

181

ADDIS ABABA UNIVERSITY

ENGINEERING GEOLOGICAL STUDY OF AWASSA TOWN AND ITS SURROUNDINGS

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the Faculty of Science,
Addis Ababa University



Approved by Board of Examiners

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In Partial Fulfilment
of the Requirement for the
Degree of Master of Science in Geology

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June, 2000

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AWASSA TOWN AND ITS SURROUNDINGS**

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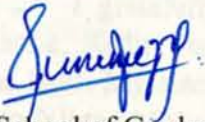
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I am indebted to the German Academic Exchange Service (DAAD) and the Addis Ababa University for their research grant. I would like to express my sincere appreciation to the Department of Geology and Geophysics, Addis Ababa University for the logistic support during fieldwork and for providing me the computer facilities.

DECLARATION

I, the undersigned, declare that this thesis is my original work, has not been presented for a degree in any other University and that all sources of material used for the thesis have been duly acknowledged.

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June, 2000

The Thesis has been submitted for examination with my approval as university advisor.

Tenalem Ayenew, Dr.



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I am indebted to the German Academic Exchange Service (DAAD) and the Addis Ababa University for their research grant. I would like to express my sincere appreciation to the Department of Geology and Geophysics, Addis Ababa University for the logistic support during fieldwork and for providing me the computer facilities.

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TABLE OF CONTENTS

	PAGE
ACKNOWLEDGEMENT	I
TABLE OF CONTENTS	II
LIST OF TABLES	V
LIST OF FIGURES	VI
LIST OF APPENDICES	XIII
ABSTRACT	IX
CHAPTER ONE	
GENERAL INTRODUCTION	1
1.1 BACK GROUND TO THE PROBLEM	1
1.2 RELEVANCE OF THE STUDY	2
1.3 RESEARCH OBJECTIVES	2
1.4 APPROACH FOLLOWED AND DIFFICULTIES ENCOUNTERED	3
CHAPTER TWO	
GENERAL OVERVIEW OF THE STUDY AREA	5
2.1 LOCATION	5
2.2 CLIMATE	5
2.3 PHYSIOGRAPHY AND DRAINAGE	7
2.4 LAND USE AND POPULATION	10
CHAPTER THREE	
GEOLOGY, TECTONICS AND STRATIGRAPHY	13
3.1 GENERAL GEOLOGICAL SETTING	13
3.2 TECTONICS	16
3.3 STRATIGRAPHY AND VOLCANISM	19
3.3.1 Basalts and ignimbrites of the plateau trap series (late Miocene)	23
3.3.2 Old alkaline and Peralkaline Silicic rocks (Late Pliocene - early Pleistocene)	24
3.3.3 Recent Basaltic lava flows, basaltic hyaloclastites and scoria cone	24
3.3.4 Recent acidic Volcanics (<1.6 Ma)	25
3.3.5 Volcano-Lacustrine deposits (Pleistocene to recent)	25
CHAPTER FOUR	
HYDROMETEOROLOGY, SURFACE HYDROLOGY, HYDROGEOLOGY AND HYDROCHEMISTRY OF THE AREA	26
4.1 GENERAL	26
4.2 HYDROMETEOROLOGICAL VARIABLES	27
4.2.1 Precipitation	27



LIST OF TABLES

4.2.2 Temperature	29
4.2.3 Humidity	29
4.2.4 Wind Speed	31
4.2.5 Sunshine Hours	31
4.2.6 Evaporation	31
4.3 SURFACE HYDROLOGY	33
4.3.1 Runoff	33
4.4 HYDROGEOLOGY	37
4.4.1 General	37
4.4.2 Hydrostratigraphy	39
4.4.3 Hydrogeological Features	42
4.4.3.1. Depth to Groundwater Level	43
4.4.3.2. Groundwater flow	45
4.5 HYDROCHEMISTRY	46
4.5.1 General	46
4.5.2 Major Cations and Anions	47
4.5.3 Water Quality	51

CHAPTER FIVE

RECENT LAKE AWASSA LEVEL FLUCTUATIONS AND LAKE WATER BALANCE

ESTIMATION	57
5.1 RECENT LAKE AWASSA LEVEL FLUCTUATIONS	57
5.2 LAKE WATER BALANCE ESTIMATION	64

CHAPTER SIX

FLOOD HAZARD MAPPING	68
6.1 GENERAL	68
6.2 FLOOD PROBABILITIES	71
6.3 ESTIMATION OF THE T-YEAR FLOOD USING EV I	72

CHAPTER SEVEN

✓ ENGINEERING GEOLOGICAL CHARACTERISTICS OF THE SOILS AND ROCKS	77
7.1 GENERAL	77
7.2 METHODOLOGY	77
7.3 LITHOLOGICAL DESCRIPTION OF THE SOILS	78
7.3.1 Clay Soils	78
7.3.2 Sandy Silt Soils	78
7.3.3 Silty Sand/Gravel Soils	79
7.4 LABORATORY ANALYSIS AND RESULTS	80
7.4.1 Mineralogical and Chemical composition	80
7.4.1.1. Mineralogical Composition	80
7.4.1.2. Organic Matter Content	80
7.4.1.3. Carbonate Content	81
7.4.1.4. Sulphate and Chloride Content and pH of the oils	82
7.4.2 Physical (Index) Properties	83
7.4.2.1. Moisture Content	83
7.4.2.2. Specific Gravity	84

LIST OF TABLES

Table 3. 1.	Stratigraphy of the central sector of the Main Ethiopian Rift	20
Table 4. 1	Average concentrations of major cations and anions and other quality parameters	48
Table 4. 2.	Soluble Sodium percent and Sodium Adsorption Ratio (SAR) for water samples from Awassa catchment	53
Table 6. 1.	Annual peak Lake Awassa levels (1970 - 1998)	70
Table 6. 2.	Flood Frequency Analysis result calculated using EVI	75
Table 7. 1.	Organic matter content of the soils of the study area	81
Table 7. 2.	Sulphate and Chloride content and pH of soils	82
Table 7. 3.	Moisture content of soils of Awassa area	84
Table 7. 4.	Specific gravity and granulometric composition of the soils of Awassa Area	85
Table 7. 5.	Ranges of grain size distribution of the various soil types	86
Table 7. 6.	Atterberg Limits and plasticity index values of the soils of Awassa Area	90
Table 7. 7.	Liquid Limit (%) determined from three - point and one -point method	91
Table 7. 8.	Activity of clay soils of Awassa Area	92
Table 7. 9.	Free swell (%) for soils of Awassa Area	94
Table 7. 10.	Compiled relevant geotechnical properties of soils of the study area	95
Table 8. 1.	Classification of soils of Awassa Area according to the USCS	97
Table 8. 2.	Engineering use Chart	99
Table 8. 3.	Chemical Analysis Data for rocks of the study Area	102
Table 8. 4.	Aggregate test results for the rhyolite sample from Kika ridge	102
Table 9. 1.	Summary of geotechnical properties of sandy silt and silty sand/ gravel soils	106

7.4.2.3. Grain Size Analysis	84
7.4.2.4. Consistency (Atterberg) Limits and Index Values	89
7.4.2.5. Activity of the Clay Soils	91
7.4.2.6. Free Swell.....	93
7.4.2.7. Other Relevant Engineering Properties	95

Fig. 2.7. Location map of the study area

CHAPTER EIGHT

ENGINEERING GEOLOGICAL CLASSIFICATION OF THE SOILS AND ROCKS	96
8.1 GENERAL	96
8.2 CLASSIFICATION OF THE SOILS	96
8.3 ENGINEERING GEOLOGICAL CLASSIFICATION OF ROCKS	100

CHAPTER NINE

ENGINEERING GEOLOGICAL MAPPING.....	103
9.1 GENERAL	103
9.2 DELINEATION AND ENGINEERING GEOLOGICAL CHARACTERISTICS OF THE MAPPING UNITS	104

CHAPTER TEN

SEISMICITY OF THE STUDY AREA.....	108
10.1 SEISMICITY OF THE AREA	108
10.2 INFLUENCE OF SOIL ON SEISMIC RESPONSE AND MOVEMENTS	112
10.3 LIQUEFACTION POTENTIAL OF THE SOILS.....	113

CONCLUSION AND RECOMMENDATION	119
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REFERENCES	124
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APPENDICES.....	131
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Fig. 4.3. Monthly total pan evaporation for the period specified as measured at Awassa station

Fig. 4.6a. Depth area curve for Lake Awassa

Fig. 4.6b. Depth volume curve for Lake Awassa

Fig. 4.7. Lake Shalla gauge levels and Tana Lake discharge

Fig. 4.8. Hydrogeological map of the Awassa catchment area

Fig. 4.9. Groundwater level map

Fig. 4.10. Types and areas of the different water bodies

LIST OF FIGURES

Fig. 2. 1.	Location map of the study area	6
Fig. 2. 2.	Structural framework of part of the Central Sector of the MER showing the Awassa Caldera Complex	8
Fig. 2. 3.	Drainage map of the study area	9
Fig. 2. 4.	Landuse and Land cover map of the study area	11
Fig. 3. 1.	Major structural features of the northern, central and southern sectors of the Main Ethiopian Rift and adjacent areas	14
Fig. 3. 2a.	Geological Map of the study area	21
Fig. 3. 2b.	A simplified Geological cross section along AA'	22
Fig. 4. 1.	Seasonal and annual variation of rainfall in Awassa Area for the period specified as recorded at Awassa station	28
Fig. 4. 2.	Annual rainfall for stations in and around Awassa catchment	28
Fig. 4. 3a.	Mean monthly temperature trends for the period specified	30
Fig. 4. 3b.	Mean monthly minimum, maximum and average temperature of Awassa area as recorded at Awassa station	30
Fig. 4. 4.	Mean monthly relative humidity as measured at Awassa station	32
Fig. 4. 5.	Monthly total pan evaporation for the period specified as measured at Awassa station	32
Fig. 4. 6a.	Depth area curve for lake Awassa	34
Fig. 4. 6a.	Depth volume curve for lake Awassa	34
Fig. 4. 7.	Lake Shallo gauge levels and Tikurwuha discharge	36
Fig. 4. 8.	Hydrogeological map of the Awassa catchment area	41
Fig. 4. 9.	Groundwater level map	44
Fig. 4. 10.	Piper tri-linear plot of the different water bodies	50

Fig. 4. 11a.	Classification of water samples from Awassa catchment for irrigation using soluble sodium percent	55
Fig. 4. 11b.	Classification of water samples based on salinity hazard	56
Fig. 5. 1.	Awassa lake level fluctuation for the period specified	58
Fig. 5. 2.	Monthly lake Awassa level and total rainfall at Awassa station	60
Fig. 5. 3.	Annual rainfall at Awassa, Arbaminch and Ziway stations for the period specified	60
Fig. 5. 4.	Monthly plots of Awassa lake level & rainfall at Awassa station	61
Fig. 5. 5.	Lakes level fluctuations of some of the Ethiopian rift valley lakes for the period specified	63
Fig. 5. 6.	Calculated and measured lake Awassa level fluctuations for the period specified	67
Fig. 6. 1.	Flood hazard map of the study site	76
Fig. 7. 1.	Particle size distribution envelopes of the various soil types	87, 88
Fig. 8. 1.	Positions of the soils of Awassa area on the Casagrande plasticity chart	98
Fig. 10. 1.	Geographical distribution of Earthquakes in the horn of Africa for the period 1960 – 1993	109
Fig. 10. 2.	Seismotectonic source zones contributing to hazard evaluation in Ethiopia and the neighbouring countries	109
Fig. 10. 3.	Seismic hazard map of Ethiopia	111
Fig. 10. 4.	The relation between shear wave velocity and the amplification factor at layered ground	114
Fig. 10. 5.	Chart for values of CN	116
Fig. 10. 6.	Relationship between stress ratio causing liquefaction and N ₁ values for silty sands for M = 7.5 earthquakes	116
Fig. 10. 7.	Field liquefaction behaviour predicted for the study site using SPT – Stress ratio chart proposed by Seed et. al. (1984)	118

LIST OF APPENDICES

Appendix A – Some of the water points Data	131
Appendix B – Hydrochemical Data	132
Appendix C – Water quality criteria	135
Appendix D – Soil and rock descriptive terms and classification schemes	136
Appendix E – Seismic hazard maps of Ethiopia	141
Appendix F – Engineering geological map, location of Sample points and long profiles, and the sections along the profiles A - A', B - B' and C - C'	142

ABSTRACT

In the present work, engineering geological study has been carried out to characterise and map the engineering geological conditions for the Awassa town and its surroundings, located 275 Km south of Addis Ababa in the central sector of the main Ethiopian rift. Hydrometeorological, hydrological, hydrogeological, hydrochemical and geodynamic conditions have been assessed in view of their relevance for future development of the town as related to Civil Engineering activities and to the rise of lake Awassa, which is becoming a threat for its future development. The engineering geological characteristics of the foundation soils and rocks has been assessed on the basis of field observations, simple in situ tests, and from the results of the laboratory analysis made on disturbed samples. Besides, borehole records from existing data have been studied in order to quantify and correlate the data from the test pits.

The main rock types in the area are volcano-lacustrine sediments and acidic volcanic rocks including ignimbrites, pyroclastic fall deposits, rhyolitic lava flows and associated obsidians. Basaltic lava flows, scorias and basaltic hyaloclastites are also locally exposed. However, the town is wholly founded on the volcano-lacustrine deposits.

The area generally gets moderate rainfall with moderate inter annual and seasonal variabilities. Lake Awassa has recently undergone lake level rise. The main causes of this phenomenon are believed to be high rainfall and reduced evaporation. The extreme value type I distribution is used for frequency analysis of annual maximum lake levels and the lake levels for different years of recurrence interval is calculated. On the bases of these calculations, a flood hazard map of the lakeshore for 200 years return period is established.

Water quality analysis results indicate that the water in the area has no immediate danger on engineering works. The waters in the area are of dominantly Na - HCO₃ type lacking high concentration of sulphate and/or chloride. However, the groundwater generally fluctuates up to 5m following rainy seasons, and may be critical for civil engineering works close to the lake.

The foundation soil is dominantly a fine-grained type though coarse-grained soils are also found at different stratigraphic positions. Three dominant soil types are identified in the field: clay soils, sandy silt soils and silty sand/gravel soils. The clay soils occupy relatively the elevated central and southeastern part of the town and the possible expansion area while the other types occur in almost the whole area. According to the Unified Soil Classification system, most of the soils belong to ML (inorganic silts, very fine sands, rock flour, silty or clayey fine sands, SM (silty sands, sand - silt mixtures) and CL (inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays). Relatively few soil samples are in groups MH (inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts), CH (inorganic clays of high plasticity, fat clays) and GM (silty gravels, gravel-sand-clay mixtures). All the soil types show thickness variation both laterally and vertically ranging from about 0.85 m to about 3.5m.

Moisture content at the time of sampling ranges from 25.1 to 29.0% for clay soils, and from 18 to 46 % for sandy silt and silty sand/gravel soils. The specific gravity of the soils is low, in the range from 2.2 to 2.5. The liquid limit and plasticity index of the clay soils range from 28 to 53% and from 7 to 24% respectively. The sandy silt soils have LL% in the range from 26.7 to 56% and PI % in the range from 1.89 to 25%.

The soils of the area are found out to be non expansive as revealed by the simple free swell test. However, some of the sandy silt and silty sand/gravel soils show reduction in volume as large as

Chapter One

29% , thus suspected to have a collapsible structure. These soils may collapse both under load and when flooded by water and may also result in differential settlement. Transmitting the weight of the structure to deeper soil or rock layer by means of piles or caissons, and if not, to make the foundation as deep as possible is believed to minimise the hazard.

From engineering geological point of view, the rocks outcropping in the town area are grouped in to two: strong (basalts and rhyolites) and weak rocks (dominantly of basaltic hyaloclastites and tuffs). From field Schmidt hammer test result, it is found out that the former ones have an unconfined strength in the range from 310 to greater than 700 Kg/cm², while the latter less than 100 Kg/cm². The strong ones are found out to be good sources for aggregates while the weaker ones are not suitable for most of civil engineering works.

Based on the field and laboratory investigation results, Engineering geological mapping at the scale of 1:10, 000 was carried out for the town area including the possible expansion area.

Chapter One

General Introduction

1.1 Back ground to the problem

In any country an important aspect of development is the availability of an adequate infrastructure. These structures are founded on or in geologic materials and the influence of the civil engineering works on the ground and vice-versa becomes so important that a sound assessment of ground conditions is vital. Moreover, geologic materials are important construction materials and the assessment of their occurrence and quality is also an important issue. In addition to the material properties, hazards such as active faulting, earthquake induced ground shaking, landslides, flooding, volcanic eruptions and groundwater conditions create problems on the safety and suitability of an engineering project. Hence, studies targeted to provide basic information for the planning of land use and for the planning, design, construction and maintenance of civil engineering works is very important. Such information is needed to assess the feasibility of the proposed land use or engineering undertaking, and for the latter to assist in the selection of the most appropriate type and method of construction, to ensure the stability of a structure in its natural setting and to aid the performance of necessary maintenance. Engineering geological research is, therefore, mainly directed towards understanding the interrelationships between the geological environment and the engineering situation; the nature and relationships of individual geological components; the active geodynamic processes and the prediction of processes likely to result from the changes being made (Price & Rangers, 1982). For each situation this implies a unique dynamic geological system of interrelated and interdependent phenomena and processes which can only be very incompletely understood. Therefore, research and studies in relation to the principal factors creating the engineering geological conditions of an individual site or area, i.e. the rocks and soils, water, geomorphological conditions and geodynamic processes is indispensable.

The area studied is in and around one of the fast growing town, Awassa, in Ethiopia. It is located in central part of the Main Ethiopian Rift, which is characterised by a number of lakes and active geodynamic processes. It is one of the important sites for settlement and development and lacks

any systematic studies related to the Engineering Geological conditions of the area. Hence the present work is carried out to fill this gap.

1.2 Relevance of the study

In Ethiopia, for many years the selection of a site for a particular land use, especially urbanisation is carried out without the basic understanding of the site geological conditions. The town of Awassa is similarly founded with out the understanding of this basic and important conditions.

Awassa is quite a recent town (founded in late 1950's), although some tribal settlements were formerly located in the area (NUPI, 1989). However, it has shown a dramatic growth since its foundation, particularly since it became the capital of the former Sidamo Administrative Region and especially its growth is fast presently being it is the capital of the Southern Ethiopian Nations, Nationalities and Peoples Administrative Regional State. The population has grown from 19,550 inhabitants in 1974 (NUPI, 1989) to more than 89,399 in 1999 (CSA, 1998). Many Engineering structures were constructed and presently there is a large need and trend to build up many new engineering projects. However, little was known about the Engineering geological conditions of the area. Moreover, the hinterland flooding due to the rise of Lake Awassa is becoming a threat for future development of the area. In view of this the present study is aimed at evaluating the overall engineering geological conditions of the area and pin pointing the main existing and potential hazards which may endanger or preclude the various types of civil engineering works that may be associated with the urbanisation.

1.3 Research Objectives

The main research objectives are:

1. To evaluate and characterise the Engineering geological conditions: hydrologic, hydrogeologic, geomorphic, and geological materials pertaining in the site and surroundings;
2. To produce, compile and evaluate geotechnical and Engineering geological

data of the foundation materials of the Awassa town, and

3. To produce an engineering geological map at a scale of 1: 10, 000 for the Awassa town area.

The specific objectives are: -

1. To describe the structure and attributes of the rocks and soils connected with engineering works.
2. To determine some of the important geotechnical properties of the foundation soil in the town.
3. To identify and evaluate specific engineering geological problems and natural hazards that may affect the success of engineering works in the town or the planned development of the town. Moreover, to suggest methodologies to minimise the effects of engineering geological problems.
4. To produce engineering geological map at a scale of 1: 10,000 accompanied with lithological and geotechnical description.

1.4 Approach followed and difficulties encountered

When this study was started little was known about the engineering geological condition of the area. Only very few site investigations were carried out at scattered places and the variation and type of Engineering geological conditions were not treated. However, related to the hydrogeological conditions of the area some important works have been carried out though some of them are regional. General accounts of the hydrogeology of the main Ethiopian Rift have been given by the Ethiopian Institute of Geological Surveys (Tesfaye Cherent, 1982), the hydrogeology of Awassa area by Dessie Nedaw (1997) and Liminological, hydrochemical, and palaeoclimatological studies have been made since the early 1960s (Talling & Talling, 1965; Baxter et al., 1965).

In the present study all the available geological, geotechnical, hydrogeological and meteorological information for the study area were collected, and the information obtained is

used in the planning and sitting of sampling points. Hydrogeological, geomorphological and geological conditions of the area were evaluated through an intensive field work and laboratory analysis of representative samples. The subsurface ground conditions were assessed with the help of new 14 test pits (2.5-5.0 m deep) concentrated in areas where geotechnical data are either scanty or non available (Appendix F). The wall of the test pits was examined to determine the sequence, thickness, classification and description of each stratum of material, and disturbed sample were obtained for laboratory analysis by trenching vertically into the trimmed side wall of the pits. A total of 48 disturbed samples were collected from these test pits and various laboratory tests were performed generally according to the procedures specified by the American Society for Testing and Materials (ASTM, 1996). Description for soils and rocks were made according to the International Association of Engineering Geologists (IAEG) (1981) recommendations. On site strength test for rocks is carried out using Schmidt hammer.

Soil testes were carried out at the laboratory of Civil Engineering Department, Technology Faculty, Addis Ababa University. Chemical analysis for some of the rocks and water samples was carried out at the central Laboratory of the Ethiopian Institute of Geological surveys, and tests for aggregate were carried out at the Building Design Enterprise.

Lack of in situ test equipment, undisturbed sampler and difficulty of transporting block samples made it difficult to carry out tests that require undisturbed samples.

Chapter Two

General overview of the study area

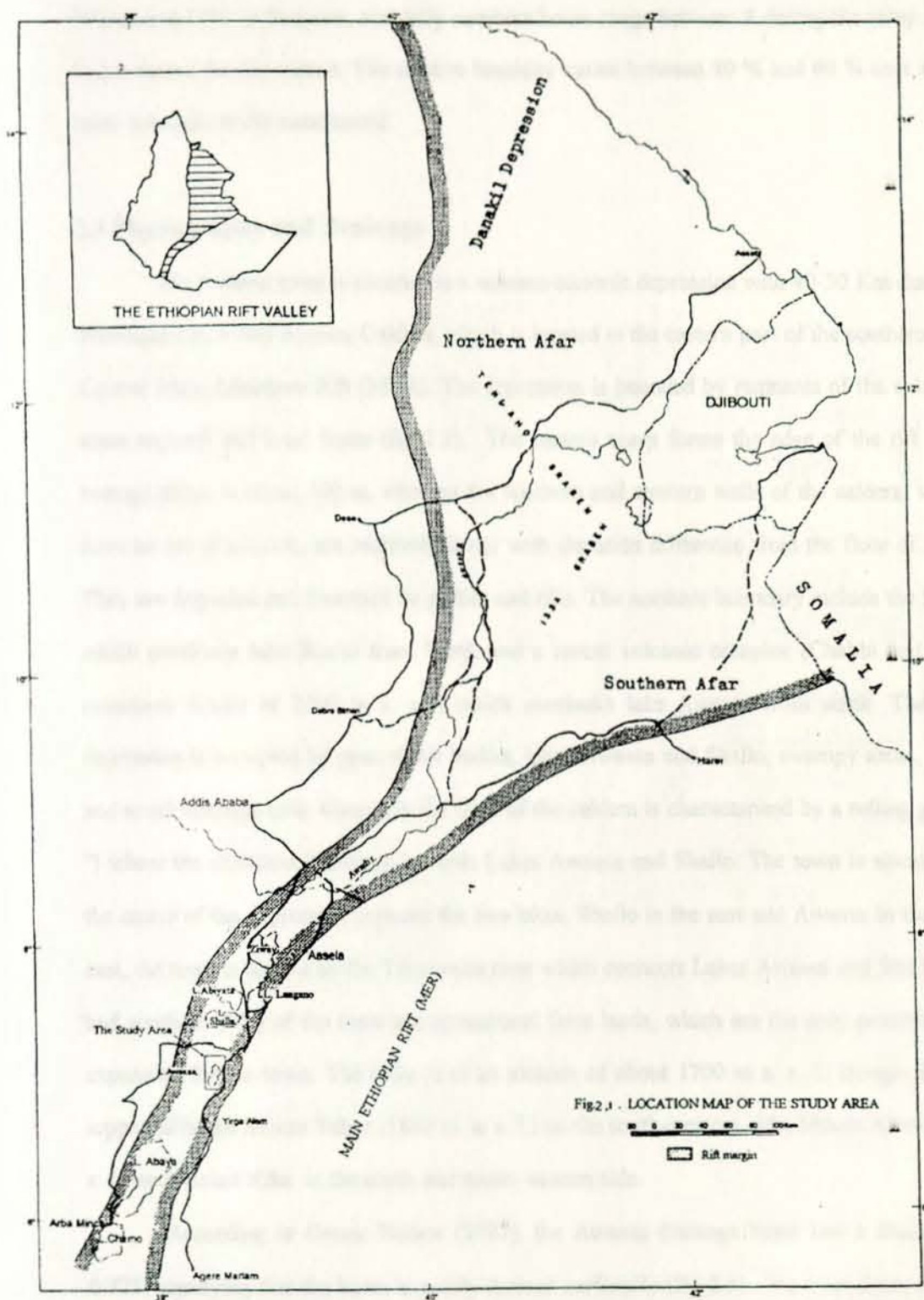
2.1 Location

The studied area is located in Southern Ethiopia some 275 km South of Addis Ababa (fig 2.1). It is bounded within the limits of $6^{\circ}48'N$ to $7^{\circ}14'N$ Latitude and $38^{\circ}16'E$ to $38^{\circ}44'E$ longitude. The area is inside the Ethiopian Rift which is one of the major portions of the so-called East African Rift System which extends from the Middle East in the North to Mozambique in the South passing through the Red Sea, Eritrea, Ethiopia, Kenya, Tanzania and Malawi with a total length of 6,500 km[?]. The investigated area is within the Awassa Catchment, which is one of the rift valley lake catchments, bounded by the Ziway-Shalla basin in the North and Northeast, the Bilate basin in the West and South and the Wabi Shebelle basin in the East. The area is accessible from Addis Ababa by the main road leading to Moyale.

2.2 Climate

The Awassa area has a sub-humid climate (FAO, 1984) and receives a mean annual precipitation of about 963 mm mainly concentrated in the eight-month-long rainy season. Rainfall is distributed in three main periods: the short rainy season ("Belg" rain) including the months of March, April and May; the big rains ("Kiremt") lasting across five months: June, July, August, September and October, and the dry spell from November to February. In the study area the wettest months are May to September with rainfall ranging from 101 to 128mm. The maximum monthly mean recorded is in July with 128 mm. The driest months are November to January (15 to 30 mm), with the lowest values recorded in December. The variability of the monthly rainfall amounts is generally high; the highest values of standard deviation characterise mainly April and May (49 and 51mm) and the months across the transition from the Kiremt to the dry spell, i. e., September and October (45 and 54 mm).

The mean temperature varies slightly through the year (17 to $20^{\circ}C$) characterising tropical climates (Billi, 1998). Daily maximum and minimum temperatures converge in August, the middle of the wet season, and diverge during the dry season. Generally temperature varies between $7^{\circ}C$ in



Winter and 33°C in Summer, and daily sunshine hours range between 4 during the rainy season and 9 hours during the dry season. The relative humidity varies between 40 % and 90 % over the year. The rainy seasons are the most humid.

2.3 Physiography and Drainage

The Awassa town is situated in a volcano-tectonic depression with 40-50 Km diameter (Giday Woldegabriel, 1986) Awassa Caldera, which is located in the eastern part of the southern sector of the Central Main Ethiopian Rift (MER). The depression is bounded by remnants of the caldera wall and some regional and local faults (fig 2.2). The eastern scarp forms the edge of the rift valley whose average throw is about 500 m, whereas the southern and western walls of the caldera, which roughly form an arc of a circle, are relatively lower with elevation difference from the floor of about 250 m. They are degraded and dissected by gullies and rills. The northern boundary include the Borbora ridge which overlooks lake Shallo from North and a recent volcanic complex (Chabbi and Urgi) with a maximum height of 2300 m a. s. l which overlooks lake Awassa from north. The floor of the depression is occupied by open water bodies, lakes Awassa and Shallo, swampy areas, Awassa town and small volcanic hills. Generally the floor of the caldera is characterised by a rolling ground (2 - 10 °) where the elevation decreases towards Lakes Awassa and Shallo. The town is situated roughly on the centre of the depression between the two lakes, Shallo in the east and Awassa to the west. On the east, the town is limited by the Tikurwuha river which connects Lakes Awassa and Shallo. The eastern and southern sides of the town are agricultural farm lands, which are the only possible directions of expansion for the town. The town is at an altitude of about 1700 m a. s. l., though there exist flat-topped hills like Mount Tabor (1800 m. a. s. l.) on the south-western side, Mount Alamura (2019 m a. s. l.) and Mount Kika in the south and south- eastern side.

According to Dessie Nedaw (1997), the Awassa drainage basin has a drainage density of 0.725, signifying that the basin is poorly drained surficially (fig 2.3). No river flows out of the basin and enters to the basin, thus the basin is closed for surficial flow. As the basin is characterised by a volcano-

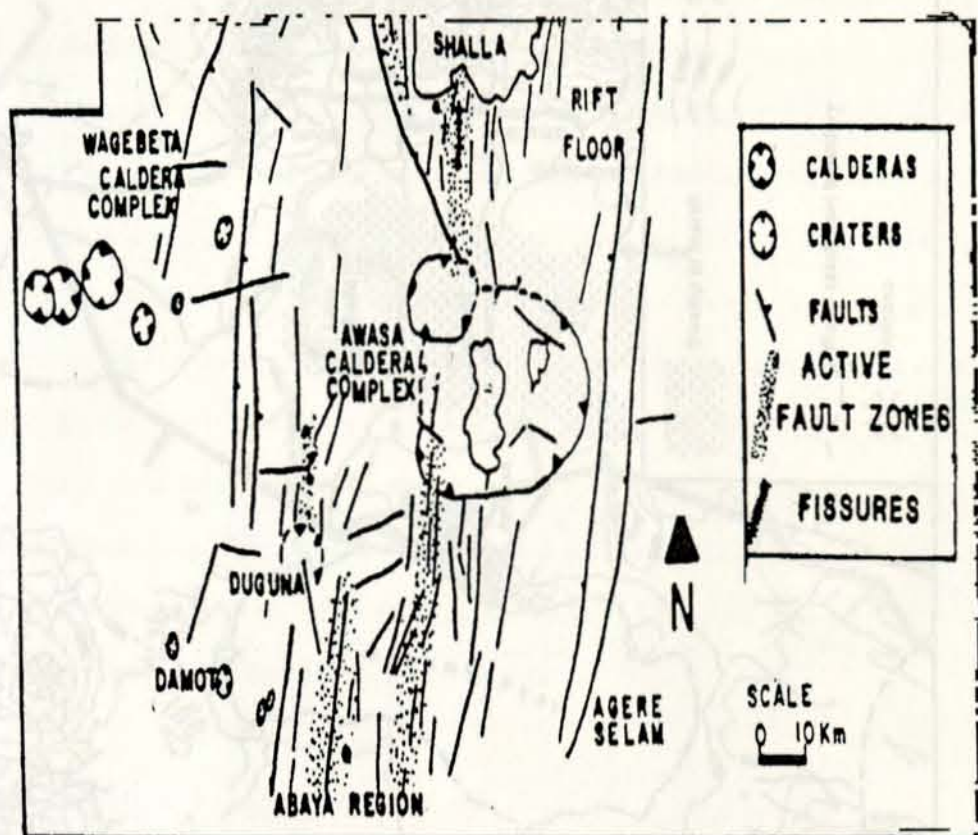


Figure 2.2 Structural framework of part of the central sector of the MER showing the Awassa Caldera Complex (modified after Woldegabriel, 1987)



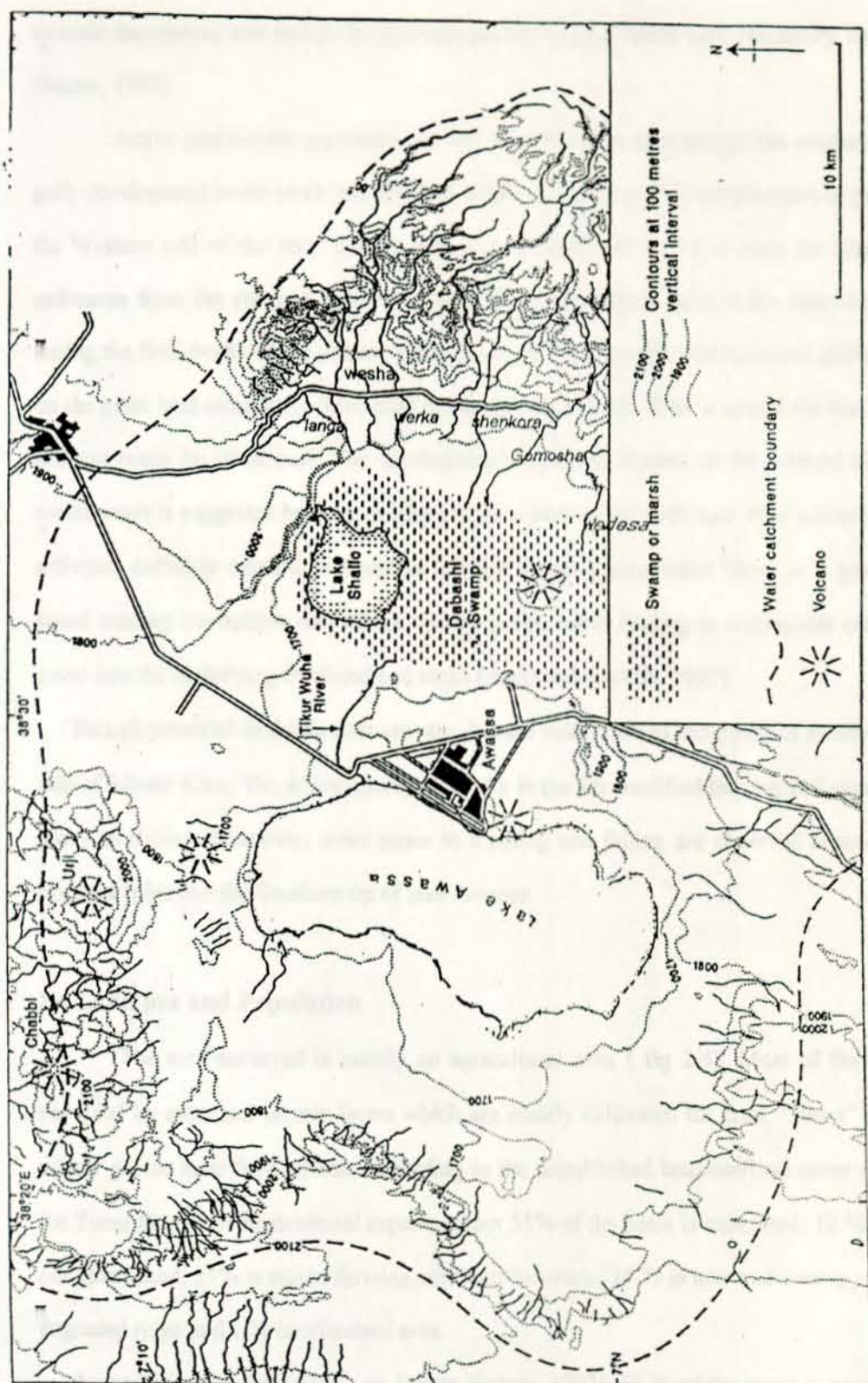


Figure 2.3 Drainage Map of the Study Area (modified after Telford, 1998)

tectonic depression, this makes the drainage pattern to be a radial type essentially centripetal (Dessie Nedaw, 1997).

Active geomorphic processes are not evident in the area except the erosional processes and gully development in the south western side of lake Awassa, and the development of ground fissures on the Western side of the lake. Gullies of 2 to 5 m wide and 3 to 4 m deep are actively discharging sediments from the elevated hills and ridges to the plain land close to the lake shore. As observed during the field works of the present study, sediments transported through these gullies are discharged on the plain land close to the lake than filling the water body. This is due to the loss of energy as the affluent reach the plain land. The development of ground fissures on the western side of the lake in recent years is suggested by some workers to be related to the 1960 and 1995 seismic activities. These activities probably resulted in fractures through which ground water flows at a greater than normal speed eroding the bottom part of the overlying soil cover leading to subsequent collapse of the soil cover into the underlying fractured bed rocks (AAU and MoWR, 1997).

Though potential landslide sites are rare, human induced mass movement is evident on the Southern side of Mount Kika. The active quarrying works in the site modified the exposed slope making the site prone to sliding. Moreover, areas prone to toppling and falling are observed around Kika along the Borbora ridge and the Southern tip of lake Awassa.

2.4 Land use and Population

The area surveyed is mainly an agricultural area (fig 2.4). Most of the flat-lying area is occupied by state and private farms which are mainly cultivated for corn. "Enset" is also one of the widely grown local food source. According to the unpublished land use/land cover map developed by the Zonal Bureau of Agricultural experts, about 55% of the basin is cultivated, 12 % is bushes, shrubs and grass land, 11% is mixed farming with agroforestry, 15 % is lake and swamp, 6% is bare soil or degraded ridge and 1 % is urbanised area.

According to Raunet (1974, in Dessie Nedaw, 1997), 50 % of the basin is covered by clay loam soil (especially the eastern half), about 30 % of the area, mainly West of Awassa, is characterise by a -loamy silty texture, 10 % is characterised by clay soil and the remaining 10 % is fine sandy loamy soil.

