pipe was installed in all the boreholes drilled. No major problem has been encountered during installation of piezometers except losing pipes, 180 m long in boreholes 5 and 9. The number of piezometers and their locations are shown in table 3-4.

3.5. LABORATORY TESTS

In addition to field identification or visual inspection, different laboratory tests have been conducted. In order to know measured strength by uniaxial compressive test, point load index tests were conducted for trachytes, basalts and tuffs. Due to their weak nature, moisture content, volume, weight and density, slake durability test was conducted only for tuff to know the durability.

Groundwater samples, were taken from eight tunnel axis boreholes and tested in AAWSA laboratory. The laboratory result of some boreholes is given in Appendix -B.

Mineralogical analysis for some rock samples is also performed. Some rocks thought to be rhyolitic lava flows were mapped late as trachytic lava flows.

3.5.1. Point Load Strength Test

Point load test is a simple field and laboratory testing method for rapid classification of strength of rocks.

Sample of irregular size or borehole cores are placed between the two cones in such a way that the load is applied in the direction of the smallest diameter. With a hydraulic hand pump oil, pressure is given to the piston to which the lower cone is mounted.

Failure load can be calculated from the maximum oil pressure recorded on the oil pressure gauge. After correction of the filler load for shape factors and size of the irregular sample, the point load strength index is calculated. The point load strength has an “index” value. The test result data can be used to classify rocks into different strength categories. But the data can not be correlated directly with uniaxial compressive strength data as obtained from carefully prepared cylindrical samples in laboratory testing. The point load test result of some boreholes is given in table 3-5.

3.5.2 Slake Durability Test

Slake durability test is a test where the durability and/or resistance to erosion of a rock mass is measured. It is used to estimate the resistance to wetting and drying of a rock sample, particularly for weak rocks which exhibit a certain degree of alteration.

Table 3-5 point load Test Results for Entoto Tunnel

No.	sample no	Rock type	Depth	D(mm)	Is	Strength designation based on point load	Uniaxial Compressive Strength(MP)	Strength based on compressive
1	en95-7	trachyte	226.66	61.2	4.3	very high	106.253	R5- very strong
		trachyte	226.99	61.2	4.4	very high	108.724	R5- very strong
		trachyte	232.54	61	4	very high	98.7	R4-strong rock
		trachyte	232.83	61	2.8	high	69.09	R4-strong rock
		trachyte	236.56	61	1.3	high	32.0775	R3-medium stro
		trachyte	236.8	61	3	high	74.025	R3-medium stro
		trachyte	263.04	63.5	6	very high	150.675	R5- very strong
		trachyte	235.09	63.5	7	very high	175.7875	R5- very strong
2	en95-13	basalt	40.8-41.1	56.8	7.5	very high	179.55	R5- very strong
		basalt	63.43-63.84	63.5	6.8	very high	170.765	R5- very strong
		Vitric tuff	61.72	63.5	6	very high	150.675	R5- very strong
		Vitric tuff	61.95	63.5	7	very high	175.7875	R5- very strong
3	en95-2	trachyte	31.85	60.9	3	high	73.9725	R4-strong rock
		trachyte	32.29	60.9	3.8	high	93.6985	R4-strong rock
		trachyte	41.9	63.2	3	high	75.18	R4-strong rock
		trachyte	42.15	63.2	2.8	high	70.168	R4-strong rock

Note: D(mm)- Smallest diameter of the sample in millimeter

Is = point load strength index in MPa

In the slake durability test fragments are placed in a sieve drum which rotates half immersed in a tank of water. The weight loss is measured after each cycle of wetting, drying and erosion. The test is suitable only for weak weathered rocks, there generally being too little effect on hard fresh rock. Since unwelded tuffs rocks are generally weak, slake durability test was conducted on such type of rocks.

Table 3-6: Slake durability test results.

Borehole	Depth	Rock Type	Strength (Mpa)	Remark
BH. 2	113.00	Tuff (fine)	7.03	R2 (weak)
	113.20	Tuff (fine)	6.83	R2 (weak)
BH 4	141.60	Tuff (coarse grained)	3.33	R1 (Very weak)
	141.90	Tuff (coarse grained)		

4.GEOLOGY

4.1 Stratigraphy

4.1.1 Regional Stratigraphy

The studied area lies in the Shoa plateau, which is formed of Tertiary volcanic rocks belonging to the trap series. It is located in the western margin of the Ethiopian Rift and consists of different volcanic rocks that range from basic to acidic composition (fig. 4-1). The geology of Addis Ababa and its surroundings had been studied by several researchers both locally and at the regional scale.

The sequence of the rock units described by Haile Selassie Girmay and Getaneh Assefa (1989) reveals the Miocene-Pleistocene volcanic succession ranging from older plateau volcanic to younger Rift volcanics. The Alaji Basalts are exposed at the crest of Entoto hills and across the Sululta plains. They form high topography (ridges) and are the oldest rocks belonging to Alaji cycle (Zenettin et al., 1974). These basalts show variations in texture from porphyritic to aphyritic. The age limit of these basalt is within the range of 20-28 Ma (Morton et al., 1979), and this forms part of the Ethiopian flood basalt of Tertiary.

In the northern part, the Entoto mountain chains are composed of rhyolite and trachyte with minor amount of obsidian rich tuffs, which are called the Entoto Silicics. They are associated with Alaji formation and rest on older basalt. The rhyolites are porphyritic with phenocrysts of quartz, sanadine and andesine-oligoclase and a groundmass of devitrified glass, iron oxide, plagioclase and quartz whereas the trachytes are composed of phenocrysts of anorthoclase, sanadine, oligoclase and magnitite and a groundmass of devirtified glass and plagioclase (Haile selassie

Girmay and Getaneh Assefa, 1989). The age range of these rock units is 21.5-22.6.Ma.

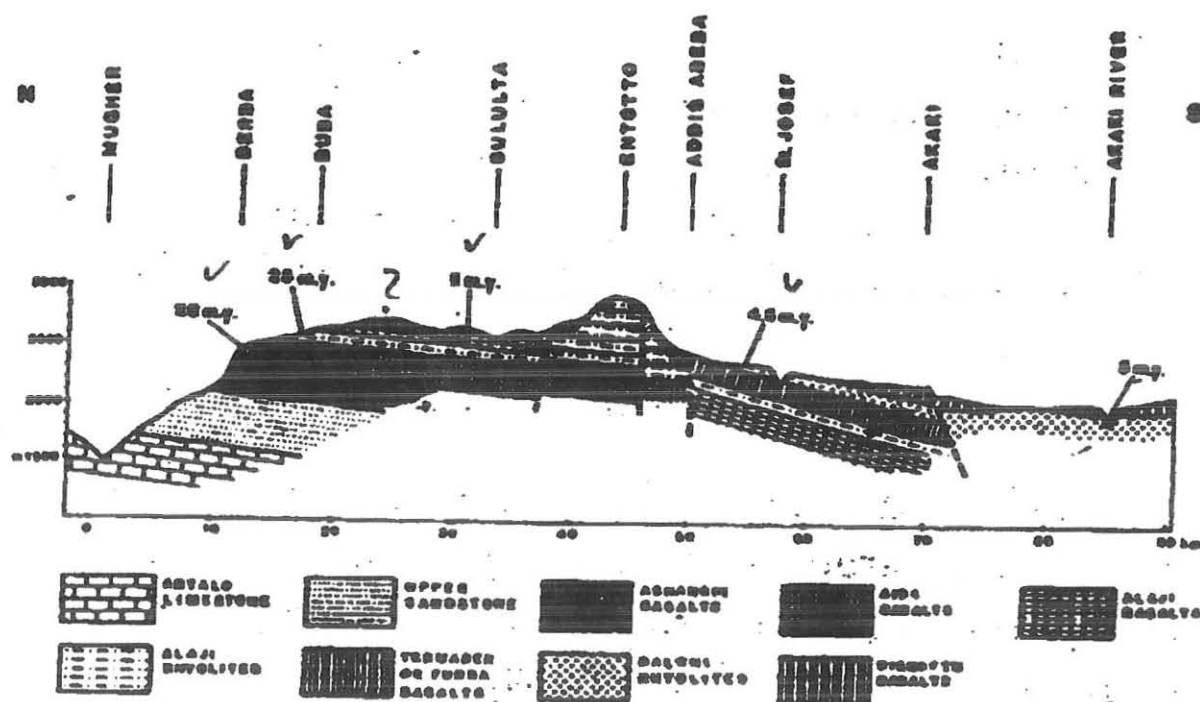


Figure 4.1 Cross section showing regional stratigraphy (after Justin-Visentin et al., 1979).

The available evidence for Early Miocene silicic volcanism in the Addis Ababa region suggests an isolated centre at Entoto, with eruption thinning both Rift ward and Plateau ward, this centre was later strongly upfaulted as a NE-SW trending horst at the margin of the developing rift (Morton et al., 1979) The Alaji Aphanitic basalt outcropping in the northeastern part near Entoto Kidanemheret church and the trachyte are separated by paleosol indicating time gap.

are filled by calcite. The K-Ar age determination of Morton et al (1979), in the tributary of Akaki river at the southwestern part of Addis Ababa indicates that the basalt is 3.6 Ma. This age corresponds to the Yerer volcanism, 3.5 Ma (Morton and Rex, 1975) which is assumed to be the source of this basalt.

The youngest of all volcanic rocks outcropping in the Addis Ababa region is the upper welded tuff. It covers the south and southwestern part of Addis Ababa and extends to the Akaki town (15 km to the southwest of Addis Ababa). The age of this rock unit determined by Morton (1974) is 3.2 Ma. It is overlain by olivine (Bofa) basalt at Akaki river near Akaki town (2.8 Ma).

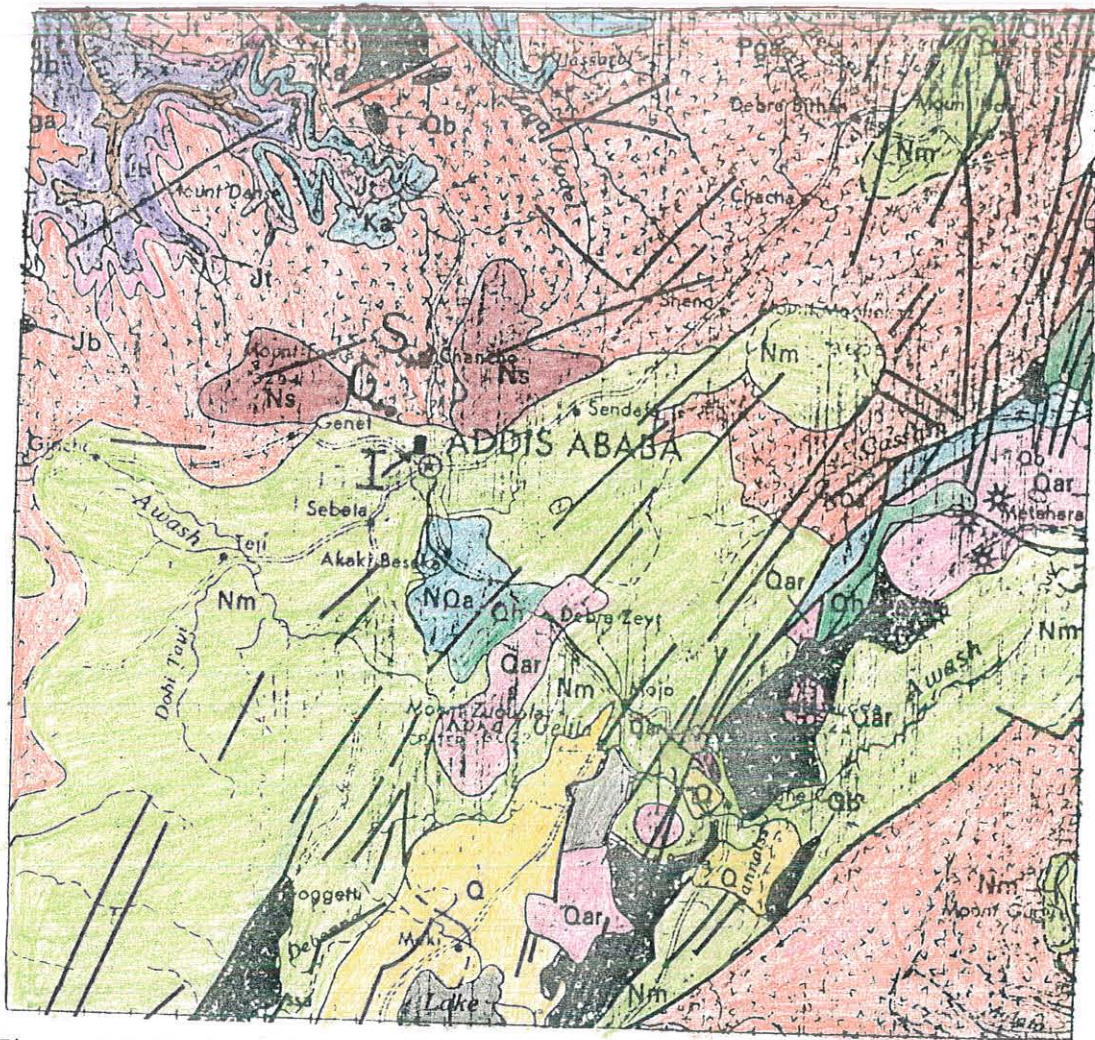


Figure 4-2 Geological map of central Ethiopia (After Kazmin, 1972).

0 40 Km.

LEGEND

- Holocene, conglomerates, Sands silts, clays, reef limestone.
- undifferentiated
- Basaltic flows and related spatter cones a/ Transitional type between alkaline and tholeiite basalt b/ alkaline olvine basalt
- siliceous domes and flows, pantellerites, obsidians, complex volcanoes of andesite-trachyte-rhyolite composition.
- Afar Group, Miocene-pleistocene, Basalts, subordinate acid lavas, ignimbrites
- Magdala group (upper Miocene-pleistocene?) Rhyolites, trachytes, rhyolites and trachytic tuffs; ignimbrites agglomerates, basalts.
- Shield Group, Miocene, Alkali basalts, tuffs, agglomerates.
- Ashangei Groups, Paleocene-Oligocene-Miocene a) Alkali olvine basalt and tuffs rare rhyolites b) Doleritic sills and gabbro-diorite intrusives.
- Ambaradam Formation, Lower Cretaceous.
- Clay it, Sandstone Conglomerate (including the upper Sandstone of Harar and Aruss)
- Agula Shale upper jurassic, Kimmeridgian Marked shales.
- Antal Limestone, Upper Jurassic Oolitic, Detrital, Fined micro-grained limestone, marls
- Abbi Beds Middle jurassic (?) Variegated shale gypsum.
- Adigt sandstone, Lower-Middle Jurassic
- Sandstones, shales, in the Abbay Basin and the Danall Alps Possibly includes parts of pzt

4.1.2. GEOLOGY OF THE TUNNEL CORRIDOR

4.1.2.1. Description of lithologic units of the studied area

The study area constitutes the northern part of Addis Ababa on the Entoto ridge, and covers an area of about 10 km². The Entoto ridge is composed of rhyolite, trachyte and tuff, which are called the Entoto Silicics of the Addis Ababa area that are associated with Alaji formation (Zenettin, et al., 1974). The age of these Alaji rhyolite, trachyte and basalt is 21.5 Ma. They rest on older basalt. Volcanism initiating the Alaji cycle occurred in Late Oligocene - Early Miocene times.