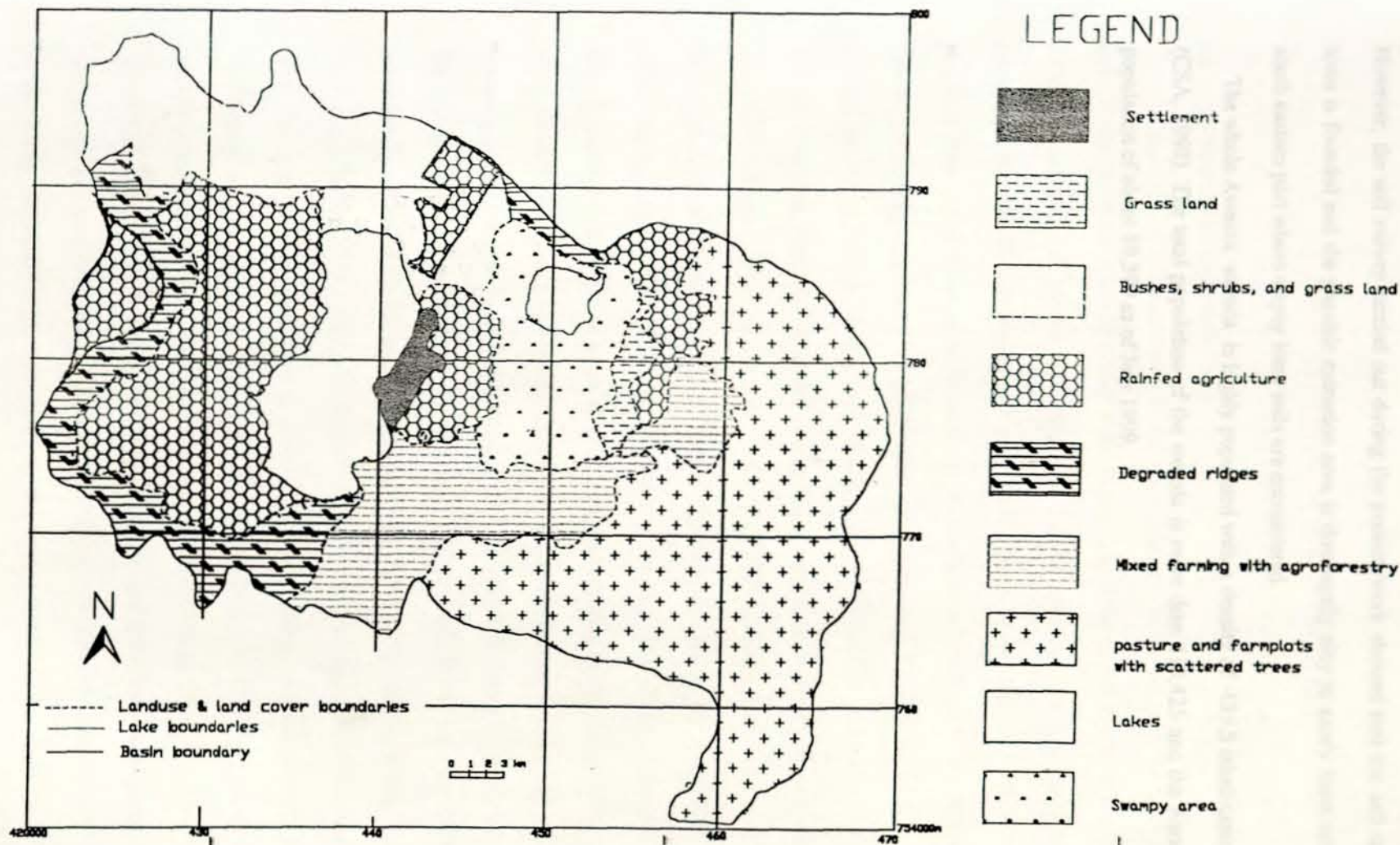


Fig.2.4 Landuse and landcover map of the study area
 (Modified after Awassa zonal bureau of agriculture)

Chapter Three

Geology, Tectonics and Stratigraphy

3.1 General Geological Setting

Volcanic and tectonic events in the study area constitute an essential part of the volcano-tectonic history of the Main Ethiopian Rift (MER). The MER is the Northern segment of the East African Continental rift system separating the 1000km wide uplifted Ethiopian volcanic province asymmetrically into the north-west and south-east Plateau. It extends in a NE/NNE to SW/SSW direction from Lake Abbe triple junction, in the southern Afar rift ($9^{\circ}30'N$), to the lake Turkana region, in the Konso highland ($5^{\circ}15'N$), and lies between long. $37^{\circ}30' - 40^{\circ}E$. It is a graben, about 800km long and up to 60- 80 Km wide, having a mean elevation of about 1600m (Mohr 1967). Its borders form flat shoulders of about 2200m high a.m.s.l. from which several rift margin shield volcanoes stalk out (Tesaye Korme et al. 1997). The MER is characterised by a great number of step faults which produced a total altitude difference of more than 1500m (Dipaola, 1972) between the top of the plateau and the floor of the rift. All these are normal step faults of various dimensions and throws, commonly arranged in an en echelon style and trending mainly along NNE- SSW and rarely along NE-SW, N-S and NW - SE directions (Mohr, 1967, Dipaola 1972) (fig 3.1). Actually, the presently (youngest) structural deformation of the Ethiopian Rift Valley is largely concentrated within 8 ± 2 km wide belt, characterised by the prominent N-S to NNE - SSW trending rift floor normal and occasionally dilatation faults, called the Wonji Fault Belt (WFB, Mohr, 1962). This fractured belt maintains a NNE (019°) structural orientation along the entire length of the rift, and has been forced into "en echelon" transpositions in order to remain within the rift margin envelope (Giday Woldegabriel, et al, in press). These sites of transposition are characterised by very recent and closely spaced normal faults, extension fractures (Chorowicz et al, 1994) and other fault-related open structures with significant volumes of fissural basalts and related differentiation products of very recent age and even historical (Dipaola, 1976). These structures are considered to be important for groundwater circulation and controlling the drainage system of the rift floor. The rift floor also comprises calderas which, according to

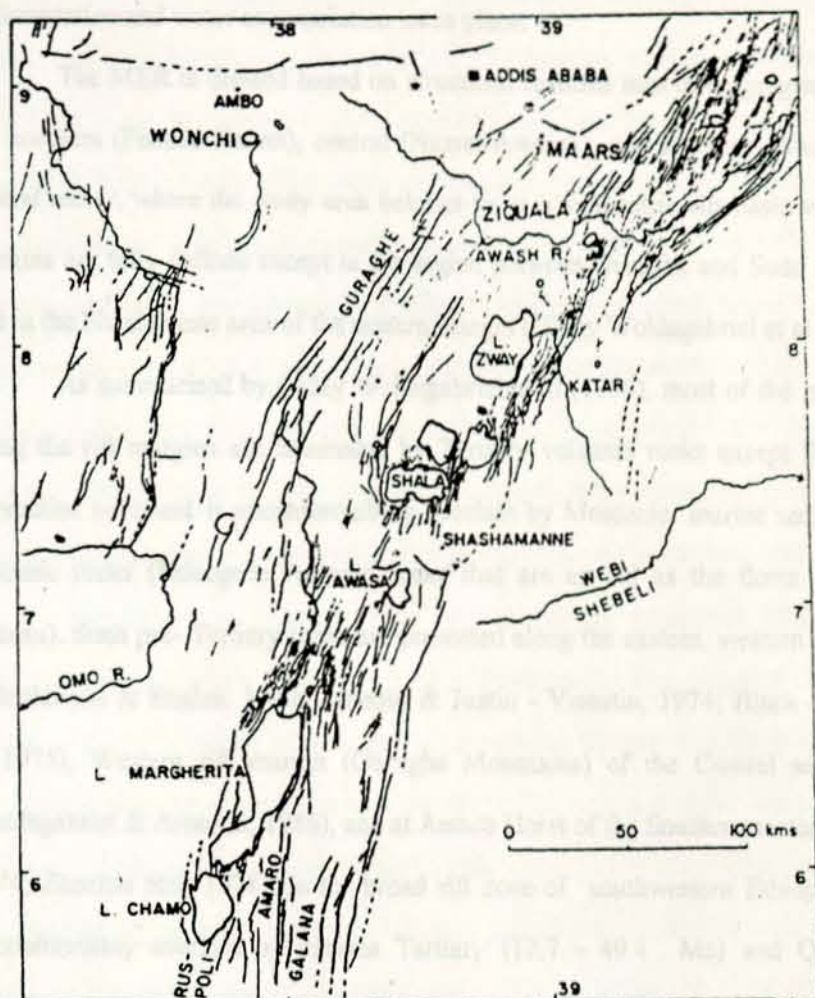


Figure 3. 1 Major structural features of the northern, central and southern sectors of the Main Ethiopian Rift and adjacent areas [after Mohr, 1980].

Mohr & Wood (1978), are generally elliptical in plan view with the long axes ranging in length between 2 and 17 km. Topographically, the axial part of the rift is the most subsided where intensive sedimentation and water accumulation takes place.

The MER is divided based on structural features into three geographic areas: represented by the northern (Fentale-Nazret), central (Nazret-Awassa), and southern (Awassa- Konso) sectors. The central sector, where the study area belongs to, is a symmetric rift basin where both sides of the rift margins are fully defined except in the region between Guraghe and Sodo of the western escarpment and in the Shashemene area of the eastern margin (Giday Woldegabriel et al., in Press).

As summarised by Giday Woldegabriel et al (1990), most of the geological sections exposed along the rift margins are dominated by Tertiary volcanic rocks except for a few locations where crystalline basement is unconformably overlain by Mesozoic marine sedimentary and / or Tertiary volcanic rocks (Paleogene volcanic rocks that are as old as the flows covering the northwestern plateau). Such pre- Tertiary rocks are presented along the eastern, western and southern Afar margins (Hutchinson & Engles. 1970; Zanettin & Justin - Visentin, 1974; Black et al., 1975; Chesse, et al., 1975), Western rift margin (Guraghe Mountains) of the Central sector of the MER (Giday Woldegabriel & Aronson, 1986), and at Amaro Horst of the Southern sector of the MER (Levitte et al., 1974; Zanettin et al., 1978). In the broad rift zone of southwestern Ethiopia, crystalline basement is unconformably overlain by various Tertiary (12.7 - 49.4 Ma) and Quaternary volcanic rocks, including the oldest known (Eocene) flood basalts in Ethiopia (Davidson & Rex, 1980; Davidson, 1983). This is contrary to previous suggestions that volcanism migrated from the northwestern plateau toward the Ethiopian rifts (MER and Afar) with time (Pilger & Rosler, 1975; Zanettin & Justin- Visentine, 1975; Morton, et al., 1979; Zanettin, et al., 1980, in Giday Woldegabriel et al., in press). On the other hand, the rift floor is covered by Mio-Pliocene to Present volcanism (Basaltic, trachytic and rhyolitic lavas, and pyroclastic flows and falls), and volcano - sedimentary products (Abbate and Saggri 1979 in Tesfaye Korme 1992).

The formation of the MER is attributed to extensional tectonics (Mohr 1967; Kazmin & Behere 1978), though the direction of relative motion between blocks (extension direction) is still controversial. From fault slip analysis two different directions have been proposed, northwest-southeast (Chorowicz et al, 1994) and northeast - southwest (Boccaletti et al., 1994). Moreover, there

is no universal agreement as to when the rift system began to develop. Many authors (Burke & Dewey 1970, and others) relate the development of the East African rift to the Afro-Arabian doming in lower Tertiary times. On the other hand, Mohr 1967; Behet, 1979; McConnell et al., 1980), described the Afro-Arabian rift system as an expression of a tectonic zone of weakness in the lithosphere which dates back to the Precambrian. According to Giday Woldegabriel et al., (1990), the development of the rift has been episodic rather than continuous, and a two-stage rift development is proposed based on structural and stratigraphic relationships from the Central sector of MER. The initial stage was characterised by the development of a series of half grabens along the rift with alternating polarity (in late Oligocene or early Miocene) and a second phase of symmetrical rifts that evolved from these half grabens in late Miocene and early Pliocene times. Accordingly, the same workers believe in the evolution of the rift from alternating half graben to full symmetrical graben with a medially located neovolcanic zone that is bifurcated to marginal grabens in the northern sector of the MER.

The formation of the rift has been intimately associated with widespread volcanism of different composition. According to Giday Woldegabriel and Aronson (1988); Hert et al (1986)(in Tenalem Ayenew, 1998), there exists a relationship between magma composition and rifting. Based on stratigraphic relationships and K/Ar dating of volcanic rocks from the escarpments, flanking plateaus, and from the rift floor of the Central sector of the Main Ethiopian rift, Giday Woldegabriel et al.,(1990) recognised six major volcanic episodes in the development of the rift over a time span from the late Oligocene to the Quaternary. The evolution of the Central sector of the MER is characterised by widespread and voluminous mafic eruptions in the Oligocene that were repeated during the Miocene time. However, during the Miocene and then volcanism was confined close to the future position of the rift.

3.2 Tectonics

The tectonic history of the present study area shares the tectonic activity and history of the MER. Batur et al, (1972) attributed the major uplift and graben faulting in the central sector of the MER to the early Pleistocene, however, Giday Woldegabriel et al., (1987) has presented abundant evidence signifying that faulting coincident or nearly coincident with the present margin has had a longer episodic history. According to the later, faulting commenced approximately in the early Miocene (as evidenced by the intercalated fluvial sediments with Oligocene lavas at Agere Selam) and

there were major episodes in the late Miocene (topographic confinement of the 8.3 Ma ignimbrite to the present main Guraghe escarpment), the Pliocene (the containment of the 3.6 Ma crystal rich tuff of the Butajira Ignimbrite with the rift margin) and the late Quaternary. The eastern rift margin in this sector of the MER is fully developed along its length except in the northeastern part of the Awassa caldera area. The age of the rocks exposed on the rift wall varies from upper Pliocene to Miocene in the middle section and Oligocene to Pliocene in the southern part of the central sector (Giday Woldegabriel, 1987). The total composite scarp height increases from the NE (Chilalo area), where the scarp rarely exceeds 100m exposing Quaternary rocks, to the SW (Agere Selam), where fault scarps of 500-800m are associated with Pliocene and other lavas (Giday Woldegabriel, 1987). This disparity, according the same author, has been caused by in filling of both lavas from the shield volcanoes at the rift shoulder flowing towards the rift and by ignimbrite from the rift floor where volcanism ultimately exceeded the rate of subsidence.

Similarly, the present western rift margin in this part of the MER has been formed by 9.7 Ma as demonstrated by the containment of during eruption of flows of these ages by the major fault scarps (Giday Woldegabriel, 1987). Accordingly, the central sector of the MER is formed in two stages: an early stage (late Oligocene-early Miocene) of a series of alternating half-grabens along the rift with major border faults on one side which was replaced in late Miocene-early Pliocene time by the Present stage of a mostly symmetrical rift. According to the same author, the present symmetrical rift was fully formed by 3.5 Ma.

The Pliocene age Awassa Caldera (Mohr, 1983 ; Giday Woldegabriel (1987); Giday Woldegabriel et al., 1990) underlain by ignimbrites (Dipaola, 1972) is by far the largest in the East African rift system, spanning half the breadth of the rift floor, approximately 50x40 km (Giday Woldegabriel, 1986). The caldera is composite and contains the younger 15 km diameter Corbetti caldera nested within it near the northern margin. It is located at the intersection of the rift axis by a major north-westerly lineament on the western plateau marked by strato volcanoes and three other small calderas, the Wagbeta caldera complex, extending 400 km from Awassa (Giday Woldegabriel, 1986).

The main Awassa caldera is asymmetrically overlapped and displaced against the eastern rift escarpment causing this rift margin arcuate, while Corbetti is along the rift axis. These calderas with

only slightly eroded walls are relatively young features along the eastern marginal graben of the rift floor. More than 500m of mostly air fall ash and ignimbrite are exposed in the caldera wall against the eastern escarpment (Giday Woldegabriel, 1987). However, the northern and southern walls of the main caldera are subdued perhaps due to rifting and burial along the MER. The northeastern wall of the Awassa caldera is poorly defined, though a NW-SE (N 45° W) fault (Werencha fault) with a down throw of about 120m to the south has exposed some of the stratigraphic successions. This scarp is beneath sediment, ignimbrite and young pyroclastics on the west of the Shashemene -Awassa highway, but east of the road it gains topographic expression eastwards. The scarp is made of dominantly of crystal rich, columnar jointed light gray ignimbrite footed and topped by pumice deposits. Blocks of ignimbrite prone to falling and toppling are evident. The Corbetti caldera is thought to be related to this transfer fault (Mohr 1976). Further east along this fault, near the eastern rift escarpment a large rhyolite volcano, collapsed on the south-west, is exposed. This was interpreted as a caldera by Dipoala (1972). The western wall of the Awassa caldera, i.e., the Abaya ridge, is well preserved and affected by the regional faults, WFB, running NNE-SSW.

The floor of the caldera is mostly covered by Lake Awassa and Shallo plus a marshy area. However, numerous smaller volcanic structures also occur within the caldera. South of Awassa town a remnant of what is probably an old rhyolite edifice rises above the plain. It is composed of rhyolite with layers of obsidian flow, tuff or pumice breccia, and ignimbrites covered by a layer of hyaloclastites all of which dip south and terminate abruptly on the north at an arcuate, eroded scarp. It is suggested by some workers (UNDP, 1973; Tesfaye Korme et al., 1997), that this scarp marks the edge of a cauldron subsidence. But evidence of the western and south-eastern caldera rim is lacking. It is also suggested that, the swampy plain east of Awassa occupies an old caldera as indicated by the arcuate arrangement of young extrusions within the caldera (UNDP, 1973).

The floor of the caldera is also highly dissected by normal faulting especially to its southern and western side (fig 3.2). Normal faults running in NNE-SSW (N 20° E) direction with down throws up to 100m are clearly visible cutting the young pyroclastic and lacustrine deposits. In the Southern side of Lake Awassa, a transverse escarpment with a northerly down throw is clearly evident, which has limited the size of the ancient large lake. Faults may act either as path ways for water movement or as flow barriers. At the foot of some of the faults which bound the basin there exist springs, hot and

cold, indicating that these faults act as conduits. Faults at the floor may possibly be filled with a weathered glassy volcanic ash and act as barriers.

The occurrence of differently oriented fractures is also noted in the caldera floor. Fractures running in N10° E to N 60° E and N100° E to N 115° E are evident especially on the flat topped hyaloclastites. The former group is dominant and run for longer distances than the latter which is infrequent and short. Most of the fractures are widely spaced, 0.5 m, filled and form irregular blocks. These fractures, especially the former ones may be signatures of the effect of the northwesterly lineament on the western plateau.

The occurrence of deep seated extension fractures in the study area is reported by Tesfaye Korme et al. (1997). According to these workers, the rhyolite ridge in the south east side of Awassa town, is a type-B volcanic neck. These ridges are short and steep hills of trachytes and/or rhyolite lava necks, commonly associated with calderas and are of the result of ejection of volcanic materials into radial or annular fractures formed by pressure exerted by the magma chamber. Their trends do not necessarily coincide with the strike of extension fractures in the region. It is suggested from their arrangement that, the young basaltic cinder cones of 0.52 ± 0.06 Ma age (Mohr, 1980) distributed in the caldera floor, may be similar injections into fractures. Columnar jointing is also observed in the ignimbrite beds located in the NW-SE transfer fault area on the northeastern and southern sides of Lake Awassa. All these various types of geological structures, especially the boundary faults, play a significant role in the recharging and circulation of water within and possibly out of the catchment.

3.3 Stratigraphy and Volcanism

A regional stratigraphy for the central sector of the MER has been provided by Dipaola (1972). Studies on specific segments of the central sector of the MER have been made by other investigators who have defined the major stratigraphic units in the region (Laury and Albritton 1975; UNDP, 1973). Subsequent detailed investigations and geochronologic studies further refined the stratigraphy of the central MER and documented the eruptive history of individual Volcanic centers within the region (Mohr et al. 1980; Giday Woldegabriel et al. 1990, 1992, in press).

Table. 3.1 Stratigraphy of the central sector of the Main Ethiopian Rift

(after Giday Woldegabriel et al., 1992)

Ethiopian Rift Valley 7° - 8°40 lat. north and adjacent plateau (Di Paola, 1972)	Central sector of main Ethiopian rift (Giday Woldegabriel et al., 1990)	Awassa caldera: Eastern wall (Giday Woldegabriel et al., 1990)	Aluto are (rift floor) Geothermal well LA-3 (EIGS, 1985)(Giday Woldegabriel et al., 1990)
Alluvium and lacustrine sediments (recent)	Silicic and mafic rocks of the Wonji Group (≤ 1.6 Ma)	Silicic and mafic rocks of the Wonji Group (≤ 1.6 Ma)	Silicic, trachytic and basaltic units of the Wonji Group (≤ 1.6 Ma)
Alkaline and peralkaline silicic flows (Holocene)			
Trachyte flows and basaltic tuff (recent Pleistocene)			
Alkaline and peralkaline silicic rocks (early Pleistocene – Late Pliocene)	Chilalo trachytes and silicic rocks (Late Pliocene, 3 - 1.6 Ma)	Chilalo trachytes (Trachytic tuff) (1.85 Ma)	Chilalo trachyte (Late Pliocene)
Alkaline and peralkaline silicic rocks (Pliocene)	Butajira ignimbrites (Pliocene, 4.2 - 3 Ma)	Butajira ignimbrites (3.55 - 3.69 Ma)	Butajira ignimbrites (Early to Mid - Pliocene)
Basalts and ignimbrites plateau trap series (pliocene - Early Eocene)	Guraghe basalts plus silicics (Miocene, 11 - 5 Ma) Shebele trachytes (Mid-Miocene, 17 - 12 Ma) Kella basalts (Oligocene, 31 - 29 Ma) Pre-Tertiary units (Mesozoic and Precambrian)	Guraghe basalts plus silicics flows (8.7 - 8.8 Ma)(Late Miocene)	

A summery of the stratigraphic nomenclature and K-Ar ages for the pyroclastic units in the MER is given by Giday Woldegabriel et al.(1992) and is presented in Table (3.1). According to this nomenclature, the oldest rhyolitic units in the region are silicic centers of mid- Oligocene to late Miocene age (31Ma and 10Ma) and several tuffaceous units ranging from Mid-to-Late Miocene age (Ca. 17-8 Ma). These silicic tuffs are intercalated with basaltic flows along the rift escarpment and shoulders of the central sector of the MER (Giday Woldegabriel et al., 1990). The Miocene silicic

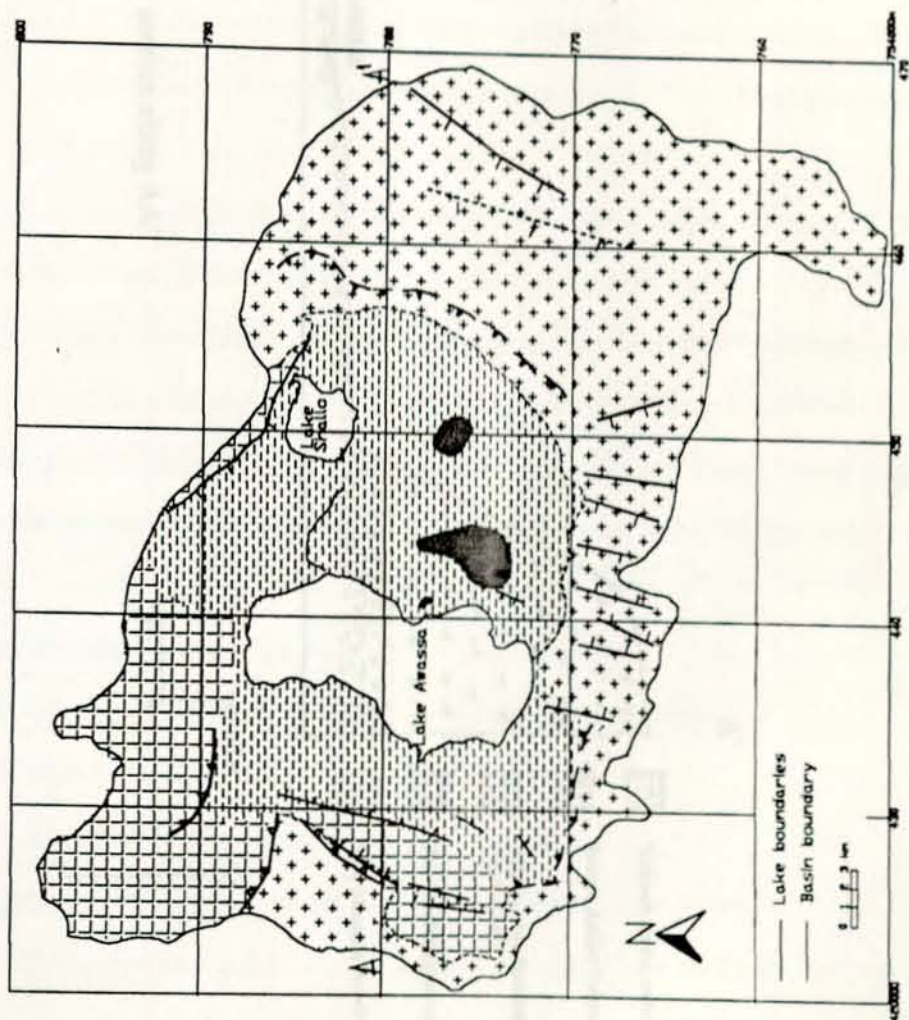
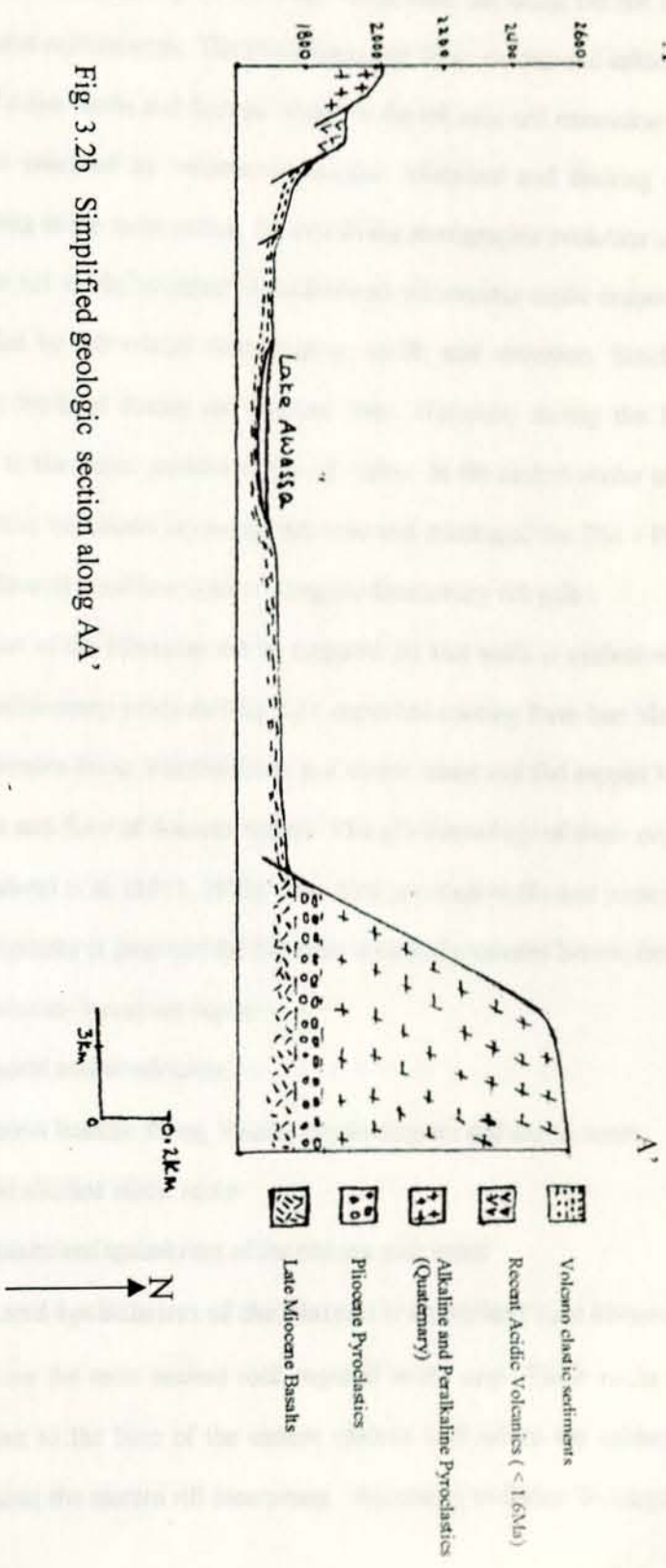


Fig.3.2a Geological map of the study area

Fig. 3.2b Simplified geologic section along AA'



rocks are smaller in volume when compared to the voluminous basaltic flows. According to Giday Woldegabriel et al. (1992), the Plio-Pleistocene units of the central sector of the MER are attributed to three time - stratigraphic groups: the Butajira ignimbrites (4.2 - 3 Ma), the Chilalo Trachytes (3 - 1.6 Ma), and the Wonji group (< 1.6 Ma), which crop out along the rift escarpments, fault scarps within the rift and caldera walls. The Plio-Pleistocene felsic centers and calderas generally occur at the intersection of major faults and fissures related to the rift axis and transverse faults. Most of the felsic eruptions were preceded by voluminous basaltic volcanism and faulting (Giday Woldegabriel, in press). According to the same author, the overall the stratigraphic evolution of the central sector of the main Ethiopian rift is characterised by widespread voluminous mafic eruptions in the Oligocene that were superseded by rift related downwarping, uplift, and extension. Similar volcanic and tectonic activities were repeated during the Miocene time. However, during the Miocene, volcanism was confined close to the future position of the rift valley. In the central sector of the main Ethiopian rift, rift - bound silicic volcanism increased with time and dominated the Plio - Pleistocene activity except for localised fissural basaltic eruptions along the Quaternary rift axis.

The part of the Ethiopian rift investigated for this work is exclusively made up of Volcanics and volcano-sedimentary products (fig 3.2), deposited starting from late Miocene to Present. several pyroclastics, rhyolite lavas, basaltic flows and cinder cones and flat topped hyaloclastites are exposed along the walls and floor of Awassa caldera. The geochronology of these deposits has been given by Giday Woldegebriel et al. (1987, 1990). Based on previous works and present field investigations, the following stratigraphy is proposed for the study area and presented below, from younger to older.

- Volcano- lacustrine deposits
- Recent acidic volcanics
- Recent basaltic flows, basaltic hayaloclastites and scoria cones
- Old alkaline silicic rocks
- Basalts and ignimbrites of the plateau trap series

3.3.1 Basalts and ignimbrites of the plateau trap series (Late Miocene)

These are the most ancient rock exposed in the area. These rocks are exposed locally at a quarry site close to the base of the eastern caldera wall where the caldera wall is asymmetrically overlapped against the eastern rift escarpment. According to Giday Woldegabriel et al., (1990) these

rocks are geochronologically grouped under Gurage basalts and consist of fine grained mugearite overlain by 30 to 50m thick welded tuff dominated by collapsed pumice clasts and sparse feldspar phenocrysts. They contain abundant welded shards which are moderately devitrified. These rocks are suggested to be the foundations on which the mega Awassa caldera was constructed, and have ages of 9.8 and 9.7 Ma, respectively (Giday Woldegabriel, 1987; Giday Woldegebriel et al., 1990). They belong to the trap series which is mostly made of basaltic lava flows with subordinately associated ignimbritic units intercalated between the basic lavas (Dipoala, 1972).

3.3.2 Old alkaline and Peralkaline Silicic rocks (Late Pliocene - Early Pleistocene)

This unit includes the rift pyroclastics and old rhyolite lava flows that are found. In the study area, these rocks are widely distributed and cover a large area. The caldera wall to the east and west are dominantly made of these formations. In the eastern wall of the caldera overlying the late Miocene welded tuff, ignimbrites and tuffs of this group are exposed. The age of these deposits ranges from 3.7 to 1.6 Ma (Giday Woldegabriel, 1987). Generally on this part of the caldera, overlying the late Miocene welded tuff, pumiceous deposits, laminated surge deposits which show cross bedding, a perlitic crystal rich tuff characterised by strongly welded shards, pumice fragments and few lithics are exposed, totally reaching a thickness of about 200 m. Rocks of these group are also exposed on the western side of Lake Awassa, especially on the western caldera wall where more than 200m thick unwelded pumice outcrop is exposed. The rhyolite ridges at Wondogenet (Mt.Abaro), at the southwestern edge of the Awassa caldera which has an age of 2.45Ma (Giday Woldegabriel, 1987) and probably the rhyolite necks at the south eastern side of Awassa town are also included in this group. These lavas show vertical sheeting and flow folding structures.

3.3.3 Recent Basaltic lava flows, basaltic hyaloclastites and scoria cone

(Recent Pleistocene)

This unit is represented by small monogenic cones which are sprinkled forming the only relief in the floor of the caldera. The scoria is found mainly associated with scoracious basalt flows. Its colour ranges from dark brown to reddish brown and the individual grains range from lapilli to bomb size. These deposits are extensively used as a construction material especially for road construction.

The flat topped hyaloclastites, generally yellowish to brown in colour, consist mainly a fine glassy material containing large (as large as 42 cm) blocks of basaltic, rhyolitic and ignimbritic material. In most cases these rocks show layering and cross bedding. These rocks are largely exposed in the eastern side of lake Awassa. For instance, mount Tabor, part of Tumura and Alamura are made of these rocks. Exposures of the basaltic flows are not common in the study area, though some are exposed in the southern side of the caldera. Mohr et al. (1980) reported an age of 0.5 ± 0.06 Ma for this rocks in the area.

3.3.4 Recent acidic Volcanics (<1.6 Ma)

These rocks are the youngest volcanic products in the study area. They are exposed in the north, east and North -western part of the water shade of lake Awassa basin. Most of these rocks consist pyroclastics, such as unwelded pumice flows, pumice falls and ashes, and rhyolites with associated obsidian flows. The products of Chabbi volcano, the rhyolite beds exposed on the NW-SE transfer fault east of Chabbi, the pyroclastic deposits overlying the Pliocene deposits on the eastern and western caldera wall, are included in this unit.

3.3.5 Volcano-Lacustrine deposits (Pleistocene to Recent)

The volcano-lacustrine deposits, which cover mainly the floor of the caldera are the only non volcanic formations in the area. These sediments are mostly of volcanic origin. Their thickness reaches up to 50m, especially on the western side of lake Awassa, as evidenced by bore holes log sections and hand dug wells. These lacustrine deposits are the result of the recession of lake Awassa which in the past occupied larger part of the floor of the caldera. Most of the lake sediments are intercalated with recent volcanic materials as seen in bore hole logs and sediment cores drilled on the lake floor (Telford, 1998).

Chapter Four

Hydrometeorology, Surface Hydrology, Hydrogeology and Hydrochemistry

4.1 General

Hydrogeological conditions, including surface hydrology affect land use planning, site selection, construction, performance, durability, cost and even the safety of engineering structures (Dearman, 1991). Ground and surface water also play important role in present day geomorphological processes, weathering, slope movements, volume changes by shrinking and swelling of susceptible soils. Groundwater may also influence methods of excavations by producing seepage forces and uplift pressures. Subsurface water is normally under pressure which increases with increasing depth below the water table to very high values. Such water pressures have a significant influence on the engineering behaviour of most rock and soil masses and their variations are responsible for changes in the stresses in these masses, which affect their deformation characteristics and failure (Bell, 1993). Moreover, rocks and soil properties are largely dependent on their moisture content.

Both surface and underground waste disposal may be influenced by hydrogeological conditions. The stability of slopes, the development of sinkholes and subsidence phenomenon is also governed by prevailing hydrometeorological and geological conditions. Hydrological conditions can also affect engineering structures through inundation, like the case in the Awassa area where the lake inundated more than 200 hectares of farm and grass land, and demolished more than 360 residential houses and governmental and non governmental organisation's buildings (MoWR, 1999). Flooding of rivers has also a paramount impact on the use of flood plains. Hydrometeorological parameters are also important in the designing of bridges, culverts and dams. Beach side erosion is also an important impact of water bodies which endangers beach side development.

The chemistry of water can also influence the development of an area, the use of engineering materials like concrete, timber and still. For instance, sulphate rich waters can attack concrete, and those of acidic waters can corrode steel structures.

Thus, in view of all the above, the aim of this chapter is to assess the overall hydrometeorological, hydrological, hydrogeological and hydrochemical condition of the waters available in the study area.

4.2 Hydrometeorological variables

Hydrometeorological variables are important in defining hydrological elements like precipitation, evaporation, runoff and infiltration which are in turn important variables required for defining and characterising the water balance of a given area. Hydrometeorological variables are also important in land use planning, design and site selection activities.

4.2.1 Precipitation

A 26 years recorded data obtained from National Meteorological Service Authority (NMSA) for Awassa station is analysed and synthesised. It is found out that the mean annual rainfall is about 963 mm. The mean monthly precipitation range from 15 to 128mm, and total annual precipitation ranges from 725 to 1200mm. There is an eight month long rainy season extending from March to October. All the rainy months have big rainfall, through the months of July, August and September are exceptionally the wettest months. The area gets more than 88% of the total rainfall during the eight rainy months and only 12% during the dry months. The monthly rainfall coefficient ranges from 0.3 (January) to 1.6 for the months of July, August and September (Dessie Nedaw, 1997). The precipitation of the area shows seasonal and inter - annual variability. For instance, rainfall amounts of above average by about 28, 20, 24, 23, and 17% were recorded in 1977, 1983, 1986, 1996 and 1998 respectively. Similarly, rainfall amounts below average by about 23, 18, 25, 21, and 22% were recorded in 1973, 1980, 1984, 1990, and 1993 respectively. Fig 4.1 shows seasonal and annual variation of rainfall in Awassa area for the periods specified.

Recorded rainfall data for the near by stations, Wondogenet, Morcho, Yirba Dubacho and Haisawaita are also available and considered (). The data show comparable range of seasonal and inter annual -variability to the Awassa station, which is preferably used in this work due to its closeness to the lake and its relatively longer record. These stations also revealed similar above annual average rainfall during the last three years, as large as 50% in the year 1996 (fig 4.2). Using the record from these stations and the corresponding influence area in the Awassa basin, the



Fig. 4.2 Annual rainfall for stations in and around Awassa basin

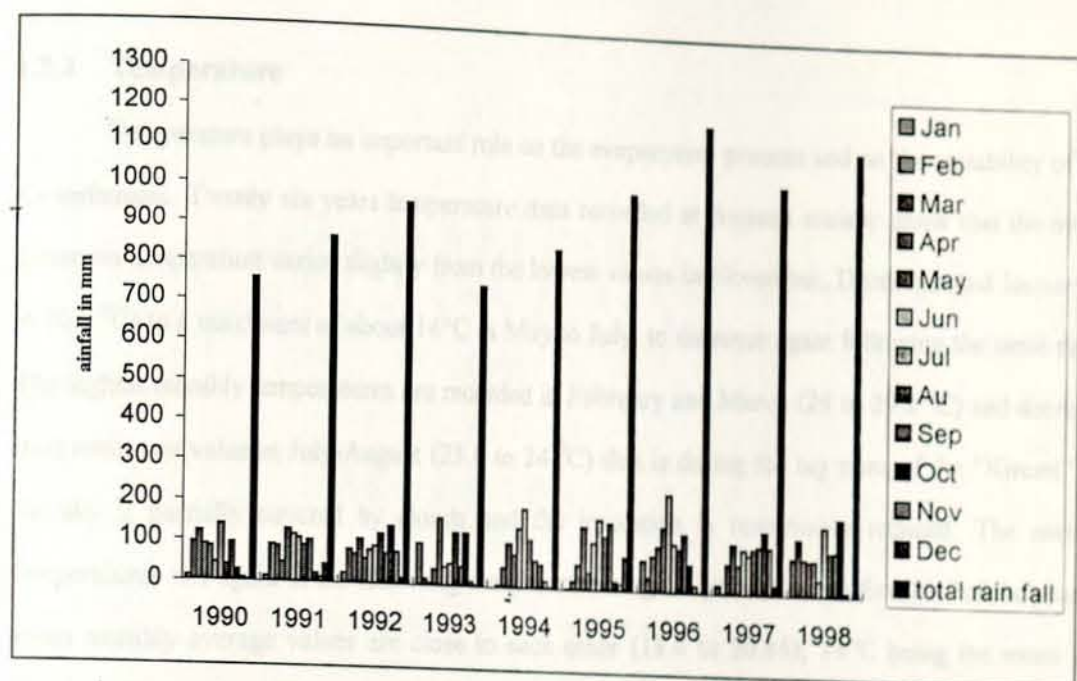


Fig 4.1 Seasonal and annual variation of rainfall in Awassa area for the period specified as recorded at Awassa station.