The Entoto ridge is located along the normal fault of the E - W trending Ambo - Kessam lineament (Moore and Davidson, 1978). It dominantly consists of trachytic lava flows at central part and basaltic lava flows as minor outcrops in northwest and southeast. The basaltic lava flow overlies the trachytic lava flows. Both welded and unwelded tuffs are rare as outcrop on the surface but frequently observed along the road cuts and on borehole logs underlying and overlying the trachytic flow. There are also quaternary sediments which include residual, colluvial and alluvial deposits in various parts. Paleosol is found near the northern portal of the tunnel.

A. Tuff

Both welded and unwelded tuffs are not widely outcropping on the surface like trachytic lava flows due to removal by erosion. But they are only found as patches in the north central part and along the road cuts. Below the surface, there is a significant amount of tuff alternating with trachytic lava flows. As seen on the core drillings, in the subsurface it covers even in a larger extent compared with the trachytic flow.

There are both thick unwelded tuff mainly in the upper part and a thin welded zone at the base of the subsurface of the study area. The layering between the soft unwelded

tuffs and hard welded tuffs is gradational and non uniform for which they are mapped together. The welded tuff has obsidian which appear as weavy lenses and / or complete layers of dark glass. This welded tuff with rock fragments and glasses (obsidian) is stronger than the fine unwelded tuff. The fine tuff with ash size material is generally weaker. Lapilli tuff contains very large rock fragments reaching upto 50 mm of trachyte and other acidic rocks. The rock fragments are sub-rounded to sub angular in shape.

Tuff is the least resistant type of rock to weathering in the study area. Weathering of tuff grades from slightly weathered to highly weathered. Due to its weak nature, it is susceptible to weathering and erosion. It disintegrates when it comes into contact with water. In highly weathered state it is friable. That is why it is not seen outcropping on the surface. But lapilli tuff with rock fragments is seen on the road cuts in the studied area, which is not in a mappable scale. Therefore, for the weathering of tuff in the area, texture, structure and topography play important role.

Generally, joint spacing ranges from 2 mm to 3 m. In most cases in tuff joint spacing is wider than in hard rocks like trachytic and basaltic lava flows. From point load test results, the strength of the unwelded tuff varies from extremely weak to weak.. Fine grained tuffs are very weak and soft. Most of the welded tuffs are hard, with medium strength scale and highly jointed.

B. Trachytic lava flow

The trachyte in the Entoto ridge is the result of lava flow along east west trending Ambo - Kessam regional fault erupted during the lower Miocene (Moore and Davidson, 1978). It is the most dominant rock in the study area. Except in some parts of either ends of the tunnel, it covers the entire part of the tunnel corridor. In the subsurface it is found alternating mainly with unwelded tuff.

D. Quaternary Sediments

Undifferentiated Quaternary sediments which include residual, colluvial and alluvial deposits occur in various parts of the area. The residual deposits occur both in mappable scale and as patches in most parts of the study area. The colluvial deposits are limited to the foot of northern and southern hills. But the alluvial deposit which is partly affected by landslide occurs in the southeastern and northern part of the area probably covering the basalt near small streams. The alluvial sediment found along the seasonal Weserbi stream (near the northern portal) has a thickness ranging from 0.2 m to 13.5 m.

From core drilling some paleosols are also observed intercalating between unconsolidated rocks. The paleosol present near the northern portal, which has a thickness of about 13 m is mapped in the cross section.(fig. 4.3).

4.1.2.2. Geological description of selected boreholes

A total of 28 boreholes were drilled into the studied area by AAWSA along the tunnel corridor. For detailed analysis nine boreholes are selected from different parts of the tunnel corridor. Both vertical and inclined drilling was conducted. The inclined boreholes targeted faults and other geological fractures and had inclination of 50° to 75° . The depth of the boreholes drilled ranges from 20 m to 290 m. In each borehole, a zone of 30 m (i.e. with 20 m above and 10 m below the center of the tunnel axis) has been carefully cored using triple tube arrangement. Permeability tests and core orientations were conducted in most boreholes.

Sixteen boreholes along the tunnel axis or adjacent to the axis are taken for drawing the tunnel zone geological cross-sections. The defined tunnel zone is 20 m above the crown and 10 m below the invert line, the length being about 4800 m. Invert elevation is 2635 m.a.s.l at upstream portal grading and 2631 m at down stream portal. The proposed tunnel alignment represents the shortest distance across Entoto ridge for the given invert elevation. Some boreholes do not reach the tunnel line. For example, borehole 27 does not reach the tunnel position while boreholes 25, 26 and 16 go only half way or less of the tunnel section. From the boreholes selected for analysis borehole 2 and 11 are selected for they are adjacent to the fault zones. Boreholes 19, 20 and 22 are taken as representative for the northern part. Borehole 25 is representative for the central part and boreholes 16, 17 and 18 represent the southern part.

The geological information above and below the invert line is very important for engineering design. Hence, a brief description of selected boreholes with special emphasis on the tunnel zone is given below. The description goes from northern part of the tunnel axis to the south.

The strength of the rocks in the boreholes is described in terms of International Society of Rock Mechanics (ISRM) classification system.* (see Appendix c).

BH-19

In borehole 19, the lithology is trachyte, the rock is moderately weathered and the strength varies from R2-R3 (weak to medium strong). The joints are close and in addition, there are also broken cores. Its surface elevation starts from 2655m.a.s.l.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0 - 12.00	trachyte	■ gray to dark gray, highly to moderately weathered, weak to medium strong (R2 - R3), fine grained, poor RQD.
12.00 - 31.00	trachyte	■ gray to dark gray, moderately weathered, medium strong (R3), porphyritic with plagioclase phenocrysts, good RQD.

* Rock strength is given as R0, R1, R2, R3 and R4 weak rocks with strength range of 5 to 25 Mpa are further subdivided into: R2⁺ = 5 - 10Mpa, R2 = 10 - 15 Mpa and R2⁺ = 15 - 25Mpa.

BH-20

In this borehole, the lithology is trachyte, dark gray to gray and pinkish color, moderately weathered, the strength is weak (R2-R2), close to moderately close jointing. At elevation of 2626 m.a.s.l. and below fine tuff is encountered which is weak and with moderately close jointing. Its collar elevation is 2667 m.a.s.l.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0 - 4.50	trachyte	▪ grayish brown, highly weathered, very weak (R1), fine-grained, poor RQD.
4.50 - 10.00	trachyte	▪ gray; moderately weathered weak (R2), close jointing, fine-grained, poor RQD.
10.00 - 12.55	clay (altered tuff)	▪ reddish brown, very weak (R1), fine grained, poor RQD.
12.55 - 18.75	trachyte	▪ light gray, fine grained, moderately weathered, weak (R2), poor RQD.
19.55 - 41.30	trachyte	▪ grayish to pinkish, fine grained, moderately weathered, weak, close jointing, good RQD.

41.30 - 46.45	tuff	▪ light gray, greenish, slightly weathered weak (R2 ⁺) fine to medium grained, good RQD.
46.45 - 47.05	trachyte	▪ greenish, slightly weathered, weak (R2 +), moderately close jointing, fine-grained, good RQD.

BH-22

The collar elevation is 2713 m.a.s.l. It mainly consists of trachyte up to the depth of 2622mts, that is about 91mts from the surface. From a depth of 73.20 to 90.73mts it is predominantly vesicular trachyte with yellowish gray, yellowish brown, and light gray colors. The weathering grades from fresh stained to moderately weathered, and the strength varies from very weak to medium strong (R1-R3). There are also broken cores at some depths below the surface. The following weak materials are encountered.

<u>Depth in meters</u>	<u>Rock</u>	<u>Description</u>
<u>below the surface</u>		
0.00 - 6.00	colluvial soil	▪ reddish brown.
6.00 - 37.45	trachyte	▪ yellowish gray, highly weathered, weak (R2), close jointing, fine to medium grained, very poor RQD.

37.45 - 63.00	Trachyte	light brown, slightly to moderately weathered, medium strength (R3), close jointing, fine to medium, very poor RQD.
63.00 - 88.50	trachyte	yellowish gray, strained, moderately weathered, medium strength (R3), fine to medium grained, poor RQD.
88.50-88.70	trachyte	highly weathered, extremely weak (R0), very good RQD.
88.70-88.90	clay	
88.90-89.10	tuff	gray color, highly weathered, very weak(R1), fine to medium grained, close jointing, poor RQD.
90.10-90.35	tuff	highly weathered, very weak (R1), close jointing, silicified trachyte, broken core.
90.75-90.94	tuff	moderately to highly weathered silicified trachyte, the strength is weak(R2⁻-R2⁺), good RQD.
90.94 and below	lapilli tuff	greenish-gray, moderately weathered, weak(R2⁻), poor RQD.

BH-2

Collar elevation is 2738m.a.s.l.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0.00 - 2.90	scoraccous basalt	▪ slightly weathered, strong to very strong (R4 - R5).
2.90 - 64.00	trachyte	▪ gray, moderately weathered, medium strength (R3), poor RQD.
64.00 - 72.60	welded tuff	▪ greenish gray, moderately weathered, weak strength (R2 ⁻), open jointing, poor RQD.
72.60 - 99.00	trachyte	▪ moderately weathered, medium strength (R3). good RQD.
99.00 - 130.05	lapilli tuff, welded tuff	▪ moderately to highly weathered, weak to medium strength (R2 ⁻ - R2 ⁺), good RQD.
130.05-134.50	indurated clay	▪ when exposed to sun light shrinkage cracks is developed and it is semi plastic material.
134.50-136.00	welded tuff	▪ dark pinkish, slightly weathered moderately strong (R3), fine to medium grained.

136.00-136.43	tuff	grayish to light green, fresh to slightly weathered, weak (R2⁺), with close jointing.
136.43-137.21	welded tuff	grayish color, fresh to slightly weathered, weak(R2)
137.21-140.95	tuff	gray to dark gray, moderately weathered weak (R2⁻), some rock fragments and phenocrysts are observed.
140.15-144.05	welded tuff	light gray, greenish gray and pinkish color, slightly to moderately strong and fine-grained tuff.
144.05-160.00	tuff	dark brown, greenish and greenish gray, slightly to moderately weathered, very weak to weak (R1-R2), close jointing and fine grained, poor RQD.

BH-25

The elevation of the top of the borehole is 2807 m.a.s.l.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0.00 - 5.00	ilty clay	▪ reddish brown.
5.00 - 8.40	trachyte	▪ highly weathered, weak (R2 ⁻).
8.40 - 18.15	trachyte	▪ yellowish gray, slightly weathered, strong (R4), fine-grained, good RQD.
18.15 - 42.15	vitric tuff	▪ yellowish gray to dark grayish, slightly weathered, weak to medium strong (R2 ⁻ - R3 ⁺) with grass shards, poor RQD.
42.15 - 48.15	trachyte	▪ dull gray, slightly weathered, weak to strong (R2 ⁻ - R4), good RQD.
48.15 - 53.10	lapilli tuff	▪ yellowish gray, moderately weathered to weak (R2 ⁺), good RQD.
53.10 - 59.25'	trachyte	▪ yellowish gray; moderately weathered, weak to medium strong (R2 ⁺ - R3).

26.30 - 30.00	tuff	▪ slightly to moderately weathered, weak to medium strong (R2 -R3).
30.00 - 50.35	trachyte	▪ light green-gray, fresh stained to slightly weathered. At 13 places there are broken cores, strength ranges from weak to medium strong, close jointing, RQD is poor.
50.35-53.35		▪ light pink-grayish pink, highly weathered, weak, close joints, very poor RQD.
53.35-53.70	complete loss	
53.70-61.20	trachyte	▪ greenish-yellow to light gray, slightly to moderately weathered, weak (R2), close jointing and broken cores observed, poor RQD.

BH-17

Collar elevation is 2644m.a.s.l. only trachyte is encountered.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0.00 - 2.50	residual soil	
2.50 - 10.00	basalt	▪ slightly weathered, strong to very strong (R3 - R4), good RQD.
10.00-12.95	trachyte	▪ highly broken core, reddish brown, moderately weathered, very weak to weak(R1-R2), close jointing, poor RQD
12.95-16.00	trachyte	▪ greenish gray, slightly to highly weathered, very weak to medium strong (R1-R3), from 13.48-14.10 broken core, core loss exists.
16.00-18.00	trachyte	▪ pinkish gray, slightly to moderately weathered, weak (R2 ⁺), contains broken core and core loss, poor RQD.

18.00-25.82	trachyte	gray slightly to moderately weathered, weak to medium strong, medium grained, closely jointed, good to very good RQD.
25.82-31.00	trachyte	greenish gray, slightly to moderately weathered, medium strong (R3) fine to medium grained. It is closely jointed.
31.00 -	trachyte	gray, moderately weathered(R3), fine-grained, close jointed, good RQD.

BH-16

The elevation of the top of the borehole is 2642 m.a.s.l. The borehole didn't penetrate the proposed tunnel line.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0.00 - 0.90	residual soil	reddish.
0.90 - 10.00	trachyte	pinkish, gray, highly weathered, poor RQD.
10.00-11.85	trachyte	gray, moderately to highly weathered, extremely weak to weak (R0-R2), close jointing. There is broken core from 10.82-11.08, RQD is poor.

11.85-13.45	trachyte	greenish gray, slightly weathered, medium strong close joints. Very good RQD
13.45-16.45	trachyte	greenish gray, slightly to moderately weathered, very weak, medium strong, contains broken core at depths of 13.90-14.11, 115.68-15.90. 20% core loss. 13.45-13.95. RQD is good.
16.45-19.45	trachyte	greenish gray, slightly to moderately weathered, weak to medium strong (R2-R3). Close joining, very good RQD.
19.45-22.45	trachyte	greenish gray, slightly to moderately weathered, weak to medium strong, close jointing, broken core at 19.69-19.80 and 21.66-22.40. RQD is poor.
22.45-25.00	trachyte	greenish gray, moderately weathered, weak(R2) broken cores at depths of 22.45-22.93 and 23.25-23.30 47% good core loss, RQD is very poor.

11.85-13.45	trachyte	greenish gray, slightly weathered, medium strong close joints. Very good RQD
13.45-16.45	trachyte	greenish gray, slightly to moderately weathered, very weak, medium strong, contains broken core at depths of 13.90-14.11, 115.68-15.90. 20% core loss. 13.45-13.95. RQD is good.
16.45-19.45	trachyte	greenish gray, slightly to moderately weathered, weak to medium strong (R2-R3). Close joining, very good RQD.
19.45-22.45	trachyte	greenish gray, slightly to moderately weathered, weak to medium strong, close jointing, broken core at 19.69-19.80 and 21.66-22.40. RQD is poor.
22.45-25.00	trachyte	greenish gray, moderately weathered, weak(R2) broken cores at depths of 22.45-22.93 and 23.25-23.30 47% good core loss, RQD is very poor.

BH-18

The top of the borehole has an elevation of 2637m.a.s.l. Unlike the other borehole consists of thick soil with thickness of 5.10mts. Significant depth for tunnel zone is 10-20mts, below the surface.