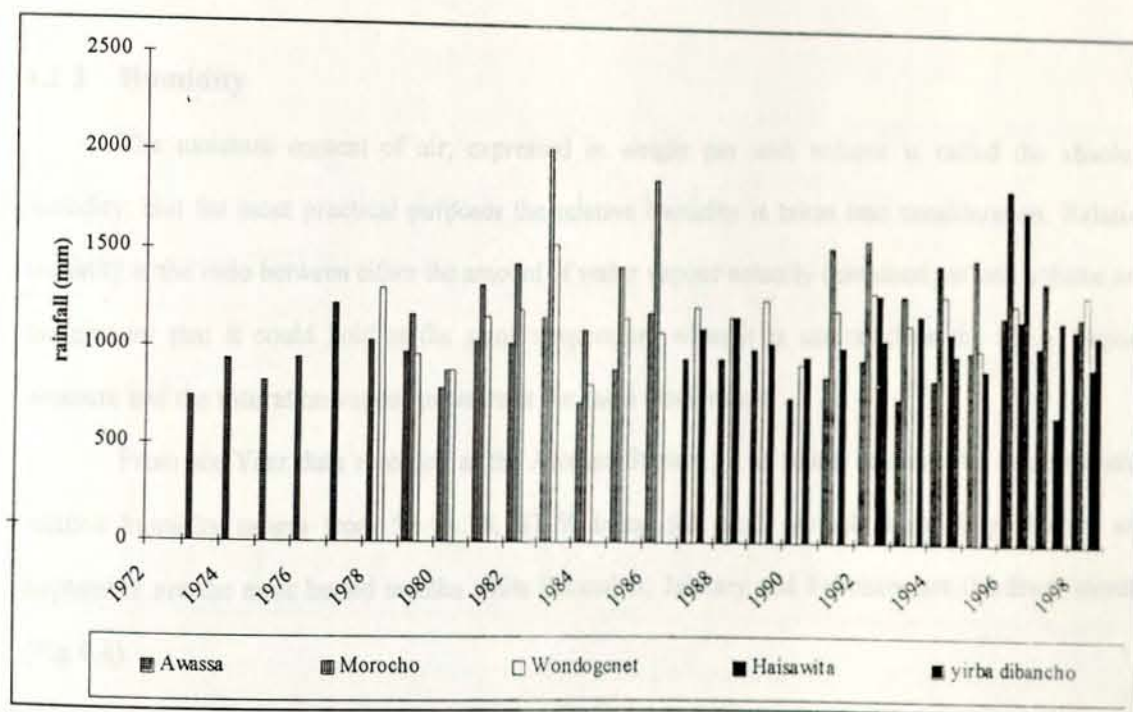


Fig 4.2 Annual rainfall for stations in and around Awassa basin

weighted mean annual rainfall of the basin, using Thiessen polygon method, is found out to be 1099mm.

4.2.2 Temperature

Temperature plays an important role on the evaporation process and on the suitability of a site for settlement. Twenty six years temperature data recorded at Awassa station show that the monthly minimum temperature varies slightly from the lowest values in November, December and January (9.1 to 10.2 °C) to a maximum of about 14°C in May to July, to decrease again following the same pattern. The highest monthly temperatures are recorded in February and March (29 to 29.2 °C) and decrease to their minimum value in July-August (23.6 to 24 °C) that is during the big rains of the "Kiremt" when the sky is partially covered by clouds and the insolation is remarkably reduced. The maximum temperatures rise again in the following months mirroring the pattern of the first half of the year. The mean monthly average values are close to each other (18.4 to 20.85), 19°C being the mean yearly average. Fig 4.3(a &b) shows the variability of mean monthly values as recorded from Awassa meteorological Station.

4.2.3 Humidity

The moisture content of air, expressed in weight per unit volume is called the absolute humidity. But for most practical purposes the relative humidity is taken into consideration. Relative humidity is the ratio between either the amount of water vapour actually contained per unit volume and the amount that it could hold at the same temperature when it is saturated or the actual vapour pressure and the saturation vapour pressure at the same temperature.

From six Year data recorded at the Awassa Station, it is found out that the mean monthly relative humidity ranges from 55 to 78, 67 % being the mean annual value. July, August and September are the most humid months while December, January and February are the driest months (Fig 4.4).

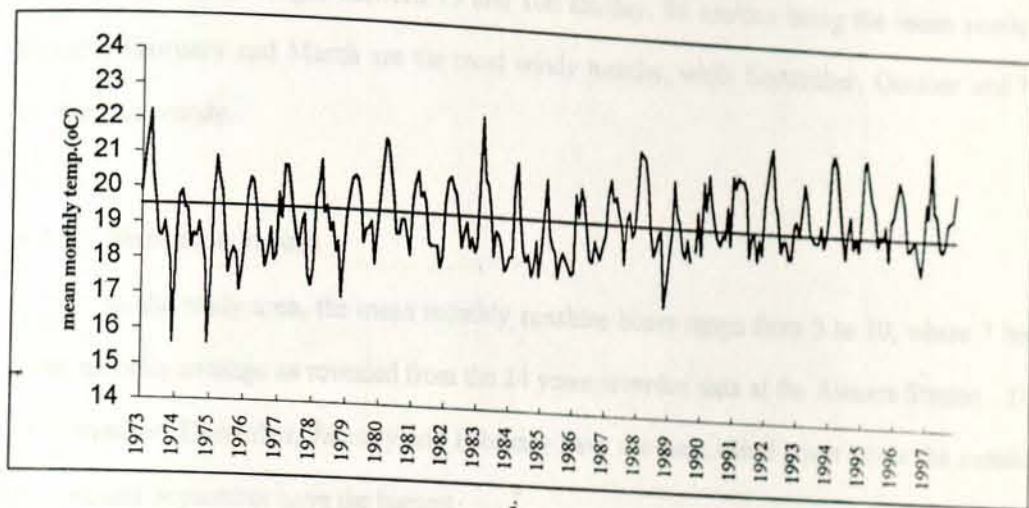


Fig 4.3a Mean monthly Temperature trends for the period specified

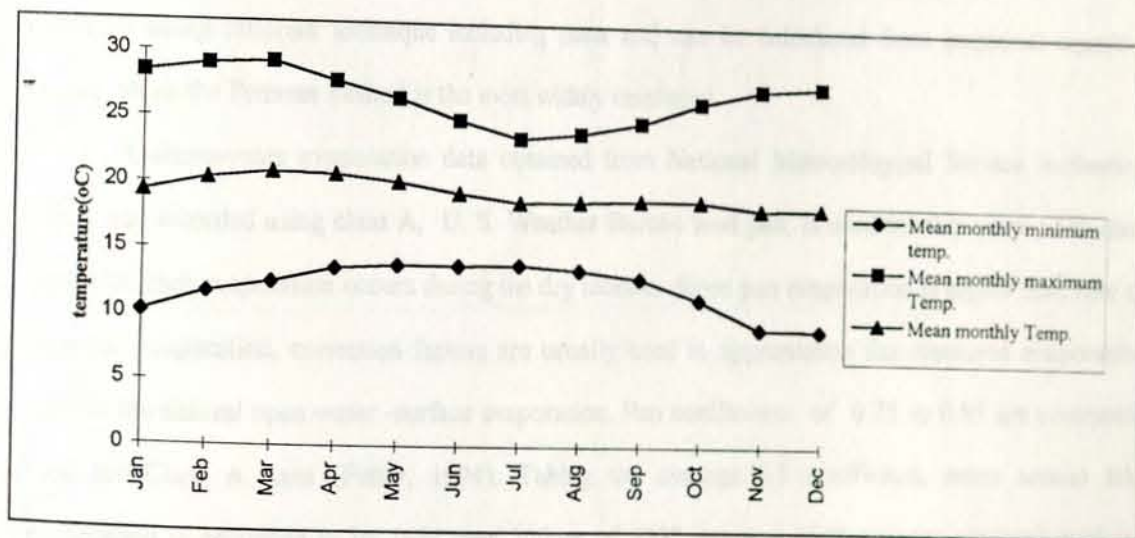


Fig 4.3b Mean monthly minimum, maximum and average temperature of Awassa area as recorded at Awassa station

4.2.4 Wind Speed

Wind speed has a strong influence on evaporation and is also important in highway and building design where it may be an important source of force acting on buildings and running vehicles. A twenty- years recorded data at the Awassa Station was analysed and it is found out that the mean monthly wind speed ranges between 73 and 106 km/day, 96 km/day being the mean yearly average. January, February and March are the most windy months, while September, October and November are the least windy.

4.2.5 Sunshine Hours

In the study area, the mean monthly sunshine hours range from 3 to 10, where 7 hours is the mean monthly average as revealed from the 24 years recorded data at the Awassa Station. The months of November, December, January and February have the least cloud cover while the months of July, August and September have the highest.

4.2.6 Evaporation

Evaporation is the mass transfer of moisture to the atmosphere from open surface water bodies including the oceans, soil and bare rock surfaces. Its rate depends on prevailing weather patterns including temperature, solar radiation, air mass saturation deficit, wind speed etc. It can be measured using different technique including pans and can be calculated from empirical equation among which the Penman method is the most widely employed.

Thirteen-years evaporation data obtained from National Meteorological Service Authority, which was recorded using class A, U. S. Weather Bureau land pan, is used for this work. The data show that high evaporation occurs during the dry months. Since pan evaporation is higher than lake or reservoir evaporation, correction factors are usually used to approximate the measured evaporation rates to the natural open water -surface evaporation. Pan coefficients of 0.75 to 0.85 are commonly used for Class A pans (Fetter, 1994). Taking the average 0.8 coefficient, mean annual lake evaporation is estimated to be 1658 mm. Values of 1358 mm and 1568 mm are obtained applying Penman and energy balance methods, respectively. However, the latter two values seem to

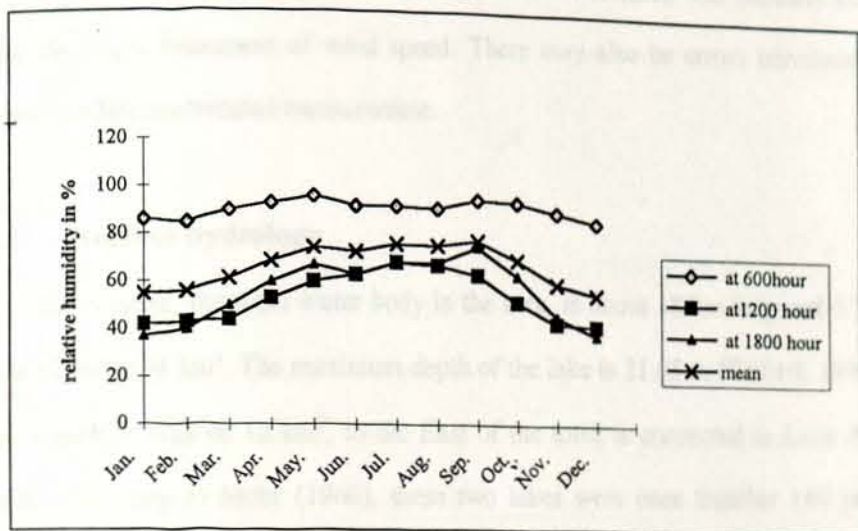


Fig 4.4 Mean monthly relative humidity (%) as measured at Awassa station

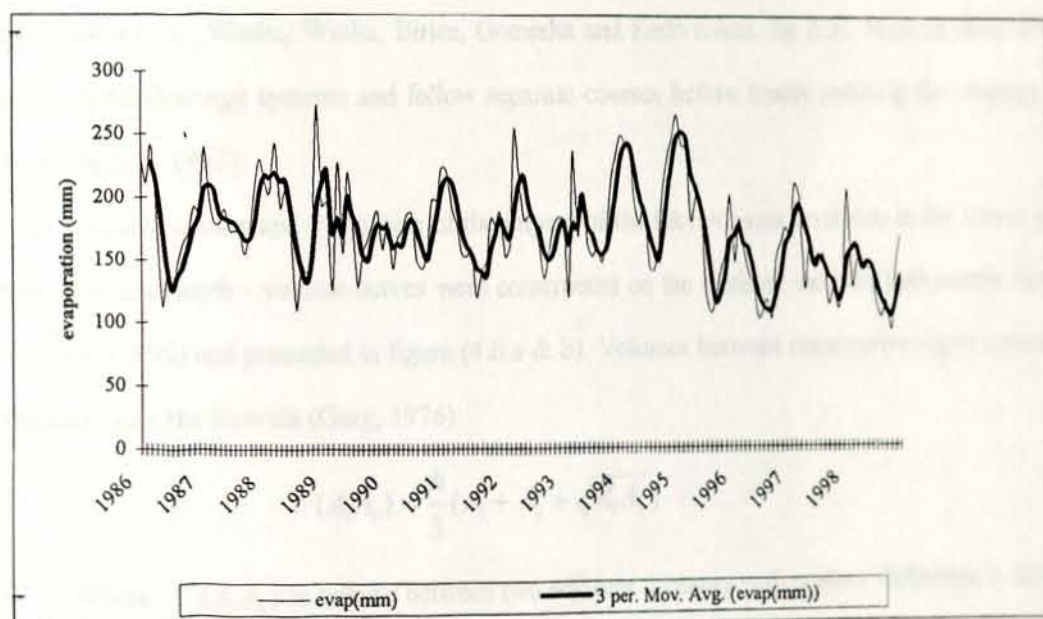


Fig 4.5 Monthly total pan evaporation for the period specified as measured at Awassa station

largely underestimated the evaporation rate as compared to the nearby Ziway-Shalla basin, which is greater than 1600 mm, hence the evaporation rate 1658 mm from recorded pan evaporation is used in this work. As seen in figure (4.5), evaporation is below average since 1996. This could be the effect of the increased rainfall during these times, which has increased the humidity of the area slightly, and also the slight decrement of wind speed. There may also be errors introduced due to measurement error, rainfall uncorrected measurement.

4.3 Surface hydrology

Lake Awassa, the main water body in the area, is about 18 km long and 5.5 km wide covering an area of about 94 km². The maximum depth of the lake is 21.65 m (Telford, 1998). Lake Shallo, which has a surface area of 12 km², to the East of the town is connected to Lake Awassa via Tikurwuha river. According to Mohr (1960), these two lakes were once together 140 years ago including the swampy area towards the east of the town which covers about 64 km². This swamp is now covered by junicas and papyrus plants and is a discharge area for the waters coming from the eastern and south eastern sides (i. e., Wesha, Werka, Birira, Gomesha and Kedo rivers, fig 2.3). Most of these streams have separate drainage systems and follow separate courses before finally entering the swampy area (Dessie Nedaw, 1997).

For characterisation and evaluation of the impact of the lake Awassa level rise in the recent years, depth- area and depth - volume curves were constructed on the basis of the lake bathymetry reported by Telford (1998) and presented in figure (4.6 a & b). Volumes between consecutive depth contours is computed using the formula (Garg, 1976)

$$V(A_1A_2) = \frac{h}{3}(A_1 + A_2 + \sqrt{A_1A_2})$$

Where $V(A_1A_2)$ is volume between two adjacent contours with contour difference h and A_1 and A_2 are areas enclosed by the upper and lower contours respectively.

4.3.1 Runoff

Considering the Awassa basin as a whole there is no river which flows out of the catchment and coming into it, so that for water balance computation of the whole catchment runoff value can

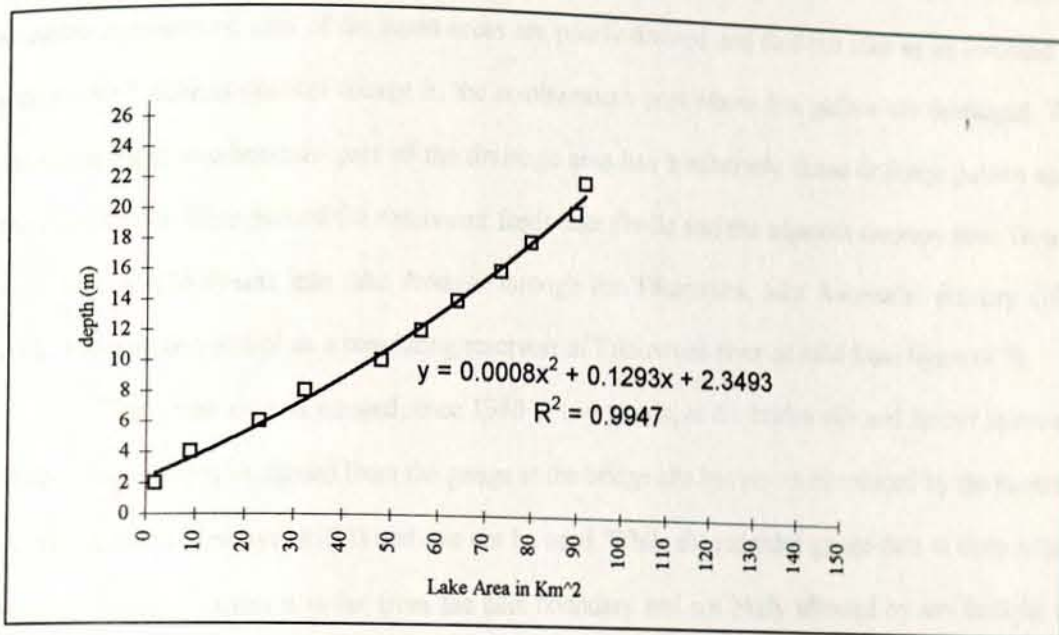


Fig 4.6a Depth Area curve for lake Awassa

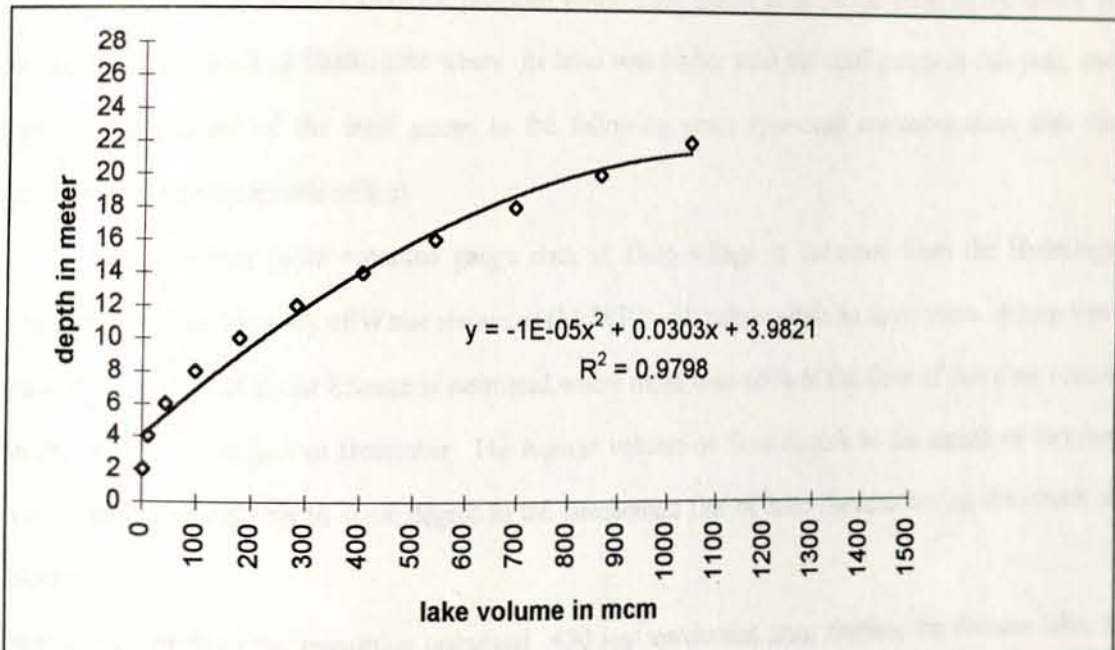


Fig 4.6b Depth volume curve for lake Awassa

be assumed zero (Dessie Nedaw, 1997). However, if we consider the surface drainage within the catchment, the runoff from the catchment area feed lake Awassa in two different ways. The northern, southern and western part of the basin areas are poorly drained and feed the lake as an overland flow without well defined channel except in the southwestern part where few gullies are developed. While the eastern and southeastern part of the drainage area has a relatively dense drainage pattern and all the runoff from these part of the catchment feeds lake Shallo and the adjacent swampy area. Overflow from lake Shallo drains into lake Awassa through the Tikurwuha, lake Awassa's primary affluent river. Lake Shallo serves as a regulating reservoir of Tikurwuha river as seen from figure (4.7).

Tikurwuha river is gauged since 1980 at two points, at the bridge site and farther upstream at Dato village. The flow record from the gauge at the bridge site has errors introduced by the backing up of lake Awassa (Tesfaye, 1982) and can not be used. While the recorded gauge data at Dato village is used in this work, since it is far from the lake boundary and not likely affected by any backing up of lake Awassa. However, as seen in figure (4.7), the level of Tikurwuha has gone above the level of Shallo since 1992, as differed from the previous years. This might arise due to some errors related to recording of the level of Shallo lake where its level was higher than the staff gauge in this year, and the re-establishment of the staff gauge in the following years (personal communication with the personnel of the responsible office).

The nineteen years recorded gauge data at Dato village is collected from the Hydrology Department of the Ministry of Water resources (MoWR) with missing data in some years. A long term mean annual flow of about 83mcm is estimated where more than 65% of the flow of this river occurs in the months of August to December. The highest volume of flow occurs in the month of October which likely contributing to some degree to the subsequent rise of lake Awassa during the month of November.

Surface runoff from the remaining ungauged 430 km² catchment area, feeding the Awassa lake, is estimated to be 62 mcm, assuming a runoff coefficient of 0.15 on the bases of the morphology, vegetation cover, and land use of the ungauged catchment. The contribution of direct surface runoff, therefore, to the lake Awassa hydrological balance is about 145 mcm, which is 62% of the total surface input.

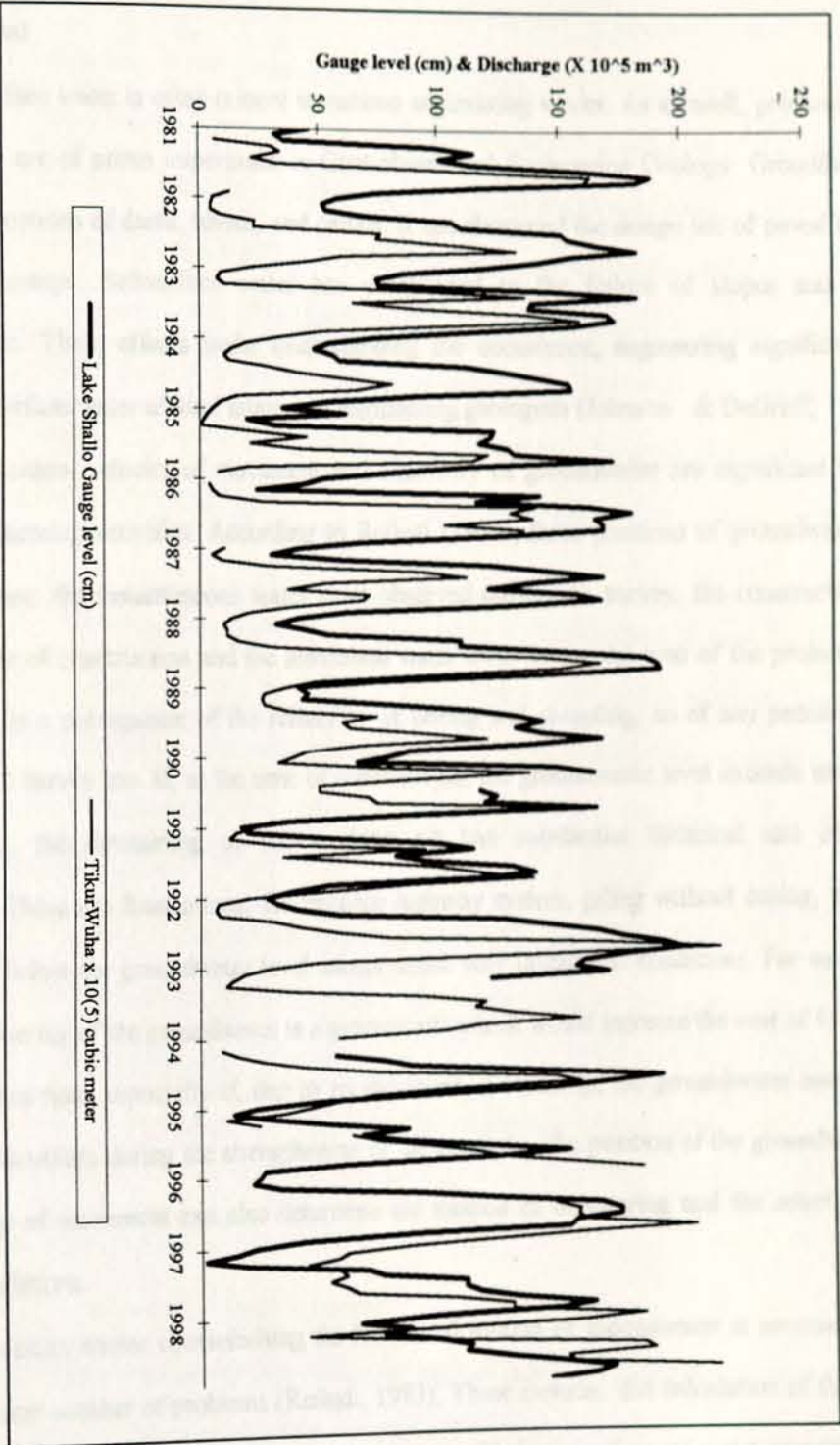


Fig 4.7 Lake Shallo Gauge levels (cm) and Tikurwuha Discharge ($\times 10^5 \text{ m}^3$)

4.4 Hydrogeology

4.4.1 General

Subsurface water is often critical in various engineering works. As a result, problems related to groundwater are of prime importance in Geotechnics and Engineering Geology. Groundwater has impaired the functions of dams, levees, and canals. It has shortened the design life of paved highways and airport runways. Subsurface water has contributed to the failure of slopes and building foundations, etc. These effects make understanding the occurrence, engineering significance, and control of subsurface water of vital interest to engineering geologists (Johnson & DeGraff, 1988).

The position, velocity of movement and chemistry of groundwater are significant for many aspects of engineering activities. According to Rethati (1983), three positions of groundwater are of prime importance: the instantaneous water level observed during the survey, the construction water level at the time of construction and the maximum water level during the time of the project. Among these, the first is a prerequisite of the reliability of boring and sampling, so of any pedological and geohydrological survey too. If, at the time of construction, the groundwater level exceeds the plane of the foundation, the dewatering of the working pit has substantial technical and economical consequences. There are foundations, for instance highway system, piling without casing, which can not be applied below the groundwater level except under very favourable conditions. For such type of foundations lowering of the groundwater is a prerequisite which would increase the cost of foundations and require extra time, especially if, due to an increased corrosivity, the groundwater must be held back from the structure during the strengthening of the concrete. The position of the groundwater level and its velocity of movement can also determine the method of dewatering and the selection of the system of foundations.

The velocity vector characterising the horizontal motion of groundwater is necessary for the solution of a large number of problems (Rethati, 1983). These include: the calculation of the swelling effect of subsurface walls, the judgement of the danger of infection of waste-water seepage systems made up of drain-pipes or shafts, the determination of the flow path of leaking water from basins and of corrosive compounds originating from industrial plants, and the assessment of the origin of waters seeping into basement and mine-spaces. Streaming groundwater may wash out the cement content of

fresh concrete (some foundations cannot be constructed in such cases) and it may increase the danger of corrosion.

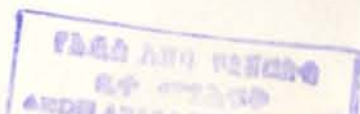
The position of groundwater is also important in relation to foundation stability against sliding, tilting and buoying, bearing capacity and settlement. The buoyant force, exerted by gravitational water filling up the pores becomes important during construction if it approaches the dead weight of the structure. With the exception of geometrical data, any other component of the bearing capacity of a foundation is a function of the groundwater stage. The upward movement of the capillary fringe due to a gradual rise of the groundwater can increase the water content of the soil, which ultimately decreases shearing strength and volume-weight of the soil. However, the nature and magnitude of the groundwater effect is a function of stratification and the physical properties of the soil. The bearing capacity of fine soils is primarily determined by cohesion which is largely determined by the water content.

Generally, the damaging role of groundwater to buildings can be active and passive (Rethati, 1983). In the first case, damage is caused by the mere presence of motion of the gravitational water, in the second category the effect is indirect: damage may be caused by the fact that the groundwater level is underneath the plane of foundation. According to the same author the damaging effect of water flowing below a foundation include:

- Deterioration of consistency of the subsoil with an increase of its compressibility
- Increase compaction of the soil, subsurface erosion and development of cavities due to leaching- out of the fine particles.
- The effect on the soil and the foundation of the chemical components carried by the water.
- Shrinkage and swelling of fine soils
- Subsidence of macro-porous and granular soils, etc.

Which of the above phenomena occur depends on the physical and chemical characteristics of the soil and the infiltrating water, the magnitude of the load on the soil, and the characteristics of the structure and the speed of flow.

A lowering of the water table itself may cause the motion of buildings, though most of the time the resulting movement is below the permitted level (Rethati, 1983). The development of high velocity



of flow during dewatering is much more dangerous. Pumping in the layers close to the ground surface lowers the groundwater level and this can initiate smaller or larger settlement of the buildings in the surroundings due to the reduction of natural stresses existing in the water supply layer. Similar cases may happen if excessive water is withdrawn from the lacustrine sediments of the study area.

The seasonal fluctuation of the groundwater table has also an important role. A change in the water level modifies the state of stress of a soil mass as well as the physical parameters of the affected layers causing deformation in the subsoil. When the groundwater level rises it diminishes the shear strength of the soils, increases its compressibility and side motion, may cause the collapse of loess soils, loose granular soils and up fills. A decrease in the level of the groundwater table may also cause a substantial shrinkage of the upper soil. Fluctuations of the groundwater table always result in soil compaction (Rethati, 1983). Fluctuations of the groundwater level in the study area reaches 4 to 5m which has a potential to cause similar effects or problems.

Hence, in view of the significance of the hydrogeological conditions, an attempt was made in this study to assess the overall hydrogeological features.

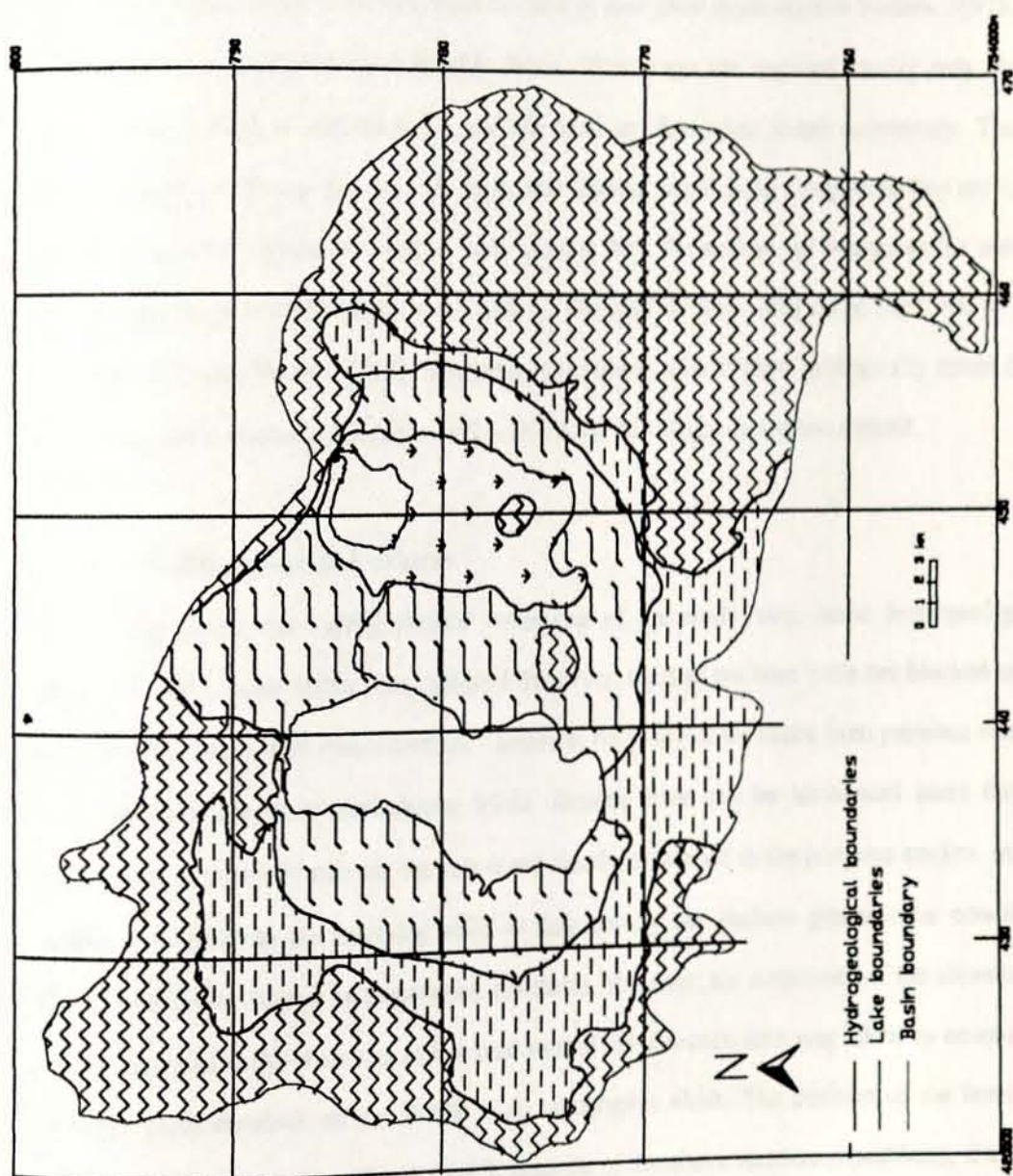
A number of hydrogeological studies have been conducted in the area (Grove et al, 1975; Tesfaye Cherenet, 1982; Dessie Nedaw, 1997; MoWR, 1999). However some of the studies are regional and the others are purpose oriented on localized areas for water supply and irrigation. Based on these studies and hydrogeological investigations carried out for the present study, it is tried to understand the hydrogeological condition around lake Awassa. In the region more than 40 bore wells and numerous hand-dug wells are found. The locations and the names of some of these wells and hand-dug wells used for this work are indicated in Appendix A. The depth of the bore wells ranges from 18 to 275m and the static water level varies from close to the surface to more than 100m. The depth of hand-dug wells reaches up to 50 m.

4.4.2 Hydrostratigraphy

From existing geological well log and geology of the area, the geological formations of the lake catchment are found out to be dominantly Quaternary acidic volcanics: rhyolites, obsidian flows and pumice falls; ignimbrites and tuffs, and volcano-lacustrine sediments, though there exist limited amount of scoria and associated lavas and hyaloclastites (fig. 3.2 & 4.8).

Volcano-lacustrine sediments:- dominantly distributed at the eastern and western side of the lake and they overlay the ignimbrites (Tesfaye Cherenet, 1982). They are characterised mainly by reworked pyroclastic materials with intercalations of water deposited volcanic ejectas and tuffs. These deposits are generally fine to medium grained and sometimes consist alternating fine and coarse beds. The lithology of the sediments is the function of their mode of origin (Tenalem Ayenew, 1998); that is whether they were quiescent lake deposits, water deposited volcanic ejecta, or coarse sediment introduced into the lakes by flood events. Some of the bore wells, especially in Shallo farm, and almost all the hand-dug wells extract water from aquifers of these materials. The groundwater is generally unconfined, and the sediments have medium to high transmissivity values, in the range from 52 to 2080 m²/day (MoWR, 1999) and yield of wells is often between 1- 8 l/s (Tesfaye Cherenet, 1982). Tenalem Ayenew (1998) has found transmissivity values of 10 to 520 m²/day and permeability values ranging from 3.1 m/day to 71.7 m²/day for similar deposits in the Ziway-Shalla basin. In the study area average thickness of the lake sediments ranges from 30 to 50 meters. The temperature of the groundwater varies from 21 to 30°C and conductivity from 230 to 2630 µs/cm (MoWR, 1999).

Recent acidic volcanics and pumice falls:- are dominantly distributed around the Corbetti caldera north of lake Awassa though they are also found to the northeast shade of the catchment, and as isolated lava flows in the caldera floor. The hydrogeologic characteristics of these deposits is highly variable and is highly controlled by the degree of fracturing and the degree of sorting of the pyroclastic material. The pumice falls are characterised by a high intergranular permeability which becomes highest when sorted (Tesfaye Cherenet, 1982). These volcanics show moderate to high degree of jointing which induces secondary porosity. Their thickness is so limited that, they are not common aquifers in the area. However, they are good areas of recharge. Tesfaye Cherenet (1982) reported permeability ranges of low to high (0.1 to 80 m/day). While Tenalem Ayenew (1998) reported values of the range of 1 to 2.5 m/day for similar volcanics in the Ziway-Shalla basin.



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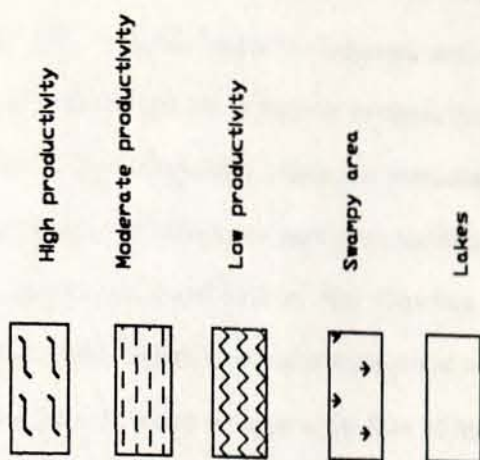


Fig.4.8 Hydrogeological map of the Awassa catchment
(adapted from Dessie Nadew, 1997)

Ignimbrites and tuffs:- outcrop in the eastern, southern and Western parts of the basin's water shade while they underlay lacustrine sediments around the lake. It is mostly well jointed especially columnar jointing and has a high to moderate permeability, depending on the degree of fracturing, but it has a low permeability where it is pumacious or massive (Tesfaye Cherent, 1982). Similar rocks in the Ziway – Shalla basin have permeabilities in the range of 0.02 to 41.6 m/day and transmissivity values between 1 and 1300 m² /day (Tenalem Ayenew, 1998). In the study area these rocks are as thick as 200m, though their higher topographic position forming the surface water shades of the basin favour them to act as recharge areas than as aquifer. Those found below the lacustrine deposits are also not that much important because of their great depth (Dessie Nedaw, 1997).

Scorias and associated basaltic flows:- these rocks are exposed locally only close to the textile factory East of Awassa town, but are good aquifers when found extensively. Though their importance as aquifers is very limited, due to their limited extent in the study area, they are found to be highly productive. There are about 9 wells tapping from the aquifer, all located in the compound of Awassa textile and Almesh cheep wood factory. The yield of these wells range from 300 to 740 liters per minute (Dessie Nedaw, 1997). According to the above author, these geologically recent deposits in most cases form unconfined aquifers with a thin cover of volcano lacustrine deposit.

4.4.3 Hydrogeological Features

To assess the hydrogeological conditions of the study area, some hydrogeological data, especially static water levels were collected. However, most of the bore wells are blocked and there is no access for water level measurements. Therefore, water levels are taken from previous studies which may introduce errors in groundwater levels. Besides error can be introduced since the depth of penetration of the wells into the aquifers is not clearly mentioned in the previous studies. In this work emphasis is given to the hand-dug wells as they portray the shallow groundwater condition which might have higher impact on engineering structures. However, the sensitivity of the altimeter used for measuring ground elevation was so low that most of the elevation data was taken by cross-referencing with the point elevation on the 10,000 scale topographic sheet. The position of the hand-dug wells were determined using Magellan PX GPS. Because of the above mentioned problems, it was not easy

to precisely define the actual local groundwater flow direction and velocity in the area. But it was possible to define a crude generalisation.

4.4.3.1. Depth to Groundwater Level

The groundwater depth generally increases as a function of the distance away from the Awassa lake. In the Southern and Southwestern side of the lake, groundwater depth increases from zero meter at loka spring, located at the shore of the lake, to more than 82 meters (MoWR, 1999) around the surface water divide of the lake catchment. The hand-dug wells measured on this side reach up to a depth of 50m and have an average water level depth of about 35m during the time of measurement (May, 1999). To the east and northeast of the lake, where the town is located, the density of hand-dug wells and bore wells is high. A Groundwater divide is identified between lake Awassa and Shallo, and the groundwater depth decreases towards both lakes. On the divide the groundwater depth goes up to 67 meters and decreases in both directions of the lake (MoWR, 1999).