<u>Depth in meters</u> <u>below the surface</u>	<u>Rock</u>	<u>Description</u>
0.00 - 9.5	residual soil	
9.5-12.05	trachyte	reddish brown, yellowish brown, highly weathered, extremely weak to very weak (R0-R1), broken cores and poor RQD.
12.05-13.80	trachyte	reddish-brown, moderately weathered, close jointing, 26%core loss RQD is poor contains broken cores at 12.4-12.5 and 13.15-13.30
13.80-18.30	trachyte	reddish brown, moderately weathered, weak(R2⁺), close jointing, contains broken core at 15.57-15.64 and 17.79-15, core loss 7-10%, RQD is poor.

18.30-19.33	trachyte	▪ reddish brown, moderately weathered, weak(R2⁺), close jointing, RQD is very poor, broken core at depths of 18.53-18.72
19.33-20.00	trachyte	▪ reddish brown, weathered, moderately close jointing, contains broken core 19.33-19.60&19.79-19.90. Very poor RQD.

Generally, the tunnel zone consists of volcanic rocks, such as trachytic lava flow and various types of tuffs (mainly ^{un}welded) differing in grain size, degree of weathering, RQD value, and jointing condition. The tunnel zone passes mainly through unwelded tuff as compared to trachytic lava flows (see Fig. 4-3).

The northern part about 700 m from the northern portal is trachyte rock but in the boreholes 20 and 22 there are fine tuffs and lapilli tuffs respectively at the bottom part a little below the tunnel invert. In the central part, the tuff extends from 700-3800 about 3100 m and in the southern part from 3800-4800, which is 1000 m of trachytic lava flow.

There are two faults near northern portal at 1000 at borehole 2 and at the south central part 3900 at borehole 11. All the portal holes lie in trachytic lava flow. But the trachyte at the southern portal shows deep weathering. It has also lower RQD value than the northern and the central part trachytic lava flow.

The southern half of the tunnel has short slope length and sharply decreases in altitude towards Addis Ababa. The trachyte flow is thicker in this part of the tunnel. The pyroclastic material (tuff) gets thinner and disappears towards borehole 11. The upper part of this area consists predominantly of trachyte and some ^{un}welded tuff. The bottom part is mainly ^{welded} tuff (fig.4-3). There are five main deposits of tuff with few thin layers intercalated within trachyte flow. The main tuff layers are dipping towards north following the topography. The tuff layer at the bottom has the largest extent than the others. The tunnel in the central part mainly passes through this tuff units.

4.2. STRUCTURES

4.2.1 Regional Structures

Structural studies of central Ethiopia, where the Addis Ababa area is located, have been carried out by some researchers using satellite images, topographic map and aerial photo interpretations, and statistical analysis (Mohr, P., 1971, Tsegaye Abebe, et al, 1995, Tsegaye Abebe, 1995). Most of the detected lineaments are dominantly related to faults, open fractures, basins and morphological depressions, defining extensive fracture systems which correspond with the three main Rift system trends of the Horn of Africa: the Red Sea (NW-SE), Ethiopia Rift (NE-SW), and the Gulf of Aden (E-W) and the fourth one is N-S trending (usually called the Mozambique belt system, commonly associated with the Pan African basement rocks) (Tsegaye Abebe, et al, 1995).

There are a number of faults in Addis Ababa area, most of whose trend coincide with the main Ethiopian rift faults. (Kebede Tsehayu and Tadesse Haile Mariam, 1990).

The western Rift margin boundary is marked by the Addis Ababa-Ambo E-W trending fault, in which the Addis Ababa area is the down thrown side. The elevation and slope is higher in the Entoto. The Filwoha section becomes nearly flat up to Mojo, with an inclination of less than 30°, except for the topographic highs related to volcanic center of Wachacha, Yerer, Furi and scoraceous cones and crator lakes in Debre Zeit - Akaki area (Haile Selassie Girmay and Getaneh Assefa, 1989).

The E-W trending Entoto ridge is affected by tectonic events that could be related to the Ambo - Kesseme fault structure. The N-E trending Filwoha fault separates, to the north, the 7 Ma old basalt overlain by welded tuffs from strongly welded trachytic tuffs to the south (Morton et al., 1979).

For Early Miocene silicic volcanism in the Addis Ababa region, Morton et al., (1979) suggest an isolated center at Entoto, with eruption thinning both the Rift ward and plateau ward: this center was later strongly upfaulted as a NE-SW trending horst at the margin of the developing rift.

4.2.2 Local Structures

The proposed Entoto tunnel corridor has been affected by faults and joints. Flow bands which are related to lava flows occur in the trachytes. The closely spaced, tight flow bands which are commonly present in trachytic lava flows are defined by various color bands. In strike and dip they differ from place to place.

4.2.2.1 Faults

There are two major faults in the studied area that are recognized from aerial photo interpretation, geophysical surveys and borehole interpretations (Fig. 4-2). One fault trending NE-SW is traced at the northern part of the tunnel corridor, crossing the tunnel axis near borehole 2. The other fault which is also a tectonic contact between trachytic and basaltic lava flows crosses the tunnel axis and located at the southern part of the studied area near borehole 11. The E-W trending fault is characterized by exceptionally high broken cores, having the lowest RQD value.

I. Joint analysis from surface observation

To apply a suitable classification in a rock mechanics for tunneling, careful field work must provide basic data on the characteristics of discontinuities (joints) in rock mass (Bieniawski, 1976; Bosscher and Connell, 1988).

Preferred joint orientation, which can introduce anisotropy in permeability and strength, has been detected through independent and random measurements of joint orientation combined with statistical techniques. Detection of preferred orientation is of particular importance as it influences the rock mass property and can affect slope stability. Hence, the orientation of joints for the lithologic units was recorded. After the detection of preferred joint orientation, the data was analysed by computer with the help of Dips software (Hoek, et al., 1997).

Joint spacing was measured as the perpendicular distance between adjacent joints or the overlapping ends of joints of a single set. Measurement was done as close to the exposure face as possible to prevent bias of the measurement towards smaller spacing (Wheeler and Dixon, 1980). For an accurate estimation of joint spacing, large size exposure was chosen.

Condition of joints which includes opening or separation of joints (distance between joint surfaces), continuity and roughness of joints as well as gouge material was typically assessed. As joints give an indication of their relative age, identification of their thickness, and abundance have also been noted in the field. The continuity of joints influences the extent to which the rock material and the joints separately affect the behavior of the rock mass. For example, open continuous joints facilitate unrestricted inflow of groundwater.

projection is plotted . This has given an idea about a possible instability problem from different joint systems with given directions and amount of dip. Rose diagram shows the strike directions of joints whereas the stereographic projection indicates amount of dip and dip direction.

Rose diagram and stereoplotting of joints of selected boreholes that are combined in group is attached in Appendix A. They can represent northern, central and southern parts of the tunnel alignment. Therefore, borehole 6,19 and 12 represent the northern part, boreholes of ,7,9 and 25 the central part, while bore- holes 12,13,14,17 and 27 represent the southern part.

Each of these borehole data is interpreted from structural point of view as follows:

A. Northern part

Borehole 6: Eighty joint planes are plotted. The rose diagram show preferred orientation in N - S direction and secondary orientation in NE - SW direction. The stereoplotting indicates joint planes are dipping in the east direction. There are also joint planes dipping towards west at 20° - 35° .

Borehole 19: Contains 22 measured joint planes. The rose diagram shows N-S preferred orientation with other minor direction in NE-SW direction. The stereo- plotting indicates east and west dipping joints at higher angle and southeast dipping at about 15° - 50° .

Borehole 22: Has a measurement of 31 joint planes. The rose diagram displays preferred orientation of NE-SW direction. The other minor ones are in E-W and NW-SE direction. The stereoplotting shows southeast dipping joints at about 45° - 80° and others dipping north about 30° - 50° .

Bore holes 6,19 and 22 with 133 joints plane numbers are being plotted to represent the northern part. The rose diagram shows preferred N-S orientation. The stereoplotting indicates high concentration of contoured poles in the west direction with joint planes gently dipping to the east (see appendix A).

B. Central part

Borehole 7: In borehole 7 the numbers of joint planes are 128. The rose diagram displays orientations in ENE-WSW direction and NNW-SSE directions. The stereoplotting shows two clusters, one dipping to the south at shallow angles 2° - 15° and another dipping to the north at about 50° - 75° .

Borehole 9: In borehole 9 there are 63 joint planes, main direction being in E-W direction and others in all directions or not preferred orientation. In the stereo-plotting it is scattered all around the circle with concentration dipping in south and north directions.

Borehole 25: Borehole number 25 contains 107 joint planes. The rose diagrams show two main preferred orientations NE-SW and (E-W) directions. There are also other joint planes with random orientation. The stereoplotting has more joint planes dipping in southeast direction than other directions. There are few joints dipping at a small angle in northwest direction.

Central part boreholes are 7, 9, and 25 and consists of 298 joint poles. In the rose diagram there are various directions but major ones are E-W and NNW - SSE directions. The stereoplotting displays high density of contoured poles in the central part of the circle with about similar dip angles, indicating shallow joint planes but directions are being scattered all over the area.

C. Southern part

Borehole 12: In borehole 12 only 20 joint planes were measured. Rose diagram shows two main preferred orientations in N-S and NE-SW directions. There are other minor directions scattered randomly. In stereoplotting northwest dipping planes have more concentrations.

Borehole 13: It has 24 joint planes. The rose diagram shows main direction in NE-SW and another set shows preferred orientation in E-W. There is another minor orientation in other direction. The stereoplotting shows southeast and northeast dipping joint planes.

Borehole 14: Consists of 138 joint planes. It shows a main preferred orientation of N-S and secondary direction striking from 25° to 70° . The stereoplotting shows east and west dipping planes at higher angle.

Borehole 17: Borehole 17 contains 55 joint planes. Main direction is N-S direction and secondary ones in NE-SW direction. The stereoplotting displays E and W dipping joint planes at higher angle and southeast joint planes dipping at low to gentle angle.

Borehole 27: Borehole 27 comprises of 74 joint planes. In rose diagram they have many directions but N-S and NE-SW are more pronounced than the others. East and west steeply dipping joints are more clustered than others.

The southern part boreholes include 12,13,14,17 and 27. The number of joint poles is 299. Joints strike in N-S direction. The stereoplotting shows high concentration of poles in the east and west margins of the circle, which infers steeply dipping joints in east and west directions (see appendix A).

4.2.3. Significance of the structures for proposed Entoto tunnel

Tectonic structures are developed both in large and small scales in the rock mass. They have either direct or indirect effect on the design and cost of the tunnel. The presence, activity, orientation, intensity, and frequency of tectonic discontinuities have a strong influence on the final development of a stability problem as well as on leakage through the rock mass (Broch, 1985)

Therefore, joints occurring along the proposed axis of the tunnel, flow bands on the rock mass and faults can reduce the stability of the tunnel. Almost all the rocks in the tunnel corridor are affected by discontinuities including faults and joints. As mentioned earlier the presence and general condition of such discontinuities have significant effect on the permeability, shear strength, compressibility, and slope stability of the rock mass with respect to the direction of the axis of the tunnel (Cording and Mahar, 1977)

The low angle between the proposed tunnel axis and the gently dipping joints in the basalt may reduce the stability of the tunnel. The commonly sub-horizontal closely spaced flow banding (sheeting) in the trachyte may also reduce the stability of the tunnel.

The presence of strongly jointed rocks at the northern portal and central part of the tunnel axis may cause unstable condition. The variously oriented joints resulted in the decrease of the resistance to erosion ,and the tectonic slicing of rock gave rise to the easiest way to gravity movement and deposition as colluvium.

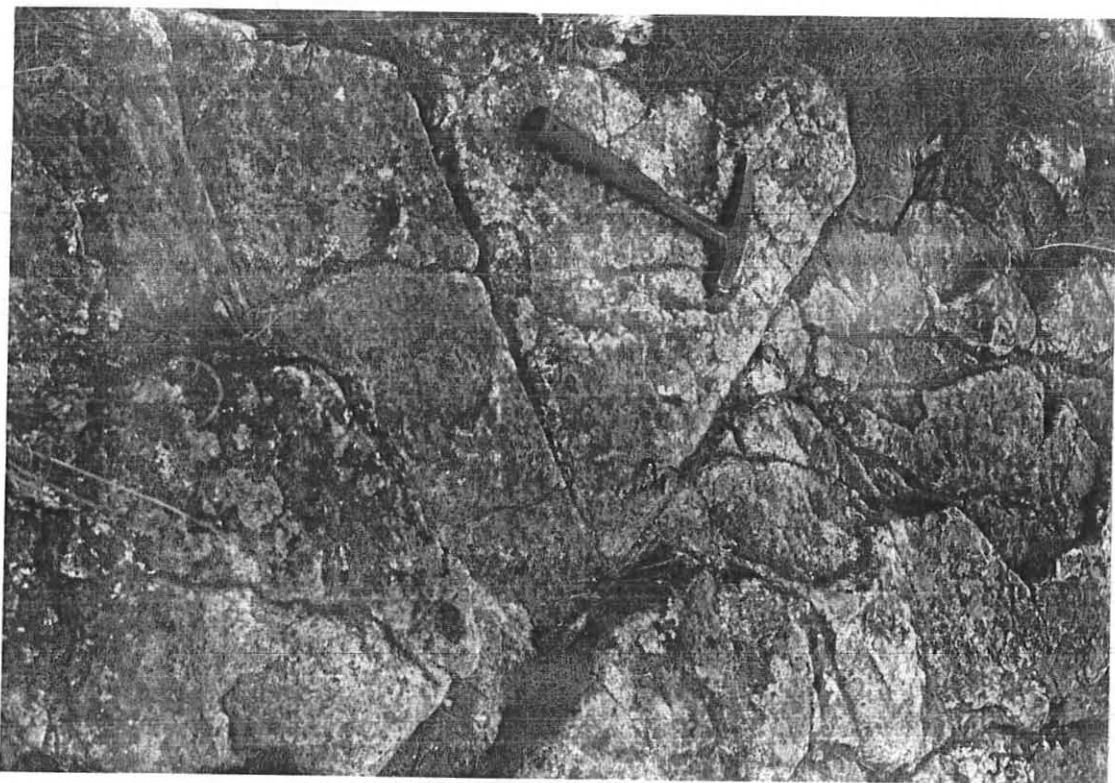


FIG. 4-4 Joint sets in the study area (photograph facing to the west).

**Table 4.1 Characteristics of sets of major joint concentrations
of Entoto tunnel area**

Section	JOINT SET	ORIENTATION (strike)	TIGHTNESS	SPACING
SECTION A	SET 1	N25°E-35°E	0-2mm wide/	extremely closely to widely spaced (5mm-1.5m)
	SET 2	N45°E-N55°E	0-5mm wide/	very closely to medium spaced (20mm - 400mm)
	SET 3	N60°W-N75°W	(0-20mm wide)	closely to very widely spaced (80mm-3m)
SECTION B	SET1	N30°E-N40°E	0-20mm wide)	extremely closely to medium spaced (10-600mm)
	SET 2	N60°E-N80°E	(0-10mm wide)	very closely to widely spaced (40-640mm)
	SET 3	N10°W-N20°W	(0-2mm)	Closely spaced (70-200mm)
SECTION C	SET 1	N30°E-N40°E	(0-20mm)	extremely closed to medium spaced (30-60 mm)
	SET 2	N40°E-N50°E	(2-5mm)	extremely closed to medium spaced (50-850mm)
	SET 3	N60°W-N80°W	(0-10mm)	very closely to widely spaced (60-180mm)
SECTION D	SET 1	N20°E-N25°E	(0-5mm)	extremely closely to widely spaced (20mm-2m)
	SET 2	N50°E-N70°E	(5-10m)	closely spaced to very widely spaced (70mm - 3m)
	SET 3	N10°W-N30°W	(0-20mm)	extremely closed to medium spaced (20-600mm)

Remark: With respect to tightness almost all joints range from tight to open.

5. ENGINEERING GEOLOGICAL AND GEOTECHNICAL PROPERTIES OF ROCKS.

5.1 Engineering Geological Mapping

Engineering geological mapping is a type of geological map which provides a generalized representation of all those components of geological environment of significance in land use planning, design, construction and maintenance as applied to civil and mining engineering (Anon, 1979).

In engineering geological mapping, the classification of rocks and soils are based on the principle that the physical or engineering geological properties of a rock, or soil, in its present state are dependent on the combined effects of mode of origin, subsequent diagenetic and tectonic history, and on weathering processes (Dearman, 1991). This approach makes it possible to determine the reasons not only for their spatial distribution as shown on the map, but also the reasons for the lithological and physical characteristics of the rocks and soils involved.

This is a basic principle of engineering geological mapping as or other geological mapping and implies not only the classification of individual rock samples but also the use of many individual rock samples, field observations and measurements to delineate uniform and continuous rock units (Anon, 1976).

A full description of a soil or rock for engineering purposes involves giving an appropriate geological name together with as much detailed information as can be gained from the examination of sample or exposure. The description may be systematic for a limited range of standard descriptive terms, and for ease of application should place the soil or rock in a limited number of groups with

definable properties, thus providing a good guide to engineering performance.

In unified classification system for both soil and rock units on engineering geological map each class represent a soil or rock unit with a narrow range of engineering properties (Dearman, 1991)..

Consequently, in this work, engineering geological description and classification was adopted, involving only:

1. A lithological classification,
2. A description of rock and soil material and mass and the assignment of descriptive names for rock and soil types.

There is an obvious need to make the classifications and descriptions simple so that they can be readily understood and applied, even by those with limited geological knowledge.

The classification in this work is based on both the field and laboratory results. The same lithological unit may be subdivided in different sub-units in an engineering geological map, if there are differences of weathering state, joint development, etc., which are influencing geotechnical property. However, concerning the studied area, the weathering state is not uniform to classify the area in different zones.

Geomorphological and hydrogeological conditions, land slips, borehole and field test information are recorded as much as possible on the map (Fig.4- 3).

slake durability test is suitable only for weak weathered rocks, there generally being too little effect on hard fresh rock. Since tuffs are generally weak slake durability test was conducted on such type of rocks. The result of point load tester gives the splitting resistance of the rock material which can be correlated only very roughly with uniaxial compression strength. Geophysical investigation and drilling data have also given valuable assistance in the determination of subsurface conditions, especially in areas in which the solid geology is poorly exposed. Seismic refraction has been used in measuring depths of overburden in the portal areas of tunnels, in locating faults, weathered zones, and in estimating rock quality.

Based on in situ and laboratory tests, rocks in the proposed tunnel area are classified into four zones. The strength of trachytic lava flow mainly ranges from medium strong to strong (R3-R4) but some samples of trachyte show weak strength, that could be due to joints and/or weathering of the rocks.

Unwelded tuffs in general fall between very weak and weak (R1-R2+), but the welded tuffs attain a strength of medium strong (R3). From the results of Schmidt hammer test performed in the field the strength of the basaltic lava flow range from 640-880 kg/cm². But highly jointed and scouraceous basalts have low strength.

Therefore, the basalt has higher unit weight and strength than other rock types in the study area. In hard rock moisture content is very low but in weak rock like tuffs some times moisture content may have a significant effect on strength.

For the correlations mentioned, there are exceptions which could be attributed to joint weathering that causes decrease of strength and/or unit weight. Other factors, which increase rock strength, are filling of joints, fractures and cavities by silica. When rocks are silicified they become strong.

5.2.1 Rock Classification Systems for Entoto tunnel

5.2.1.1 General

In estimating the preliminary costs and in the excavation of a tunnel, factors such as design of the route, cross-sections, construction time, and method of excavation play a crucial role. Hence, the classification of rock types in relation to their suitability for tunneling is a very important aspect and required the identification of rock masses with similar behavior from the engineering view point (Arogyaswamy, 1992).

A classification system for rock masses is essential to ensure understanding and communication among those concerned with a given tunneling project, such as the owner, the engineer, the contractor, the rock mechanics engineer and the engineering geologist (Bieniawski, 1978).

These aims should be fulfilled by ensuring that a classification system has the following attributes.

- a) is simple and understandable;
- b) each term is clear and the terminology used is widely acceptable;
- c) only the most significant properties of rock masses are included.
- d) is based on measurable parameters, which can be determined by relevant tests quickly and cheaply in the field.

- e) it is based on a rating system, which can weigh the relative importance of classification parameters.
- f) is general enough so that the same rock will possess the same classification, regardless of how it is being used.

In general, a rock mass classification has the following purposes in the tunneling application.

1. To divide a particular rock mass into groups of similar behavior;
2. To provide a basis for understanding the characteristics of each group;
3. To yield quantitative data for the design of tunnel support;
4. To provide a common basis of communication.

The need for a suitable classification in the field of rock mechanics has long been recognized and, in fact, numerous proposals have been made. Nevertheless, although some of the existing classifications have a considerable potential some are fully satisfactory in terms of the above pre-requisites (Bieniawski, 1979).

For example, the rock mass classification systems currently in use internationally in the field of tunneling are: Rock Load (Terzhagi, 1946), stand-Up Time (Lauffer, 1958), Rock Quality Designation or RQD (Deere et al, 1967), Rock Structure Rating or RSR (Wickham, et al, 1972), Rock Mass Rating-RMR or Geomechanics (Bieniawski, 1973), and Q-system (Barton et al, 1974) to mention. But, many of these classifications are based on rock material characteristics only and do not fully explain the property of rocks for tunneling purposes.

The two classification systems that appear to be in widest use on an international basis in tunneling and which do not rely on performance monitoring (though they can be used in conjunction with monitoring) are the RMR and Q-systems. For tunnel support selection purpose these two systems are therefore considered in the study.

Bieniawski's(1973) Geomechanics classification evolved from several earlier systems and has undergone several changes (1974,1975,1976,1979 and 1989) since its first introduction in 1973, Bieniawski (1989) rates the following six parameters in his RMR system: uniaxial compressive strength of rock material, RQD, spacing of joints, condition of joints, groundwater conditions and orientation of joints.

The common parameter used in both systems is RQD. Bieniawski also includes joint spacing and orientation, while the Q-system considers the number of joint sets. Orientation is included implicitly in the Q-system by classifying the joint roughness and alteration of only the most unfavorably oriented joint sets or discontinuities (Barton, 1988).

The very detailed treatment of joint roughness and alteration, perhaps the strongest feature of the Q-system, is not particularly emphasized in the RMR-Geomechanic classification. In addition, in the RMR system, rock stress is not used specifically as a parameter though it is apparently used when selecting support measures (Bieniawski, 1979)

In the Q-system, the ratio (unconfined compression strength/ major principal stress) is evaluated when treating rock stress problems. The rock burst

problem can be quite accurately predicted in hard rocks. The Q - system also accounts for loosening caused by shear zones and faults, and squeezing and swelling ground. (Barton, 1988).

According to one report of comparison between rock mass classification systems for tunneling, only the Q-system was found to be applicable to the worst conditions encountered during excavation. When considering stand-up time versus unsupported span. Bieniawski's system still seem to be conservative compared with the Q-system (Einstein et al., 1983).

The Q-system is more detailed than any of the other methods as regards the factors such as joint roughness (or degree of planarity), joint alteration (filling), and relative orientation. The use of joint roughness and joint alteration represents one of the strongest features of the method. It also seems to be a factor that is virtually ignored in the other classification schemes. For example, in the RMR system, although data for all joint set and discontinuities are collected, only the average data are incorporated in the numerical ratings. Furthermore, in the RMR it is impossible to separately vary the degree of joint roughness and the degree of infillings as obviously may occur in practice. The Q-system is essentially a weighting process, in which the positive and negative aspects of a rock mass are assessed.

Much of the problem in proposing a classification system is to select a set of parameters of greatest significance. It is believed that there is no single parameter or index, which can fully and quantitatively describe a jointed rock mass for tunneling purposes. Various factors have different significance and only if taken together can they describe satisfactorily a rock mass.

material is weak, the rock material properties will influence the behavior of the rock mass. The determination of the uniaxial compressive strength of rock material is a simple process for which standard techniques are available. Hence, point load test was applied for trachyte, basalt and tuff. The result is given in table 3-5.

- *Rock Quality Designation (RQD)*: is a measure of drill core quality as obtained from boreholes. The RQD is a quantitative index based on a modified core recovery procedure which "incorporates" only three pieces of hard, sound core which are 100mm or greater in length. Shorter lengths of core are ignored as they are considered to be due to close shearing, jointing or weathering in the rock mass (Bieniawski, 1979). RQD can vary from 0 to 100, but in actual case RQD values of 0 and 100 are uncommon. The average range of RQD for each borehole is given in (table 5-1).
- *Spacing and Orientation of joints*: are of paramount importance for the stability of structures in jointed rock mass. The presence and spacing as well as the dip and strike of joints reduce the strength of a rock. The data on spacing of joints was obtained from a joint survey discussed before. Considering joint spacing and joint orientation, there is a big variation in the proposed tunnel zone. Three joint sets and other joints, which are randomly oriented, are observed in the area (see table 5-1, and appendix E).
- *Condition of Joints*: includes separation of joints (distance between joint surfaces), continuity and roughness of joints as well as gouge material. Tight joints with rough surfaces and no gouge have a high strength. On the other hand, open continuous joints will facilitate unrestricted inflow of ground water. The continuity of the joints influences the extent which the rock materials and

Table 5-1 Ruir Along Tunnel Axis boreholes

Borehole name	en86-21	en86-19	en86-20	en86-22	en86-15	en86-2	en86-24
Tunnel zone depth	4 to 15	6 to 31	22 to 47	73 to 98	135 to 160	130 to 160	113 to 143
TRCD	40 to 45	45 to 50	25 to 30	40 to 45	70 to 75	45 to 50	70 to 75
2 joint spacing	c	c	mc	c	mc	c to mc	mc
3 joint condition	smooth	m rough	m smooth	sm to rough	rough	m smooth	m smooth
4 joint orientation	unfavourable	m unfavourable	unfavourable	unfavourable	unfavourable	unfavourable	favourable
5 ground water condition	well	well	unfavourable	well	well	well	well
6 strength	R2+	R2+ to R3	R1 to R2+ for tuff and R2+ for trachyte	R1 to R3 for trachyte R0 to R1 for tuff	R3 for trachyte R2(-) to R2+ for tuff	R0 to R2(-) for tuff R3 for welded tuff	R1 to R2(-) for tuff R3 for trachyte
Lithology	Trachyte	Trachyte	22.2 to 25.35 tuff 25.35 to 41.3 trachyte	3 to 80.94 trachyte 80.94 to 96 tuff	35 to 148 trachyte 148 to 152 tuff	mainly tuff with inter alation of welded tuff	26 to 140 trachyte the rest tuff
Remark		Broken cones seen	41.3 to 46.45 tuff 46.45 to 47.05 for trachyte		152 to 160 trachyte		

(Continued)

en86-25	en86-10	en86-27	en86-11	en86-12	en86-17	en86-16	en86-18
152 to 162	200 to 230	75 to 105	60 to 89	20 to 50	10 to 31	5 to 25	10 to 20
60 to 65	80 to 85	85 to 70	20 to 25	35 to 40	35 to 45	30 to 40	25 to 30
c to mc	w	c	c	c	c	c	c
rough	rough	m rough	m rough	rough	m rough	m rough	m rough
m. favourable	favourable	m. unfavourable	unfavourable	unfavourable	m. unfavourable	unfavourable	unfavourable
well	well	well	well	well	dry	dry	well
R4 for trachyte	R2(-) tuff	R3 for trachyte and R0 to R2(-) for tuff	R2 to R3 for trachyte	R2 to R3 for trachyte	R1 to R3 for trachyte	R1 to R3 for trachyte	R2+ trachyte
R2 to R3 for tuff	R0 to R2(-) for tuff	R3 for trachyte and R0 to R2(-) for tuff					
152 to 173 trachyte	tuff	75 to 98 trachyte	trachyte	trachyte	trachyte	trachyte	trachyte
173 to 172 tuff	ash alternates	98 to 105 tuff					

Table 5 -2a. RMR Classification for Entoto Tunnel

sample No.	Rock Type	Depth (m)	RMR Value						
			strength	RQD	Spacing of Discontinuity	Conditions of Discontinuity	Ground Water	Adjustment for Joint	Total
En96-21	Trachyte	4-15	4	8	8	25	7	-5	47
En96-19	Trachyte	6-31	4	8	8	25	7	-5	47
En96-20	Tuff and Trachyte	22-47	2	8	10	25	7	-5	47
En96-22	Tuff and Trachyte	73-98	2	8	8	20	7	-5	40
En96-15	Tuff and Trachyte	135-160	2	13	10	30	7	-5	57
En96-2	Tuff and Trachyte	130-160	2	8	10	25	7	-5	47
En96-24	Tuff and Trachyte	113-143	4	13	10	25	7	-2	57
En96-25	Tuff and Trachyte	152-182	2	13	10	30	7	-5	55
En96-26	Tuff and Trachyte	220-250	2	8	10	25	7	-5	47
En96-10	Tuff	200-230	1	17	15	25	7	-2	63
En96-27	Tuff and Trachyte	75-105	2	13	8	25	7	-5	50
En96-11	Trachyte	60-89	4	3	8	25	7	-10	37
En96-12	Trachyte	20-25	4	8	8	30	7	-5	52
En96-17	Trachyte	10-31	4	8	8	25	15	-5	55
En96-16	Trachyte	5-25	4	8	8	25	15	-5	55
En96-18	Trachyte	10-20	4	8	8	25	7	-5	47

Note 2: Table of appendix D, gives a description of "FAIR" or the conditions assumed where the tunnels to be driven against the dip of a set of joints dipping between 45-90 using the description for "Tunnels and Mines" in table of appendix D gives an adjustment rating of -5.

Even though, the RMR classification falls on the FAIR ROCK boundary some modifications have to be made for highly fractured and extremely weak and friable tuffs. In some boreholes hard trachyte was encountered. For this reason recommendations for GOOD and POOR conditions of rocks should be considered due to the high variability of the Entoto tunnel zone rocks.

5.2.1.3. The Rock Mass Quality Index (Q-System)

The concept of rock mass quality (Q) system which is introduced by Barton et al., (1974) is used for determination of rock mass characteristics and tunnel support requirements. This method is considered to be the one that provides the best approximation to the actual conditions to be encountered in the tunnel. The numerical values of Q ranges from 0.001 in the case of very poor, very weak rocks to 1,000 for very competent massive rocks (Barton et al., 1974).