The threat to the foundation of the town due to high water table conditions is anticipated only close to the lake, though for high rise buildings and sensitive engineering structures care should be taken since water level fluctuations may reach up to 4 to 5 meters. In addition, the water level fluctuations in lake Awassa may cause a local rise and fall of the water table in the surrounding area and even a reversal in flow direction.

In this work, it was planned to measure the range of fluctuation of the groundwater table in response to the rise of lake level and to assess its impact on the geotechnical properties of the foundation soil of the town. However, the attempt is not successful due to the dry period which prevailed in 1999. As defined by Rethati (1983), high groundwater level occur in the middle of late spring, after rainy seasons, and low water levels occur in late Autumn, after dry seasons, and higher drops of the groundwater table usually occurs after higher level of groundwater table conditions. He also pointed out that the limited amount of rainfall that spells during the dry seasons has no importance in recharging the groundwater. If similar conditions are assumed for the study area, the measurements made for this work (Appendix A) in early May after the high water table condition in the spring of 1998, due to high precipitation and increased lake level, may portray the lower water table condition which may be used for design purposes. The lack of a continuous recording

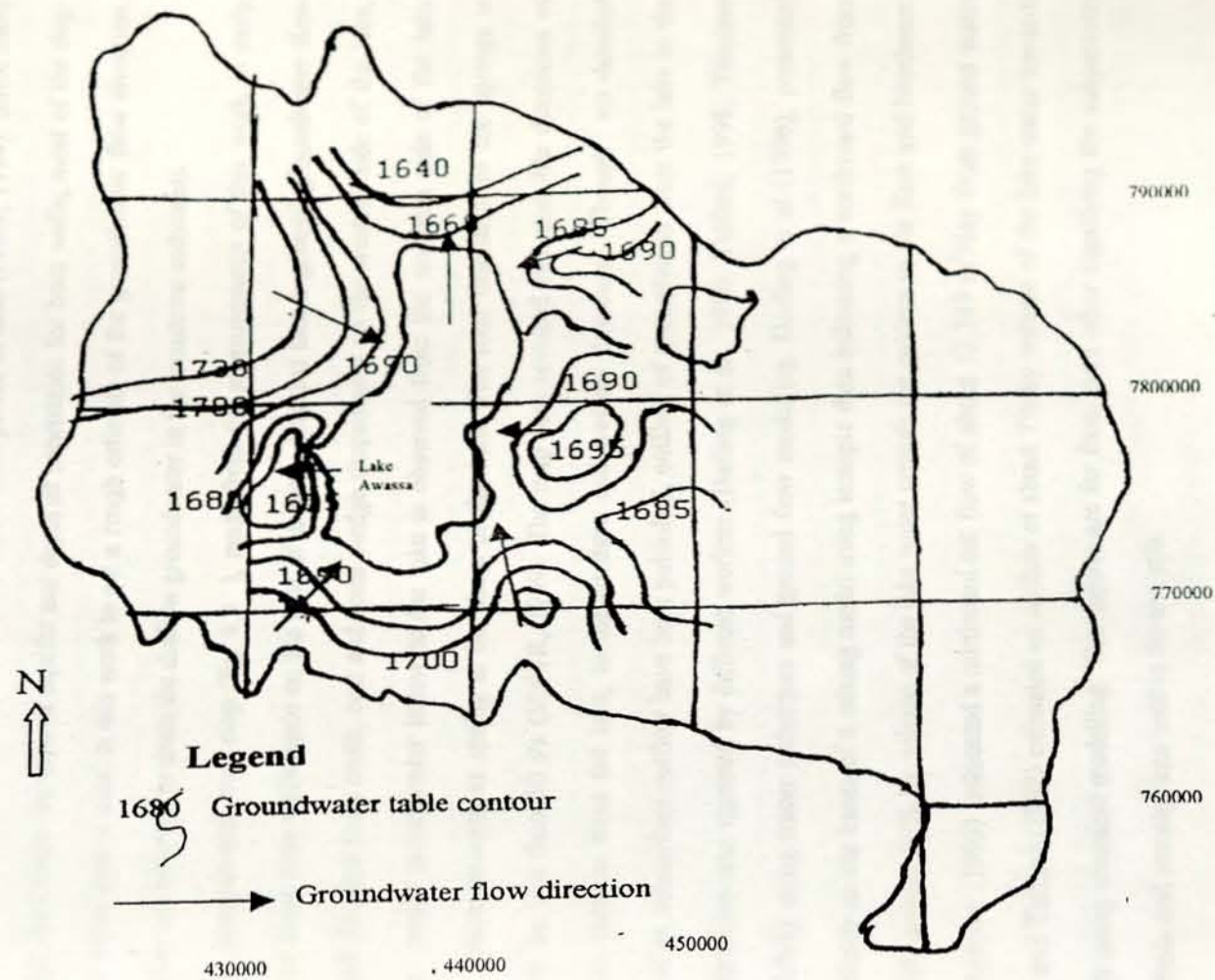


Figure 4.9 Groundwater Level Map
(Reduced from 1:50,000 scale map)

system of groundwater tables in the area made it difficult to predict the time and magnitude of high and low water levels, which are more important for engineering design.

4.4.3.2. Groundwater flow

Groundwater flow directions can be determined using groundwater table map constructed from water-level readings made in a number of water points, each of which is open only in the aquifer of interest, and all the readings should be made within a short period of time (Fetter, 1994). Since there is not enough data about the type of aquifer and depth of penetration for bore wells, most of the dug-wells water table data is used in this work to get a rough estimate of the groundwater flow direction. The dug-wells are believed to drain the shallow groundwater in the lacustrine sediments.

The groundwater level map (fig. 4.9), made from the measurements of this work in early May and for some bore wells made on the time of drilling, revealed that a general groundwater flow towards lake Awassa from south, east and some wedge shaped area on the western side of the lake, though the major groundwater inflow to the lake is observed from the southern side of the lake catchment which emerges as spring at the shore of the lake. The total discharge of the springs is estimated to be less than 50 l/s (MoWR, 1999). The map also revealed two possible directions of groundwater outflow from the lake, to the northern and western direction. Though no detailed estimates of its subsurface outflow have been published, outflow of groundwater from the lake in the northern direction was reported by different workers (Darling et al., 1996; Telford, 1998; Tenalem Ayenew, 1998) using tracer techniques and ground flow modelling. Darling et al (1996), presented strong evidence on the bases of a mixing model using isotopic data indicating a northward flow from the lake with around 50% by volume of the lake water outside the caldera to the East and Northeast. Tenalem Ayenew (1998) estimated a northward out flow of about 12,345 m³/day from ground water modelling and Telford (1998) estimated an outflow of about 7% by volume of the lake water towards lake Shalla using chemical modelling. These figures are far from each other signifying the subjectivity of the methods and assumptions used in the analysis.

4.5 Hydrochemistry

4.5.1 General

The chemistry of water (the water quality) has a significant effect on the development of an area. The chemical and biological characteristics of water determine its use for domestic, industrial or agricultural purposes. Besides the chemical constituents of water especially the groundwater may have a negative impact on engineering structures. The corrosive nature of some groundwater and the complex technical task in protecting underground structures against corrosion has been documented since long time. Recent researches on concrete also revealed the effect of the chemistry of water. The concomitant presence of sulphate and chloride ions in sea water and arid saline soils and groundwater causes deterioration of reinforced concrete structures due to reinforcement corrosion and sulphate attack. The reaction between the cement phase - tricalcium aluminate (C_3A) - and sulphate ions forms ettringite and gypsum if the (C_3A) content is above a certain level, which leads to degradation of concrete into a non-cohesive granular mass and disruptive expansion (Amoundi et al., 1994). Corrosion of reinforcement with subsequent cracking and spalling of the concrete cover is promoted by the penetration of oxygen and exacerbated by the ingress of free chloride ion from the pore electrolyte solution (Buenfeld et al., 1984).

Rethati (1983) categorised materials that are corrosive to concrete, mortar and certain stones and bricks according to their effects as follows, considering only the components of groundwater:

- a) Dissolving materials: those soft waters with a total salt content of less than 200mg/l, total hardness of 6 German degrees, or of alkalinity less than 2.5.
- b) Materials causing double decomposition: those acidic waters ($p^H < 7$), water containing alkaline bases (if $p^H > 9$), and salts that leach out, dissolve or transform the components of cement : magnesium-, ammonium-, zinc-, copper- salts (generally above 1000 mg/l), chlorides, nitrates and acetates (above 500 mg/l).

- c) Materials causing swelling: those solutions containing sulphate ions (SO_4) above a concentration of 400 mg/l and salts (e.g. sodium carbonate) which appearing in crystal form in concrete exerts a larger crystal pressure than the tensile strength of concrete.

Rethati (1983) also pointed out corrosion accelerators: streaming and repeated exchange of corrosive fluids, repeated drying out and wetting of the structure, repeated fluctuations of temperature and permanent contact between corrosive groundwater and concrete. These all show the significance of chemical characteristics of water in engineering practices.

Hence, an adequate solution to the problem of corrosion is possible only if an appropriate chemical and hydrogeological data is available. In this work it is attempted to better understand the chemical behaviour of both surface and groundwater using previous data and chemical analysis made for this work. Besides the chemical data is used to constrain and quantify the water balance components of the lake water and to assess the possibility of the use of the lake water for irrigation, which at the same time may reduce the problem of town ward inundation of the lake due to the lake level rise.

4.5.2 Major Cations and Anions

The water bodies in the Awassa catchment show a large variability in their cation and anion contents as shown in table 4.1. In almost all the samples the dominant cation is sodium, followed by potassium in surface water bodies in the rift floor and hot springs, and by calcium in upland rivers, cold springs, shallow dug wells and bore wells. Magnesium constitute the minimum in all the samples. Bicarbonate is the most dominant anion followed by chloride and sulphate. The water bodies in the floor of the caldera are of Na-HCO_3 type (fig.4.10) while the water bodies in the mountainous areas show the equal importance of Ca^{2+} and Mg^{2+} . The pH of the water samples in the basin ranges from 6.6 to 9.2. The high bicarbonate concentration in the existing pH range could be attributed to the dissociation of water and fluid rock interaction in the form of hydrolysis under the presence of soil carbon dioxide (Fetter, 1994) and/or the flux of mantle carbon dioxide which occurs in most parts of the rift (Berhanu Gezaw et al., 1996). The surface water bodies and cold springs are soft waters while the groundwaters and hot springs are in the range of medium to hard waters.

Type		Na ⁺ mg/l	K ⁺ mg/l	Ca ²⁺ mg/l	Mg ²⁺ mg/l	Cl ⁻ mg/l	F ⁻ mg/l	HCO ₃ ⁻ mg/l	NO ₃ ⁻ mg/l	CO ₃ ²⁻ mg/l	SO ₄ ²⁻ mg/l	SiO ₂ mg/l	Tot. Hardness mg/l	Temp. °C	pH	Cond. mS/cm	TDS mg/l
rivers draining the eastern side of the water shade	Average	26.02	8.37	18.68	3.67	4.96					4.90		49		7.56		
	Maximum	42.99	12.12	41.08	8.14	8.51					13.93		136		8.2		
	Minimum	10.81	5.08	8.22	2.19	2.48					1.44		28		7.1		
	stan. dev	13.75	3.16	14.60	2.52	2.23					5.11						
Shallo Swamp	Average	38.5	9.4	6.9	2.5	9.9	3.3				2.9		27		7.4		
	Maximum	82.3	21.1	11.6	4.13	18.1	5.5				6.24		46		7.8		
	Minimum	3.22	1.96	2	1.22	4.61	1.1				0.48		10		6.6		
	st. dev	40.2	10.3	4.8	1.5	7.2	3.1				3				0.7		
Tikurwoha river	Average	81	12.1	9.3	3.4	12.8		84.5			2.8		35		7.21		
	Maximum	178	22	13	5	19		151			4.25		47		7.7		
	Minimum	29.9	6.7	4	2	5.32		18			0.96		19		6.57		
	st. dev	60.1	6.2	3.5	1.3	6.9		223			1.55						
Lake Awassa	Average	184.3	33.4	10.3	5.4	33.1	8.2	384		104	17.1	90.1	38		8.9	841.6	908
	Maximum	235	45	50.5	15.2	48	9.7	445		185	43.2	130	77		9.2	1050	1063
	Minimum	137	24	3	1.2	13.8	5.9	288		13	0.01	43	20		8.7	741	765
	st. dev	26.8	6.6	11.4	3.2	8.8	1.5	54.7		77.15	17.1	28.9			0.2	95.5	128.6
Cold Sprigs	Average	24	4.4	12.8	5.7	7.3	0.9	46.3	7.6		8.5		56	20.8	7.6	242.8	155.4
	Maximum	37	5	24	10.7	18.1	2.1	146	13		46		104	28.3	8.3	305	311
	Minimum	11	3.91	6	3	4	0.1	4.88	0.4		0		27	20.5	6.8	208	104
	st. dev	13	0.55	5.4	2.6	4.5	0.8	65.8	4.4		15.6			2.5	0.6	33.5	71.93
Shallow Dugwells	Average	140.5	16.3	26.7	10	33.3	6.5	363		4.28	23.4		95				
	Maximum	270	23	47	20	62	10	743		18	110		167				
	Minimum	19	6	15	2	10	0.8	100		0	5		58				
Bore wells	Average	153.3	12.3	21.6	10.2	21.1	4.1	400	3.5		17.4	101.8	106.4	24.9	7.95	723	
	Maximum	235	20	36.8	21.6	60	11	915	8.8		70	150	170	31	8.7	1205	
	Minimum	18.1	3	6	1.2	2.5	0.1	0	0		0	88	20	20	6.8	90	
Hot springs	Average	414	39.5	25.1	5.8	80.3	4.9				104				8.2	2377.3	1765
	Maximum	714.7	67.3	44.1	9.1	153.8	12				198				8.6	3610	2713
	Minimum	180	19.9	5.6	1	11.5	1				0.96				7.6	1598	801
	st. dev	212.6	15.1	14.2	3.1	49.2	4.7				74.7				8.2	1079.9	917.3

Table 4. 1. Average Concentrations of Major cations and anions and other water quality parameters.

The major ion chemistry data also show some degree of change in concentration of natural waters in the basin as they flow from the mountains towards lake Awassa (table 4.1). The most dilute waters are found in streams, cold springs and groundwater in the upland areas. Lake Shallo (swamp) and the Tikurwuha show concentrations intermediate between upland waters and lake Awassa. Telford (1998) suggested evaporation concentration and the addition of saline thermal waters for the geochemical evolution of these waters.

The concentration of shallow groundwaters, measured in wells and bore wells, especially circulating in the lacustrine sediments, revolves around that of the lake, suggesting that these waters are either influenced by the lake water mixing or/and the chemistry of the aquifer material that is the lacustrine sediments. The most concentrated waters in the catchment are from the thermal springs which can be accounted for by the high degree of rock water interaction at high temperature.

The river water samples draining the mountainous areas show a highly variable ionic composition. However, the variability is larger for cations than anions which are dominated by bicarbonates. The data show the equal importance of Ca^{2+} and Mg^{2+} to Na^+ in these water bodies reflecting variations in the lithology of the rocks which they drain, where more basic volcanics exist. These waters have average pH of about 7.5 and $\text{Ca}^{2+} / \text{Mg}^{2+}$ ratio greater than unity.

Shallo swamp, Tikurwuha and Lake Awassa have lower concentrations of Ca^{2+} as compared to the other water bodies in the basin. This which could be due to the precipitation of calcite from solution and selective removal by micro-organisms. The presence of up to 27% calcite in Awassa lake and Shallo swamp sediments was reported by Telford (1998). The pH of these water bodies ranges from 6.6 to 9.2, and they get more alkaline towards the lake (8.9 on average).

Lake Awassa is found to be fresh (dilute) despite its location in a topographically closed basin as a terminal lake. It is an oligosaline (Telford ,1998) bicarbonate dominated lake where $\text{Na}^+ + \text{K}^+$ and $\text{HCO}_3^- + \text{CO}_3^{2-}$ constitute greater than 80%. Groundwater outflow towards the lower-lying and deep lake Shalla has been suggested (Telford, 1998) as the most plausible mechanism for maintaining the low salinity of the lake. Total dissolved solids values reported at different times show the dilution of the lake with time (Appendix B). This could be due to the high inflow than normal into the lake of low TDS waters.

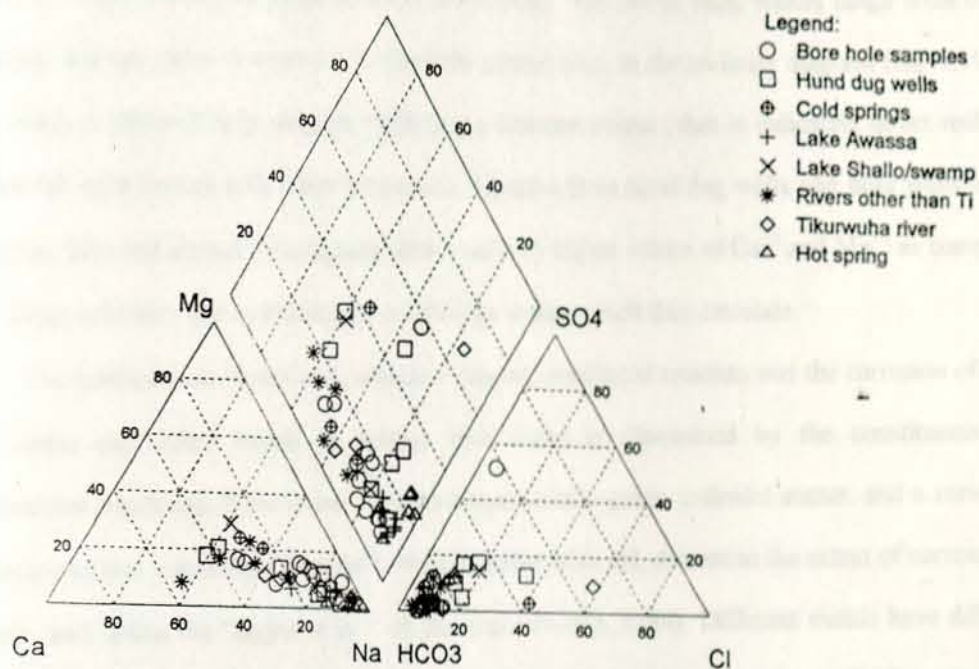


Figure 4.10 Piper tri - linear plot of the different water bodies

Shallow hand dug wells, cold springs, and bore wells also show high degree of variability in the major cations reflecting their degree of water rock interaction and variations in the lithology of the rocks within which they circulate. In all of these water bodies $\text{Ca}^{+2} > \text{K}^+$ which can be due to the dissolution of calcite in the soil horizons and/ or cation exchange: Na^+ for Ca^{+2} . The occurrence of exchangeable Ca^{+2} in the soils as large as 22% has been revealed in recent works carried out by Ministry of Water Resources (from archives of MOWR). The pH of these waters range from 6.5 to 8.7 and the average value is around 7.9. The cold springs plot, in the tri-linear diagram (fig. 4.10), in similar fields to those of river samples, reflecting a common origin, that is indicating direct recharge from rainfall with limited rock water interaction. Samples from hand dug wells and bore wells to the south of the lake and around Wondogenet show relatively higher values of Ca^{+2} and Mg^{+2} as compared to the others, probably due to differences in lithology within which they circulate.

The deterioration of concrete, asbestos- cement, reinforced concrete and the corrosion of iron, steel, copper and other metals in contact with water is determined by the constituents and environmental conditions of the water. Natural waters contain gases, colloidal matter, and a variety of electrolyte and non - electrolyte materials; these, together with pH, determine the extent of corrosion in a system and define the "aggressivity" of the water (WHO, 1984). Different metals have different range of stability under natural water systems. The Presence of anions that form soluble compounds with a metal increase the "corrosivity" of the water with respect to that metal, where as anions that form insoluble compounds may increase that metal's passivity (WHO, 1984). Comparing the chemical constituents of the waters of the study area with the limits given by Rethati (1983), as discussed in section 4.5.1, it is found out that the impact of the waters of the study area on civil engineering structures is little or none since the waters are generally far from the limiting conditions.

4.5.3 Water Quality

Whether a water of a given quality is suitable for a particular purpose depends on the criteria or standards of acceptable quality for that use (U.S environmental protection agency, in Todd 1980). In line with this quality limits, especially chemical constituents, of water supplies for drinking, industrial and irrigation purposes are used to asses the suitability of the water bodies in the study area.

Most of the water bodies within the Awassa catchment area have chemical constituents within the limit of the acceptable and allowable ranges for drinking water set by WHO (1984) except for some ions: fluoride, silica and in some cases bicarbonate. The cold springs and rivers are suitable for drinking where the fluoride content is exceptionally within the acceptable and allowable limits (table 4.1. and Appendix B). The source of high fluoride concentration in most of the waters of the main Ethiopian rift valley is not yet well known. However, it is suggested to be from chemical weathering of acidic rocks and magmatic intrusions (Berhanu Gizaw, 1996).

The suitability of water for irrigation is contingent on the effects of the chemical constituents of the water on the plant and the soil. Salts may harm plant growth physically by limiting the uptake of water through modification of osmotic pressure , or chemically by metabolic reactions such as those caused by toxic constituents. On the other hand salts may cause changes in soil structure, permeability and aeration which indirectly affect plant growth (Todd, 1980). The suitability of the waters of the study area for irrigation is assessed based on the irrigation water classifications schemes which employ sodium concentration. Sodium concentration is usually used in classifying an irrigation water because of its effect in reducing soil permeability and hardening of the soil (Tamiru Alemayehu, 1992).

The Sodium Adsorption Ratio (SAR) as recommended by Richard (1954, in Tamiru Alemayehu, 1992), given in equation 4.5.3, and the soluble sodium percent as defined by equation (4.5.3b, Wilcox 1955) are calculated for some of the water samples of the study area and it is tried to evaluate their suitability for irrigation using the Wilcox (1955) classification method (table 4.2, fig 4.11 a & b). It is found out that most of the water samples, except for some dug wells, rivers and cold springs, are doubtful to unsuitable for irrigation according to the above classification system. The SAR values also showed medium to high salinity risk in using this waters for irrigation. Particularly the Awassa lake water is totally unsuitable for irrigation.

	EC $\times 10^{-6}$	Na ⁺	SAR
BH-1	1245.8	72.9	6.7
BH-2	1162.3	70.9	6.1
BH-3	1259.5	76.4	7.5
BH-4	138.2	70.9	1.8
BH-5	1175.8	74.7	7.0
BH-6	265.9	51.3	1.5
BH-7	903.1	82.4	8.0
BH-8	1155.5	90.7	13.9
BH-9	1139.0	88.5	11.8
BH-10	909.2	87.7	10.2
BH-11	1113.0	77.7	7.4
BH-19	198.2	62.4	1.4
DW-1	596.8	44.2	1.7
DW-2	1040.3	66.4	4.9
DW-3	383.2	55.1	2.1
DW-4	239.6	51.6	1.1
DW-5	1020.1	91.0	12.9
DW-6	1453.2	83.6	10.8
DW-7	796.9	82.8	7.4
DW-8	1404.2	80.5	9.3
CS1	241.8	71.2	2.7
CS3	239.7	48.9	1.3
CS4	113.0	51.3	0.9

Rv1	308.6	69.6	2.8
Rv2	846.5	95.7	18.1
Rv3	308.0	83.8	4.7
Rv4	229.0	64.2	2.0
Rv5	567.9	85.7	6.8
La-1	1198.0	94.9	18.5
La-2	1120.0	93.9	16.3
La-3	893.0	94.8	15.9
La-4	1054.0	85.3	9.1
La-5	1074.0	93.7	15.8
La-6	1158.0	93.1	15.5
La-7	915.0	90.6	11.3
La-8	836.0	87.8	9.2
La-10	756.0	88.0	8.8
La-11	881.0	90.1	10.9
La-12	1303.0	74.4	6.7
La-13	1074.7	86.4	9.7
La-14	871.5	95.0	16.3
La-15	938.6	95.5	18.2
La-16	859.0	95.0	16.3
La-17	846.9	88.3	9.6
Las-15	195.0	73.3	2.5
Las-16	504.0	81.7	5.3
Las-17	39.0	48.7	0.4

Table 4.2 Soluble sodium percent and sodium adsorption ratio (SAR) for water samples from Aawssa catchment

$$SAR = \frac{Na}{\sqrt{\frac{(Ca+Mg)}{2}}} \quad 4.5.3a$$

$$\%Na = \frac{(Na + K)}{Ca + Mg + Na + K} \times 100 \quad 4.5.3b$$

$$1 \text{ meq/l of cations} = EC \times 10^6$$

The water quality standards for industrial use depend on the function of the industry. Recommended water quality criteria for selected industrial processes are given in Appendix C. According to this criteria, most of the waters of the study area are unsuitable for food processing, brewing and carbonate beverage industries which require more or less potable water. This is due to their higher concentrations of fluoride, silica and bicarbonate. Most of the waters are also unsuitable for paper industries and laundering. However, they can be used for textile, tanning and washing purposes.



Fig. 4.11a. Classification of water samples based on sodium and hardness (after WHO, 1958)

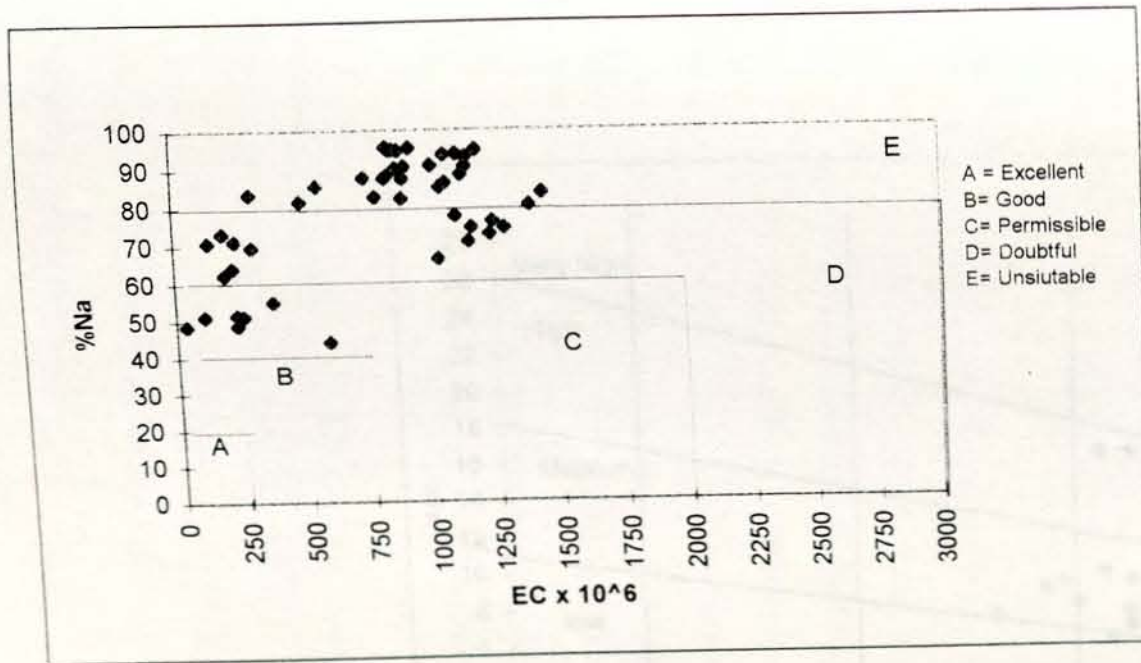


Fig 4.11a Classification of water samples from Awassa basin for irrigation using soluble sodium percent
(after Wilcox , 1955)

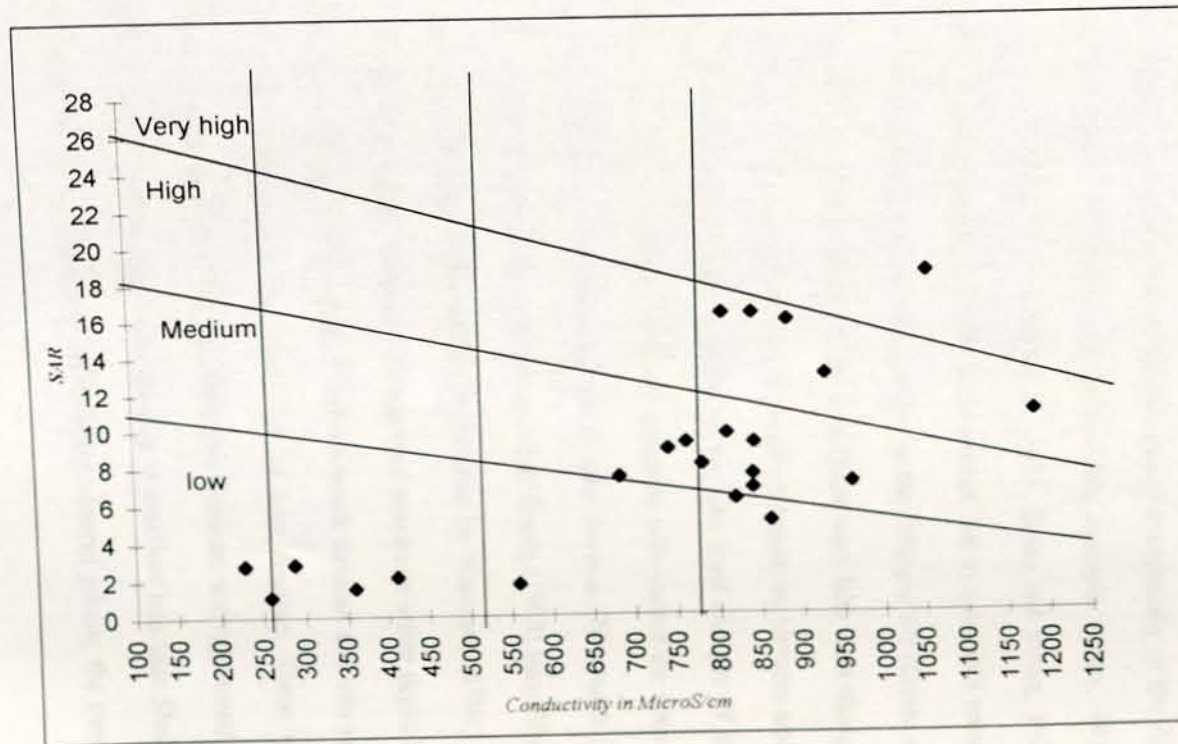


Fig 4.11b Classification of water samples based on salinity hazard (after Wicox, 1955)

Chapter five

Recent Lake Awassa level fluctuations and Lake water balance estimation

5.1 Recent Lake Awassa level fluctuations

Late Quaternary lake level changes in the Ethiopian rift especially in the Ziway-Shalla basin (north of the study area) has been studied and reported (for example, Leaky, 1931; Nilson, 1940; Bishop, 1976; Grove and Goudie, 1970; Grove et al., 1975; Gasse and Street, 1978a; Street, 1979; Benvenuti et al; 1995; Tenalem Ayenew, 1998). Some studies led to a detailed reconstruction of Late Quaternary lake level oscillations and climatic changes in the Ethiopia rift (Gasse and Street, 1978a; Street, 1979; Benvenuti et al., 1995). However, the Late Quaternary lake level changes in lake Awassa is not studied in detail, probably due to lack of well preserved lacustrine terraces and strand lines.

Relatively few historical data are available on the lake level changes of lake Awassa before 1969. As pointed out by Telford (1998), European explorers who visited the area at the close of the 19th century provide the first written descriptions of lake Awassa. Neumann (1901) found lakes Awassa and Shallo united, at least in the wet season, while Bourq (1903) travelling in 1902, drew the lakes as being separate, with lake Shallo smaller than drawn by Neumann (1901). Baxter et al (1965 in Telford et al. 1998), reported the change in the recorded maximum water depth from 21.6m in 1937 to 17.8m in 1964. Grove et al (1975) reported undated beach terraces cut into volcanic debris up to 40m above the modern lake level to the north-west of lake Awassa. These volcanic debris are probably Holocene in age (Telford, 1998) as Holocene volcanic activity would have buried earlier terraces. This rise is insufficient to cause lake Awassa to overflow into lake Shallo, for which a 60m rise is required, and there is no evidence of an overflow channel joining the two lakes (Grove et al., 1975).

Lake level measurement of Lake Awassa was started in 1969 with daily and monthly records. The monthly lake level records since October 1969 till December 1998 are made available for this work from MoWR. Within the recorded time a total increment of about 3.22 meters is observed. This corresponds to an average annual increment of 0.11 meter of the lake level. The maximum level recorded so far was 3.82 meters in the 13th of Nov. 1998. The monthly

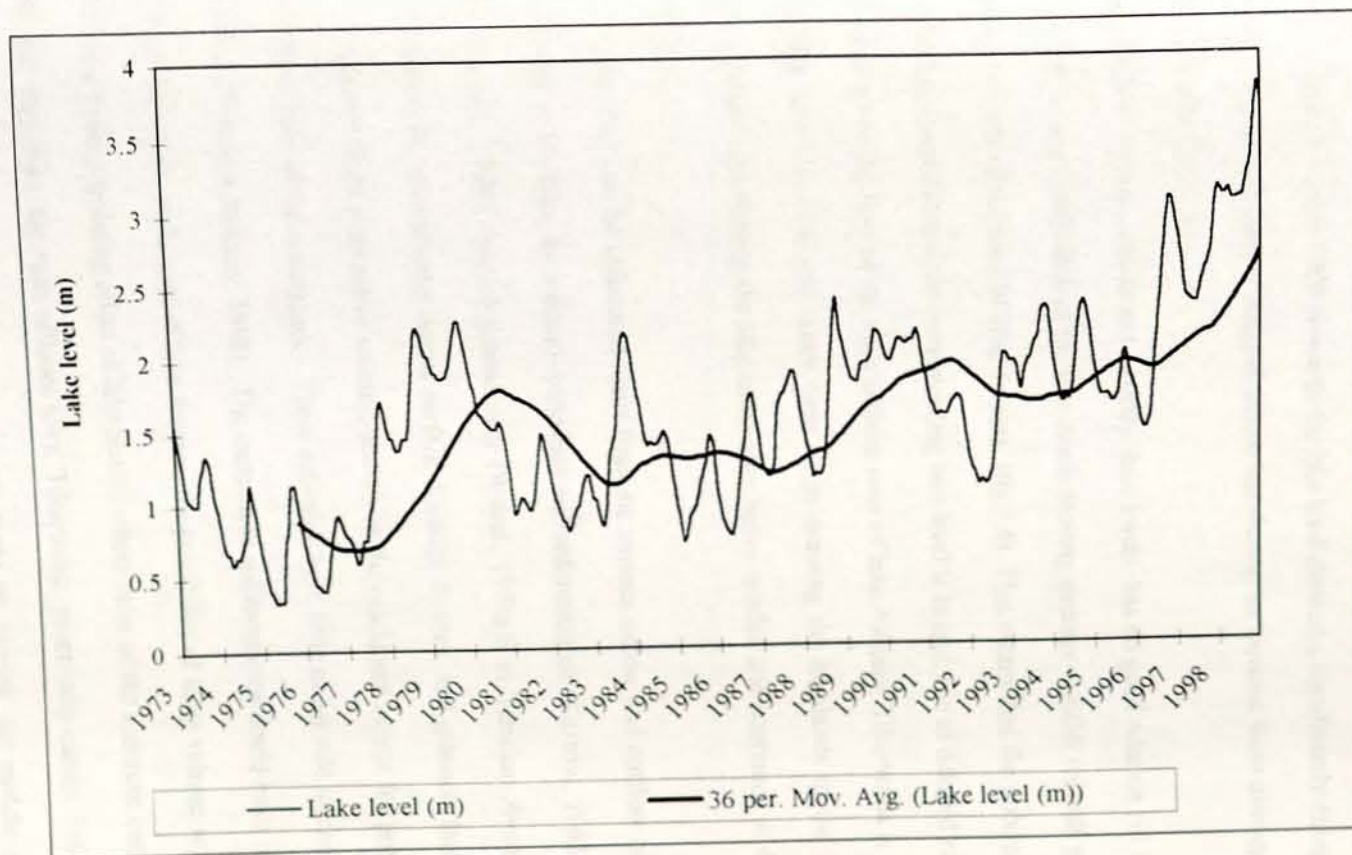


Fig 5.1 Awassa Lake level fluctuation for the period specified

lake level records indicate that the lake had no clearly defined trend before 1996 (fig 5.1). The fluctuation recorded is generally in accordance with temporal changes in rainfall in the basin. For instance, sharp increases in the level of the lake were observed in 1977, 1983 & 1986 corresponding to the increased amount of precipitation above average by about 28%, 20% and 24% respectively (fig 4.2 and 5.1). However, since 1996 on-wards the lake level showed a significantly rising trend. During this period the rainfall recorded at Awassa station has shown an increase from average value of 17% to 23% as shown in fig (5.3).

Rainfall at Awassa station and monthly lake levels has no good relation (-0.1) as seen in fig (5.2). However, a one month shift of the five month moving average rainfall records gives a better fit with monthly records of the level of lake Awassa (fig 5.4). This means that the poor correlation of the monthly catchment rainfall and the corresponding lake level is in part due to delayed runoff. This time lag is expected since the flow of the main affluent river of lake Awassa, Tikurwuha is regulated by the Shallo swamp which in turn gets water from rivers draining the highlands of the Eastern plateau. Moreover, groundwater feeding the lake starts after heavy rainfall and continues to do so for delayed periods of time.