The six parameters taken into consideration for classifying the rock mass quality are the following:

1. By the RQD system the intensity of the jointing is estimated
2. The number of joint sets (J_n) present. It is often affected by foliation, schistosity, cleavage, bedding, etc.
3. The roughness of the most unfavorable joint faces (J_r).
4. The extent of alteration and the nature of the filling in the unfavorable joint (J_a). This examination provides an approximation of the shear strength of the rock mass.
5. The estimate of joint water reduction factor (J_w) which again affects the shear strength.
6. The stress reduction factor (SRF) which accounts for the loading on the tunnel caused either by the dislodging of rocks containing clay parting or due to unfavourable stress strength ratios in the case of massive rock.

Taking into account the sum total of the above mentioned observations, the rock mass quality classification system is given by:

$$Q = (RQD/J_n) * (J_r/J_a) * (J_w/SRF)$$

The three pairs of ratios (RQD/J_n , J_r/J_a and J_w/SRF) represent block size, minimum interlock shear strength, and active stress respectively. These are fundamental geotechnical parameters (Barton, 1989). In explaining the meaning of the paired parameters used to determine the value of Q , Barton et al., (1974) offered the following comments which are helpful in the estimation of ratings in the study.

1. The first quotient, i.e., (RQD/J_n) represents the over all structure of the rock mass, and it happens to be a crude measure of the relative block size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400.
2. The second quotient (J_r/J_a) represents the roughness and the degree of alteration of the joint walls or filling materials. This quotient is high for rough, unaltered joints in direct contact. But when rocks have thin clay mineral coatings and fillings with no rock wall contact, the conditions are extremely unfavorable to tunnel stability.
3. The third quotient (J_w/SRF) consists of two stress parameters. The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may in addition cause softening and possible outwash in the case of clay filled joints. The parameter SRF is a measure of: (1) loosening load in the case of excavation through shear zones and clay bearing rock, (2) rock stress in competent rock, (3) squeezing or swelling loads in plastic incompetent rock.

Undoubtedly, there are several other parameters, which could be added to improve the accuracy of the classification system. One of these would be joint orientation. Although many case records included the necessary information on

structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected. The parameters J_n , J_r and J_a appear to play a more important general role than orientation, because the number of joint sets determines the degree of freedom for block movement and the frictional and dilational characteristics can vary more than the down-dip gravitational component of unfavorably oriented joints. If joint orientation had been included, the classification system would be less general, and its essential simplicity lost.

The detailed rock mass descriptions of the six individual parameters and their numerical ratings are given in table 5-3 and table 5-4. The Tunneling Quality Index Q , for a rock mass of the proposed tunnel was obtained from field observation and borehole logs

When referring ratings given by Barton, et al., (1974), the rating value for three joint sets plus random is 12 (table 1, Appendix E). But for the portal areas it becomes $J_n \times 2$ (see table 5-3). The joints in the study area are rough or irregular (some are smooth, planar and highly weathered with surface staining), whose rating value given by Barton, et al (1974) of the rock mass is 1.5.

The numerical value of RQD is used directly in the calculation of Q . The RQD values ranging from 20% to 85% . For the rock masses in most part of the tunnel zone, the average value of RQD is variable. It ranges from VERY POOR to GOOD quality. Based on its value the tunnel axis can be classified into four zones. The Q - System classification is made for each zone separately. But the average Q value of three of the four zones is similar. Hence, the recommended supporting method will be similar for these zones (table 5-3 and table 5-4).

Considering joint alteration number, the rock mass in the study area is highlyweathered having silty or sandy clay coatings whose rating is 3.0. From the packer permeability test results, there is large inflow or high pressure in competent rock mass with unfilled joints. The rating designed for such rock mass is 0.5. Although it needs to measure the stress condition of the area, from the topographic condition, it is estimated to have medium stress, with the rating value of 1.0.

Therefore, by applying the rating value of these parameters the total Q value calculated for each zone ranges from 0.1 to 4 (see table 5-3 and table 5-4) .

The recommended supporting methods are discussed in the next chapter.

5.3. GROUNDWATER CONDITIONS

5.3.1. The effect of groundwater in tunneling

Groundwater frequently provides some of the most difficult problem during tunneling operation. The problem may be directly or indirectly caused by the percolation of water towards the tunnel (Bell, 1993). The greatest groundwater hazard in underground work is the presence of unexpected water bearing zone, depending on the location of hydrological boundaries. Obviously, the location of the water table, and its possible fluctuations are the major considerations.

Table 5-4:- Rock mass classification with Q system along the tunnel axis boreholes.

	Zone 2				Zone 1							Zone 4	Zone 3			
Borehole number	21	19	20	22	15	2	24	25	26	10	27	11	12	17	16	18
RQD/Jn	1.77	1.97	1.14	1.77	6.04	5.62	6.04	5.62	3.95	6.87	5.62	1.87	1.50	1.66	1.45	1.14
Jn/Ja	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Jw/SRF	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Q	0.44	0.49	0.28	0.44	1.51	1.40	.51	1.40	.98	1.72	1.40	0.46	0.39	0.41	0.36	0.28
Span/ESR	2.34	2.34	2.34	2.34	2.34	2.34	.34	2.34	.34	2.34	2.34	2.34	2.34	2.34	2.34	2.34
Q _{aver}	0.41				1.42							0.46	0.36			

As a general statement, which is valid for this type of geological environment, it is possible to say that the main aquifers that could be found in such a peculiar area are:

- elluvial and alluvial sediments,
- weathered volcanic rocks,
- permeable or pervious layer between lava flows,
- unconsolidated tuffs and ashes,
- joints and fractures in the volcanic rocks,
- shear zones and major tectonic discontinuities.

The main groundwater reserve of the region is localized in the fractured volcanic rocks(Vernier et al., 1985; Kebede Tsehayu and Tadesse Haile Mariam, 1990). Several different flows with unconformity favored the occurrence of multi-layer aquifer system. Several contact springs concentrated on the northern part of the city showing multi-layer volcanics.

The groundwater flow of the region is mainly governed by the present day land form of Addis Ababa which in turn, is influenced by geological processes especially faulting and denudation. The layers of lava flows are dipping nearly southwards and so do the unconformity surfaces. For this reason the general groundwater flow is from the north to the south. The flow of eastern side is from east to west and that of the western part is from west to east. The main groundwater reserve exists to the south of the city following the lower topography.

5.3.3. Groundwater along the tunnel alignment

The data used for groundwater studies relating to aquifer characteristics, groundwater head variation and groundwater flow were obtained from in-situ packer permeability tests and piezometric measurements that were surveyed by AAWSA and geophysical exploration done by EIGS staffs for the project.

5.3.3.1. Groundwater flow and piezometry

Groundwater table monitoring was carried out by continuous water level recorder to analyze seasonal fluctuation of groundwater in boreholes drilled at Entoto tunnel. The nature of the aquifer is also analyzed from the geophysical survey results.

The aquifer in the studied area consists of volcanic rocks. This area is generally characterized by highly weathered and slightly fractured to fresh trachytic lava flows, tuffs and basaltic lava flows. Permeability is higher in the intensely weathered and fractured zones. The massive rocks are virtually impervious.

Permeability of trachytic lava flow is higher than that of the unwelded tuff. In trachyte the average value is 10^{-4} cm/s. Whereas in tuff the dominant value is 10^{-5} cm/s. But in some zones where there is intensive fracturing of rocks, the values are higher than this (table 3-4).

The data on groundwater head measurements shows that at the portals and the central part of the tunnel axis, minimum and maximum water levels are at 25.25 m and 100 m respectively above the tunnel axis (Fig. 5-1).

5.4. MASS MOVEMENT

Stability of slopes is a critical factor in the excavation of tunnels. Underground excavations cause subsidence in overlying rock and occasionally continuing gradual surface deformations (Kelly and Mares, 1993). The intensity and extent of instability depends on the physico-mechanical properties of the rocks, strike and dips of beds; the shape, size and depth of excavation, the rate and the method used for driving and supporting the slope.

Instability of a slope is the result of gravitational movement of rock mass induced by a disturbance of the equilibrium in a slope. Slope movements are controlled by the geological structures, the hydrogeological regimen and any changes in that regime, the geomorphological history and human activity. These can, in turn be caused by various factors, such as changes in slope inclination and height, changes in water content, rock weathering or seismic activity. Therefore, the stability of adjacent areas must be assessed including likely surface deformations and changes in the hydrogeological regimen.

Mass movements can occur in terrains ranging from vertical cliffs to slopes as gentle as one or two degrees. Having a complex morphological feature, the Addis Ababa region has been an active zone of sliding, even though the degree of movement varies from place to place. The northern morphological unit (with a slope of 30-35 degrees) is characterized by rock slides (Lulseged, 1990). Scree deposits (debris) soil and rock materials are seen in the northwestern part of the city at the foot of steps of Entoto ridge. The variously oriented joints cutting the rocks resulted in the decrease in the resistance to erosion and the tectonic slicing of the rocks gave rise to the easiest way to gravity movement.

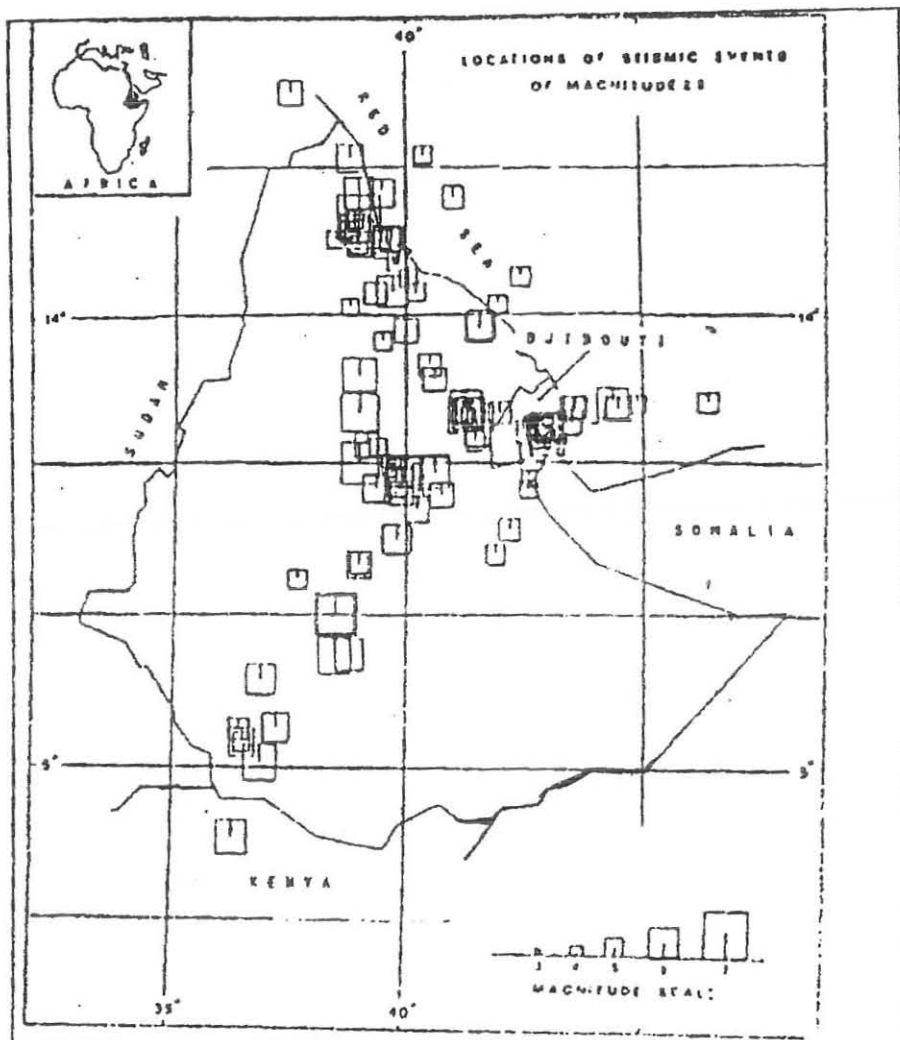


Fig.5-2 - Epicenter location of earthquakes with magnitude equal to or greater than 5 (Pierre Gouin, 1976).

The work by the same author indicates that the seismicity of Addis Ababa and its surroundings, by virtue of its proximity to the East African Rift (EAR) system, is related to the overall process of rifting and extension. From seismic risk map of Ethiopia, Addis Ababa lies in the second risk zone (see Fig. 5-3).

According to historical seismic occurrences, and interpretations worked by Laike Mariam Asfaw (1990), tremors have been reported in Addis Ababa since its foundation nearly a hundred years ago. For example, an earthquake that occurred in 1906, south of Addis Ababa, caused panic in the city. However, it was noted at that time that the damage due to this earthquake was light mainly because the Ethiopian mud houses withstand strong shaking (Kebede Tsehayu and Tadesse Haile Mariam, 1990).

The main centers of activity in other directions, notably to the NE, lie farther away. Although these centers are relatively remote, the strongest shakings in Addis Ababa, in recent times were caused by earthquakes originating in the region. The 1961 and 1977 earthquakes that occurred about 200 km northeast of Addis Ababa (near the western escarpment) were strongly felt in the city.

The intensities felt in the different parts of the city vary. For example, the 1961 Karakore earthquake, which had a magnitude of 6.7 was felt in Addis and caused light damage to buildings and other structures. But the Abware area had consistently felt the strongest shaking than most other parts of Addis Ababa for it lies on the Filwoha fault.

On earthquake shook the city of Addis Ababa, on 28 July 1979, the tremor was felt more strongly in the southern subrubs of the city, and at Kaliti and Sebeta an intensity as high as 6 was reached. Some regions notably Entoto, the Southwest, Yeka, Abware and Filwoha districts felt the tremor more strongly than others. This might be due to topographic difference and the effect of faulting. Therefore, the intensities felt in different parts of the city reported were interpreted in terms of topographic, geologic and tectonic formations.

The Entoto area is the oldest part of the city, where in the past been reporting relatively high intensities following earthquakes that occurred outside Addis Ababa. It is a well known phenomenon that topography plays an important role in amplifying seismic motion and the high intensity felt in this district is, therefore, considered to be owing to the topographic conditions.

Hence, considering the seismicity and earthquake risk of Addis Ababa and its surroundings, it is better to take an appropriate hazard mitigation measures. This can possibly be done by designing the lining that can resist earthquakes, especially at the localities, which are most vulnerable.

6. EXCAVATION AND SUPPORTING METHODS

A careful design of water tunnel requires the use of engineering judgement with a clear evaluation of the geotechnical and insitu stress conditions. There are a number of theoretical analysis that should lead to a safe design only if the local conditions and local inherent changes are properly evaluated and understood (Broch et al., 1987). Based on the evaluation of the ground conditions and the rock mass classification, selected excavation and supporting methods have been recommended.

6.1. EXCAVATION METHOD

Excavation of a tunnel is a very complicated and costly process. The method and mechanics to be adopted for excavation certainly depend upon the nature and types of rocks present along the excavation route, which can be known only through geological investigation. This was one of the main objectives of the engineering geological investigation.

Some specifications from the geological investigation allowed to select the method of excavation depending on the changes in the rock mass properties. However, tunnel excavation in volcanic terrain is very difficult due to unpredictable nature of its property so it needs to define precisely what would be encountered during excavation. (Burgess and Knights, 1994).

When considering the proposed site, the rock succession in the tunnel is heterogeneous. It is represented by intercalation of volcanic rocks (trachytes and tuffs) which are weathered and fractured to various degrees. At places, indurated

clays are found sandwiched between the volcanic rocks. In addition, weak zones associated with faults are determined; discontinuities such as jointing properties are observed and analyzed for the purpose of rock mass classification.

Eventhough, on the basis of the RMR classification and rock mass quality systems the materials fall on the FAIR ROCK boundary, some modifications have to be made for highly fractured and extremely weak and friable tuffs. Near boreholes 2 and 11 at the fault zone, the condition is very poor. On the other hand, at some boreholes hard trachytes were encountered. For this reason recommendations for GOOD and POOR conditions of rocks are considered due to the high variability of the Entoto tunnel zone rocks.

Mechanical cutting using moles or blasting is used in hard rock tunneling (Bell, 1995; Kumar, 1983). Excavation using rock blasting methods involve exploding suitable quantities or right type of explosive usually followed by a mucking period during which the blasted fragments are removed. Tunnel boring machine is more sensitive to changes in rock properties than conventional drilling and blasting techniques. Consequently, their use in rock masses which have not been thoroughly investigated involves a high risk. Apart from ground stability and support, the most important economic factors in machine tunneling in hard rocks are cutter costs and penetration rate.

Hence, depending on the RMR rating and Q value of the rock mass in the area, the excavation technique recommended is by drill and blast, either as full

face scale blast or as a crack blasting (see tables 6-1 and 6-2). However, full face driving should be used with caution because the rocks in the tunnel zone are geotechnically variable.

Top heading and bench with 1.5-3 m advances in top heading is necessary. The tunnel can be excavated from portals, using drill and blast and direct hoe excavation. The majority of the tunnel will be excavated in hard grade R3/R4 that can be easily excavated although requiring blasting.

6.2. SUPPORTING METHOD AND LINING DESIGN

Rock mass quality, together with the support pressure and the dimensions, and purpose of the underground excavation, are used to estimate the type of suitable support (Bell, 1995).

The proposed 4.8 km Entoto tunnel is designed to the horseshoe cross section of 3.75 m width by 3.3 m height. Its invert elevation is 2640 m.a.s.l. at upstream portal and elevation 2631 m.a.s.l. at down stream portal, after reviewing the upstream portal site it has been recommended that the invert be lowered to 2635 m.a.s.l.

The general design criteria for the pressure tunnel is based on the evaluation of the rock conditions and the correct assessment of rock mass behavior. Hence, the following three foremost fundamental aspects are properly considered in

connection with the geological and geotechnical environment of the proposed area: stability of the tunnel walls, leakage due to high permeability of the rock mass, and hydraulic fracture of the surrounding rock mass.

1. Stability Conditions

Tunnels with no lining are normally accepted where rock is self-supporting and no possible rock loosening is expected due to internal pressure or to action of water, the rock being able to withstand all pressure conditions. The same criteria are applied for a shotcrete lining, as long as the deformation of the rock mass does not induce major cracking of the shotcrete layers (Broch, et .al 1987).

Following the aforementioned criteria, shotcrete can be used as a lining in sector of very jointed and weathered rocks to avoid progressive erosion of the rock; in zones of fractured hard rock for controlling the falling of small rock bolts and in order to prevent a continuous loosening of the mass. In disturbed zones, where the RQD value of the rock mass falls in the VERY POOR class, shotcrete lining with a certain thickness is used.

2. Water Leakage

Most of the serious difficulties encountered during tunneling operations are directly or indirectly caused by the percolation of water towards the tunnel and most of the techniques for improving groundwater conditions are directed towards its control. This may be achieved by using drainage, compressed air, grouting, or

freezing techniques (Bell, 1995).

Hence, design criteria consider, with respect to water leakage, the imperviousness of a water tunnel in the rock mass itself. Unless an impervious lining is placed in areas of high permeability or the permeability of the rock mass is reduced by means of grouting, there will always be the possibility of having water leakage into the rock mass that could reach the ground surface. As the permeability of rock is normally negligible, it is the jointing and the faulting of the rock mass and the type and amount of joint infilling material, that are of importance when an area is evaluated for leakage.

When assessing the expected water leakage conditions of the proposed tunnel, in addition to the permeability of the rock mass, it was essential to consider the location of the water table with respect to the tunnel internal water pressure head. The average permeability of the rock mass at the proposed tunnel is $> 10^{-5}$ m/s. and the water table is above the tunnel axis. Whenever, tunnels enter such saturated bodies of rocks below the surface, water is likely to flood the excavation. Therefore, a considerable grouting program can be implemented to reduce the permeability of the rock mass around the tunnel, where the degree of fracturing is very high.

3. Hydraulic Fracturing

Hydraulic fracture of a rock mass surrounding a water tunnel occurs when the tensile stress, induced by the water pressure, exceeds the confining stresses within the rock and causes rupturing of the mass, either along pre-existing

Table 6.1 Tunnel Support condition using RMR

Tunnel Conditions	Rock Condition	Tunnel Support.
Good	Hard trachyte	Rock Bolts
Fair	Fairly Competent tuff and highly jointed trachyte	Rock bolts, Shotcrete or Steelsets and shotcrete
Poor	Extremely and Very weak, friable tuff, and in some cases highly fractured weathered trachytes	Steelsets, Spilling and shotcrete.

Table 6-2- Support Recommendation using the Q - system.

Zone	Support category	Q - value	De=(SPAN/SR F)	Tunnel support
1 (Central)	21	2 - 1 1.42 (poor)	2.34	Rock Bolts and shotcrete ; a systematic bolting (untensioned , grouted) (at 1m spacing) with a 50 - 90mm reinforced steel fiber (mesh).
2 (Northern portal)	25	0.41 (very poor)	2.34	untensioned , fully grouted systematic bolting (at 1m spacing) and shotcreted with reinforced mesh 50mm.
3 (Southern Portal)	25	0.36 (very poor)	2.34	untensioned , fully grouted systematic bolting (at 1m spacing) and shotcreted with reinforced mesh 50mm.
4 (fault zone)	25	0.46 (very poor)	2.34	untensioned , fully grouted systematic bolting (at 1m spacing) and shotcreted with reinforced mesh 50mm.

between categories, additional classification system is used to rationalize the bolt spacing and shotcrete on concrete arch thickness for each category. This compromises solution uses tailored to fit the case records giving detailed dimensions of bolt patterns and shotcrete or concrete linings used in volcanic terrain of Nairobi Water Supply Project (Burgess and Knights, 1994). The simple design concepts for rationalizing the support recommendations are given in Appendix F. Based on the value of "Q", classes of support were designed to suit the expected site conditions. The classes were based on the use of shotcrete as permanent support. (table 6-2).

Small variations in support methods also occur in each category and are due to rock mass differences, since a given value of Q is not unique, but a combination of several variables. In order to separate the more important variations in support practice, the additional factors (RQD)/Jn and Jr/Ja should be evaluated in addition to the overall quality Q. Two excavations having the same rock mass quality Q, may in one case be bolted and in another only shotcreted (Cording and Mahar, 1978).

The conditional factor RQD/ Jn describing block size will normally separate these two cases. For instant, rock masses with RQD/ Jn values larger than 10 will tend to be massive to blocky requiring only bolting, while values less than 10 are likely to represent blocky and jointed rock which can often be adequately treated with shotcrete (Barton. et al, 1974).

The conditional factor Jr/ Ja describing inter block shear strength may play an important role. In some cases the equivalent dimension (De) which is equal to

7. CONCLUSION AND RECOMMENDATION

Geological mapping of the proposed Entoto tunnel corridor indicated that the area is mainly composed of trachytic lava flow and tuff; basaltic lava flow and Quaternary sediments as minor outcrops. These Entoto silicic rocks are associated with Alaji formation, whose volcanism ended in Late Oligocene - early Miocene times.

The rocks of the studied area have been affected by discontinuities such as joints and faults. Primary flow band developed mainly in trachytes.

Entoto ridge, through which the tunnel passes, is located along the normal faults of the E-W trending Ambo - Kessam lineaments, which determines the orientation of some discontinuities in the area. The orientation of NE-SW trending joints and the faults crossing the tunnel axis near the northern portal, coincide with the Main Ethiopian Rift. There are also other structures which trend in the N-S direction.

The major intersecting faults that were determined by aerial photo interpretation, geophysical investigation and bore hole data have significant effect on the fracturing of the rock mass in the tunnel zone. There is an increase in the intensity of jointing near faults indicating that they are broadly synchronous.

Joints occurring parallel to the proposed axis of the tunnel, sub-horizontal joints, and the weakness of the rocks near and at faults can reduce the stability of the tunnel. Eventhough the chemistry of the groundwater at the proposed area has no significant effect on the tunnel support, the permeability of the rock mass and the presence of the water table above the tunnel zone is a problem for the tunnel work.

Consideration of the rock mass properties indicated that the most preferable tunneling excavation method are drilling and blasting. For hard, fractured rock materials, the principal means of support used can be bolting , and shotcrete. The support and excavation methods are selected using the RMR and Q- System classifications.

The rock mass of the tunnel zone is predicted to have a RMR value falling in the FAIR ROCK class. But with some modification locally, recommendation is given also for GOOD and POOR conditions. The Q- value is also between 0.4 and 4. Two different standard primary support classes were therefore need to be considered.

To fulfill the requirements for a smooth lining for the proposed water tunnel, a lining of concrete is recommended. In addition some parts of the tunnel are located in faulted zone and permeable rock mass conditions. Grouting the lining needs to be designed to resist full hydrostatic (leakage) and geostatic loading. Preliminary calculations indicate that a 50-90 mm thick concrete lining would be sufficient for the tunnel of 3.75 m finished diameter (appendix G).

It should be recognized that, at present, many of the index properties in a rock mass are difficult to assess fully at the exploratory stage. Moreover, it is even more difficult to predict the ground behavior in volcanic terrain based on the index properties particularly during exploratory phase. Besides, when the drilling is poorly executed and hole alignment is bad, the degree of over break and need for supporting may increase considerably.

Therefore, the recommend excavation and supporting method based on the evaluation of the ground conditions and the rock mass classifications may

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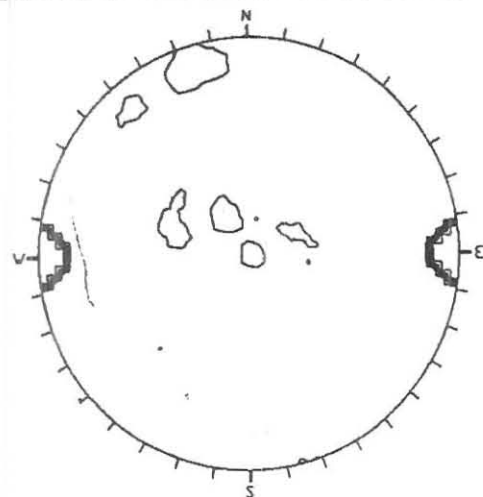
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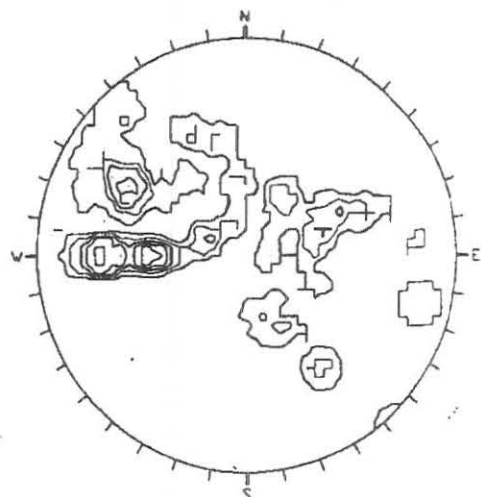
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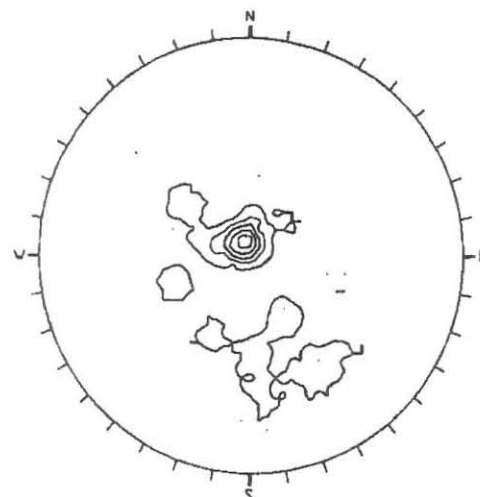
Appendix A: Figures showing rose diagram and stereo plotting of Selected boreholes



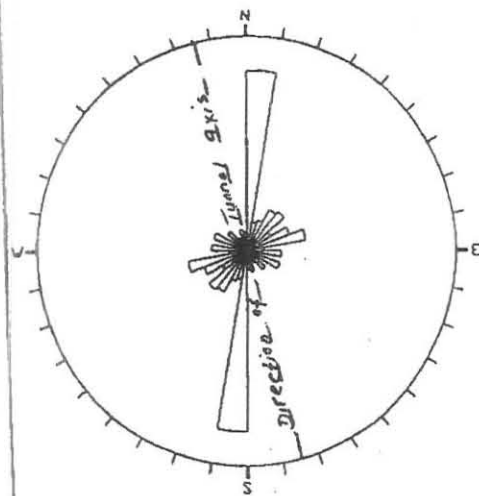
EN-SOEN



EN-NRCN

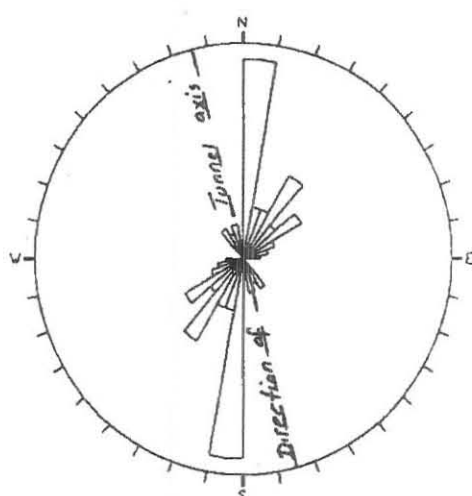


EN-CRCN



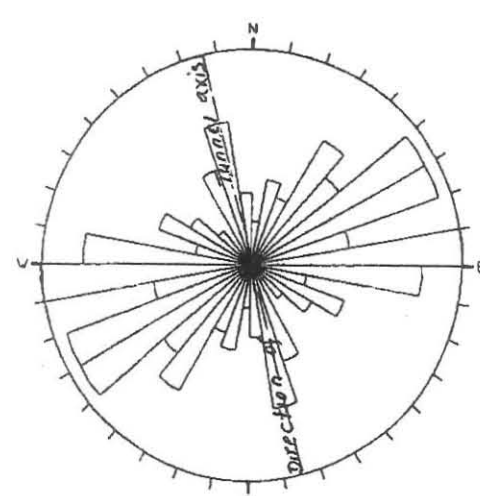
EN-SORS

SOUTHERN PART
NO. OF POLES 299



EN-NRRS

NORTHERN PART
NO. OF POLES 133



EN-CRRS

CENTRAL PART
NO. OF POLES 298

SOUTHERN PART OF ENTOTO
STEREOPLOTHING AND ROSE DIAGRAM.

NORTHERN PART OF ENTOTO
STEREOPLOTHING AND ROSE DIAGRAM.

CENTRAL PART OF ENTOTO
STEREOPLOTHING AND ROSE DIAGRAM.

Appendix B: Chemistry of the groundwater taken from some boreholes along the tunnel axis
(Source : AAWSA).