A lake level can be influenced apart from the various inflow and outflow components of the water balances of the lake, by volcano- tectonism and sedimentation (Karrow, 1963; Slay, 1973a; in Tenalem Ayenew, 1998), delayed groundwater (Winter, 1978a,b in Tenalem Ayenew, 1998), long distance transfer of groundwater across surface drainage divides, and climatic factors (Street, 1978, 1980). At present there is no active volcanic activity in the catchment except for geothermal activities in the northern side of the watershed. These activities have little or no role in changing the level of the rift lakes (Tenalem Ayenew, 1998). The increase in sedimentation could have influenced the level of rift lakes. However, in the case of lake Awassa, the possibility of large volume sediment transport is unlikely. Due to the regulating effect of lake Shallo, where most of the sediment coming from highland erosion get deposited, the main affluent river, Tikurwuha, practically carries little or no sediment. This is confirmed by a single time measurement made on August, the middle of the wet season. Samples taken at different depths were filtered using 0.45 μm filtering paper and the suspended sediment load was calculated to be 1350 PPM. A total sediment transport of about 1660 PPM is estimated assuming and

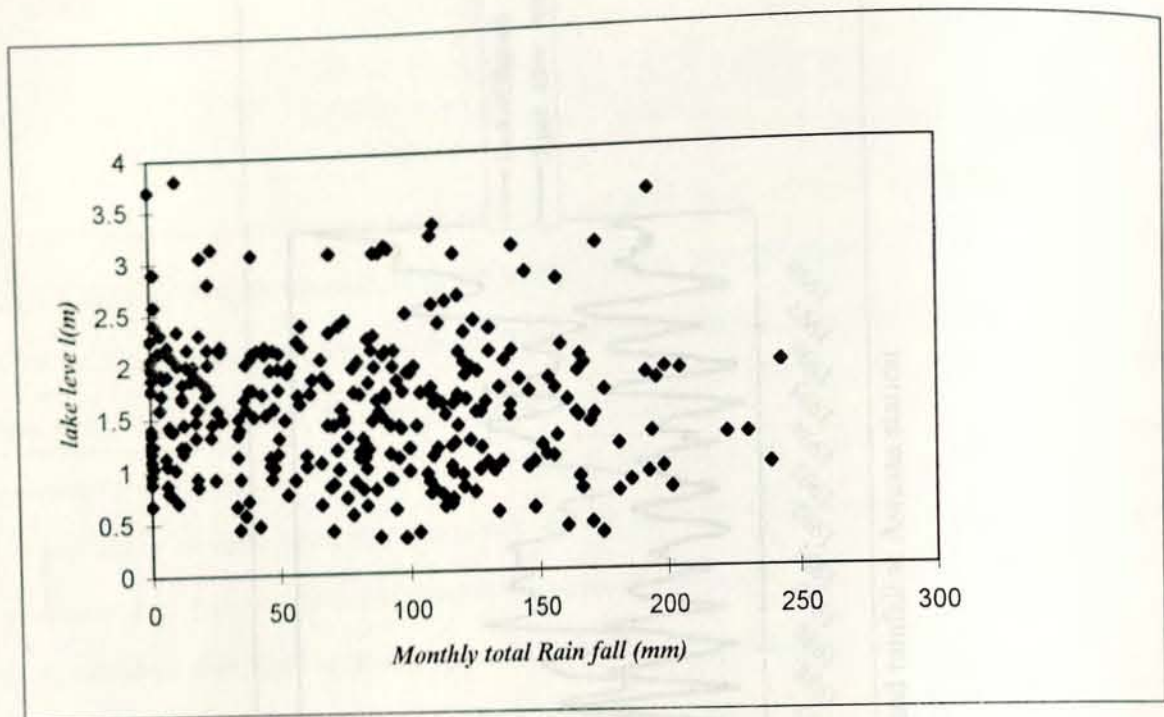


Fig 5.2 Monthly lake Awassa level and rainfall at Awassa station (1973 - 1998)

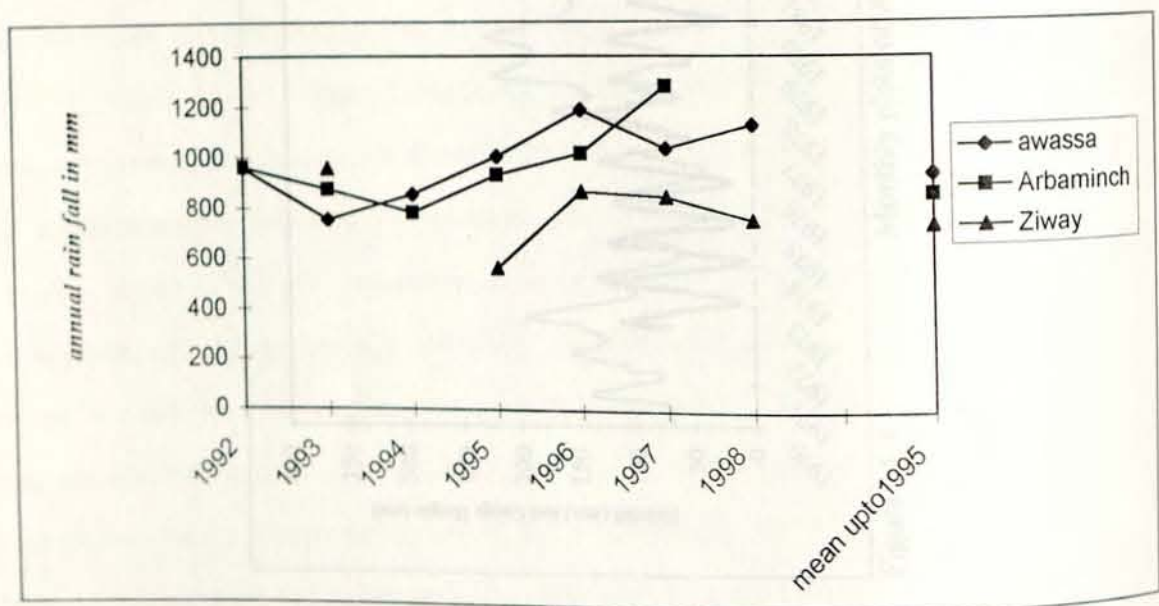


Fig 5.3 Annual rainfall at Awassa, Arbaminch and Ziway station for the period specified

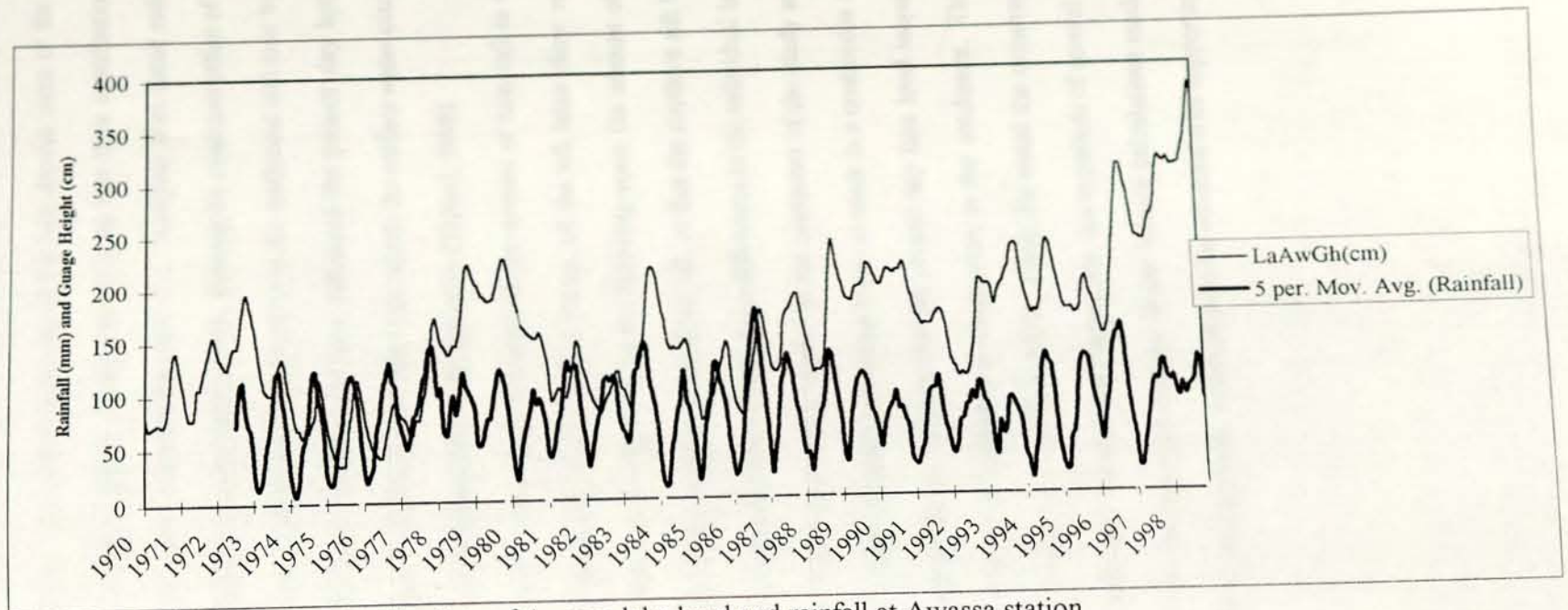


Figure 5.4 Monthly plots of Awassa lake level and rainfall at Awassa station

adding 15% to the suspended sediment transport to allow for the bed load contribution. A rough estimate on the total sediment transport can be made by adding 10 - 20% to the suspended sediment transport to allow for the bed load contribution (Raghunath, 1985). The ephemeral rivers draining the southern, southwestern and northwestern side of the lake deposit most of the sediment on the plain close to the lake boundary before reaching the lake, so that their contribution is minimal. Besides, Telford (1998) estimated a sedimentation rate of 1.7 mm/year from dated sediment core obtained by coring the lake bottom at a water depth of 12m, showing the minimum effect of lake sedimentation.

The increased occurrence of earthquakes in the catchment and near by basins; however, may resulted in new fractures which might have influenced the present day hydrologic regime of lake Awassa. But groundwater input to the lake from outside the surface water catchment will be negligible as lake Awassa is the elevated lake in the rift valley (Telford, 1998).

In 1996 there was a large increment in the amount of rain fall in the central and southern sector of the Ethiopian rift (fig 5.4 and .). Hence, all the rift lakes show relatively high levels as a response of the high inflow (fig 5.5.). In the following years the amount of rainfall was still above average especially to the southern sector of the rift, so that the southern rift lakes Awassa, Abaya and Chamo show an increasing trend in spite of the differences to the individual lake characteristics as seen in fig (5.5). The subsequent excess rainfall in the catchment of the study area supported by reduced evaporation will result residuals to increase hence to result in a continuous rise of the lake level. It is, therefore, more plausible to conclude that the present day lake level increment is due mainly to the change in the amount of rainfall and evaporation in the catchment. This is in agreement to the conclusion by Makin et al (1975 in NUPI, 1989). He noted the occurrence of cyclic or long term variations, in addition to the seasonal fluctuations, due to periods of several years when the net surplus is above or below the long term average. Water balance calculations made on monthly basis showed good estimates of the lake level comparable to the recorded ones supporting the above conclusion (fig 5.6).

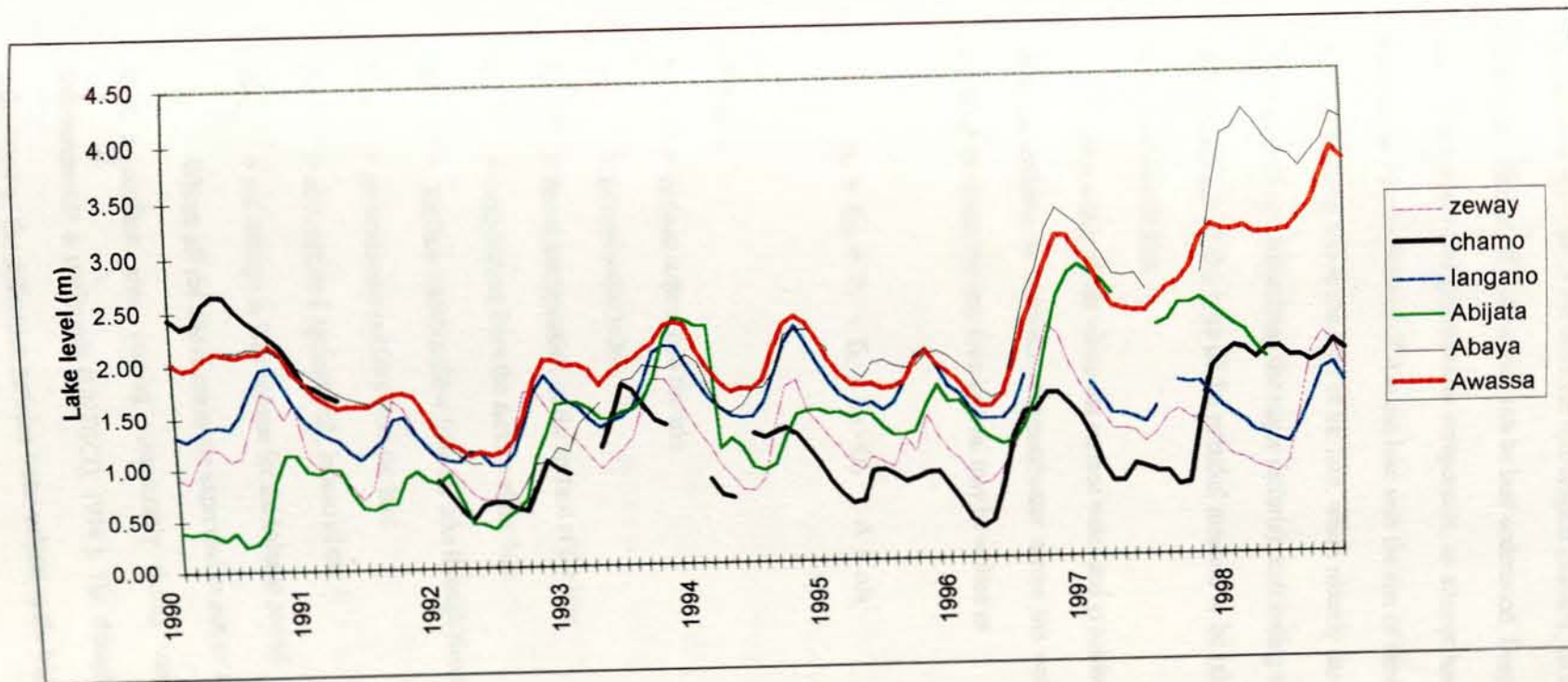


Fig 5.5 Lakes level fluctuations of some of the Ethiopian rift valley lakes for the period specified

5.2 Lake water balance Estimation

Lake water balance studies are used to estimate the flux of water, inflow and out flow, to lakes and identify the role of the different hydrological elements, so that the major factors that contribute to periodical lake level fluctuations can be best understood. Despite limitations on the hydrological data for surface water and groundwater components, an attempt has been made in this part of the study to estimate the water balance of Awassa lake with the aim of identifying the major factors that contribute to the periodical rise in the level of the lake, which recently has become a treat for further development of Awassa town. Identifying the major factor(s) contributing to the lake level rise, definitely, will help in the selection of the type of the remedial measure to be taken in order to alleviate the problem of hinterland inundation.

In a water shade where the surface water and groundwater divides coincide and hence with no external inflows or outflows of groundwater across the water shade boundary, the water balance equation for lakes for any time interval may be written as

$$S_I + G_I + P_L - E_L - S_O - G_O \pm A = \Delta S_L$$

Where

- S_I = surface inflow into the lake
- G_I = groundwater inflow
- P_L = direct precipitation on the surface of the lake
- E_L = evaporation from the surface of the lake
- S_O = surface water outflow from the lake through rivers
- G_O = groundwater outflow from the lake
- A = abstraction (agricultural , industrial etc.)
- ΔS_L = net change in lake volume for the balance period.

Where all the above terms are expressed in mm or volumetric measurements, though for lakes with a surface area varying considerably during water level fluctuations, using volumetric measurements is preferable (UNESCO, 1974). The change of water volume for the balance period is calculated as the difference between water volume at the beginning and at the end of the period.

For closed or endorheic (those without surface outflow) lakes, where the underground flux components (G_i and G_o) do not contribute significantly to the balance, the equation for mean water balance under equilibrium condition can be reduced to the form

$$S_i + P_L = E_L$$

However, many studies showed the importance of groundwater in the hydrological balance of lakes (Winter, 1978a in Tenalem Ayenew, 1998). In this study the unmeasured components of the groundwater flux are considered as a residual of the total annual water balance and incorporated in the ΔS_L term. As a result the general form of the annual water balance equation for the closed Awassa lake can be written as:

$$S_i + P_L - E_L = \Delta S_L \quad \begin{array}{l} \text{(since there is no significant amount of abstraction} \\ \text{from/to the lake)} \end{array}$$

Using the values 145mcm (= runoff from ungauged area plus mean annual discharge of Tikurwuha river as discussed in section 4.3.1) for S_i , 88mcm ($= 0.963 \times 91 \times 10^6 \text{ m}^3$, where 91Km^2 is the average lake area taken for the balance period) for P_L , and 151mcm ($= 1.658 \times 91 \times 10^6 \text{ m}^3$) for E_L , ΔS_L will be 82 mcm. This amounts to an equivalent water depth of 900 mm. This figures show that the lake is not in equilibrium condition, and require either groundwater outflow to take place to maintain balance or/and an increase in lake volume. As seen from the recorded data, there is a raising of the lake level, esp. since 1996, and also the occurrence of groundwater outflow towards the lower lying and deep Shala lake was reported by different workers using isotopic evidence (Darling et al., 1996) and chemical and groundwater flow modelling (Telford, 1998; Tenalem Ayenew, 1998).

Monthly water balance estimation is attempted and compered with the measured mean monthly lake level (gauge height). The estimation is carried out considering the whole inflow and out flow components with simple assumptions as discussed below.

Inflow components:

- Total monthly rainfall on the lake surface (computed from measured rainfall data as recorded at Awassa station)
- Total monthly Tikurwuha river discharge (measured data at Dato village)

- Runoff from ungauged catchment area computed using a runoff coefficient of 0.15.

Out flow components:

- Total monthly evaporation from the lake surface (computed from measured pan evaporation data as recorded at Awassa station and using a pan coefficient of 0.8)

Groundwater inflow and outflow is assumed to be equal in all the months, though not true in reality, due to difficulty to estimate accurately. However, since groundwater outflow probably remains relatively constant in relation to the other water balance components (Street, 1980), the above assumption, $G_o = G_i$ will subdue the effect of groundwater inflow. Hence, the role of groundwater inflow for the recent lake level rise can be assessed indirectly from the result of the monthly water balance computation by making comparison to the measured change in volume of the lake (or lake level) with the calculated one. That is, a good correlation between them shows the minor role of groundwater inflow to the recent lake level rise and to the errors of the estimation and assumption of the components and this computation. This is what is found out in the present study.

The result (fig 5.6) show a good correlation (0.99) between the calculated and measured lake levels supporting that the lake level rise is due to the change in environmental conditions especially rainfall and evaporation. However, this does not mean that the groundwater inflow will not increase as rainfall increases. Both groundwater and surface runoff increase as rainfall increases and also a change in vegetation cover of the catchment with time will play a role in the increment of the runoff. But what the result shows is that the major cause of the recent lake level rise is due to change in environmental conditions in the catchment area, especially rainfall and evaporation. The other components are results of these environmental conditions.

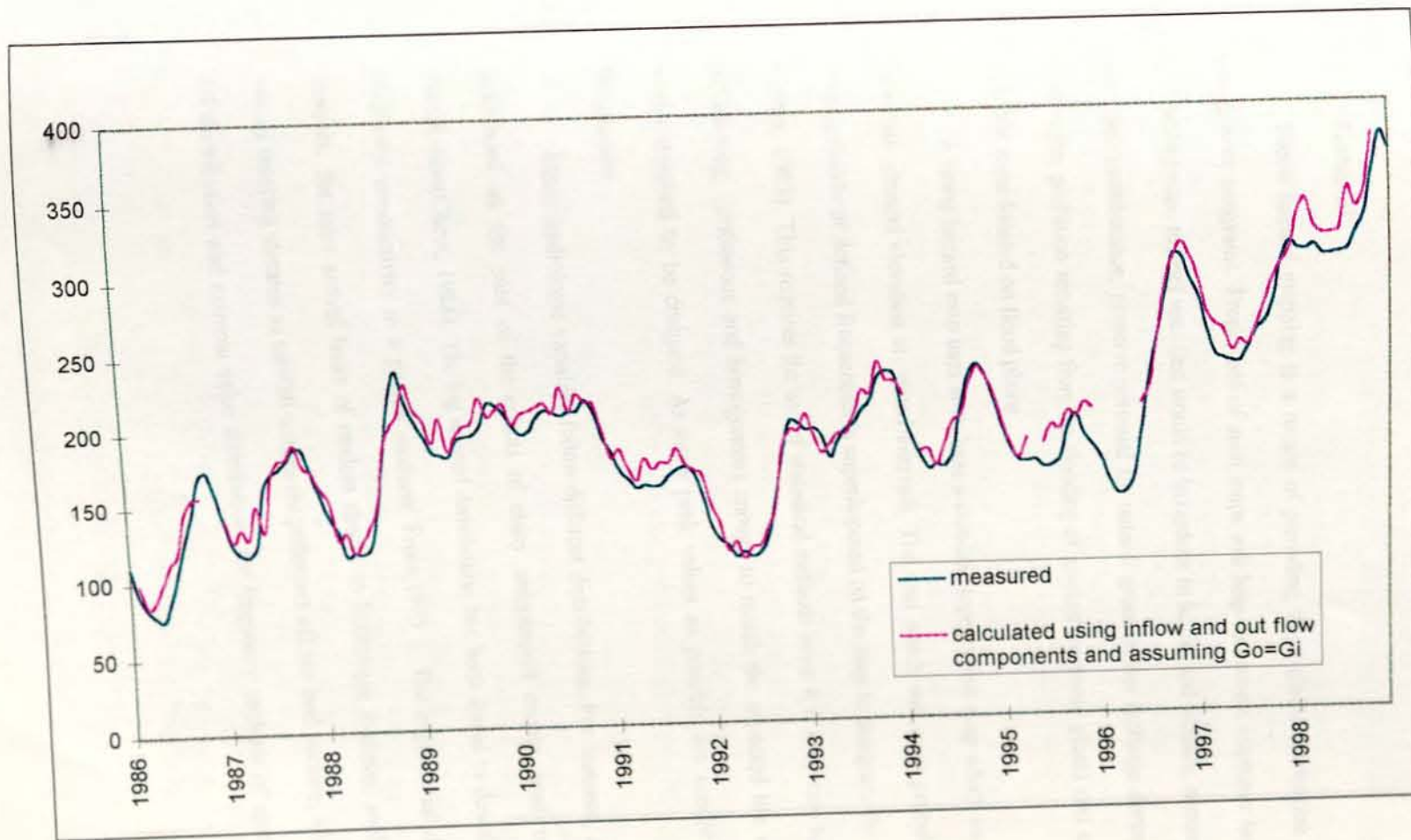


Fig 5.6 Calculated and measured lake Awassa level fluctuations for the period specified

Chapter Six

Flood hazard mapping

6.1 General

Flood hazard mapping is a means of providing flood plain information for planning and management programs. Proper use of such maps will help to prevent improper land development in flood plain areas, restrict uses that would be hazardous to health and welfare, encourage adequate lake shore line maintenance, preserve potential for natural groundwater recharge during flood events and avoid water pollution resulting from the flooding of sewage treatment plants and solid waste disposal sites that were located on flood plains.

A flood hazard map uses at its base a standard topographic map which includes contours that define the ground elevation at stated intervals. The area which will most probably be inundated by various floods of defined frequencies is superimposed on the map to designate the 'flood hazard map' (Foster, 1983). This requires the use of statistical methods since it is very rare to have a satisfactory record long, continuous and homogeneous enough to match the expected life of many engineering works required to be designed. As many peak values as possible are needed in assessing flood frequencies.

Many hydrologic variables follow different distributions. For instance, annual precipitation, calculated as the sum of the effects of many independent events, tend to follow the normal distribution (Chow, 1988). The log normal distribution has been found to describe the distribution of hydraulic conductivity in a porous medium (Freez, 1975). The exponential distribution is used to describe the inter arrival times of random shocks to hydrologic systems, such as slugs of polluted runoff entering streams as rainfall washes the pollutants off the land surface, while the log-parson type III distribution and extreme value distribution, for frequency analysis of annual maximum floods (

Storms, floods, and droughts are extreme events in hydrologic systems. The magnitude of such an extreme event is inversely related to its frequency of occurrence, where very severe events occurring less frequently than more moderate events. In order to relate the magnitude of such extreme events to their frequency of occurrence, frequency analysis of hydrologic data, through the use of probability distributions, is commonly employed (Chow, 1988). The hydrologic data series analysed are assumed to be independent and identically distributed, and the hydrologic system producing them is considered to be stochastic, and space and time independent. This requires a careful selection of hydrologic data to be used so that the assumption of independence and identical distribution are satisfied. In practice, this is often achieved by selecting the annual maximum of the variable being analysed with the exception that successive observations of this variable from year to year will be independent (Chow 1988). The results of flood peak frequency analysis have been used for many engineering purposes: for the design of dams, bridges, culverts, and flood control structures, to determine the economic values of flood control projects, and to delineate flood plains and determine the effect of encroachments on the flood plain.

In the study area, the raising of Lake Awassa level has caused economic and social damages. The lake has already inundated part of the southern and western part of the Awassa town including private and governmental buildings, agricultural land, grazing land and recreation sites along the beach. As it has the potential for further engineering and related damage a flood risk analysis and flood hazard mapping is attempted in this work. This work may assist official and private bodies in making decisions and alternative plans concerning the development of specific lands subject to flooding. The most frequently used probability distribution, the extreme value type I or the Gumbel's method is adopted for this method gives the prediction of floods of a particular frequency exceeding the observed floods by a safe margin than the other methods (Ranghunath, 1985), and only peak readings could be taken. It is also this method that is used in the National Urban Planning Institute of the country (NUPI, 1989).

The annual maximum hydrologic data series is used in this work. This series takes only a single maximum value in each year of record so that the number of data values equals the record length in years. Using a 29-years of peak lake level records (table 6.1) and based on the assumption that level fluctuations are mainly due to the difference in inflow and out flow, and that the selected

Year	Maximum gauge level
1970	1.46
1971	1.57
1972	1.96
1973	1.65
1974	1.15
1975	1.18
1976	0.93
1977	1.74
1978	2.25
1979	2.3
1980	2.09
1981	1.48
1982	1.2
1983	2.25
1984	2.08
1985	1.48
1986	1.84
1987	1.92
1988	2.39
1989	2.2
1990	2.2
1991	1.83
1992	2.04
1993	2.34
1994	2.38
1995	2.03
1996	3.12
1997	3.15
1998	3.82
mean	2.00

Table 6.1 Annual Peak Lake Awassa Levels (1970 - 1998)

annual peaks are independent of one another the extreme value type I (EV I) probability distribution function is used.

6.2 Flood Probabilities

Flood frequency analysis entails the estimation of the peak discharge (level in the case of lakes) which is likely to be equalled or exceeded on average once in a specified period, T years. This is called the T -year event and the peak, Q_T , is said to have a return period or recurrence interval of T years. The return period, T years, is the long-term average of the intervals between successive exceedences of a flood magnitude, X_T , measured over a very large number of occurrences. However, those intervals may vary considerably around the average value T . Thus the return period of an event of a given magnitude may be defined as the average recurrence interval between events equalling or exceeding a specified magnitude.

The probability $P = P(X \geq X_T)$ of occurrence of the event $X \geq X_T$ in any observation may be related to the return period in the following way (Chow, 1988): for each observation, there are two possible outcomes: either "success" $X \geq X_T$ (probability P) or "failure" $X < X_T$ (probability $1-P$). Since the observations are independent, the probability of a recurrence interval of duration t is the product of the probabilities of $t-1$ failures followed by one success, that is, $(1-P)^{t-1} P$, and the expected value of t , $E(t)$, is given by:

$$\begin{aligned} E(t) &= \sum_{t=1}^{\infty} t(1-p)^{t-1}p = p + 2(1-p)p + 3(1-p)^2p + 4(1-p)^3p + \dots \quad 6.2a \\ &= p[1 + 2(1-p) + 3(1-p)^2 + 4(1-p)^3 + \dots] \end{aligned}$$

The expressions within the brackets has the form of the power series expansion

$$(1+x)^n = 1 + nx + \frac{n(n-1)}{2}x^2 + \frac{n(n-1)}{6}x^3 + \dots$$

$x = -(1-p)$ and $n = -2$ so equation (6.2a) may be written as

$$E(t) = \frac{p}{[1 - (1-p)]^2} = \frac{1}{p}$$

Hence $E(t) = T = 1/P$ that is the probability of occurrence of an event in any observation is the inverse of its return period:

$$P(X \geq X_T) = 1/T$$

and if $F(X)$ is the probability of $X < X_T$:

$$F(X) = 1 - P(X) = (T(X) - 1) / T(X)$$

and the probability that a T -year return period event will occur at least once in N years is the complement of the situation where no T -year event occurs in N years, which requires a sequence of N successive "failures", so that

$P(X < X_T \text{ each year for } N \text{ years}) = (1-P)^N$ and the complement of this situation will be

$P(X \geq X_T \text{ at least once in } N \text{ years}) = 1 - (1-P)^N$, and since $P = 1/T$

$$P(X \geq X_T \text{ at least once in } N \text{ years}) = 1 - (1 - 1/T)^N$$

6.3 Estimation of the T -year flood using EV I

The equation of the Gumbel extreme value distribution (EV I) is given by (Chow, 1988)

$$F(x) = \exp\left[-\exp\left(\frac{x-u}{\alpha}\right)\right] \quad -\infty \leq x \leq \infty \quad 6.3a$$

where $F(X)$ is the probability of an annual maximum $X \leq X_T$ as defined previously, and the parameters α and u are related to the moments of the data series of X values and given as follows:

$$\alpha = \frac{\sqrt{6}}{\Pi} S$$

where S = the sample standard deviation

$$u = \bar{x} - 0.5772\alpha$$

$\bar{x}$ = the sample mean

The parameter u is the mode of the distribution (point of maximum probability density). A reduced variant y can be defined as

$$y = \frac{x-u}{\alpha}$$

substituting the reduced variant in to equation (6.3a) gives

$$F(x) = \exp[-\exp(1-y)]$$

solving for y

$$y = -\ln \left[\ln \left(\frac{1}{F(x)} \right) \right] \quad 6.3b$$

$$\begin{aligned} \text{since } 1/T &= P(X \geq X_T) \\ &= 1 - P(X < X_T) \\ &= 1 - F(X_T) \end{aligned}$$

$$\text{so } F(X_T) = (T-1)/T$$

and substituting into equation (6.3b)

$$y_T = -\ln \left[\ln \left(\frac{T}{T-1} \right) \right] \quad 6.3c$$

for the EV I distribution, X_T related to y_T by the following equation (Chow, 1988)

$$X_T = u + \alpha y_T \quad 6.3d$$

According to the same author, it is also possible to calculate by an alternative method, using frequency factors as follows:

The magnitude of X_T of a hydrologic event may be represented as the mean $\bar{\mu}$ plus departure ΔX_T of the variant from the mean

$$X_T = \bar{\mu} + \Delta X_T \quad 6.3e$$

The departure may be taken as equal to the product of the standard deviation δ and a frequency factor K_T , that is, $\Delta X_T = K_T \delta$. The departure ΔX_T and the frequency factor K_T are functions of the return period and the type of probability distribution to be used in the analysis. Equation (6.3e) may, therefore, be expressed as

$$X_T = \bar{\mu} + K_T \delta$$

which may be approximated by

$$X_T = \bar{x} + K_T S \quad 6.3f$$

where K_T for the extreme value type I distribution is given by the following expression as derived by Chow (1953):

$$K_T = -\frac{\sqrt{6}}{\Pi} \left\{ 0.5772 + \ln \left[\ln \left(\frac{T}{T-1} \right) \right] \right\}$$

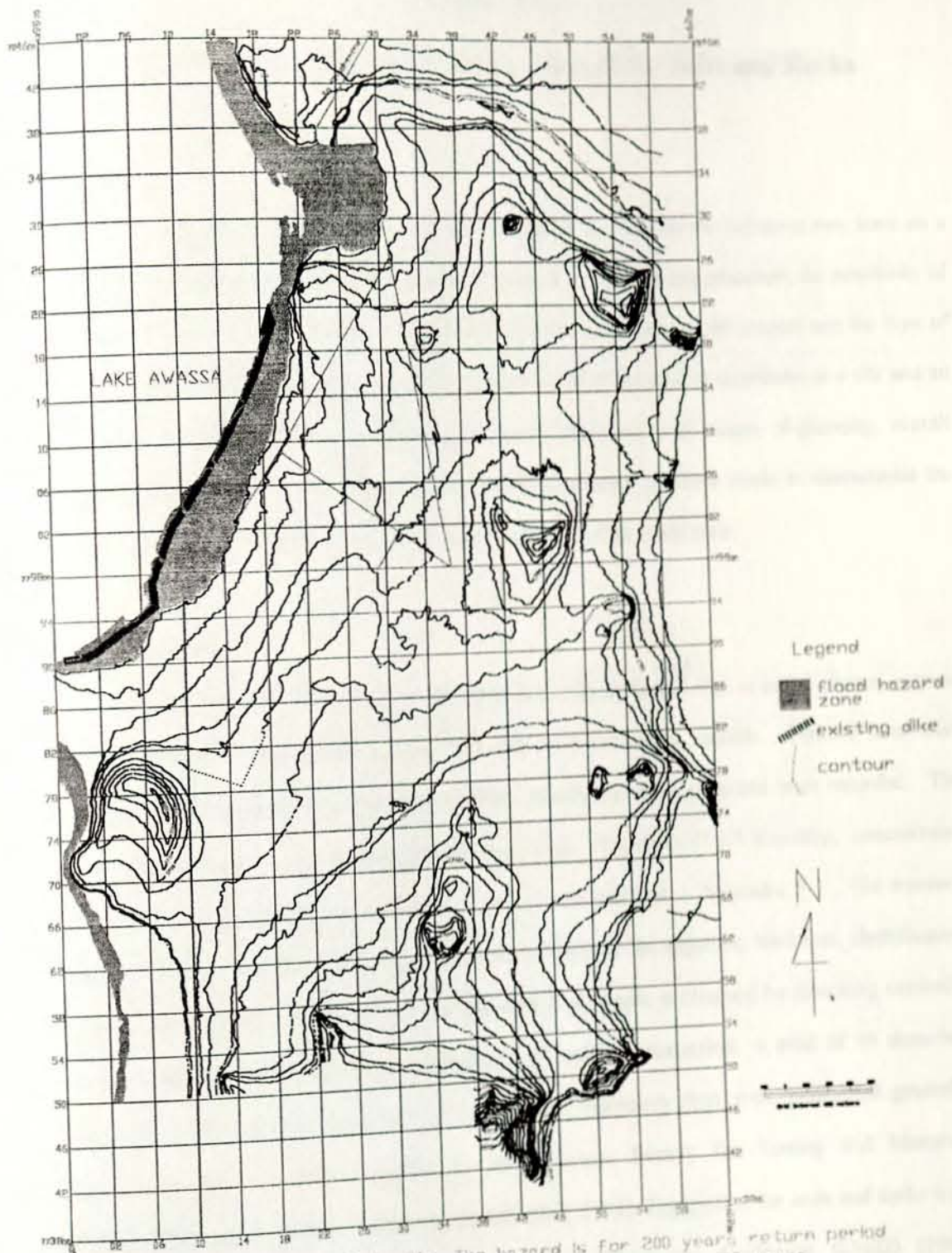
In the study area, from 29- years of peak lake level records (table 6.1), the mean and standard deviations are computed to be 2.00 and 0.628 respectively. Using these statistical data lake levels are calculated for different return periods using equations 6.3a and 6.3f which gave the same result. Using 1678.4 m a.s.l. as the elevation of the zero staff gauge level (NUPI, 1989), lake area and lake volume for the different return period lake levels are calculated and presented in table 6.2.

The flood risk zones of two hundred years return period is bounded on the topographic base map (fig 6.1), assuming that there is no dike and having as many interpolated points as possible. On the other hand, the return period of the lake level which would safely be protected by the existing dike (crest elevation 1681 m a.s.l., NUPI, 1989) is estimated to be about 10 years, for a free board of 0.25m above wave water level, which is above the acceptable risk (0.5 for a design life of 50 years) to provide a physical protection. Around cities flood protection structures like levees are generally designed using return periods in the range of 50 - 200 years with corresponding risk of failure in the range of 0.6 - 0.2 for a design life of 50 years (Chow, 1988). Since selection of 100 or 200 year flood implies a certain level of acceptable risk, 0.4 or 0.2 respectively, it is a common practice to provide a physical protection for all floods up to the magnitude of the 100 or 200 years flood. This requires, a raising of the crest level of the existing dike by additional 1 or 1.5 m with a freeboard of not less than 1m above the design lake level.

Return period in years	lake level in meter (a.s.l.)**	Lake Volume in mcm	lake Area in Km2	Gauge height (m)			actual prob. of exceedence in N=29 year
				Calculated using Gumbel Method	95% confidence limit (above zero staff gauge)		
					Upper(X)	Lower(X)	
2	1680.30	1064.63	94.18	1.90	1.94	1.86	1.000
5	1680.85	1122.14	95.80	2.45	2.52	2.38	0.998
10	1681.22	1161.03	96.84	2.82	2.91	2.73	0.953
20	1681.57	1198.96	97.81	3.17	3.29	3.06	0.774
50	1682.03	1248.96	99.04	3.63	3.78	3.48	0.443
100	1682.37	1287.09	99.93	3.97	4.15	3.80	0.253
200	1682.71	1325.64	100.80	4.31	4.51	4.11	0.135
400	1683.05	1364.69	101.65	4.65	4.88	4.43	0.070
500	1683.16	1377.37	101.92	4.76	4.99	4.53	0.056
1000	1683.50	1417.12	102.74	5.10	5.36	4.85	0.029

** Calculated using 1678.4m a.s.l. as the height corresponding to the zero staff gauge level (NUPI, 1989)

Table 6. 2. Flood frequency analysis result calculated using EV I.



Chapter Seven

Engineering Geological Characteristics of the Soils and Rocks

7.1 General

The importance of the ground conditions at a particular site and the influence they have on a project depend on a number of factors, such as the type of structure being proposed, the sensitivity of the structure or any installed machinery to ground movements, the size of the project and the type of soils and rocks underlying the site. Hence, the consideration of the ground conditions at a site and an appreciation of their significance to proposed project is important at all stages of planning, overall design, detail design and construction. In this context an attempt has been made to characterise the Engineering Geological characteristics of the soils and rocks of the study area.

7.2 Methodology

All the available information for the study area was collected, that is aerial photos, general geology maps, site investigation reports, bore hole and excavation records. Natural cuts were inspected and foundation excavations for buildings, sewers and other purposes were recorded. The information obtained is used in planning the location of new 14 test pits (2.5-5.0 m deep, concentrated in areas where geotechnical data are either scanty or non available (Appendix F). The trimmed surface of the side wall of the test pits is examined to determine the sequence, thickness, classification and description of each stratum of material, and disturbed sample is obtained by trenching vertically into the trimmed side wall of the pit with a cut and uniform crosssection. A total of 48 disturbed samples are collected from these test pits and various laboratory tests were performed generally according to the procedures specified by the American Society for Testing and Materials (ASTM,1996) and the British Standards Institute (BSI, 1988). Description for soils and rocks were made according to the International Association of Engineering Geologists (IAEG) (1981) recommendations. Except for the chemical analysis and pH determination, which were carried out at the laboratory of Building Design Enterprise, all soil testing was carried out at the laboratory of

Department of Civil Engineering, Addis Ababa University. A total of more than 44 bore holes (6 - 30m depth) and more than 20 pits was analysed and used for the present work.

7.3 Lithological Description of the Soils

The area the Awassa town is founded, is largely covered by thick (> 30m) soil. In the present study the soil investigation was carried out only in the existing built up areas and possible areas of expansion especially to eastern and southeastern side of the town. From the limited filed investigations (pitting) and a study of available site investigation reports, the soils of the study area are lithologically classified into the following types:

Clay Soils

Sandy Silt Soils

Silty Sand /Gravel Soils

7.3.1 Clay Soils

These type of soils cover the central, the eastern and the south-eastern part of the study area and generally occur in the relatively elevated parts. They commonly overlay sandy silt soils and in few places silty sand soils and weathered ignimbritic or tuffaceous rocks. Their thickness ranges from 0.85m to 3.5m. They are generally dark brown in color, though light gray and dark gray varieties are also encountered. They exhibit a range of consistency from medium stiff to hard and have low to medium plasticity and toughness. They are slightly cracked, columnar to prismatic and sometimes sub-angular blocky. These soils usually contain significant amount of sand and silt, hence silty and sandy clays are not uncommon. They are supposed to be residual soils, i.e. highly weathered products of the underlying ignimbritic rock.

7.3.2 Sandy Silt Soils

These soils occur in almost all part of the investigated area at different stratigraphic positions. They especially outcrop extensively to the northern and north-eastern part of the town and usually interstratified with silty sand soils. They underlay the clay soils to the eastern, central and south-eastern part of the town. These soils have various colors including dark yellowish brown and light gray in dry state and olive yellow color when wet. They are massive in structure and sometimes show

yellowish mottling and contain carbonate concretions as large as 12 cm in diameter. These soils are a result of high degree of weathering of the underlying ignimbritic rocks and/or lake deposits especially those below the ignimbritic layer.

7.3.3 Silty Sand/Gravel Soils

The silty sand soils occur in similar areas as the sandy silt soils. They are commonly found interstratified with the sandy silts and become dominant close to the lake. They are loose, massive and usually moist having light gray color. They are found usually below sandy silt soils except close to the lake where they may be found on top of the former ones. In some places they overlay immediately the weathered ignimbritic unit and it is suggested that these soils also have similar origin to the sandy silt soils.