No.BH2.	Physic Chemical Parameters	Value
1	Temperature°C	-
2	Turbidity (F.T.U)	-
3	Colour (Pt.Co.Units)	-
4	Odour	-
5	Taste	-
6	PH	12.3
7	Total Dissolved solids (mg/l)	115
8	Conductivity (us/cm)	167
9	Total Alkalinity CaCo ₂	46
10	Total Hardness as CaCo ₃	32
11	Carbon Dioxides as CaCo ₃	30
12	Magnesium Hardness as CaCo ₃	2
13	Carbon Dioxides as Co ₂	Nil
14	Oxygeodissolved	5.3
15	Nitrate as No ₃	0.6
16	Nitrate as No ₂	Nil
17	Ammonia as NH ₃	Nil
18	Iron as Fe	Nil
19	Manganese as Mn	Nil
20	Chloride as Cl	5.0
21	Fluoride as F	0.06
22	Hexavalent Chromium as Cr	Nil
23	Copper as Cu	
24	Phosphate as Po ₄	0.826
25	Silica as SiO ₂	22.4
26	Sulfate as So ₄	11.7
27	Saturation Index (SI)	

Appendix B(Continued)

No.BH27	Physic Chemical Paremeters	Value
1	Temprature°C	-
2	Turbidity (F.T.U)	-
3	Colour (Pt.Co.Units)	-
4	Odour	-
5	Taste	-
6	PH	10.8
7	Total Dissolved solids (mg/l)	67
8	Conductivity (us/cm)	97
9	Total Alkalimity CaCO_2	30
10	Total Hardness as CaCO_3	104
11	Carbon Dioxides as CaCO_3	40
12	Magnesium Hardness as CaCO_3	64
13	Carbon Dioxides as CO_2	
14	Oxygeodissolved	6.6
15	Naitrate as NO_3	4.5
16	Nitrate as NO_2	Nil
17	Ammonia as NH_3	Nil
18	Iron as Fe	0.21
19	Mangansse as Mn	Nil
20	Chloride as Cl	10.0
21	Fluoride as F	0.19
22	Hexavalent Chromium as Cr	Nil
23	Copper as Cu	
24	Phosphate as PO_4	0.077
25	Silica as SiO_2	26.6
26	Sulfate as SO_4	8.6
27	Saturaction Index (SI)	

Appendix B(Continued)

No.BH12	Physic Chemical Paremeters	Value
1	Temprature °C	-
2	Turbidity (F.T.U)	-
3	Colour (Pt.Co.Units)	-
4	Odour	-
5	Taste	-
6	PH	11.9
7	Total Dissolved solids (mg/l)	853
8	Conductivity (us/cm)	1244
9	Total Alkalimity CaCO_2	252
10	Total Hardness as CaCO_3	120
11	Carbon Dioxides as CaCO_3	100
12	Magnessium Hardness as CaCO_3	20
13	Carbon Dioxides as CO_2	Nil
14	Oxygeodissolved	6.2
15	Naitrate as NO_3	1
16	Nitrate as NO_2	
17	Ammonia as NH_3	1.153
18	Iron as Fe	0.22
19	Mangansse as Mn	Nil
20	Chloride as Cl	4.5
21	Fluoride as F	0.35
22	Hexavalent Chromium as Cr	Nil
23	Copper as Cu	0.5
24	Phosphate as PO_4	0.086
25	Silica as SiO_2	16.7
26	Sulfate as SO_4	9.5
27	Saturaction Index (SI)	

Appendix C: Field estimates of uniaxial compressive strength (After Hoek, et al, 1997).

Grade*	Term	Uniaxial comp. strength (MPa)	Point load index (MPa)	Field estimate of strength	Examples**
R6	Extremely strong	> 250	>10	Rock material only chipped under repeated hammer blows, rings when struck	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100-250	4-10	Requires many blows of a geological hammer to break intact rock specimens	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
R4	Strong	50-100	2-4	Hand held specimens broken by a single blow of geological hammer	Limestone, marble, phyllite, sandstone, schist, shale
R3	Medium strong	25-50	1-2	Firm blow with geological pick indents rock to 5 mm, knife just scrapes surface	Claystone, coal, concrete, schist, shale, siltstone
R2	Weak	5-25	***	Knife cuts material but too hard to shape into triaxial specimens	Chalk, rocksalt, potash
R1	Very weak	1-5	***	Material crumbles under firm blows of geological pick, can be shaped with knife	Highly weathered or altered rock
R0	Extremely weak	0.25-1	***	Indented by thumbnail	Clay gouge

* Grade according to ISRM (1981).

**All rock types exhibit a broad range of uniaxial compressive strengths which reflect the heterogeneity in composition and anisotropy in structure. Strong rocks are characterised by well interlocked crystal fabric and few voids.

***Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.

Appendix D: Ratings for Rock Mass Rating system.

Rock Mass Rating System (After Bieniawski, 1989).

CLASSIFICATION PARAMETERS AND THEIR RATINGS											
Parameter			Range of values								
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred				
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa		
		Rating	15	12	7	4	2	1	0		
2	Drill core Quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	<25%				
		Rating	20	17	13	8	3				
3	Spacing of discontinuities		> 2 m	0.6-2 m	200-600 mm	60-200 mm	< 60 mm				
		Rating	20	15	10	8	5				
4	Condition of discontinuities (See E)	Very rough surfaces Not continuous No separation Unweathered wall rock		Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous				
		Rating	30	25	20	10	0				
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10-25	25-125	> 125				
		(Joint water pressure) (Major principal stress)	0	< 0.1	0.1-0.2	0.2-0.5	> 0.5				
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing				
		Rating	15	10	7	4	0				
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)											
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable					
Ratings	Tunnels & mines	0	-2	-5	-10	-12					
	Foundations	0	-2	-7	-15	-25					
	Slopes	0	-5	-25	-50						
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS											
Rating	100 - 81		80 - 61		60 - 41		40 - 21		< 21		
Class number	I		II		III		IV		V		
Description	Very good rock		Good rock		Fair rock		Poor rock		Very poor rock		
D. MEANING OF ROCK CLASSES											
Class number	I		II		III		IV		V		
Average stand-up time	20 yrs for 15 m span		1 year for 10 m span		1 week for 5 m span		10 hrs for 2.5 m span		30 min for 1 m span		
Cohesion of rock mass (kPa)	> 400		300-400		200-300		100-200		< 100		
Friction angle of rock mass (deg)	> 45		35-45		25-35		15-25		< 15		
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions											
Discontinuity length (persistence)	< 1 m		1-3 m		3-10 m		10-20 m		> 20 m		
Rating	6		4		2		1		0		
Separation (aperture)	None		< 0.1 mm		0.1-1.0 mm		1-5 mm		> 5 mm		
Rating	6		5		4		1		0		
Roughness	Very rough		Rough		Slightly rough		Smooth		Slickensided		
Rating	6		5		3		1		0		
Infilling (gouge)	None		Hard filling < 5 mm		Hard filling > 5 mm		Soft filling < 5 mm		Soft filling > 5 mm		
Rating	6		4		2		2		0		
Weathering	Unweathered		Slightly weathered		Moderately weathered		Highly weathered		Decomposed		
Rating	6		5		3		1		0		
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**											
Strike perpendicular to tunnel axis						Strike parallel to tunnel axis					
Drive with dip-Dip 45-90°			Drive with dip-Dip 20-45°			Dip 45-90°			Dip 20-45°		
Very favourable			Favourable			Very favourable			Fair		
Drive against dip-Dip 45-90°			Drive against dip-Dip 20-45°			Dip 0-20°-irrespective of strike*					
Fair			Unfavourable			Fair					

*Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

**Modified after Wickham et al. (1972).

Appendix E: Ratings for Rock Mass Quality (Q-system).

Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al., 1974).

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0-25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25-50	
C. Fair	50-75	
D. Good	75-90	2. RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.
E. Excellent	90-100	
2. JOINT SET NUMBER	J_s	
A. Massive, no or few joints	0.5-1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $(3.0 \times J_s)$.
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use $(2.0 \times J_s)$.
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 1 m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.
c. No rock wall contact when sheared		
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0	
I. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	(nominal) 1.0	
J. Slickensided, planar	(nominal) 1.0	
4. JOINT ALTERATION NUMBER	J_a	ϕ degrees (approx.)
a. Rock wall contact		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	1. Values of ϕ , the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25-35
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25-30
D. Silty, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20-25
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less in thickness)	4.0	8-16

Support of underground excavations in hard rock

s (continued).

DESCRIPTION	VALUE	NOTES
4. JOINT ALTERATION NUMBER	J_a	ϕ degrees (approx.)
a. Rock wall contact before 10 cm shear		
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25-30
G. Strongly over-consolidated, non-softening clay mineral fillings, continuous ≤ 5 mm thick	6.0	16-24
H. Medium or low over-consolidation, softening clay mineral fillings (continuous ≤ 5 mm thick)	8.0	12-16
J. Swelling clay fillings, i.e. montmorillonite (continuous ≤ 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0-12.0	6-12
c. No rock wall contact when sheared		
K. Zones or bands of disintegrated or crushed rock and clay (see G, H and I for clay conditions)	6.0	
L. Thick continuous zones or bands of clay	8.0	
M. Zones or bands of silty or sandy-clay, small clay fraction, non-softening	8.0-12.0	6-24
N. Thick continuous zones or bands of clay	5.0	
P. & R (see G, H and I for clay conditions)	10.0-13.0	
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm ²) ≤ 1.0
A. Dry excavation or minor inflow, i.e. ≤ 5 l/m locally	1.0	
B. Medium inflow or pressure, occasional misting of joint fillings	0.66	1.0-2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	
D. Large inflow or high pressure	0.33	2.5-10.0
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2-0.1	2.5-10.0
F. Exceptionally high inflow or pressure	0.1-0.05	> 10
6. STRESS REDUCTION FACTOR		SRF
a. Weakness zones intersecting excavation, which may cause lowering of rock mass when tunnel is excavated		
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)		
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth ≤ 50 m)		
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)		
D. Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)		
E. Single shear zone in competent rock (clay-free) (depth of excavation ≤ 50 m)		
F. Single shear zone in competent rock (clay-free) (depth of excavation > 50 m)		
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)		
1. Factors C to F are crude estimates, increase J_w if damage installed.		
2. Special problems caused by ice formation are not considered.		
1. Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.		

(continued).

DESCRIPTION			VALUE	NOTES
4. STRESS REDUCTION FACTOR			SRF	
b. Competent rock, rock stress problems				
	σ_1/σ_3	σ_1/σ_3		2. For strongly anisotropic virgin stress field
H. Low stress, near surface	> 200	> 13	2-5	(a) measured; when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_1/σ_3 to 0.8; and σ_1/σ_3 to 0.6. When $\sigma_1/\sigma_3 > 10$, reduce σ_1/σ_3 and σ_1/σ_3 to 0.6 and 0.6, where
J. Medium stress	200-10	12-0.66	1-0	σ_1 = unconfined compressive strength, and
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10-5	0.66-0.33	0.5-2	σ_3 = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
L. Mild rockburst (massive rock)	5-2.5	0.33-0.16	5-10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10-20	3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5-10	
O. Heavy squeezing rock pressure			10-20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5-10	
R. Heavy swelling rock pressure			10-15	
ADDITIONAL NOTES ON THE USE OF THESE TABLES				
When making estimates of the rock mass Quality (Q), the following guidelines should be followed in addition to the notes listed in the tables.				
1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to RQD for the case of clay-filled rock masses: $RQD = 115 - 3.3 J$, (appendix), where J = total number of joints per m. If $RQD < 100$ use $3.3 \times J \times 2.5$.				
2. The parameter J , representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage, or bedding, etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few joints visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J .				
3. The parameters J_1 and J_2 (representing shear strength) should be relevant to the weakest significant joint set or clay-filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_1/J_2 is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_1/J_2 should be used when evaluating Q . The value of J_1/J_2 should in fact relate to the surface most likely to allow failure to initiate.				
4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock stress/rock strength. A strongly anisotropic stress field is not favourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factors situation.				
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated on the uniaxial and uniaxial strengths appropriate in the present and future in situ conditions. A very conservative estimate of the strength could be made for those rocks that increase rate when exposed to moist or saturated conditions.				

Appendix F: Support of under ground excavations in hard rock.

Guidelines for excavation and support of 10 m span rock tunnels in accordance with the *RMR* system (After Bieniawski, 1989).

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I – Very good rock <i>RMR</i> : 81-100	Full face, 3 m advance	Generally no support required except spot bolting		
II – Good rock <i>RMR</i> : 61-80	Full face, 1-1.5 m advance. Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III – Fair rock <i>RMR</i> : 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None
IV – Poor rock <i>RMR</i> : 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
V – Very poor rock <i>RMR</i> : < 20	Multiple drifts 0.5-1.5 m advance in top heading Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert	150-200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close in- vert

Appendix G (continued).

Classification and Prediction of Support for Six of the Case Records Described by Cecil (1970)

Case No.	1. DESCRIPTION OF ROCK MASS 2. Nature of instability 3. Purpose of excavation, location, reference	SPAN m	Height m	Depth m	Support used	RQD J_n	J_r J_a	J_w SRF	Q	ESR	SPAN' ESR m	Roof Support Recommendation
8	1. 50 m length of closely spaced, tight diagonal joints in leptite. Planar, smooth joints. 1 joint set, 5–30 cm, spacing. No water present. 2. Minor overbreak when blasting. 3. Tailrace tunnel, Seitevare Hydro, N. Sweden (ref. Cecil 1970).	9	9	140	None	70 2	1.0 1.0	1.0 1.0				Category 0
24	1. 60 m length, including a 1 m wide shear zone in mylonite. Crushed mylonite and non-softening clay seams and joint fillings. Intersecting joint set. 2 joint sets plus random, 5–30 cm spacing. Minor water inflows (< 3 l/min). 2. Wedge shaped roof fall. 3. Headrace tunnel, Vietas Hydro, N. Sweden (ref. Cecil 1970).	12.5	6.5	60	Rock bolts, wire mesh and shotcrete	60 6	1.0 n	1.0 2.5		35 1.6 5.6		Category 22 = B 1 m + S (mr) 2.5–5 cm
39	1. 50 m length, shear zone in quartzite, "sugar cube" rock structure. Planar, smooth, unaltered joints. 3 joint sets, < 5 cm, spacing. 5–10 l/min water inflow. 2. Major roof falls, progressive formation of dome- and vault-shaped crown. Also falls from the face. 3. Headrace tunnel, Rendal Hydro, Norway (ref. Cecil 1970).	8	6	200	Cast concrete arch, immediately after mucking out	20 15	1.0 1.0	0.66 5		1.3 1.6 7.8		Category 31 = CCA 20–30 cm + B 1 m
43	1. 25 m length, 3 m wide shear zone in thinly laminated schist, swelling montmorillonitic clay seam in shear zone, some chlorite joint coatings. Planar slickensided joint walls. 1 joint set, 5–30 cm spacing. Ground water seepage along cased de-air hole may have contributed to swelling process. 2. Complete collapse of tunnel during operation of power plant. Vault-shaped crown opening. 3. Tailrace tunnel, Sällsjö Hydro, N. Sweden (ref. Cecil 1970).	9	8	110	Original 6–8 cm shotcrete tailed. Permanent support after failure with cast concrete arches	20 2	0.5 12	1.0 2.5		0.18 1.6 5.0		Category 31 = CCA (sr) 30 cm + B 1 m
48	1. 15 m length, overthrust shear zone in schist, in which there was a 3 cm thick clay (non softening) and graphite seam. Shear zone was 50–100 cm wide and contained smooth, slickensided graphite-coated joint surfaces. 1 joint set, 5–30 cm spacing. Insignificant water inflow. 2. Wedge-shaped roof fall. 3. Tailrace tunnel, Bergvattnet Hydro, N. Sweden (ref. Cecil 1970).	6.5	4.5	50	Rock bolts, wire mesh and two shotcrete applications	10 2	1.0 10	1.0 5		0.17 1.6 5.6		Category 31 = B 1 m + S (mr) 5 cm
56	1. 20 m length, 10 m wide vertical shear zone in granite. Rock crushed and frequently altered to earthy-gravel. Some remnant joint surfaces coated with clay (non-softening). Rock adjacent to zone blocky and loose. Irregular slickensided joint surfaces, 5–30 cm spacing. Large water inflows after blasting carried fault zone debris into tunnel, left open voids up to 1 m wide. Note: Tunnel located within 10 km of a major overthrust shear, locally vertical and low angle shear zones occur. 2. Progressive roof fall-out to form a large vault-shaped opening. 3. Headrace tunnel, Stensjöfaller Hydro, N. Sweden (ref. Cecil 1970).	5.9	4.5	100	No support immediately after blasting. Eventually two shotcrete applications	10 20	1.5 6	0.33 2.5		0.10 1.6 4.1		Category 34 = S (mr) 7.5 cm
										0.017 1.6 3.7		

Note: Right-hand column: "Roof Support Recommendation" is obtained from Tables 11, 12, 13, and 14

Key: S = shotcrete, B = systematic bolting, sb = spot bolting, CCA = cast concrete arches, mr = mesh reinforced, sr = steel reinforced, clm = chain link mesh.

Bolt spacing is given in metres. — Shotcrete or concrete thickness is given in centimeters.

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Engineering Classification of Rock Masses for the Design of Tunnel Support

Appendix G: (continued)

Classification and Prediction of Support for Six of the Case Records Described by Cecil (1970)

N	DESCRIPTION OF ROCK MASS 2. Nature of instability 3. Purpose of excavation, location, reference	SPAN m	Height m	Depth m	Support used	RQD J_n	J_r J_n	J_w SRF	Q	ESR	SPAN/ ESR m	Roof support recommendation	
60	1. 20 m length, 1 m wide zone of sheared granite with clay seams (non-softening) slide boundary is a thin (< 1 cm) clay seam and thinly sheared material that lie in contact with massive rock. Planar, slickensided joints. 1 joint set, 5–30 cm spacing. Insignificant inflow of water. See note, case 56. 2. Wedge-shaped roof fall. 3. Headrace tunnel, Stensjöfallet Hydro. N. Sweden (ref. Cecil 1970).	5.9	4.3	85	Rock bolts, and shotcrete	80 2	0.5 6	1.0 2.5		1.3	1.6	3.7	Category 21 = B 1 m + S 2.5 cm
66	1. 80 m length, open horizontal sheeting joints in granite, partially filled with sand sized material. Planar, rough surfaced joints. 2 joint sets, 5–30 cm spacing. Insignificant water inflow. See note, case 56. 2. Overbreak above springline. 3. Access tunnel, Stensjöfallet Hydro. N. Sweden (ref. Cecil 1970).	7	4.5	15–20	Rock bolts and shotcrete	70 4	1.5 2	1.0 5		2.6	1.3	5.4	Category 21 = B 1 m + S 2.5 cm
67	1. 50 m length, close vertical jointing cutting across schistose rock structure in schistose metagreywacke. Sandy, gravelly joint fillings. Planar smooth surface joints. 1 joint set plus random (for schistosity planes), 5–30 cm spacing. Water inflows up 1000 l/min. 2. Large overbreak in intrados, some roof falls. 3. Railrace tunnel, Stensjöfallet Hydro. N. Sweden (ref. Cecil 1970).	5.9	4.8	100	Shotcrete	20 3	1.0 2	0.2 1.0		1.7	1.6	3.7	Category 21 = S 2.5 cm
70	1. 10 m length, strongly sheared granite, very tight vertical structure. Planar, rough surfaced, unaltered joints. 1 joint set, 5–30 cm spacing. Insignificant water inflow. 2. Stable, minor overbreak, no roof falls. 3. Collector tunnel, Mo i Rana Hydro. N. Norway (ref. Cecil 1970).	8	5.7	15	None	40 2	1.5 1.0	1.0 2.5		12	1.6	5.0	Category 0 Note: Very tight structure may imply higher stress, i.e. SRF = 1.0 Hence Q = 30
74	1. Approx. 2 km length, massive granite, widely spaced, tight, vertical joints. Planar, smooth-surfaced unaltered joints. 1 joint set, 1–3 m spacing. Insignificant water inflow. 2. No overbreak in chambers, but overbreak at intersections. 3. Waste water treatment plant, Kåppala. Sweden (ref. Cecil 1970).	12	12.5	≤ 100	None in chambers	100 2	1.0 1.0	1.0 1.0	50	1.3	9.2	Category 0.9 = NONE or sb	
75	1. 300 m length, massive gneiss, few joints. Planar, rough-surfaced, unaltered joints. > 3 m spacing. Insignificant water inflow. 2. Minor overbreak, no falls or slides. 3. Wine and liquor storage rooms. Stockholm (ref. Cecil 1970).	20	24.5	18	Bolts at intersections 50 spot bolts in about 300 m of chamber	100 2 × 3 100	1.0 1.0 5	1.0 1.0 1.0	16.7	1.0	12.0	Category 14 = B 1.5–2 m + cfm Category 0.5 = None or sb	

Note: Right-hand column "Roof Support Recommendation" is obtained from Tables 11, 12, 13, and 14.

Key: S = shotcrete, B = systematic bolting, sb = spot bolting, CCA = cast concrete arches, mr = mesh reinforced, sr = steel reinforced, cfm = chain link mesh.

Bolt spacing is given in metres. — Shotcrete or concrete thickness is given in centimetres.

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Appendix G (continued)

Support category	Q	Conditional factors RQD/I_n	I_r/I_a	SPAN/ESR (m)	P kg/cm ² (approx.)	$S_r/AN/ESR$ (m)	Type of support	Note See p. 229
25	1.0—0.4	> 10	> 0.5	—	2.25	1.5—4.2	B (urg) 1 m + mr or cim	I
		≤ 10	> 0.5	—			B (urg) 1 m + S (mr) 5 cm	I
		—	≤ 0.5	—			B (tg) 1 m + S (mr) 5 cm	I
26	1.0—0.4	—	—	—	2.25	3.2—7.5	B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
		—	—	—			B (urg) 1 m + S 2.5—5 cm	I, IX
27	1.0—0.4	—	—	≥ 12 m	2.25	6—18	B (tg) 1 m + S (mr) 7.5—10 cm	I, IX
		—	—	< 12 m			B (urg) 1 m + S (mr) 5—7.5 cm	I, IX
		—	—	> 12 m			CCA 20—40 cm	VIII, X, XI
		—	—	< 12 m			+ B (tg) 1 m	
		—	—	< 12 m			S (mr) 10—20 cm	VIII, X, XI
		—	—	< 12 m			+ B (tg) 1 m	
28* See note XII	1.0—0.4	—	—	≥ 30 m	2.25	15—38	B (tg) 1 m + S (mr) 30—40 cm	I, IV, V, IX
		—	—	≥ 20, < 30			B (tg) 1 m + S (mr) 20—30 cm	I, II, IV, IX
		—	—	< 20 m			B (gr) 1 m + S (mr) 15—20 cm	I, II, IX
		—	—	—			CCA (sr) 30—100 cm + B (tg) 1 m	IV, VIII, X, XI
29*	0.4—0.1	> 5	> 0.25	—	3.0	1.0—3.1	B (urg) 1 m + S 2—3 cm	—
		≤ 5	> 0.25	—			B (urg) 1 m + S (mr) 5 cm	—
		—	≤ 0.25	—			B (tg) 1 m + S (mr) 5 cm	—
30	0.4—0.1	≥ 5	—	—	3.0	2.2—6	B (tg) 1 m + S 2.5—5 cm	IX
		< 5	—	—			S (mr) 5—7.5 cm	IX
		—	—	—			B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
31	0.4—0.1	> 4	—	—	3.0	4—14.5	B (tg) 1 m + S (mr) 5—12.5 cm	IX
		≤ 4, ≥ 1.5	—	—			S (mr) 7.5—25 cm	IX
		< 1.5	—	—			CCA 20—40 cm + B (tg) 1 m	IX, XI
		—	—	—			CCA (sr) 30—50 cm + B (tg) 1 m	VIII, X, XI
32 See note XII	0.4—0.1	—	—	≥ 20 m	3.0	11—34	B (tg) 1 m + S (mr) 40—60 cm	II, IV, IX, XI
		—	—	< 20 m			B (tg) 1 m + S (mr) 20—40 cm	III, IV, IX, XI
		—	—	—			CCA (sr) 40—120 cm + B (tg) 1 m	IV, VIII, X, XI

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* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

(continued) Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality
(Q range: 0.1—0.001)

Support Category	Q	Conditional Factors RQD/J_n	SPAN/ESR (m)	P Kg/cm ² (approx.)	SPAN/ESR (m)	Type of support	Note See p. 229
33*	0.1—0.01	≥ 2	—	6	1.0—3.9	B (tg) 1 m + S (mr) 2.5—5 cm S (mr) 5—10 cm S (mr) 7.5—15 cm	IX
34	0.1—0.01	≥ 2	≥ 0.25	6	2.0—11	B (tg) 1 m + S (mr) 5—7.5 cm S (mr) 7.5—15 cm S (mr) 15—25 cm CCA (sr) 20—60 cm + B (tg) 1 m	IX VIII, X IX IX VIII, X, XI
35	0.1—0.01	—	≥ 15 m	6	6.5—28	B (tg) 1 m + S (mr) 30—100 cm CCA (sr) 60—200 cm + B (tg) 1 m	IX, IX, XI VIII, X, XI, II
See note XII	—	—	≥ 15 m	—	—	B (tg) 1 m	IX, XI, III
—	—	—	< 15 m	—	—	+ S (mr) 20—75 cm CCA (sr) 40—150 cm + B (tg) 1 m	VIII, X, XI, III
36*	0.01—0.001	—	—	12	1.0—2.0	S (mr) 10—20 cm S (mr) 10—20 cm + B (tg) 0.5—1.0 m	IX VIII, X, XI
37	0.01—0.001	—	—	12	1.0—6.5	S (mr) 20—60 cm S (mr) 20—60 cm + B (tg) 0.5—1.0 m	IX VIII, X, XI
38 See note XIII	0.01—0.001	—	≥ 10 m ≥ 10 m	12	4.0—20	CCA (sr) 100—300 cm CCA (sr) 100—300 cm + B (tg) 1 m	IX VIII, X, II, XI
—	—	—	< 10 m	—	—	S (mr) 70—200 cm + B (tg) 1 m	IX VIII, X, III, XI

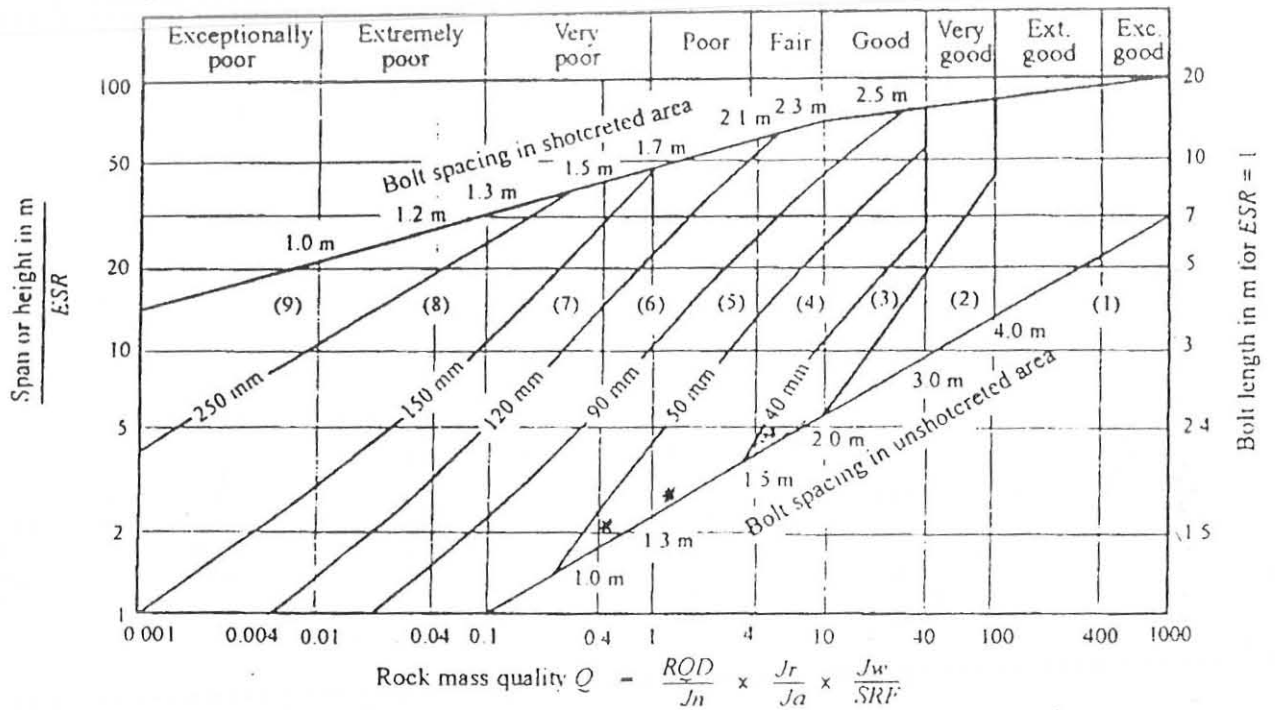
* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Engineering Classification of Rock Masses for the Design of Tunnel Support

Supplementary Notes for Support Tables

- I. For cases of heavy rock bursting or "popping", tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.
- II. Several bolt lengths often used in same excavation, i.e. 3, 5 and 7 m.
- III. Several bolt lengths often used in same excavation, i.e. 2, 3 and 4 m.
- IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2—4 m.
- V. Several bolt lengths often used in some excavations, i.e. 6, 8 and 10 m.
- VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4—6 m.
- VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25—40 cm) as permanent support.
- VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.
- IX. Cases not involving swelling clay or squeezing rock.
- X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.
- XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e. > 1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e. $RQD/J_n < 1.5$, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when $RQD/J_n < 1.5$, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.
- XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR > 15 m only).
- XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 (SPAN/ESR > 10 m only).

Appendix G: (Continued).



* Q. value for the proposed tunnel

REINFORCEMENT CATEGORIES

- 1) Unsupported
- 2) Spot bolting
- 3) Systematic bolting
- 4) Systematic bolting with 40-100 mm unreinforced shotcrete
- 5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
- 6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
- 7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
- 8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
- 9) Cast concrete lining

Declaration

I, the undersigned, declare that this thesis is my work and that all sources of material used for the thesis have been duly acknowledged.

Name **Girma Wolderufael**

Signature



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