The silty gravel soils are commonly found at depths below 3m and become important more towards the lake. They have light gray color and contain rounded pumice clasts as large as 20 cm embedded in the fine soils. These soils are supposed to be waterline (reworked) pyroclastic ejectas. In some places gravely pumice fall deposits are encountered near to the surface showing reverse grading. These may show the relatively recent recession of the lake.

All the above lithological types show thickness variations both laterally and in the vertical direction.

Deep site investigation bore holes and dug wells revealed the occurrence of relatively thin layer (1.5m) of ignimbritic rock at depth ranges from 4 to 8m underlined by lacustrine sediments of loose sandy silt and silty and gravely sand soils. These soils occur continuously or interstratified to highly to moderately weathered weak tuff. However, to the south-eastern side, around the textile factory, basaltic rocks which are vesicular on top are found below the silty/sandy soils underlying the ignimbritic rocks.

In the investigated area in addition to the above mentioned lithological units scorias dominated by lapilli and block size materials were encountered at two points forming small hills. They are considered as soils from engineering point of view but since they cover a very small area not investigated in detail in the present work.

7.4 Laboratory analysis and results

7.4.1 Mineralogical and Chemical composition

7.4.1.1 Mineralogical Composition

Determination of non clay mineral types was done on sand and gravel fractions of few samples using magnifying glass. Accordingly, the soils of the study area mainly consists of quartz, feldspar, micas and rock fragments as would be expected in a volcanic terrain characterised by silicic volcanics.

The types of clay minerals present is assessed indirectly. The Atterberg limits of the soils, the position of the soils on the Casagrande plasticity chart and the activity of the soil can give an indication of which minerals are present. Accordingly, using the suggestions proposed by Hertz and Kovacs (1991, Appendix D) and Mitchell (1976, Appendix D) kaolinite, illite and Ca-montmorillonite are the most probable types of clay minerals constituting the soils of the study area. However, for practical works mineralogical test is recommended.

7.4.1.2 Organic matter content

The organic matter content may have a marked influence on the physical properties of soils. The natural water content of organic soils is high. The liquid and plastic limit tend to increase whereas the plasticity index decreases with increasing organic content. Organic soils are also highly compressible. Hence, the organic matter content of the soils of the study area have been determined for a number of selected samples by the ignition loss method, according to ASTM D 2974-87 procedure, and presented in Table 7.1. The clay soils show the highest ignition loss value in the range from 4.5 to 7.2%. The sandy silt and silty sand soils have similar values ranging from 2.9 to 5%. Usually soils containing organic matter greater than 25% by weight are considered as organic soils (Alkhafaji and Andersland, 1992). Therefore, the soils in the study area can be considered as inorganic soils.

Sample	Depth (m)	Lithological type	organic matter content (%)
Tp4-S1	0.5 - 0.9	Clay soil	4.9
Tp5-S1	0.45 - 1.2		5.2
Tp5-S2	1.2 - 2.3		4.8
Tp7-S1	0.65 - 1.05		5.7
Tp7-S2	1.05 - 1.4		5.3
Tp10-S1	0.4 - 0.5		5.2
Tp10-S2	0.5 - 1.3		7.2
Tp14-S1	0.4 - 0.9		4.5
Tp1-S1	0.2 - 0.75	Sandy silt soil	4.2
Tp1-S4	4.0 - 5.0		3.1
Tp3-S1	0.7 - 1.7		4.6
Tp3-S2	1.7 - 2.1		3.4
Tp4-S3	2.5 - 3.3		4.7
Tp5-S3	2.3 - 4.25		3.1
Tp11-S2	1.85 - 3.1		5
Tp14-S2	0.9 - 2.55		3.9
Tp14-S3	3.0 - 3.8		3.1
Tp1-S2	0.75 - 1.4	Silty sand/gravel soil	3.2
Tp1-S3	1.4 - 4.0		3.2
Tp4-S2	0.9 - 1.4		2.9
Tp6-S4	4.2 - 5.0		4
Tp10-S4	2.6 - 5.0		3.3

Table 7.1 Organic matter content of the soils of the study area

7.4.1.3 Carbonate Content

A soil's carbonate content affects the physical and chemical conditions of the soil environment. Carbonate in soils occurs primarily in the form of calcite, CaCO_3 , which acts as a cementing agent, forming aggregates out of the fine clay platelets (Mathwson, 1988). This results in a decrease in the plasticity and increase in the strength of the soil.

The presence of calcium carbonate (CaCO_3) on the soil samples of the study area is tested by dropping 20% hydrochloric acid on to the surface of them. If the soil is calcareous, effervescence results due to the reaction between the hydrochloric acid and the carbonates. From the intensity and duration of the reaction the carbonate content can be estimated.

Soils of the study area show varying degrees of intensity and duration of effervescence, from nil to fairly strong and from nil to not lasting, respectively, according to the way of description suggested by the Swiss Society for Testing Materials. According to the correlation suggested by the same society (Kedzi, 1974, Appendix D), these ranges of intensity and duration of the effervescence

suggest a carbonate content of less than 5%. The soil samples relatively close to the lake Awassa and Tikurwuha river and shallo lake have higher values relative to the others. However, carbonate concretions are common at depths of 2.5 to 5m in most of the test pits. Besides, the occurrence of as large as 27% of CaCO_3 is reported by Telford (1998) from a cored lake bed sediment.

7.4.1.4 Sulphate and Chloride content and pH of the soils

The ground or groundwater may contain chemicals capable of causing damage to the building materials of engineering structures and may cause their serious deterioration. The principal constituents that cause concrete to deteriorate are sulphates, which are most common in clay soils and acidic waters. The concomitant occurrence of chloride (Cl^-) is also reported to be injurious for concrete. In view of this, the sulphate and chloride, as well as the pH of some of the soil samples are determined in the laboratory according to the BSI (1988,1990) procedures and presented in table 7.2. The result show that the soils of the study area have less than 0.08% sulphate, less than 0.004% chloride and a pH of around 8. However, pH values as low as 5.8 are reported by recent investigations carried out by MoWR (1999). The same work, also reported that the soils of the study area have cation exchange capacity (CEC) in the range from 2 to 43 meq/100gm.

Total sulphate contents of more than 0.2 percent by weight in soils are potentially aggressive (BRE, 1981). Hence, the sulphate content of the soils of the area investigated is not likely to cause any damage to building materials.

Sample	Depth (m)	Sulphate content (%)	Chloride content (%)	pH
Tp1-S4	4.0 - 5.0	0.0192	-	-
Tp7-S2	1.05 - 1.4	0.08	0.002	8
Tp8-S2	1.85 - 2.45	0.0064	0.004	8
Tp12-S2	0.9 - 2.6	0.0064	0.002	-

Table 7.2 Sulphate and Chloride content and pH of soils

7.4.2 Physical (Index) Properties

Soils are usually heterogeneous and complex materials. The complexity is contributed by their existence in almost innumerable varieties by its combination of solid, liquid and gases, where in many instances the solid particles vary in size from big boulders to colloidal size. To understand the behaviour of the soils, methods of evaluations have been centred around their physical property as the physical characteristics are of importance to the engineers. The physical properties of soils which serve mainly for identification and classification are commonly known as index properties.

For the disturbed soils samples collected from the field, determination of the following physical properties was carried out.

7.4.2.1 Moisture content

The moisture content of a soil has a profound influence on its properties. It affects the shear strength, the consolidation properties and its response to compaction. It may also affect future swelling or shrinkage of the soil. Moisture content tests are therefore frequently carried out and are routinely included as part of the standard procedure for most tests.

The moisture content of some of the representative samples is determined according to the ASTM (D2216) procedure and presented in Table 7.3. The weight of the natural soil sample was carried out in the field immediately after sampling to minimise the loss of moisture as a result of long stay and high temperature condition of the area. The result revealed that the clay soils have a moisture content of 25.1- 29%, silty sand soils have 24.6 to 45.7% and the sandy silt soils have 18 to 42.7%. The low moisture content of the clay soils could be due to the loss of moisture by evaporation since they are close to the surface.

Sample	Depth (m)	Lithological Type	Moisture content(%)
Tp5-S2	1.5	Clay soils	27.7
Tp6-S2	1.1		25.1
Tp10-S2	1		29
Tp1-S4	4.9	sandy silt soils	22.4
Tp2-S2	0.65		29
Tp2-S3	1.6		42.3
Tp3-S2	2.1		32.7
Tp4-S4	4		33.6
Tp5-S3	4.1		18
Tp6-S3	2.85		36.3
Tp8-S4	4.2		42.7
Tp1-S3	2.1	Silty sand/gravel soils	28.7
Tp3-S3	4.5		24.6
Tp6-S4	5		45.7

Table 7.3 Moisture Content of soils of Awassa area

7.4.2.2 Specific Gravity

The specific gravity determination of the soils was carried out for all samples according to the ASTM (D 854) procedure. An accurate value of specific gravity is required to compute the void ratio of a soil, unit mass of soil and in Stock's law in hydrometer analysis. In the present study, since the specific gravity is required for the latter use, specific gravity determination is carried out on that portion of the sample that passes the 2.0 mm sieve (table 7.4). This analysis showed that the clay soils have a specific gravity in the range from 2.19 to 2.50 and that of silty sands and sandy silt soils is from 2.18-2.48. These values are less than the normally used values (2.5-2.8), which could be due to the presence of some organic matter and /or due to the fraction of the soil that is used for analysis. For determination of the specific gravity of soil masses usually that portion of the soil passing 4.75 mm sieve is used.

7.4.2.3 Grain Size Analysis

Soils are commonly found in a wide range of particle sizes and shapes. The particle size distribution, usually referred to as the grading, is an important property of soils especially coarse grained soils. It is used for classification and to infer a number of engineering properties of these soils.

Sample	Depth (m)	Lithologic Type	Specific gravity	Gravel	Sand	silt	clay
					%		
Tp4-S1	0.5 - 0.9	Clay Soils	2.19		49	39	12
Tp5-S1	0.45 - 1.2		2.31		45	48	7
Tp5-S2	0.2 - 2.3		2.31		31	57	12
Tp6-S1	0.45 - 0.85		2.29		21	59	20
Tp6-S2	0.85 - 1.75		2.4		24	65	11
Tp7-S1	0.65 - 1.05		2.32		22	66	12
Tp7-S2	1.05 - 1.4		2.5		8	58	34
Tp8-S1	0.45 - 0.5		2.35		22	48	30
Tp10-S1	0.4 - 0.5		2.31		31	59	10
Tp10-S2	0.5 - 1.8		2.38		26	59	15
Tp13-S1	0.45 - 1.8		2.33		35	52	13
Tp14-S1	0.4 - 0.9		2.31		44	48	8
Tp1-S1	0.2 - 0.75	Sandy silt soils	2.21		47	45	8
Tp1-S4	4.0 - 5.0		2.33		24	67	9
Tp2-S2	0.35 - 0.75		2.42		23	49	28
Tp2-S3	0.75 - 3.0		2.43		30	63	7
Tp3-S1	0.7 - 1.7		2.33		47	46	7
Tp3-S2	1.7 - 2.1		2.28		34	58	8
Tp4-S3	2.5 - 3.3		2.29	12	16	58	14
Tp4-S4	3.3 - 4.0		2.32		26	61	13
Tp5-S3	2.3 - 4.25		2.33		34	62	4
Tp6-S3	1.75 - 4.2		2.35		25	62	13
Tp8-S4	3.75 - 4.2		2.48		46	48	6
Tp9-S1	0.45 - 0.95		2.3		26	59	15
Tp9-S2	0.95 - 1.55		2.38		44	50	6
Tp9-S3	1.55 - 3.9		2.31		22	67	11
Tp9-S4	3.9 - 4.75		2.27		38	59	3
Tp10-S3	1.3 - 2.6		2.39		27	68	5
Tp11-S1	0.25 - 1.85		2.44		20	51	29
Tp11-S2	1.85 - 3.1		2.37		30	56	14
Tp12-S2	0.9 - 2.6		2.35		24	66	10
Tp13-S2	1.8 - 3.25		2.42		32	62	6
Tp13-S3	3.25 - 3.8		2.36		14	64	22
Tp14-S2	0.9 - 2.55		2.45		20	58	22
Tp14-S3	3.0 - 3.8		2.37		21	74	5
Tp1-S2	0.75 - 1.4	Silty Sand/gravel Soil	2.34		71	27	2
Tp1-S3	1.4 - 4.0		2.23		55	39	6
Tp2-S1	0.1 - 0.35		2.2		77	19	4
Tp3-S3	2.1 - 5.0		2.37	16	42	38	4
Tp4-S2	0.9 - 1.4		2.18		73	20	7
Tp6-S4	4.2 - 5.0		2.29	34	34	29	3
Tp8-S2	1.85 - 2.45		2.36		56	41	3
Tp8-S3	2.45 - 3.75		2.42	3	68	25	4
Tp10-S4	2.6 - 5.0		2.32		53	39	8
Tp11-S3	3.1 - 5.0		2.4	37	45	16	2
Tp12-S1	0.2 - 0.9		2.31		56	40	4
Tp12-S3	2.6 - 3.7		2.26	44	30	23	3
Tp12-S4	4.2 - 4.4		2.22	62	14	22	2
Tp13-S4	3.8 - 4.55		2.33	26	30	40	4

Table 7.4 Specific gravity and granulometric composition of the soils of Awassa area

Grain size analysis was carried out for all soil samples brought from the test pits according to the ASTM (D 422) procedure. The analysis consisted first drying the soils sample and then pulverising it before letting run through a nest of sieves. The soil passing the No. 10 sieve was subjected to hydrometer analysis and the result obtained was expressed by a plot of percent finer (passing) by weight verses the size of the soil particles in millimetres on a log scale. From the curve thus drawn the relative amount of each type of soil constituting the sample was computed. The Grain size analysis results for all the samples tested in shown in Table 7.4 and their range of distribution in fig 7.1. The size range of each group was adopted from ASTM (> 4.75 mm-gravel; $4.75 - 0.075$ mm - sand, $0.0075 - 0.002$ mm silt, and less than 0.002 mm clay).

All the soil samples are composed of a relative proportion of different size groups as naturally expected, although one group dominates the others in relative percentage. Silt and sand size materials predominate in most of the samples with fewer clay and gravel size materials. The range of grain size distribution for the different lithological type soils is presented in Table 7.5. Though, the soils grouped under the clay lithologic type contain large proportion of silt and sand size materials as compared to the clay size materials, they were identified as clay soils in the field on the basis of field identification tests. This is confirmed as discussed latter through the routine laboratory analysis and using the Unified Soil Classification System. This could be due to either the silt is of finer silt type approaching clay size material or the clay size materials are of active type dictating the property of the soil mass.

	Gravel %	Sand %	Silt %	Clay %
Clay Soils	-	8 - 49	39 - 66	7 - 34
Sandy Silt Soils	-	14 - 47	45 - 74	3 - 29
Silty sand/gravel soils	3-62	14 - 77	16 - 41	2 - 8

Table 7.5 Ranges of grain size distribution of the various soil types

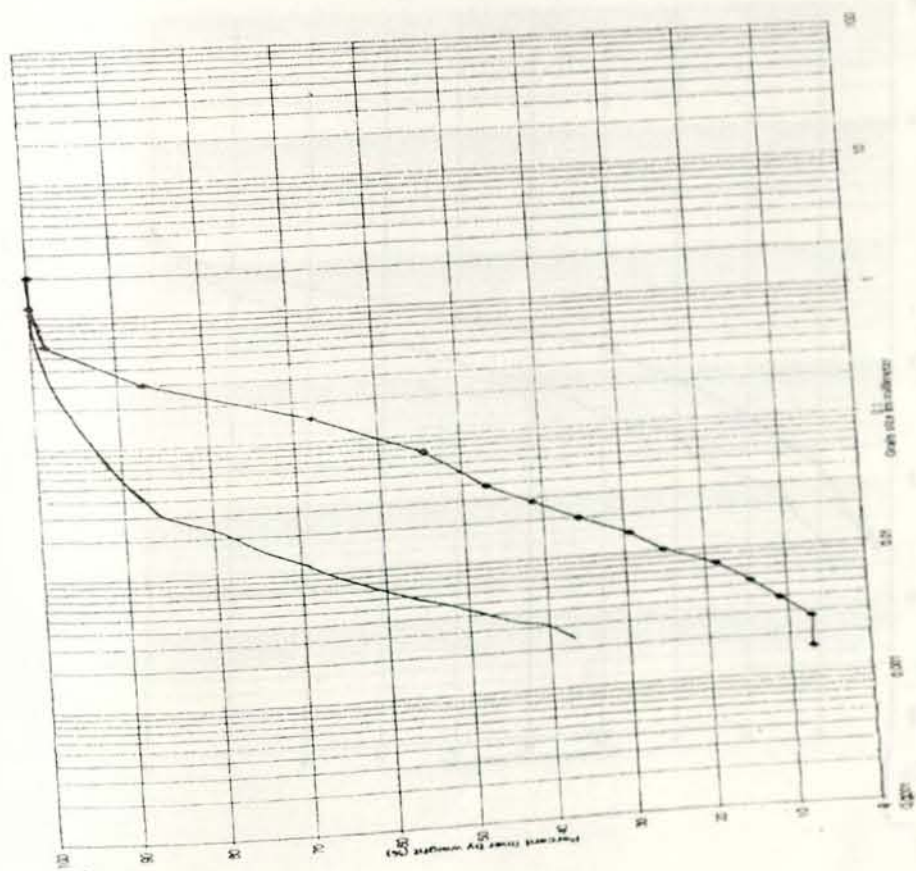
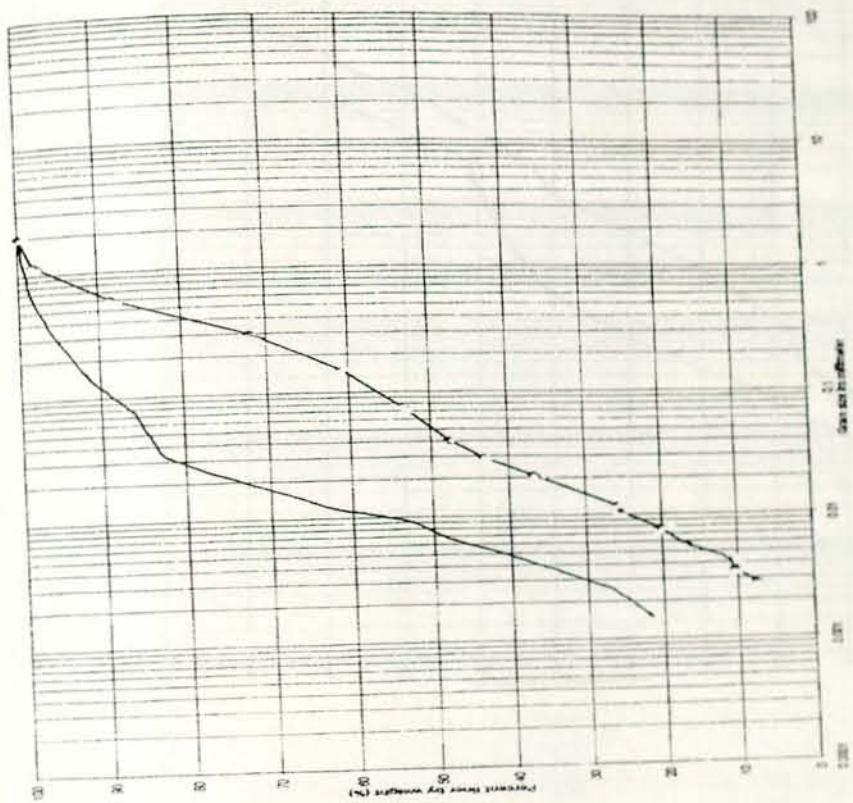
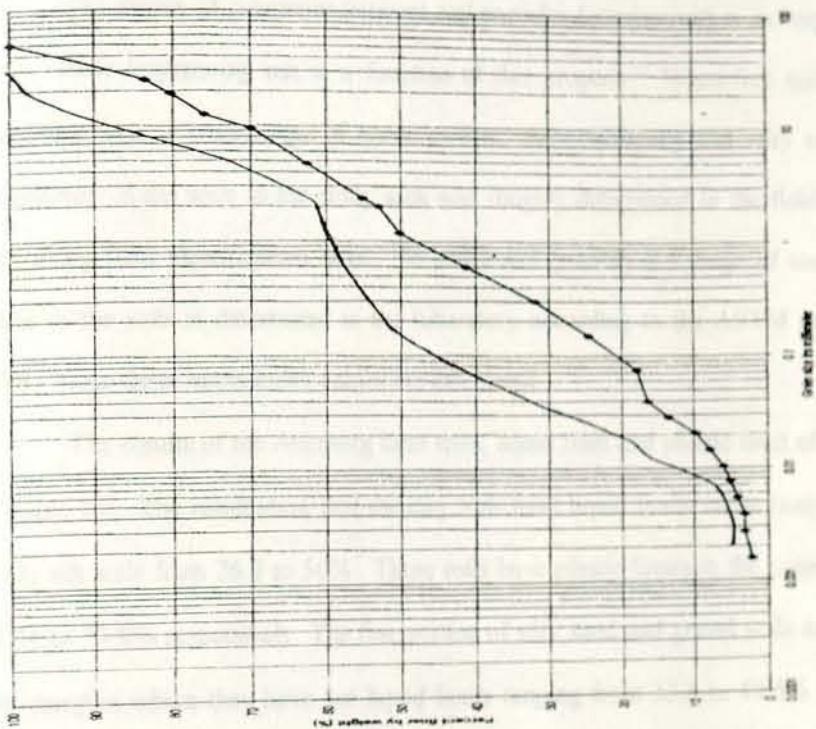
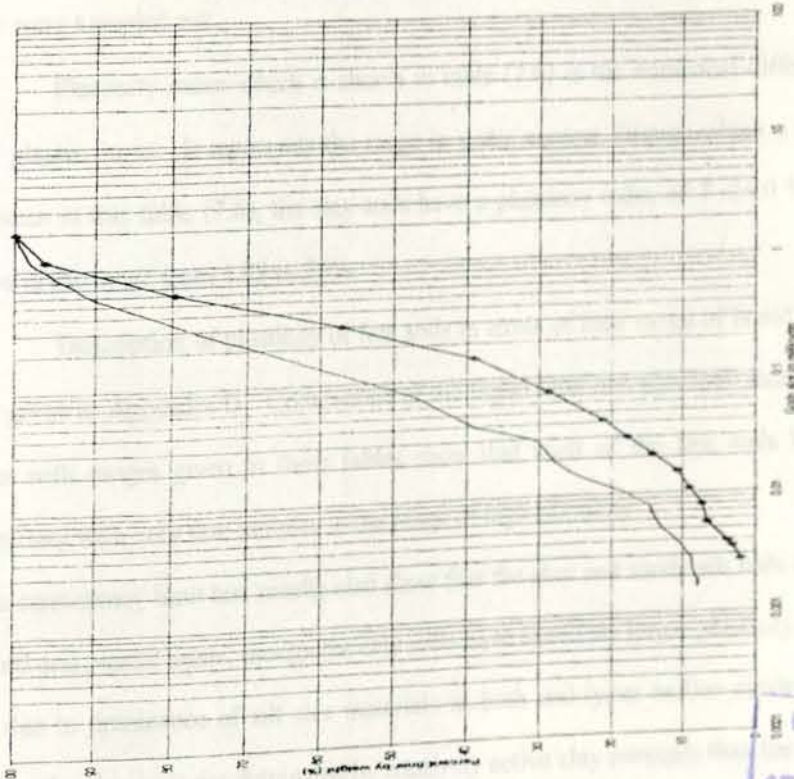


Fig. 7.1 Particle size distribution envelopes of the various soil types



Silty Gravel Soils



Silty Sand Soils

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7.4.2.4 Consistency (Atterberg) Limits and Index Values

The degree of coherence between soil particles (consistency) is an important property of fine soils. Their engineering use is a function of this property. When fine soils (silts and clays) are remoulded over a wide range of water content, the consistency can vary to a great extent. The consistency of the soils in the study area was roughly determined in the field as soft, firm, stiff and hard using field identification tests. To prove and quantify the range of consistency, the Atterberg limits of the soils is determined in the laboratory according to the ASTM procedure, designation D 4318. From these limits index values are calculated.

The results of the Atterberg limit tests, liquid limit and plastic limit of soil samples are shown in Table 7.6. The result show that the clay soils have liquid limits in the range from 28.0 to 53% and sandy silt soils from 26.7 to 56%. These soils have plastic limits in the range from 13.2 to 29% and 24.24 to 33.9% respectively. The fine portion of silty sand and gravel soils are non plastic except for few samples where they have got liquid limits ranging from 33.8 to 49.5% and plastic limits in the range from 25.87 to 31.13%. The comparison of the plastic limit with the moisture content of the soils, for which moisture content is determined, indicates that these soils were above their plastic state when they were sampled out.

Plasticity index which is shown in table (7.6) is the numerical difference between the liquid and plastic limits. It represents the range in water content through which a soil is in a plastic stage. As seen in this table (7.6), the clay soils have a plasticity index of 7 -24.0 % and the sandy silt soils have in the range from 1.89 to 25%.

Description of plasticity of fine soils in terms of their range of liquid limit and plasticity index are given in Appendix D. Comparison of the liquid limit and plasticity index of the soils of the study area with ranges given in these tables show that most of the fine soils have low to intermediate plasticity with very few samples in the range of high plasticity.

The consistency limit test results also show that the clay and sandy silt soils have comparable range of liquid and plastic limits, though the clay soils have relatively larger plasticity index values. This could be due to dominance of silt size materials in both soil types and/or minor difference in mineralogy where the clay soils are dominated by relatively active clay minerals than the sandy silt soils.

Sample	Depth (m)	Lithologic Type	LL	PL	PI
			%		
Tp4-S1	0.5 - 0.9	Clay Soils	29	22	7
Tp5-S1	0.45 - 1.2		28	21	7
Tp5-S2	0.2 - 2.3		30.55	21.37	9.18
Tp6-S1	0.45 - 0.85		35.9	24.07	11.83
Tp6-S2	0.85 - 1.75		46.3	26.58	19.72
Tp7-S1	0.65 - 1.05		35.7	24.84	10.86
Tp7-S2	1.05 - 1.4		51.9	28.2	23.7
Tp8-S1	0.45 - 0.5		40.8	23.25	17.55
Tp10-S1	0.4 - 0.5		33	22	11
Tp10-S2	0.5 - 1.8		49.85	27.22	22.63
Tp13-S1	0.45 - 1.8		53	29	24
Tp14-S1	0.4 - 0.9		30.16	13.22	16.94
Tp1-S1	0.2 - 0.75	Sandy silt soils	31.4	29.21	1.89
Tp1-S4	4.0 - 5.0				
Tp2-S2	0.35 - 0.75		52	26	25
Tp2-S3	0.75 - 3.0		51	31	20
Tp3-S1	0.7 - 1.7		28.2	24.24	3.96
Tp3-S2	1.7 - 2.1		26.7	25.13	1.57
Tp4-S3	2.5 - 3.3		53	31	21
Tp4-S4	3.3 - 4.0		39.6	33.86	5.74
Tp5-S3	2.3 - 4.25				
Tp6-S3	1.75 - 4.2				
Tp8-S4	3.75 - 4.2		56	32	24
Tp9-S1	0.45 - 0.95		28.5	25.62	2.88
Tp9-S2	0.95 - 1.55		47.4	29.28	18.12
Tp9-S3	1.55 - 3.9		35.9	30.54	5.36
Tp9-S4	3.9 - 4.75				
Tp10-S3	1.3 - 2.6		33	29.79	3.21
Tp11-S1	0.25 - 1.85		50	29.02	20.98
Tp11-S2	1.85 - 3.1		38.5	30.35	8.15
Tp12-S2	0.9 - 2.6				
Tp13-S2	1.8 - 3.25		48.8	31.37	17.43
Tp13-S3	3.25 - 3.8		45.9	29.98	15.92
Tp14-S2	0.9 - 2.55		37	28.54	8.46
Tp14-S3	3.0 - 3.8				
Tp1-S2	0.75 - 1.4	Silty Sand/gravel Soil	49.5	31.13	18.37
Tp1-S3	1.4 - 4.0		33.8	25.87	7.93
Tp2-S1	0.1 - 0.35				
Tp3-S3	2.1 - 5.0				
Tp4-S2	0.9 - 1.4				
Tp6-S4	4.2 - 5.0				
Tp8-S2	1.85 - 2.45				
Tp8-S3	2.45 - 3.75				
Tp10-S4	2.6 - 5.0				
Tp11-S3	3.1 - 5.0		44.55	30.59	13.96
Tp12-S1	0.2 - 0.9				
Tp12-S3	2.6 - 3.7				
Tp12-S4	4.2 - 4.4				
Tp13-S4	3.8 - 4.55				

Table 7.6 Atterberg limits and plasticity index values of the soils of Awassa area

In this work, it is also attempted to evaluate the application of the one point method(Murthy, 1992) for the determination of the liquid limit of the soils of the study area. It is found out that the one point method using Equation (7.4.2.4a) gives relatively comparable values to the liquid limit of the soils determined by three point method (table 7.7).This method can be used for simple practical applications but it will not totally replace the three point method.

$$LL(\%) = w \left(\frac{N}{25} \right)^n \quad 7.4.2.4a$$

Where w (%) is the water content corresponding to the number of blows N , and

n , an index whose value vary from 0.121 to 0.068. An average value of 0.1 gives best estimate for the soils of the study area.

LL (%) is the liquid limit determined by one point method.

7.4.2.5 Activity of the Clay Soils

From the result of Atterberg limits and particle size distribution determination, it is tried to estimate the activity of the clay soils of the study area (table 7.8). The activity of clay soils is simply the ratio of the plasticity index to the clay fraction in percent of the whole soil. The result show that the clay soils of the study area have activity values of 0.55 to 2.12., i.e. in the range of inactive to highly active clays according to the classification by Head (1984, Appendix D).

Sample	Depth (m)	Lithologic Type	Activity
Tp4-S1	0.5 - 0.9	Clay Soils	0.55
Tp5-S1	0.45 - 1.2		1.05
Tp5-S2	0.2 - 2.3		0.77
Tp6-S1	0.45 - 0.85		0.59
Tp6-S2	0.85 - 1.75		1.79
Tp7-S1	0.65 - 1.05		0.91
Tp7-S2	1.05 - 1.4		0.70
Tp8-S1	0.45 - 0.5		0.59
Tp10-S1	0.4 - 0.5		1.1
Tp10-S2	0.5 - 1.8		1.51
Tp13-S1	0.45 - 1.8		1.8
Tp14-S1	0.4 - 0.9		2.12

Table 7.8 Activity of clay soils of Awassa area

7.4.2.6 Free Swell

This test is a very simple laboratory test proposed by Gibbs and Holtz (1956 in Girmay Mahari, 1992). It is defined as the increase of the soils volume from a loose dry powder form when it is pored into water expressed as the percentage of the original volume. The test gives a fair approximation of the degree of expansiveness of the soil sample. For the present work, the test is carried out by poring very slowly of 10 cubic centimetre of that part of the dry soil passing No. 40 sieve into a 100 cubic centimetre graduated measuring cylinder and letting the content stand for approximately twenty four hours until all the soil completely settles on the bottom of the cylinder. Then the final volume of soil is noted.

$$\text{Free swell (\%)} = \frac{\text{Final volume} - \text{Initial volume of soil}}{\text{Initial volume}} \times 100$$

Table 7.9 shows the free swell values for the tested samples. The soils of the study area exhibit different ranges of free swell values, - 29 to 30%. According to Head (1984 in Lulseged Ayalew, 1992), soils with free swell values less than 50% are not likely to show expansive properties hence the soils of the study are can be considered as non-expansive. However, the negative values mean that there are reduction in volume due to hydro- compaction which is commonly a property of collapsible soils. Collapsible soils are soils that are susceptible to large decrease in bulk volume when they become saturated. In most cases the deposits are characterised by loose structures of bulky shaped grains, often in the silt to fine sand size range (Mitchell , 1976). They may be residual, water deposited, or aeolian deposits (Dudely, 1970 in Mitchell ,1976).

Large negative values are observed for sandy silt and silty sand/gravel soils especially at depths of below 2m which could be of collapsible soil types. The confirmation of these soils weather they belong to collapsible type or not requires detail investigation using undisturbed samples. At this point it is worth mentioning that precaution should be taken in the foundation depth of large scale engineering structures since these soils are distributed almost in all of the investigated area.

Sample	Depth (m)	Lithologic Type	free swell (%)
Tp4-S1	0.5 - 0.9	Clay Soils	-5
Tp5-S1	0.45 - 1.2		7.5
Tp5-S2	0.2 - 2.3		2.5
Tp6-S1	0.45 - 0.85		4
Tp6-S2	0.85 - 1.75		15
Tp7-S1	0.65 - 1.05		0
Tp7-S2	1.05 - 1.4		30
Tp8-S1	0.45 - 0.5		20
Tp10-S1	0.4 - 0.5		11.5
Tp10-S2	0.5 - 1.8		25
Tp13-S1	0.45 - 1.8	Sandy silt soils	0
Tp14-S1	0.4 - 0.9		0
Tp1-S1	0.2 - 0.75		-4
Tp1-S4	4.0 - 5.0		-17.5
Tp2-S2	0.35 - 0.75		30
Tp2-S3	0.75 - 3.0		17
Tp3-S1	0.7 - 1.7		0
Tp3-S2	1.7 - 2.1		0
Tp4-S3	2.5 - 3.3		-17.5
Tp4-S4	3.3 - 4.0		4.5
Tp5-S3	2.3 - 4.25		-29
Tp6-S3	1.75 - 4.2		4
Tp8-S4	3.75 - 4.2		16.5
Tp9-S1	0.45 - 0.95		0
Tp9-S2	0.95 - 1.55		20
Tp9-S3	1.55 - 3.9		-20
Tp9-S4	3.9 - 4.75		-20
Tp10-S3	1.3 - 2.6		14
Tp11-S1	0.25 - 1.85		17.5
Tp11-S2	1.85 - 3.1		10
Tp12-S2	0.9 - 2.6	Silty Sand/gravel Soil	-20
Tp13-S2	1.8 - 3.25		20
Tp13-S3	3.25 - 3.8		25
Tp14-S2	0.9 - 2.55		4
Tp14-S3	3.0 - 3.8		10
Tp1-S2	0.75 - 1.4		15
Tp1-S3	1.4 - 4.0		-11
Tp2-S1	0.1 - 0.35		7.5
Tp3-S3	2.1 - 5.0		0
Tp4-S2	0.9 - 1.4		0
Tp6-S4	4.2 - 5.0		-25
Tp8-S2	1.85 - 2.45		-16
Tp8-S3	2.45 - 3.75		-17.5
Tp10-S4	2.6 - 5.0		-10
Tp11-S3	3.1 - 5.0		-10
Tp12-S1	0.2 - 0.9		10
Tp12-S3	2.6 - 3.7		-12.5
Tp12-S4	4.2 - 4.4		-20
Tp13-S4	3.8 - 4.55		-15

Table 7.9 Free swell (%) for soils of Awassa

7.4.2.7 Other Relevant Engineering Properties

Due to lack of undisturbed soil sampler and In situ test equipment, determination of soil properties that require undisturbed samples were not carried out in the present work. However, from existing site investigation reports and recommendations a summary of the range of values of some of these properties for the soil types in the study area is presented in table (7.10). In situ test results are also included.

Soil type	Clay soils	Sandy silt soils	Silty sand/gravel soils
Shrinkage limit(%)	15 - 28	0 - 30	0 - 23
Dry unit weight (Kg/m^3)	791 - 1350	658 - 1360	947 - 1340
Natural unit weight (Kg/m^3)	1197 - 1750	816 - 1830	1159 - 1730
Void ratio	1.05 - 2.01	1.002 - 2.57	0.89 - 1.85
Unconfined compressive Strength (Kg/cm^2)	1.4 - 3.5	0.8 - 3.7	0.9 - 2
Angle of internal friction(ϕ)	22 - 32	23 - 38	21 - 36
Cohesion (Kg/cm^2)	0.01 - 0.05	0.00 - 0.03	0.00 - 0.03
Standard penetration test (SPT)(blows)	11 - 40	13 - 47	19 -36

Table 7.10 Compiled relevant geotechnical properties of soils of the study area

(source: Archives of Building Design Enterprise and Building Collage Soil Testing Departments)

Chapter Eight

Engineering Geological Classification of the soils and rocks

8.1 General

Engineering geological classification involves grouping of soils and rocks into categories which exhibit similar properties so as to provide a systematic method of description. The classification usually takes into consideration both the field and laboratory results then with a joint guidance it is possible to identify, characterise and delineate uniform soil and rock types.

8.2 Classification of the soils

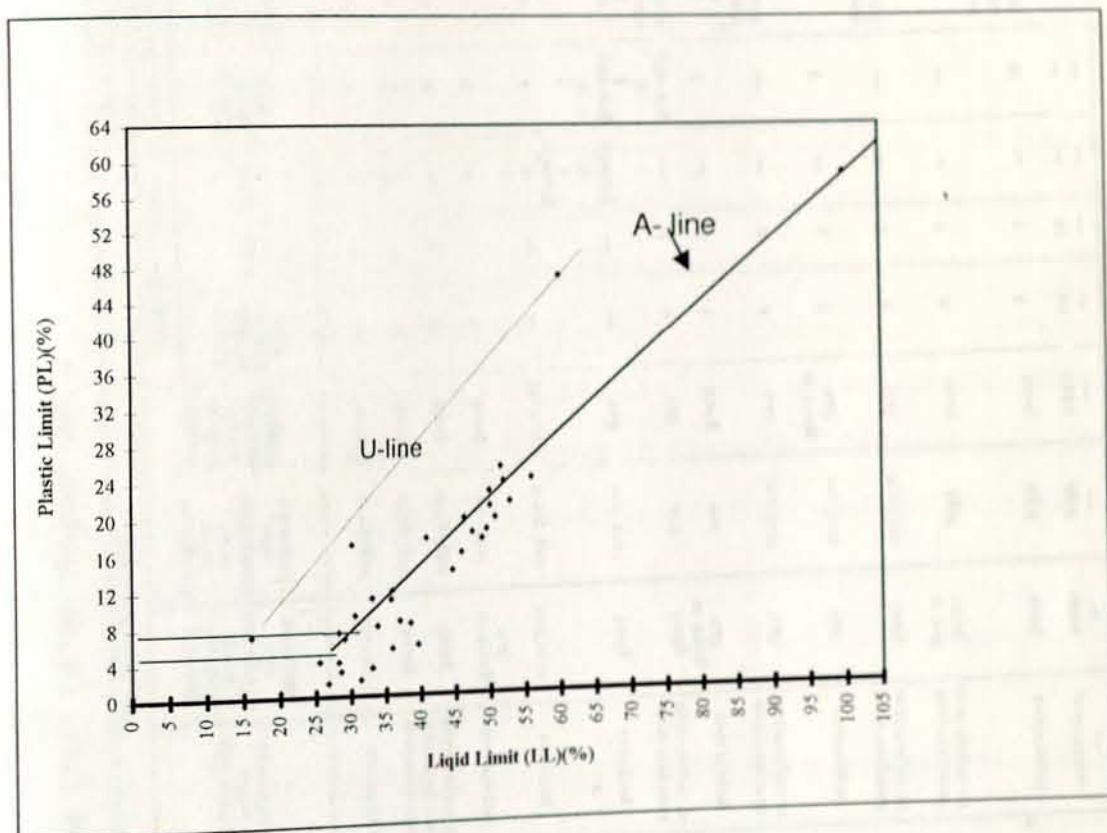
The classification of soils into different groups has been employed differently in different disciplines. However, a classification that has engineering applications is widely employed in engineering geological and civil engineering disciplines. These classifications involve a combination of particular size and measures of plasticity and are a textural - plasticity soil classification. The most widely used ones are those that divide soils in an orderly, easily remembered system of groups, or classes, that have similar physical and engineering properties and that can be identified by simple and inexpensive tests (Johnson, 1988). These groups ideally provide estimates of both the engineering characteristics and performance of soils for design and construction engineers.

In this work the Unified Soil Classification System (USCS) is employed. This system was developed co-operatively by the U.S. Army Corps of Engineers (USAE) and the U.S. Bureau of Reclamation (USBR) and published in 1953 by both agencies (USAE, 1953; USBR, 1953). It has since been adopted by the American Society for Testing and Materials (ASTM, 1983 in Johnson, 1988). It is routinely used world-wide and is accepted as for international geotechnical communication (Mirza, 1982 in Johnson, 1988).

The USCS method is based on grading of the constituent particles and plasticity finer than 0.425 mm. Accordingly, when more than half of the soil sample is larger than the No. 200 sieve, it is classified as coarse grained and is further subdivided by sieving and gradation. When more than half

Sample	Depth (m)	Lithologic Type	Gravel	Sand	silt	clay	USCS	Classification
				%			Symbol	description
Tp4-S1	0.5 - 0.9	Clay Soils		49	39	12	CL-	Sandy silty clay
Tp5-S1	0.45 - 1.2			45	48	7	CL-	Sandy Silty clay
Tp5-S2	0.2 - 2.3			31	57	12	CL	Sandy lean clay
Tp6-S1	0.45 - 0.85			21	59	20	CL	Lean clay with sand
Tp6-S2	0.85 - 1.75			24	65	11	CL	Lean clay with sand
Tp7-S1	0.65 - 1.05			22	66	12	CL	sandy lean clay
Tp7-S2	1.05 - 1.4			8	58	34	CH	Fat Clay
Tp8-S1	0.45 - 0.5			22	48	30	CL	Lean clay
Tp10-S1	0.4 - 0.5			31	59	10	CL	Sandy lean clay
Tp10-S2	0.5 - 1.8			26	59	15	CL	Lean clay
Tp13-S1	0.45 - 1.8			35	52	13	CH	fat clay
Tp14-S1	0.4 - 0.9			44	48	8	CL	Sandy lean clay
Tp1-S1	0.2 - 0.75	Sandy silt soils		47	45	8	ML	Sandy Silt
Tp1-S4	4.0 - 5.0			24	67	9	ML	Silt with sand
Tp2-S2	0.35 - 0.75			23	49	28	MH	Sandy elastic silt
Tp2-S3	0.75 - 3.0			30	63	7	MH	Sandy elastic silt
Tp3-S1	0.7 - 1.7			47	46	7	ML	Sandy silt
Tp3-S2	1.7 - 2.1			34	58	8	ML	Sandy silt
Tp4-S3	2.5 - 3.3		12	16	58	14	SM	sandy silt
Tp4-S4	3.3 - 4.0			26	61	13	ML	Silt With sand
Tp5-S3	2.3 - 4.25			34	62	4	ML	Sandy silt
Tp6-S3	1.75 - 4.2			25	62	13	ML	Sandy silt
Tp8-S4	3.75 - 4.2			46	48	6	MH	Sandy elastic Silt
Tp9-S1	0.45 - 0.95			26	59	15	ML	Silt with sand
Tp9-S2	0.95 - 1.55			44	50	6	ML	Sandy silt
Tp9-S3	1.55 - 3.9			22	67	11	ML	Silt with sand
Tp9-S4	3.9 - 4.75			38	59	3	ML	Sandy silt
Tp10-S3	1.3 - 2.6			27	68	5	ML	Silt with sand
Tp11-S1	0.25 - 1.85			20	51	29	MH	Elastic silt with sand
Tp11-S2	1.85 - 3.1			30	56	14	ML	Sandy silt
Tp12-S2	0.9 - 2.6			24	66	10	ML	Silt with sand
Tp13-S2	1.8 - 3.25			32	62	6	MS	Sandy Silt
Tp13-S3	3.25 - 3.8			14	64	22	ML	Silt
Tp14-S2	0.9 - 2.55			20	58	22	ML	Silt with sand
Tp14-S3	3.0 - 3.8			21	74	5	ML	Silt with sand
Tp1-S2	0.75 - 1.4	Silty Sand/gravel Soil		71	27	2	SM	Silty Sand
Tp1-S3	1.4 - 4.0			55	39	6	SM	Silty Sand
Tp2-S1	0.1 - 0.35			77	19	4	SM	Silty Sand
Tp3-S3	2.1 - 5.0		16	42	38	4	SM	Silty sand with gravel
Tp4-S2	0.9 - 1.4			73	20	7	SM	Silty Sand
Tp6-S4	4.2 - 5.0		34	34	29	3	SM	Silty sand with gravel
Tp8-S2	1.85 - 2.45			56	41	3	SM	Silty sand
Tp8-S3	2.45 - 3.75		3	68	25	4	SM	Silty sand
Tp10-S4	2.6 - 5.0			53	39	8	SM	Silty sand
Tp11-S3	3.1 - 5.0		37	45	16	2	SM	Silty sand with gravel
Tp12-S1	0.2 - 0.9			56	40	4	SM	Silty sand
Tp12-S3	2.6 - 3.7		44	30	23	3	GM	Silty gravel with sand
Tp12-S4	4.2 - 4.4		62	14	22	2	GM	Silty gravel
Tp13-S4	3.8 - 4.55		26	30	40	4	SM	Silty sand with gravel

Table 8.1 Classification of soils of Awassa area according to USCS



of the soil sample is smaller than the number 200 sieve, it is classified as fine - grained and is subdivided primarily by liquid limit values and degree of plasticity commonly using the plasticity chart or A- line diagram. This classification is presented in Appendix D.

The soils of the study area, according to this classification scheme, are dominantly fine grained soils with intercalation of coarse grained soils. Some of the soils fall above the A-line and most below the A-line, showing the dominance of silty soils in the area (table 8.1 and fig 8.1). Most of the soils belong to ML (inorganic silts, very fine sands, rock flour, silty or clayey fine sands), SM (silty sands, sand - silt mixtures) and CL (inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays) and few are in groups MH (inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts), CH (inorganic clays of high plasticity, fat clays) and GM (silty gravels, gravel-sand-clay mixtures).

In table(8.2) a general performance characteristics of USCS group is presented. From this table, it can be seen that the soils of the area which mostly belong to ML, SM and CL groups in the USCS are in the range from poor to fair for most civil engineering works except for SM soils which can be good to some of civil engineering works.

The lithological types described in chapter (7) well much with the dominant classification groups. The clay soils include the CL and CH groups, the silty sand and silty gravels are grouped in the USCS as SM and GM respectively. The sandy silt soils grouped as ML and MH. Hence, the description given in chapter (7) for the lithological types applies also to these groups.

8.3 Engineering Geological Classification of Rocks

The bases for the classification of rock units in the study area is restricted on their lithology and index values for the material strength as determined by the use of Schmidt hammer test and simple field hammering tests. In the area investigated, the build up areas and the possible areas of expansion of Awassa town, three different types of rocks outcrop: basalts with associated scoria, rhyolites and hyaloclasites. A chemical analysis is carried out on representative fresh basalt and rhyolite samples and presented in table (8.3). The analysis is carried out for identification and assessment of occurrence of defective substances which impair the use of these rocks for engineering purpose.

Basalts: these rocks outcrop on the south-eastern side of the town close to Textile Factory. They are black in colour and occur in a form of medium to very large blocks forming a small ridge running N 110° S along with the associated scoria. The basalts have a vesicular texture and is more of scoracious type. As seen in table (8.3) these rocks contain lower amounts of SiO₂ hence are of good rocks as concrete aggregates. When these rocks are hammered they give strong rebound and require many blows of a geological hammer to break intact rock specimens, hence described as very strong rocks (Grade R₅) according to Hoek's Classification (Appendix D). However, from the results of Schmidt hammer test performed in the field the strength of these rocks range from 290 - greater than 650 Kg/cm². Except for their limited volume, these rocks can be good sources of aggregate for concrete and asphalt.

Rhyolites: These rocks form isolated ridges in the eastern and south-eastern side of the town. When fresh they are gray in colour. They are fine grained in texture and may require petrographic or chemical analysis for identification. The chemical analysis result is presented in table (8.3). They are fractured and slightly weathered in most of the top part and along the fracture planes. Those in the eastern side are normally in medium to very large block forms forming small hills. The rhyolite ridges to the south-eastern side are also fractured but not in separated blocky forms. The fracture spacing ranges from less than 2 cm to greater than 1m. They show flow foliation oriented in N 70° E which adds the anisotropy of these rocks. Due to this intense fracturing they are easily quarried for road construction as well as for concrete aggregate. These rocks are described as very strong rocks (Grade R₅), according to the Hoek's classification (Appendix D), on the bases of field hammering test. They give a strong rebound and require many blows to break them. From field Schmidt hammer test result, it is found out that these rocks have a strength in the range from 310 to greater than 700 Kg/cm². Since the rhyolite ridges are widely used as a source of aggregate, various aggregate tests were carried out in this work to evaluate their performance (table 8.4). As seen in this table these rocks are of good quality as an aggregate source except for the high silica content which may be active for alkali- silica reaction when they are used as concrete aggregate.

Table 8.3 Chemical analysis data for rocks of the study area

Sample Type	Location	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	Na ₂ O	K ₂ O	MnO	TiO ₂	P ₂ O ₅	H ₂ O	LOI
Basalt	Tumura	47.92	15.87	12.32	8.89	5.46	1.68	0.12	0.21	4.02	0.59	0.27	0.21
Rhyolite	Kika	75.82	9.94	4.30	0.14	0.13	3.55	4.31	0.13	0.21	0.06	0.68	<0.01

Table 8.4 Aggregate test result for the rhyolite sample from Kika ridge

Test type	Test result	Maximum allowable	procedure
Soundness test by sodium sulphate	% loss 0.86	% loss 12	ASTM C 33
Abrasion	% wear 10.18	% wear 50	ASTM C 33
Stripping	above 95%	above 95%	ASTM D1664
Flakiness Indices	23%	35%	BS 882
Bulk specific gravity	2.42		
Bulk specific gravity(SSD)	2.43		
Apparent specific gravity	2.46		
Water absorption	0.67%		

Hyaloclastites and tuffs: As described in chapter (3) these rocks form isolated flat topped hills in the study area concentrated in the eastern and south eastern side. They are light gray to yellowish in colour and contain large blocks of different rock fragments embedded in fine grained matrix. They are highly weathered and affected by two sets of joints oriented N 30° E and N 115° S. The joints are filled in most of the places with calcite and clay like materials and are close to widely spaced (2 cm to 1m). In some places these deposits show layering. According to the same classification system used for others, these rocks are described as rocks of very weak to weak strength (group R₁ - R₂) on the bases of field hammering test. From Schmidt hammer rebound test performed in the field, these rocks have compressive strength values of less than 100 Kg/cm².

Chapter Nine

Engineering Geological Mapping

9.1 General

Engineering Geological mapping is a type of geological map which provides a generalised 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, 1972).

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 (IAEG, 1981). 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.

IAEG(1981) has suggested the following classification based on lithology and mode of origin for the purpose of engineering geological mapping:

The Engineering Geological Type (ET) : has the highest degree of physical homogeneity. It should be uniform in lithological character and physical state. These units can be characterised by statistically determined values derived from individual determinations of physical and mechanical properties and are generally shown only on large-scale maps.

Lithological Type (LT): is homogeneous throughout in composition, texture and structure, but usually is not uniform in physical state. Reliable values of average mechanical properties cannot be given for the entire unit; usually only a general idea of engineering properties, with a range of values, can be presented. These units are used on large-scale, and where possible, on medium-scale maps.

Lithological Complex (LC): comprises a set of genetically related lithological types developed under specific palaeogeographical and geotectonic conditions. Within a lithological complex the spatial arrangement of lithological types is uniform and distinctive for that complex, but a lithological

complex is not necessarily uniform in either lithological character or physical state. In consequence, it is not possible to define the physical and mechanical properties of the whole lithological complex, but only to give data on the individual lithological types comprising the complex, but only to give data on the individual lithological types comprising the complex and to indicate the general behaviour of the whole lithological complex. The lithological complex is used as a mapping unit on medium - scale and some small-scale maps.

Lithological Suite (LS): comprises many lithological complexes that developed under generally similar palaeogeographical and tectonic conditions. It has certain common lithological characteristics throughout which impart a general unity to the suite and serve to distinguish it from other suites. Only very general engineering geological properties of a lithological suite can be defined. These units are only used on small- scale maps.

A full description of a soil or rock for engineering purposes involves the assignation of the appropriate geological name together with as much detailed information as can be gained from the examination of a sample or exposure. The description may be systematic if a limited range of standard descriptive terms is used and for ease of application each descriptor may be classified. On the other hand, a classification should place the soil or rock in a limited number of groups each with definable properties, which can give a good guide to engineering performance (IAEG, 1981).

In engineering geological mapping, a three-part approach to engineering geological description and classification has been adopted (IAEG, 1981), involving: (i) a taxonomic classification of rock and soil units delineated on engineering geological maps at various scales, (ii) a lithological classification, and the assignment of fundamental lithological names to rock and soil units, and (iii) a description of rock and soil material and mass and the assignment of descriptive names for rock and soil types. In the present study the latter two are used.

9.2 Delineation and Engineering Geological Characteristics of the Mapping units

The domain of the engineering geologists is in most cases within the top few meters of the earth's surface. Hence, surfacial deposits are of critical importance in an engineering geological map. The term "soil" is used for all material which can be excavated without the use of explosives or

equipment for loosening , and are mapped in an area if their thickness exceeds 1m (Soesters and Rangers, 1980). According to this authors, the thickness of 1m is chosen for two reasons: first, the surface morphology of the underlying bedrock is usually recognisable through the overlying soil if its thickness is less than 1m and not recognisable when the thickness exceeds 1m. Second, the engineering geological importance of a soil cover of less than 1m is rather limited. In the study area, however, the top 30 to 50 cm thick soil is not critically evaluated and may not be considered for mapping purposes since people in most cases excavate to a depth of this much level for different purposes including for foundation.

Based on the IAEG (1981) recommendation, above described, the surface engineering geological mapping of the Awassa town area at a scale of 1:10,000 is carried out (Appendix F) on the bases of identifying the main lithological units and/or combination of units having similar engineering geological properties, as mapping units. The mapping units are characterised and described based on the information obtained from field observation and sampling followed by systematic laboratory tests such as index properties. Considering the scale and purpose of the map and available information four mappable units are delineated and their engineering geological characteristics are described as follows.

Lean and Fat Clays : this lithological unit comprises the soil groups CL, CH and the CL-ML soils. They are dark brown in colour with dark gray varieties. They are slightly cracked, columnar to prismatic and sometimes have sub angular blocky structure. Their thickness varies both vertically and laterally, ranging from 0.85 to more than 3 meters. These soils have natural unit weight in the range from 1197 to 1750 Kg/m³ and dry unit weight from 791 to 1350 Kg/m³. The liquid limit for this type ranges from 28.0 to 53%, plastic limit from 13.2 to 28.2%, shrinkage limit from 15 to 28%, plasticity index from 7 to 24%, organic content from 4.5 to 7.2% and free swell less than 30, hence they are non expansive. This soils have unconfined strength in the range from 1.4 to 3.5Kg/cm², angle of internal friction, cohesion and natural void ratio in the range from 2 to 32 degrees, 0.01 to 0.05 kg/cm² and 1.05 to 2.01, respectively, as obtained from existing site investigation reports. These soils are not suitable as foundation or fill material. They usually contain appreciable amount of silt and sand size materials.

Sandy Silt and silty Sand/Gravel soils : This unit comprise the soil groups MS, SM, and GM. Due to similarities in their engineering properties, except for Atterberg limits where SM/GM soils are commonly non plastic, and distribution in the area, these lithological units are mapped as one unit. They commonly occur interstratified to each other. They are light gray and olive yellow in colour in dry and wet state respectively. These soils become dominant in all the sites of the study area below a few meters from the surface. The sandy silt soils are the most dominant groups. These soil groups are suggested to be of collapsible type showing reduction in volume when saturated. The important engineering properties are presented in table (9.1). These soils are fair to good for most of civil engineering works.

Table 9.1 Summary of geotechnical properties of sandy silt and silty sand/gravel soils

Properties	Sandy Silt	Silty Sand/gravel soils
Liquid limit (%)	26.7 - 56	non plastic
Plastic limit (%)	24.24 - 33.9	
Plasticity Index (%)	1.89 - 25	
Shrinkage limit(%)	0 - 30	0 - 23
Natural unit weight (Kg/m^3)	816 - 1830	1159 - 1730
Unconfined Comp. Strength (Kg/cm^2)	0.8 - 3.7	0.9 - 2
Angle of internal friction (degree)	23 - 38	21 - 36
Cohesion (Kg/cm^2)	0.00 - 0.03	0.00 - 0.03
Void ratio	1.002 - 2.57	0.89 - 1.85
Standard Penetration Test (SPT) value	13 - 47	19 -36

Strong rocks (Basalts and Rhyolites): these units form simple ridges in the study area. They are fractured and commonly occur in medium to large blocky forms. Their strength ranges from strong to very strong, according to the Hoek's classification system, and are weathered on top parts and along fractures. From Schmidt hammer rebound tests performed in the field they have a compressive strength in the range from 290 Kg/cm^2 to greater than 650 kg/cm^2 . The basalts are black in colour and vesicular and scoracious while the rhyolites are light gray and showing flow folding structures. These rocks are not suitable for foundation from topography point of view. However, they are good sources of aggregates.

Weak rocks (Hayaloclastites and tuffs): these rocks forming simple peaks distributed here and there, are very weak to weak rocks ($R_1 - R_2$) and highly weathered. They form flat tops and contain

dominantly fine grained material with blocks and boulders of various rock types. They are affected by two sets of fractures oriented N 30° E and N 115° S which are mostly filled by calcite and clay like material, as discussed in chapter 3. These rocks are unsuitable for foundation.

In addition to the above mappable units, the scorias are mapped as separate units. They are reddish brown in colour forming small hills, and are dominantly of lapilli to block size. They are widely used as a source of construction materials especially for road construction.

Chapter Ten

Seismicity of the Study area

10.1 Seismicity of the area

Engineering works and earth processes interact in two distinct and important ways. First, natural processes influence how structural and other development occurs. Second, structures and development can alter natural processes, creating changes that may be undesirable (Johnson & DeGraff, 1988).

Forecasting, or prediction, the interaction of engineering works with earth processes is necessary for safety and reliability. Lowrance (1976 in Johnson & DeGraff, 1988) points out that being safe, or free of risk, is a matter of degree rather than being absolute. An element of risk is involved in engineering works, as it is with most human endeavours. Measuring risk for geologic processes is based on knowledge and principles. Acceptability of that measured risk is generally dictated by social, political, and economic factors. Therefore, determining acceptable risk requires the engineering geologist to participate in formulating public policy (Johnson & DeGraff, 1988).

The Ethiopian rift system, where the study area belongs, the Gulf of Aden and the Red sea are Characterised by a significant Seismic activity (fig 10.1). They are characterised by several intermediate size ($5.5 < \text{Magnitude (M)} < 6.5$) and a few larger earthquakes ($6.5 < M < 7.5$) (Fekadu Kebede, et al, 1996, 1997).

Since seismic instrumental recording started in Ethiopia in 1959, a number of destructive earthquakes have occurred in and around the country. This includes the earthquakes that occurred in and around the study area. For example, the Awassa earthquake in 1960 with $M_p^* = 6.3$; the 1983 Wendo-Genet and the 1985 Langano earthquakes with $M_b = 5.1$ and $M_b = 4.6$, respectively; the 1987 earthquake episode in the south most rift of Ethiopia, with maximum magnitude $M_s = 6.2$ (Fekadu Kebede and Like Mariam Asfaw, 1996, 1997) and the 1995 Awassa earthquake $M_b = 5.0$ (Atalay Ayele, 1998). These recent events indicate the level of seismic activity in the study area and MER.

The occurrence of the various major zones of seismicity and crustal deformation contributing to seismic hazard in the region is noted long ago. However, seismic source zones were

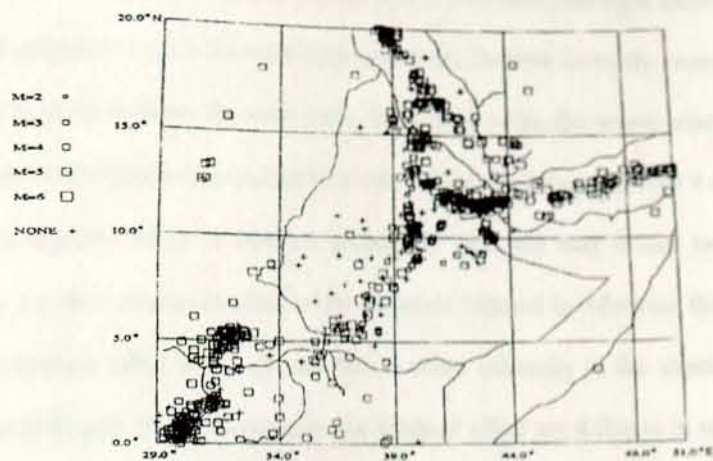


Fig 10. 1. Geographical distribution of earthquakes in the Horn of Africa for the period 1960-1993(after Atalay Ayele, 1998)

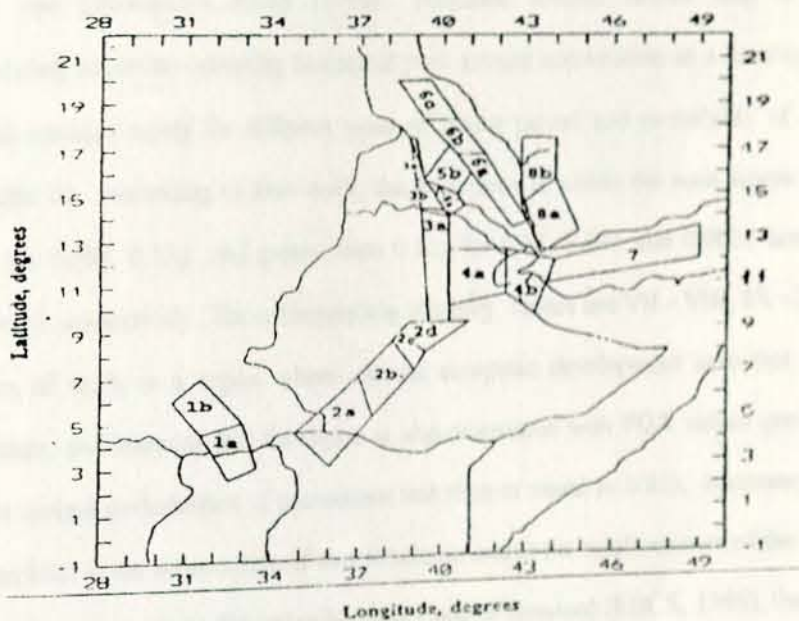


Fig.10.2

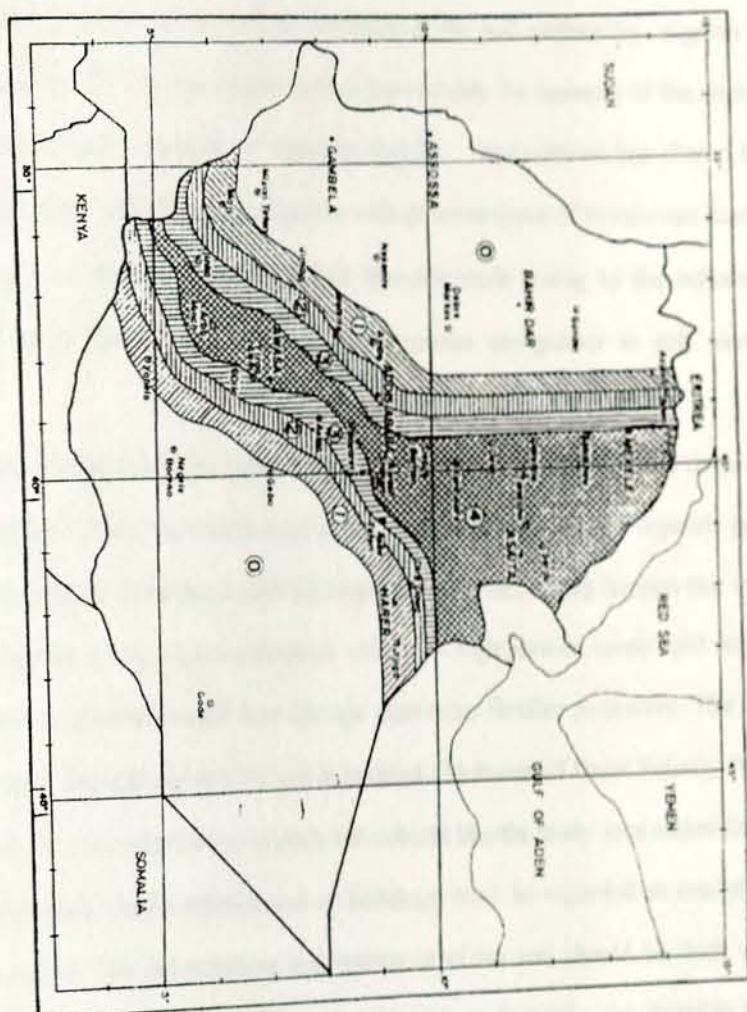
Seismotectonic source zones contributing to hazard evaluation in Ethiopia and the neighbouring countries. Numbers 1-8 in the figure show seismic source zones while alphabets (a, b, etc.) are intended to indicate sub-sources within each gross source.
(after Fekadu Kebede et al, 1996)

not identified until recently. Fekadu Kebede et al (1996) identified eight source zones in the region (fig 10.2) and presented briefly the main seismo-tectonic features currently known for each seismic zone. According to these workers, the study area, Awassa, is within the source zone 2 which is characterised by large and intermediate-size earthquakes with a focal depth ranging from 9 to 15 km.

Earthquake waves or motions propagated to a site may induce two kinds of influence on structures, i.e. the vibrational effect of the structure induced by vibration through foundation and the static deformation effect or the ground failure effect indirectly to the structure by the failure of the foundation (HU and SM). Since these two kinds of effect are different in mechanism and in the way they affect the structures, commonly two sets of maps to show the results of these two effects, one for ground failure and another for ground motion is employed. However, in the case of the study area only the latter type of map which is constructed on the regional bases is only available. Fekadu Kebede and Likemariam Asfaw (1996) presented seismic hazard map of Ethiopia and the neighbouring countries indicating horizontal peak ground acceleration as a function of g and modified Mercalli intensity levels for different years of return period and probability of not being exceeded (Appendix E). According to their work, the study area is within the zone where the horizontal PGA values are $0.08g$, $0.12g$ and greater than $0.12g$ for 0.01 , 0.005 and 0.0033 annual probabilities of exceedence, respectively. The corresponding intensity values are VII - VIII, IX - X, and X-XI. Since the area of study is a region where various economic development activities are currently being undertaken, and knowing that the region is also associated with PGA values greater than or equal to 0.1 (for annual probabilities of exceedence less than or equal to 0.01), necessary precautions should be taken both in the construction of new structures and in the reinforcement of the existing ones.

According, to the Ethiopian building Code of standard (EBCS, 1995), the study area is within Zone 4 with α_0 (the bedrock acceleration ratio) of 0.1 (Fig 10.3).

The seismic hazard maps so far developed for the country and/or the region, however, are more regional and do not consider the site surface geology effects (personal communication with Laikemariam Asfaw). However, the effect of site geology is an important factor in changing the seismic wave movements and hence the seismic response of foundations.



10.2 Influence of Soil on Seismic Response and Movements

Soils rest on bedrock through which seismic waves are transmitted to them. During an earthquake the behaviour of the subsoil with respect to the bedrock is similar to that of a structure with respect to the soil. The thickness of the various soil strata and their elastic properties determine the type of seismic movement which will be produced at the soil surface by a given movement on the surface of the bedrock. The soil characteristics thus modify the intensity of the seismic movement and also modify its frequency content or response spectra. Observations has shown that the effects of earthquakes at comparable distances in places with different types of terrain can have varying intensity as much as several degrees on the modified Mercalli scale owing to the influence of the subsoil (Mohammed Ezzat Sobaih, 1999). Generally intensities are greater in soft terrains than in firm terrains.

The characteristics of the subsoil also have an important effect on the frequency content of the surface movement. Since the reaction of a structure to an earthquake depends on the value of its characteristic period in accordance with the response spectrum, it may happen that in soft terrains with a response spectrum giving high acceleration values for high periods, more rigid structures (those with short characteristic periods) suffer less damage than more flexible structures. The opposite is applied for buildings with foundations on firm soil or bedrock (Mohammed Ezzat Sobaih, 1999).

Hence, in areas where there is thick soil column like the study area where the foundation soil is more than 30m thick, the foundation soil of buildings must be regarded as one of their fundamental structural members. The deformations and resistance of the soil should be dealt with as part of the design in the same ways as beams or walls. Since it is generally not possible to change the soil conditions, the structure and foundations should be sited to the specific characteristics of the soil at the building site.

The amplification factor of the ground for peak acceleration is affected by the shear wave velocity of the surface layer (Midvikawa and Kobayashi (1980 in Mohammed , 1996) Figure (10.4) shows this relation. For the study area the shear wave velocity of the surface layer is evaluated using the modified Imai's formula (1975) by Stuenkel et al (1984 in Mohammed , 1999):

$$V_s = 89.9 \times N^{0.3}$$

Where

N - the number of blows in standard penetration test

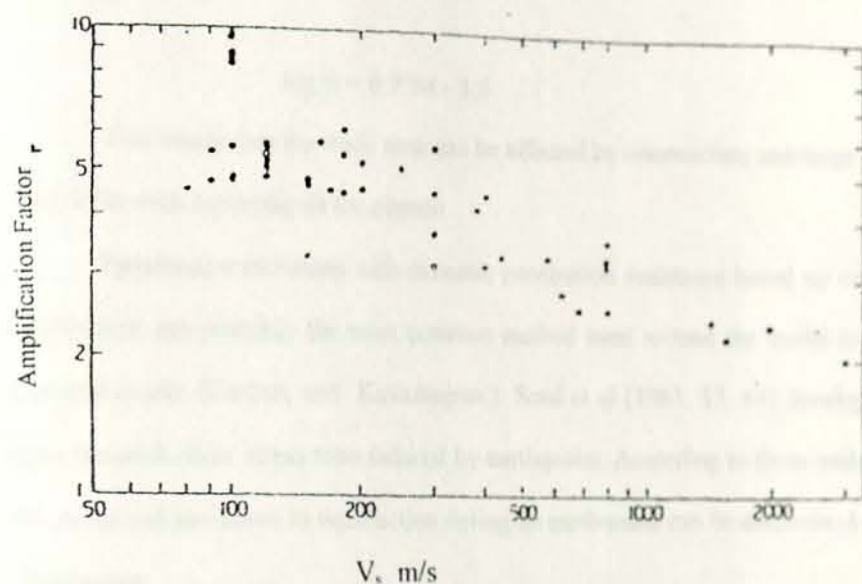
V_s - the horizontal shear wave velocity

Considering only N values of the top 5 - 8m thick soil column for an average N value of 17, V_s is calculated to be 210 m/sec. This shear wave velocity corresponds to an amplification factor of 5.5 in fig (10.4), which is in agreement to the commonly used amplification factor for Holocene unconsolidated deposits, $F = 4.5 - 5.5$, (Mohammed, 1998,1999). This means that the horizontal ground motion in the study area may exceed 0.55g for annual probabilities of exceedence less than or equal to 0.01. Hence, the seismic coefficient that has to be used in the design of structures in the area has to consider the amplification of the soil column.

10.3 Liquefaction Potential of the soils

In a narrow sense the word liquefaction means a complete loss of shear strength which can occur when a loose cohesionless soil is subjected to shear stress, either monotonic or cyclic. In a broader sense, this term has also been used to denote a partial loss of shear strength due to build-up of pore water pressure (Kezdi, 1988) . It results from the transfer of intergranular forces to the water which fills the pores. The resistance of these soils depends on the pressure transmitted by the contact between the particles. Therefore, when part of the intergranular pressure is transferred to the water, the soil loses a corresponding portion of its resistance. When the pore pressure induced by the earthquake passes away, concentrated currents of water rising through crevices are generated and carry sand up to the soil surface, building it up into mounds resembling small volcanoes, which are characteristic signs of such phenomena (Mohammed Ezzat Sobaih, 1999).

The vulnerability to liquefaction during an earthquake depends on various factor among which the magnitude of the stresses or strains induced in it is an important. Seed and Idriss (1971), based on contemporary records of damage due to the earthquakes experienced at Niigata in Japan for about 370 years, discovered that severe damages (owing to liquefaction) were only caused in three instances when the ground acceleration exceed 0.13g, though the duration of shaking is a



contributive factor. Moreover, for a given magnitude of earthquake, M , there appears to be a limiting epicentral distance, R , beyond which liquefaction is unlikely to occur. The limiting distance, R in Km, for $M \geq 6$ may be expressed as (Kuribayashi and Tatsuoka 1975 in Kezdi, 1988):

$$\log R = 0.77M - 3.6$$

This means that the study area can be affected by intermediate and large earthquakes (≥ 6.5 M) even occurring on the plateau.

Empirical correlations with dynamic penetration resistance based up on past performance in earthquakes are probably the most common method used around the world to evaluate liquefaction potential in situ (Carlton, and Kavazanjian,). Seed et al (1981, 83, 84) developed correlations based upon the peak shear stress ratio induced by earthquake. According to these authors, the index used to determine soil resistance to liquefaction during an earthquake can be determined conveniently from the relationship:

$$\frac{\tau_{av}}{\delta'_o} = 0.65 \times \frac{a_{max}}{g} \times \frac{\delta_o}{\delta'_o} \times r_d$$

Where a_{max} = maximum acceleration at the ground surface

δ_o = total overburden pressure at depth under consideration

δ'_o = effective overburden pressure at depth under consideration

r_d = a stress reduction factor which decreases from a value of 1 at the ground surface to a value of about 0.9 at a depth of 35ft.

The soil penetration resistance used in the correlation is normalised standard penetration resistance, N_1 , where N_1 , is the penetration resistance the soil would have under an effective overburden pressure of 96 KPa. The value of N_1 , can be determined from the value of N measured in a standard SPT investigation by means of the formula:

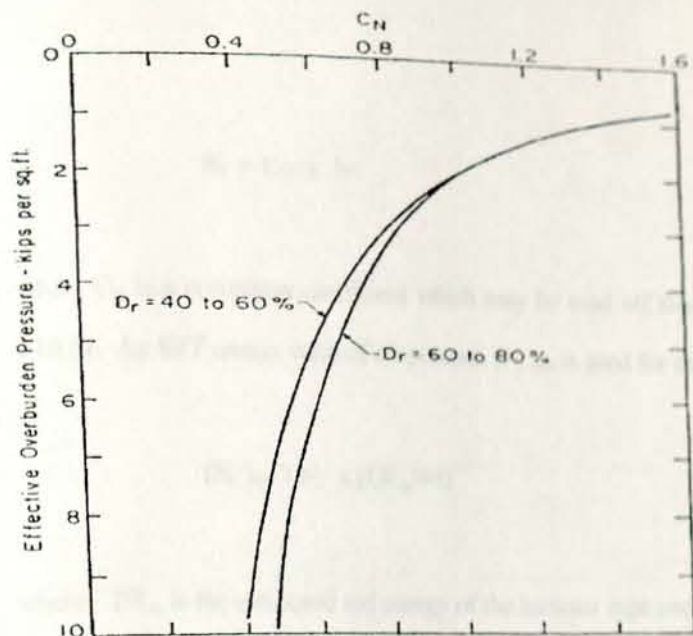


Fig 10.5 Chart for values of C_N (after Seed et al, 1984) 1ksf = 48 kpa

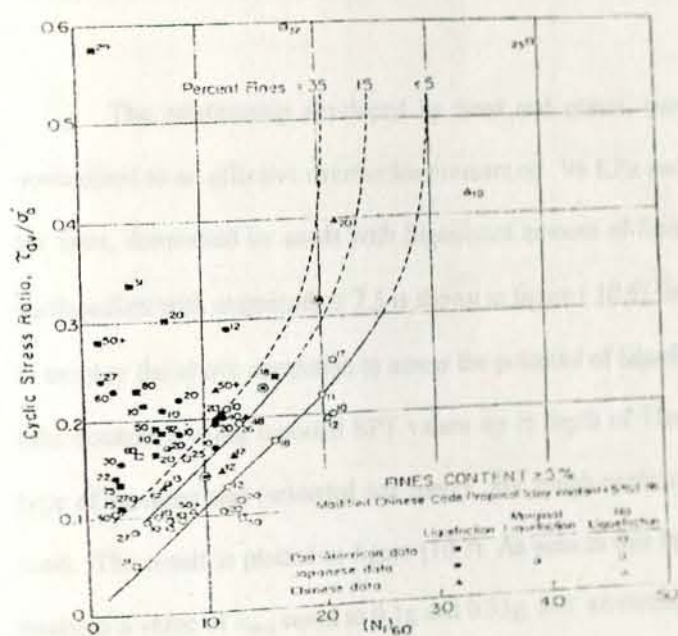


Fig. 10.6 Relationship between stress ratio causing liquefaction and N_1 values for silty sands for $M=7.5$ earthquakes (after Seed et al, 1984)

$$N_1 = C_N \times N$$

where C_N is a correction coefficient which may be read off from charts such as that shown in figure (10.5). An SPT energy ratio of 60 percent, $(N_1)_{60}$ is used for the correlation as defined below:

$$(N_1)_{60} = N_1 \times (ER_m/60)$$

where ER_m is the estimated rod energy of the hammer type used in SPT.

$(N_1)_{60}$ is used in the correlation because of the fact that no procedure is likely to deliver 100% of the theoretical free fall energy intended when the test was first developed, and uses instead a standard energy delivered to the drill stem equal to 60 % of the theoretical free fall energy (Seed et al., 1984).

The relationship developed by Seed and others, between τ_{av}/σ'_o and the SPT blow count normalised to an effective overburden pressure of 96 KPa and an SPT energy ratio of 60 % , $(N_1)_{60}$, for sites, dominated by sands with Significant amount of fines, which have and have not liquefied in earthquakes with magnitude $\cong 7.5$ is shown in figure (10.6). In the present study it has been attempted to employ the above correction to assess the potential of liquefaction of the soils which are close to the lake boundary using reported SPT values up to depth of 15m. However, no data is found about the type of hammer and estimated rod energy. For rough evaluation uncorrected N_1 value is used in this work. The result is plotted in figure (10.7). As seen in this figure liquefaction is very unlikely. In the analysis a value of a_{max} equal to 0.1g and 0.55g, and an earthquake magnitude of 7.5 is assumed.

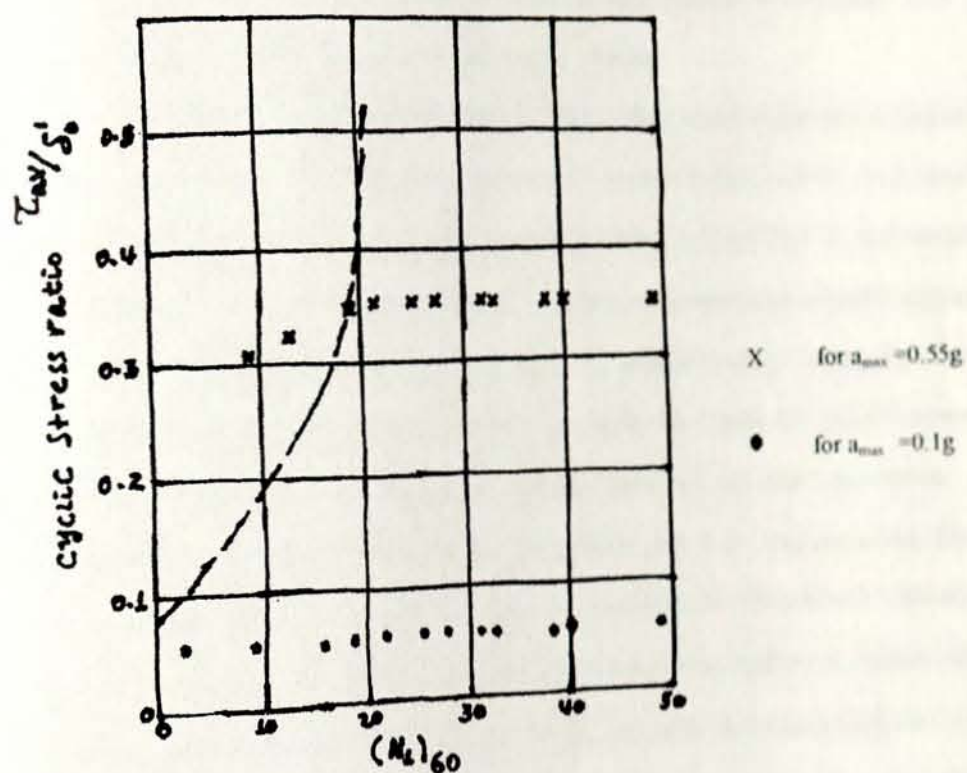


Figure 10.7 Field liquefaction behavior predicted for the study site using SPT- stress ratio chart
Proposed by Seed et al (1984), for $M = 7.5$ earthquake

Conclusion and Recommendation

Based on the increasing demands of peoples to construct new infrastructure in the town of Awassa, in relation to the development of the town being the capital of the Southern Nation Nationalities and Peoples Region, and the threat of the lake Awassa level rise to inundate part of the town, an engineering geological study addressing hydrogeological, hydrological and geotechnical conditions is carried out in and around the town of Awassa.

The area is within the Main Ethiopian Rift where recent tectonism, volcanism and crustal deformation is active. There are many signatures of these activities, and the study area is located in one of these signatures: Awassa mega caldera. The area is characterised by voluminous deposits of silicic volcanic products (Miocene to Recent) to the large part and some lacustrine deposits occupying the floor of the caldera. The area is dissected both by regional faults bordering the Ethiopian rift and many small to large faults within the caldera. The faults have generally NE/SW orientation though faults oriented transverse to the regional faulting directions are also encountered. The area is geomorphologically characterised by rolling type ground with some volcanic peaks, fault scarps and remnants of the old Awassa caldera wall and the peaked walls of the nested Corbbeti caldera. The floor of the caldera is occupied by water bodies, marshy areas, and dotted volcanic edifices. Active geodynamic processes, currently visible, are lacking except for the ground fracturing to southwestern side of the lake Awassa, and fumarolic activities on the Corbbeti caldera to the north of the study area.

The area gets an average precipitation of about 963 mm concentrated in the eight month long rainy season with moderate seasonal and inter-annual variability. The variabilities are sometimes as large as more than 25 % of the mean. Especially an increased amount of rainfall is recorded since 1996 with a concomitant decrement in total evaporation and rise in lake level, which has resulted enormous loss of private, government and non government property.

The fluctuation of the lake level with rainfall of the catchment is poorly correlated. However, a few months shifting of the rainfall gives a better fit evidencing the role of delayed runoff and groundwater inflow. The routing effect of lake Shallo and the swampy area on the amount of discharge of Tikurwuha river plays an important role for the time lag.

The water bodies in the study area generally show a large variability in their cation and anion content though most of them are a Na - HCO₃ type waters. Some degree of geochemical evolution (in concentration) is evident as the water flows from the uplands towards the rift and finally reaching the lake. The variability of the chemistry of the water bodies is a simple reflection of the degree of water rock interaction, the chemistry of the aquifer, external source of solutes via hot springs and degree of soil water interaction and removal of chemical constituents from the system through precipitation and extraction by micro-organisms.

The groundwater flow condition of the area is not assessed in detail due to lack of sensitive equipment to measure elevation and in accessibility of most of the water points. However, based on the available data, it is possible to say that groundwater generally flows towards the two lakes occupying relatively the lower elevations and finally reaching to its destination, lake Awassa. Groundwater is found relatively at shallow depths (< 40m) and in most cases in unconfined condition. A danger to foundations close to the lake and even towards the centre, due to large scale fluctuation of the groundwater table up to 5m, is anticipated and necessary precautions should be taken in both in the construction of new structures and reinforcement of the existing ones.

The chemical constituents of the waters of the study area made them unsuitable for various purposes. In most of the cases the above maximum concentration of fluoride, bicarbonate, silica and total hardness made most of the water bodies unsuitable for drinking, brewing, food processing etc. The use of these waters for irrigation may result salinity hazard, except for few dug wells and springs. Especially lake Awassa is totally unsuitable for irrigation. Their effect on construction materials: concrete brick and masonry is found out to be negligible. However, the degree of "aggressivity" is a function of the material type.

Water balance estimation of the lake revealed the non-equilibrium condition of the lake where a net surplus in subsequent years of high rainfall under reduced evaporation resulted a significant rise in the lake level. Besides the similar observations made on the southern rift valley lakes, and the monthly water balance computation result also support the effect of such environmental change as a major cause of the recent lake level rise. This is found out to be in agreement to the conclusion of Makin et al, (1975 in NUPI, 1989). Hence, remedial measures to protect the hinterland inundation of

the lake due to level rise should consider and evaluate the role, variability and periodicity of such micro environmental changes.

The selection for method of controlling the hinterland inundation of lake Awassa requires a detail study involving risk and economic analysis and environmental impact assessment, which is beyond the scope of present study. However, it is possible to forward some recommendation. A number of possible solutions are available including: transferring of water to neighbouring catchment, using the lake water for irrigation, strengthening of the existing dyke, construction of detention structures etc. However, transferring of water to neighbouring catchment may have ecological and environmental impact, besides it requires a constant running cost for pumping of water since it is not possible to transfer water to near by catchment area by gravity for topographic constrains. The lake water is found out to be unsuitable for irrigation as discussed above so that this will not be a solution. Hence, it seems more plausible to strengthen the existing dyke for an acceptable risk, safety factor and return period of lake level with a deeper cut off trench and appropriate construction material to protect vulnerable areas, supported by the construction of detention structures on the main affluent river, Tikurwuha, and applying soil conservation methods in the catchment. The runoff from the town could be collected using a good network of ditches, which collect water from the town, and it can be transferred to the lake after the necessary treatment. The development of vulnerable areas should be carefully done on the bases of risk analysis. Especially siting of economical and social facilities in these areas should be avoided or relevant engineering design should be applied.

The foundation where the town lies is found out to be a thick soil stratum ($> 30\text{m}$). Three dominant lithological types are identified in the field: Clay soils, sandy silt soils and silty sand/gravel soils. This is confirmed through a routine laboratory analysis. The fine soils are dominant in most of the sites and the coarse soils are more restricted towards the lake and at depth. The origins of these soils is suggested to be either lacustrine deposits and/or residual soils. The clay soils occupy relatively the elevated central and south-eastern part of the town and the possible expansion area. While the other types occurring in almost all the area. They commonly underlay the clay soils. These soils are dominantly massive showing no lamination and so on. The clay soils are mostly fractured and blocky in structure. According to the USCS classification scheme, the clay soils contain the groups CH, CL

and few CL-CM soils. The sandy silt soils embrace soil groups ML, MH and MSL, and the silty sand /gravel soils contain the groups SM, GM and SML soils.

Mineralogical and Chemical analysis of the samples indicated the occurrence of quartz, feldspars, mica flakes and rock fragments, and presence of Kaolinite, illite and Ca- montmorillonite is suspected indirectly from the other parameters of the clay soils. The soils are generally inorganic and their carbonate content is below 5 %. The content of Sulphate and Chloride are negligible in the soils of the area and hence, it is unlikely to create concrete deterioration.

Index properties of soils was determined using the procedures designated by ASTM (1996). Moisture content at the time of sampling ranges from 25.1 to 29.0% for clay soils, and from 18 to 46 % for sandy silt and silty sand/gravel soils, indicating that some of the samples were above their plastic limit at the time of sampling. The specific gravity of the soils is low, in the range from 2.2 to 2.5. The liquid limit and plasticity index of the clay soils ranges from 28 to 53% and from 7 to 24% respectively. The sandy silt soils have LL% in the range from 26.7 to 56% and PI % in the range from 1.89 to 25%. The silty sand /gravel soils are almost non plastic except for few samples where the fines portion have got some degree of plasticity. The clay soils are in the range of inactive to highly active types.

The soils of the area are found out to be non expansive as revealed by the simple free swell test. However, some of the sandy silt and silty sand/gravel soils show reduction in volume as large as 29% , thus suspected to have a collapsible structure. Making some insitu tests like insitu density, and laboratory tests on undisturbed samples like consolidation is recommended to better characterise the compressibility of these soils and to measure the specific subsidence. These soils may collapse both under load and when flooded by water and may also result differential settlement. To minimise the hazard it is advantageous to transmit the weight of the structure to deeper soil or rock layer by means of piles or caissons, and if not, to make the foundation as deep as possible whereby the thickness of the layer subject to subsidence hazard and, consequently, the absolute magnitude of the subsidence , might be reduced. In order to prevent subsidence, water had to be prevented from entering the zone under the foundation slab, at least in concentrated quantities.

The occurrence of strong to very strong and some very weak to weak rocks forming small peaks and ridges is evident in the area. Basalts and rhyolites belong to the first group while the

hyaloclastites and tuffs in the second. The strong varieties are found out to be suitable as source of construction material especially for aggregate source, while the latter ones are unsuitable to most of engineering uses. They are highly weathered and very weak.

Mappable units are selected on the bases of the scale and purpose of the map. Lithological units and engineering properties employed for the selection. Hence, four units are identified and mapped: clay soils, sandy silt and silty sand/gravel soils, basalts and rhyolites and Hyaloclastites. In addition, the scorias though occupy very small area, mapped as one unit from the point of their importance as construction materials.

✦ The seismicity of the area requires attention in the designing of structures. The thick soil column amplifies the seismic movement as large as 5 times, hence may affect the structures very severely. The seismic coefficient to be used in the design, therefore, should consider the characteristics of the foundation soil: consistency, compaction, homogeneity, damping and amplification properties. Knowledge of the foundation soil is essential for earthquake - resistant design. In the case of a design in which only static loads are considered, it may be more economical to compensate for an insufficient knowledge of soil properties by providing a greater margin of safety in the design. However, for a proper earthquake resistant design it is necessary to know the mechanical properties of the foundation soil, since in some cases a soil which behaves well under static loads will present serious problems when subjected to seismic loads, and it will not always be possible to remedy those problems by using a larger safety factor for static loads.

A better characterisation and definition of the seismic response of the foundation soils requires intensive investigation applying geophysical methods which could be an area of future research. Applying the geophysical methods especially seismic survey to define the shear wave response of the foundation soils will make it possible to carry out a microzonation of the study area and its surroundings.

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35. Basaltic Andesite	1974	1974	1974
36. Basaltic Andesite	1974	1974	1974
37. Basaltic Andesite	1974	1974	1974
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74. Basaltic Andesite	1974	1974	1974
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81. Basaltic Andesite	1974	1974	1974
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83. Basaltic Andesite	1974	1974	1974
84. Basaltic Andesite	1974	1974	1974
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87. Basaltic Andesite	1974	1974	1974
88. Basaltic Andesite	1974	1974	1974
89. Basaltic Andesite	1974	1974	1974
90. Basaltic Andesite	1974	1974	1974
91. Basaltic Andesite	1974	1974	1974
92. Basaltic Andesite	1974	1974	1974
93. Basaltic Andesite	1974	1974	1974
94. Basaltic Andesite	1974	1974	1974
95. Basaltic Andesite	1974	1974	1974
96. Basaltic Andesite	1974	1974	1974
97. Basaltic Andesite	1974	1974	1974
98. Basaltic Andesite	1974	1974	1974
99. Basaltic Andesite	1974	1974	1974
100. Basaltic Andesite	1974	1974	1974

Appendix A

Some of the Water Points data used for this work

No.	Locality	Grid reference		GWL, m	Source
		UTME	UTMN		
1	State farm unit 2	445176	780670	1689	1
2	EWUCA compund	443273	779488	1689	1
3	Shalo 1 B	443150	787200	1693	1
4	Boko (Alamura)	441515	771015	1696	1
5	Catholic hospital	440951	774122	1684	1
6	Wanza wood factory	442779	781335	1686	1
7	Ceramic factory	444253	782354	1688	1
8	Awassa collage	442024	779632	1686	1
9	Poultry Awassa	443347	775988	1686	1
10	Awassa office	443859	779850	1693	1
11	Wondo Kosha	434120	790051	1661	2
12	Sedine	439497	770573	1684	1
13	Samma	429345	785535	1784	2
14		426350	778528	1672	2
15	W-1 Shallo farm	443836	786728	1684	1
16	W-3 Shallo farm	444229	788916	1689	1
17	W-4 Shallo farm	443190	785010	1681	1
18	W-5 Shallo farm	443342	784426	1681	1
19	W-6 Shallo farm	442423	785532	1683	1
hand dug wells					
1	Jara gelcha	431007	774098	1681	
2	Jara gelcha	430394	774008	1680	
3	Erbora	430939	775542	1674	
4	Jirmancho	430704	776448	1670	
5	Multea	428233	777750	1695	
6	Awassa farm 3	431045	779974	1676	2
7	Kerresso	432842	778920	1675	2
8		441854	778911	1687	2
9	East of shemena	427574	773352	1691	2
10	Gebereal sefer	442284	778500	1693	
11	South of bus station	443125	778802	1695	
12	Cheffe sefer	445820	779757	1691	
13	Cheffe sefer	446143	779859	1685	
Cold springs					
1	Lake spring	437076	772052	1680	
2	Gemota Gale	444757	771017	1690	

1 - Southern Nation Nationalities and peopels Regional state Water, Mines and Energy Bureau
2 - MoWR (1999)

APPENDIX B

HYDROCHEMICAL DATA

Environmental parameters, and major cations and anions data for water samples from bore wells (from the specified sources)

No	Index No	Date of Sampling	Locality Name	2nd rd	Total Hardness	Total	Ca	Mg	Fe	Cl	F	NO3	Br	CO3	SO4	Source
					mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	
1	BH-1	15/03/08 E.C.	Textile Fact. No 7	444751	777656	170	500	28	8.2	540	201.4	12	21.6	32	21.6	0.7
2	BH-2	15/03/08 E.C.	Textile Fact. No 8	444725	777636	170	500	28	8.3	820	183.5	10	32	21.6	32	0.36
3	BH-3	14/03/08 E.C.	Textile Fact. No 1			150	510	28	7.8	940	200.2	20	28	19.2	0.22	70
4	BH-4	07/03/07 E.C.	Textile Fact. No 4			20	25	21	5.8	90	18.08	7.2	0.26	6	1.2	29
5	BH-5	04/03/07 E.C.	Textile Fact. No 8			140	500	31	7.8	900	197	8	36	14.3	1.1	48
6	BH-6	03/03/02 E.C.	Tobacco F1	445052	7775508	90	140	20	7.1	300	27.08	3	11.87	16	4.8	2.7
7	BH-7	30/06/07 E.C.	Togbe No 1	447105	791452	80	400	25	7.9	780	104	12	20	7.2	0.1	0.25
8	BH-8		Shilo Farm Unit 2.1	443215	798975			98			1014	12	15	4	10	10
9	BH-9		Shilo Farm Unit 2.2	443273	779486			98			795	175	14	8	10	10
10	BH-10		Water Res. Compound	445781	777224			95	8.3		1048	190	15	25	15	3
11	BH-11	20/01/07	Amelia Unit 2 farm			156	8				38.8	15.58	0.07	28	2.2	9.78
12	BH-12	12/07/06 E.C.	Chopwood Fact.(Almash)	442666	791844	138	260	22.1	8.4		34	4	12.64	0.04	32.5	2.11
13	BH-13	12/07/06 E.C.	Amelia-SOS 01			34	444	22.9			8	3.30	9	2.5	433.84	3.08
14	BH-14	15/06/06 E.C.	Amelia-SOS 01			80	750	25.5	9.2	1205			60	8.75	9.15	0.9
15	BH-15	05/06/06 E.C.	Camencia 2			48	12	21.3	8.3	259			5	1.46	14.64	2.28
16	BH-16	28/06/06 E.C.	Almash	441515	771015	138	20	25.7	8.7	1076	538		10	3.25	8.8	0.05
17	BH-17		Almash Borehole					94			19	16	10	3	83	0
18	BH-18		Wondogenet													1

Environmental parameters, and major cations and anions data for water samples from hand dug wells (from the specified sources)

No	Index No	Date of Sampling	Locality Name	2nd rd	Total Hardness	Total	Ca	Mg	Fe	Cl	F	NO3	Br	CO3	SO4	Source
					mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	
1	One-1		South of Amasia			187	50	18	12		38	0.8	248		0	53
2	One-2		South of Amasia (Dare School)			175	150	15	42	17	57	5.8	143		12	29
3	One-3		SE of Amasia			90	101	18	45	6	10	1	203		0	5
4	One-4		Wondogenet			58	19	18	15	5	14	3	100		0	5
5	One-5		Almash Borehole			48	200	23	13	2	38	10.1	523		18	102
6	One-6		Thar Mafra			119	270	19	8	82	10	500		0	110	35
7	One-7		Thar Mafra			83	140	20	4.5	36	8	34		0	38	2
8	One-8	Aug. 1999	East of Lake Amasia	441752	778111	137					48	5.11	743	0.35		22

1 - Southern Nations Nationalities and Peoples Regional State Health and Energy Bureau

2 - Textile Chemical (1982)

3 - May 1999, water quality Development & Rehabilitation Project Feasibility study and engineering design phase 2

Environmental parameters, and major cations and anions data for water samples from Tikurwuha river, cold and hot springs (from the specified sources)

Index No	Date of Sampling	Total Hardness mg/l	SiO ₂ mg/l	Temp °C	pH	Cond mS/cm	TDS mg/l	Na mg/l	K mg/l	Ca mg/l	Mg mg/l	Cl mg/l	F mg/l	HCO ₃ mg/l	NO ₃ mg/l	Br mg/l	CO ₃ mg/l	SO ₄ mg/l	Source
Rv1	August, 1999	47		20	6.72	288			44	9.2	13	3.5	14	1.31	151		nil		2 This work
Rv2	1996	19							178	14	4	2			453.54				Debbie Nedaw
Rv3	Nov-85	25						54.3	8.602	10.02									3.8424 Tilihun And Harrison In Telford (1998)
Rv4	Jul-83	41						29.9	6.647	11.6232	2.9172	5.3175							0.9606 Tilihun And Harrison in Telford (1998)
Rv5		41	0			7.7		41.1	99	22	8	5	19	25	18		210		4.25 Scripps 1977 in Tesfaye 1982
CS1	August, 1999	34.8		28.3	6.76	229			37	4.4	9	3	4	0.96	139	7.27			1.1 This work
CS2	28/01/99	50		25.6	7.4	234	116				11.2	5.35	5	1.97	4.88	0.44	nil	nil	
CS3	28/01/99	50		24.9	7.9	208	104				11.2	5.35	6	0.19	4.88	0.11	nil	nil	
CS4	28/01/99	61	103			7	311		24	5	13	3	6	0.6	346		5	5	
CS5	28/01/99	72		20.6	7.8	229	115				16	7.78	6.5	0.1	9.96	0.68	nil	4	
CS6	28/01/99	27	77		7.4		164		11.0	3.91083	6.01	3.04	18.08	2.09			1.92123 TELFORD 1998		
TS1	1971	46		25.2	8.3	252	125				12	3	5	0.84	7.32	5.28	0.03	nil	
TS2	1971	104		25.3	8.2	305	153				24	10.7	7.5	0.23	12.2	13.2	0.02	46	
TS3		157					960		180	48.1032	6.01	4.01	25.88	0.95			0.96061		
TS4		96		8.2	8.6	3610	2713		670	48.1032	35.07	4.01	153.85	7.41			184.918		
TS5	Aug. 66	154		7.9	8.6	1598	801		390	32	13	1	11.5	2.16	nil	5.28	0.06		
TS6	1971	150		8	7.9	1924	1649		540	43.0192	5.61	1.34	68	12	885		nil	91	
TS7	1971			7.6					715	24.2472	38.10	3.65	134.70				177.714 wood and Talling 1988		
TS8	Nov. 95			8.1					245	24.2472	38.08	9.12	51.75				198.267 Baumann et al. 1975 in Telford 1998		
TS9	Nov. 95							215.0	19.941	38.076	9.11625	60.23					7.2 Baumann et al. 1975 in Telford 1999		
TS10	Nov. 95							329.6	35.19	44.088	8.14385	75.5085					94.6191 Telford (1998)		
TS11	Nov. 95							197.9	53.176	21.042	9.11625	40.413					1.27.28 Telford (1998)		
TS11	Nov. 95							654.8	67.232	23.046	7.04948	127.975	2.09				4.3227 Telford (1998)		
TS11	Nov. 95							270.1									166.184 UN		

Environmental parameters, and major cations and anions data for water samples from lake Awassa (from the specified sources)

Index No	Date of Sampling	River, spring & Lake		Total Hardness mg/l	SiO ₂ mg/l	Temp. °C	pH	Cond. mS/cm	TDS mg/l	Na mg/l	K mg/l	Ca mg/l	Mg mg/l	Cl mg/l	F mg/l	HCO ₃ mg/l	NO ₃ mg/l	Br mg/l	CO ₃ mg/l	SO ₄ mg/l	Source	
		Name																				
La-1	May-61	Lake Awassa	34	72				1050	1063	235.0	44.965	4.4088	4.74045	34.032						26.8968	Talling&Talling(1965) In Telford (1998)	
La-2	Mar-64	Lake Awassa	23	96			8.8	840	1008	217.9	40.664	11.6232	1.2155	29.778						43.227	Wood & Talling(1988) in Telford (1998)	
La-3	Aug-66	Lake Awassa	77	130			9.2	882		175.2	33.235	4.008	3.1603	32.614						29.2983	Wood & Talling(1988) in Telford (1998)	
La-4	Mar-73	Lake Awassa	34				8.7	843		185.1	36.754	6.012	15.1938								Goff & Awetur (1974) in Telford (1998)	
La-5	Feb. 76	Lake Awassa	40	106			9.2			212.2	32.453	4.8096	5.3482	30.1325	9.69					0.4803	Von Damm & Edmond(1984) in Telford (1998)	
La-6	"	Lake Awassa(near it)	43	109			9.15		904	224.8	39.1	7.014	5.46975	34	9					90	2 Tesave (1982)	
La-7	Jun-83	Lake Awassa	51							169.9	35.19	8.2164	5.46975								0.9606	Tilahun And Harrison(1989) In Telford
La-8	Feb-85	Lake Awassa	21	75			8.8	762		151.0	30.107	11.022	5.71285	34.3865							2.4015	Demseke & Amba (1990) in Telford (1998)
La-9	Nov-85	Lake Awassa	46							175.9	33.626	8.2164									30.2589	Taylor & Zinabu (1989) in Telford (1998)
La-10	12-Mar-91	Lake Awassa	43	43			8.8	741	800	137.0	26.979	8.6172	5.8344	13.8255							35.0019	Elizabeth et al. (1994)
La-11	11-Nov-92	Lake Awassa					8.7		765	164.8	20.107	9.018	5.1051	31.905	7.6	427					0.4803	Berhanu (1996)
La-12	Nov-95	Lake Awassa	73							197.7	43.01	50.5008	9.84555	48.212							0.4803	Telford (1998)
La-13	1995	Lake Awassa	22					810		190	40	22	4.4	45	8.8	44.5	1				15	Tenalem(1998)
La-14	1996	Western shore of	20							175	26	3	3.5								185	Desse Nedawt (1997)
La-15	1996	Eastern shore of	21							192	24	3	3.3								146	Desse Nedawt (1997)
La-16	1996	Northern shore of	21							173	25	3	3.36								175	Desse Nedawt (1997)
La-14	Aug-99	Lake Awassa	50		20.5		8.89	805		156	27	11	5.4	30	5.92	420	3.01				13	This work
La-15	11-Nov-92	Lake Shalo	26				6.6		210	29.9	5.083	7.014	2.06635	7.09	1.14						0.4803	Berhanu (1996)
La-16	Feb-76	Lake Shalo	46	42			7.7		82.3	21.114	11.6232	4.1327	18.07925	5.51							1.9212	Von Damm & Edmond(1984) in Telford (1998)
La-17	Aug-66	Swamp	10	80			7.8	48		3.2	1.935	2.004	1.2155	4.6085							6.2439	Wood & Talling(1988) in Telford (1998)

1 - Southern Nation Nationalities and peoples Regional state Water, Mines and Energy Bureau

3 - Tesfaye Cherant(1982)

4 - MoWR (1999), water supply Development & rehabilitation project Feasibility study and engineering design, group 2

Major cations and anions data for water samples from streams (from the sources specified)

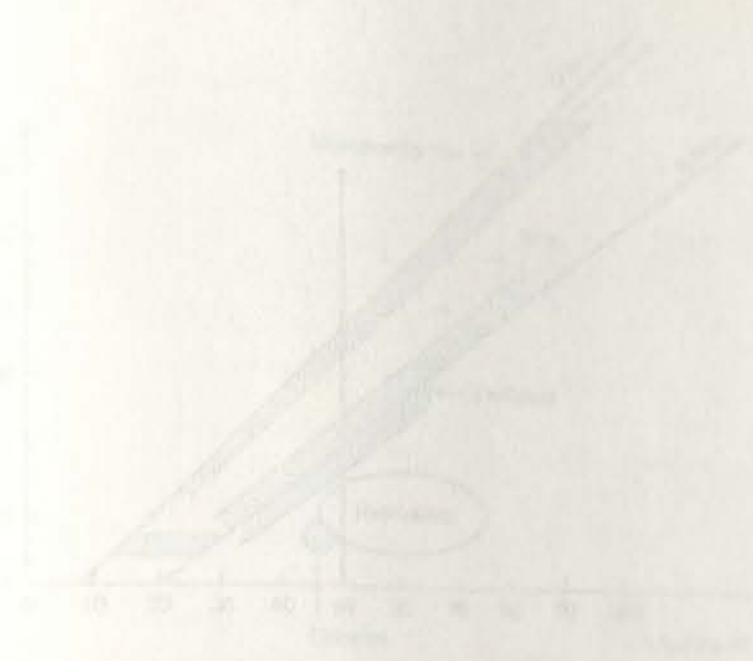
Location	Date	Na	K	Ca	Mg	Cl	HCO ₃ + CO ₃	F	SO ₄	pH	Source
B. Works	Feb-76	1.95	0.28	0.48	0.24	0.11	2.11	0.05	0.03	8.6	3.1 Von Damm & Edmond In Telford 1998
Small area	Aug-66	0.57	0.15	0.26	0.3	0.14	1.17	0.12		8.8	2 Wood & Talling 1988
Trunk flow	1971	0.87	0.13	1.3	0.22	0.14	2.05	0.29			2.1 Baumann et al 1975 in Tesfaye 1982
on windings	Mar-80	0.8	0.14	0.41	0.22	0.07	1.72	0.03			Tilahun & Harrison 1975 in Telford 1988
on windings	Apr-80	0.47	0.21	0.42	0.18	0.14	1.3	0.07			Tilahun & Harrison 1975 in Telford 1988
on windings	Nov-95	1.87	0.21	2.05	0.63	0.24		0.07			Telford 1998

TABLE WATER QUALITY CRITERIA FOR INDUSTRIAL USE.
(Water Quality Criteria, U.S. Dept. Interior, 1968).

Determinand	Steam Generation		Textile Industry	Paper Industry	Chemical Industry	Petro-chemical Industry	Iron & Steel Industry	Cement Industry	Leather Industry
	Low pressure	High pressure							
1. Elements									
Al	5.0	0.01							
Ca				20.0	68.0	75.0			
Cl				200	500	300			
Cu	0.5	0.01	0.01					60	
Fe	1.5	0.01	0.1	0.1	0.1	1.0	250	250	
Mg				12.0	19.0	30.0	25	0.1	
Mn	0.3		0.01	0.05	0.1				
							0.5	0.01	
2. Inorganic Compounds									
NH ₃ - N	0.07	0.07							
NO ₃ - N					5.0				
PO ₄					100				
SiO ₂	30.0	0.01		50.0			250	250	
							35.0		
3. Quality Parameters									
Hardness as CaCO ₃	20		25	100	250	350	100	150	
Alkalinity	140				125		400		
HCO ₃	17.0				128				
Total dissolved solids	700	0.5	100		1000	1000	600		
Suspended solids	10	0	5	10.0	30	10	500		
Dissolved oxygen	2.5	0.007							
pH Units			6 - 10	6 - 8	6 - 9	5 - 9	6.5 - 8.5	6 - 8	

Parameters	Acceptable value
Turbidity	1-10
Color	5-10
Taste and Odour	Inoffensive
Dissolved solids	850mg/l
Total hardness as CaCO ₃	10-250mg/l
Total alkalinity as CaCO ₃	30-250mg/l
Total solids	1000mg/l
pH	>7.5
Sodium (Na) ⁺	300mg/l
Potassium (K) ⁺	-
Magnesium (Mg) ²⁺	40mg/l
Calcium (Ca) ²⁺	80mg/l
Iron (Fe) ²⁺	0.2mg/l
Manganese (Mn) ²⁺	0.2mg/l
Chloride (Cl) ⁻	300mg/l
Fluoride (F) ⁻	1mg/l
Nitrite (NO ₂) ⁻	-
Nitrate (NO ₃) ⁻	20mg/l
Bicarbonate (HCO ₃) ⁻	300mg/l
Carbonate (CO ₃) ²⁻	-
Sulphate (SO ₄) ²⁻	-
Silica (SiO ₂) ²	50mg/l

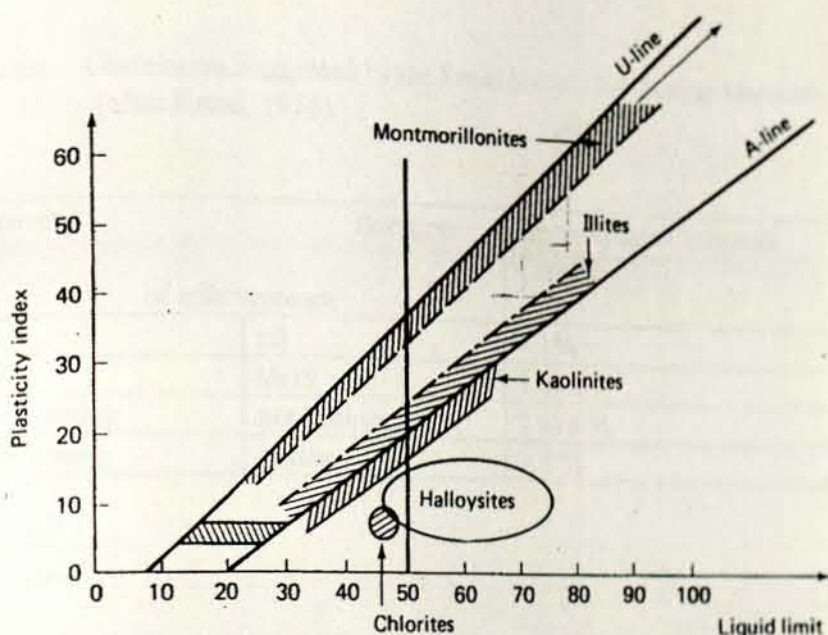
Table 8 .Water quality criteria for food processing
food canning and freezing industries.
Source-American Water Works Association
Water quality and Treatment, McGraw-Hill
New York, 1971



APPENDIX D

SOIL AND ROCK DESCRIPTIVE TERMS AND CLASSIFICATION SCHEMES

The following terms and classification schemes are used in this report. The terms are defined in the text and the classification schemes are given in the tables. The terms are used in the text and the classification schemes are given in the tables.



Location of common clay minerals on Casagrande plasticity chart (Holtz & Kovacs 1981).

Table Atterberg limit values for the clay minerals (Mitchell 1976).

Mineral*	Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)
Montmorillonite (1)	100-900	50-100	8.5-15
Nontronite (1) (2)	37-72	19-27	
Illite (3)	60-120	35-60	15-17
Kaolinite (3)	30-110	25-40	25-29
Hydrated Halloysite (1)	50-70	47-60	
Dehydrated Halloysite (3)	35-55	30-45	
Attapulgite (4)	160-230	100-120	
Chlorite (5)	44-47	36-40	
Allophane (undried)	200-250	130-140	

*(1) Various ionic forms. Highest values are for monovalent; lowest are for di- and trivalent. (2) All samples 10% clay, 90% sand and silt. (3) Various ionic forms. Highest values are for di- and trivalent; lowest are for monovalent. (4) Various ionic forms. (5) Some chlorites are nonplastic.

Table Correlation Suggested by the Swiss Society for Testing Materials
(after Kezdi, 1974)

Intensity of effervescence		Duration	Approximate Carbonate Content
nil	nil		< 1%
weak	short		1 to 2 %
fairly strong	not lasting		2 to 4 %
very strong	lasting		> 5 %

Terms for the Description of fine soils (after IAEG, 1981)

a) using liquid limit

Term	Range of liquid limits (%)
Low plasticity	under 35
Intermediate plasticity	35 - 50
High plasticity	50 - 70
Very high plasticity	70 - 90
Extremely high plasticity	over 90

b) using Plasticity index

Term	Plasticity index (%)
Non plastic	under 1
Slightly plastic	1 - 7
Moderately plastic	7 - 17
Highly plastic	17 - 35
Extremely plastic	over 35

Activity of clays (after Head, 1984)

Description	Activity
Inactive clays	< 0.75
Normal clays	0.75 - 1.25
Active clays	1.25 - 2
Highly active clays	> 2

Activities Of Various Minerals (after Holtz & Kovacs 1981).

Mineral	Activity
Na- montmorillonite	4 - 7
Ca- montmorillonite	1.5
Illite	0.5 - 1.3
Kaolinite	0.3 - 0.5
Halloysite (dehydrated)	0.5
Halloysite (hydrated)	0.1
Attapulgite	0.5 - 1.2
Allophane	0.5 - 1.2
Mica (muscovite)	0.2
Calcite	0.2
Quartz	0

Field estimates of uniaxial Compressive strength (After Hoek et al, 1995)

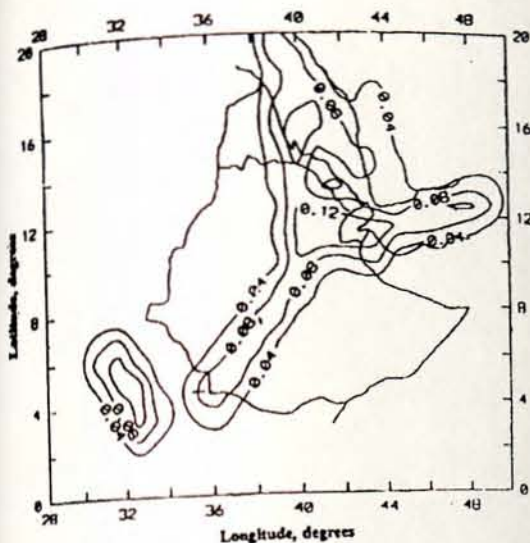
Field estimates of uniaxial compressive strength.					
Grade*	Term	Uniaxial comp. strength (MPa)	Point load index (MPa)	Field estimate of strength	Examples**
R6	Extremely strong	> 250	>10	Rock material only chipped under repeated hammer blows, rings when struck	Fresh basalt, chert, diabase, gneiss, granitic 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, rock salt, 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 groups

* 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.

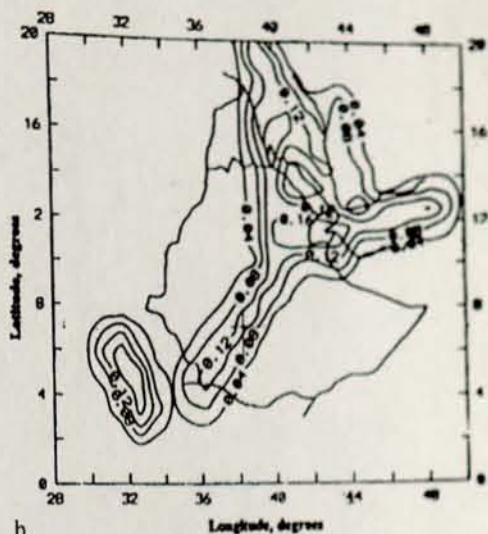
*** Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.

APPENDIX E – Seismic Hazard Maps of Ethiopia



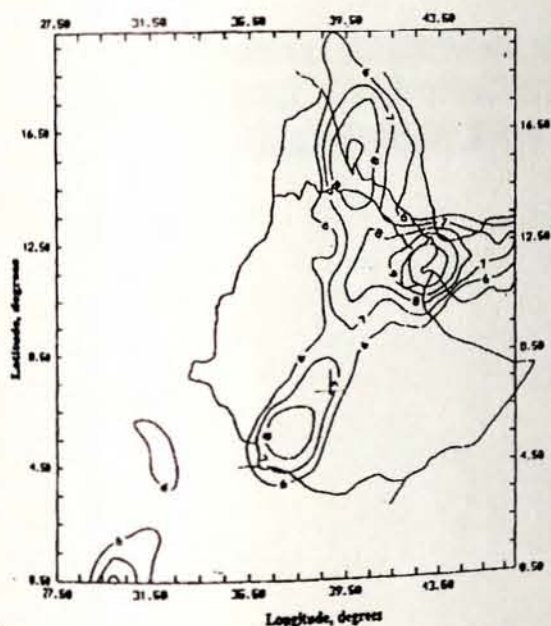
a

Seismic hazard map of Ethiopia and the neighbouring countries. The hazard is for an annual probability of exceedence of 0.01. Contours indicate horizontal peak ground acceleration as a fraction of g.



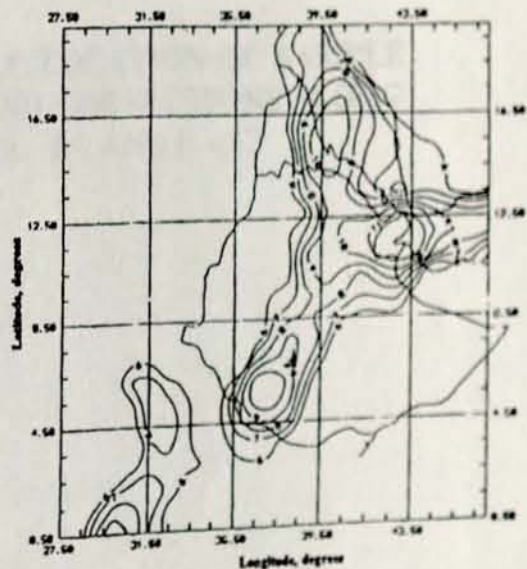
b

Seismic hazard map of Ethiopia and the neighbouring countries. The hazard is for an annual probability of exceedence of 0.005.



c

Seismic hazard intensity map of Ethiopia and the neighbouring countries. The hazard is for 100 years return period with 90% probability of not being exceeded. Contours indicate different Modified Mercalli Intensity levels.



d

Seismic hazard intensity map of Ethiopia and the neighbouring countries. The hazard is for 200 years return period with 90% probability of not being exceeded.

Seismic hazard maps of Ethiopia and The neighboring countries
(after Fekadu Kebede, 1996)

APPENDIX F

**ENGINEERING GEOLOGICAL MAP, LOCATION OF SAMPLE
POINTS AND LONG PROFILES, AND THE SECTIONS ALONG
THE PROFILES A - A', B - B', AND C - C'**

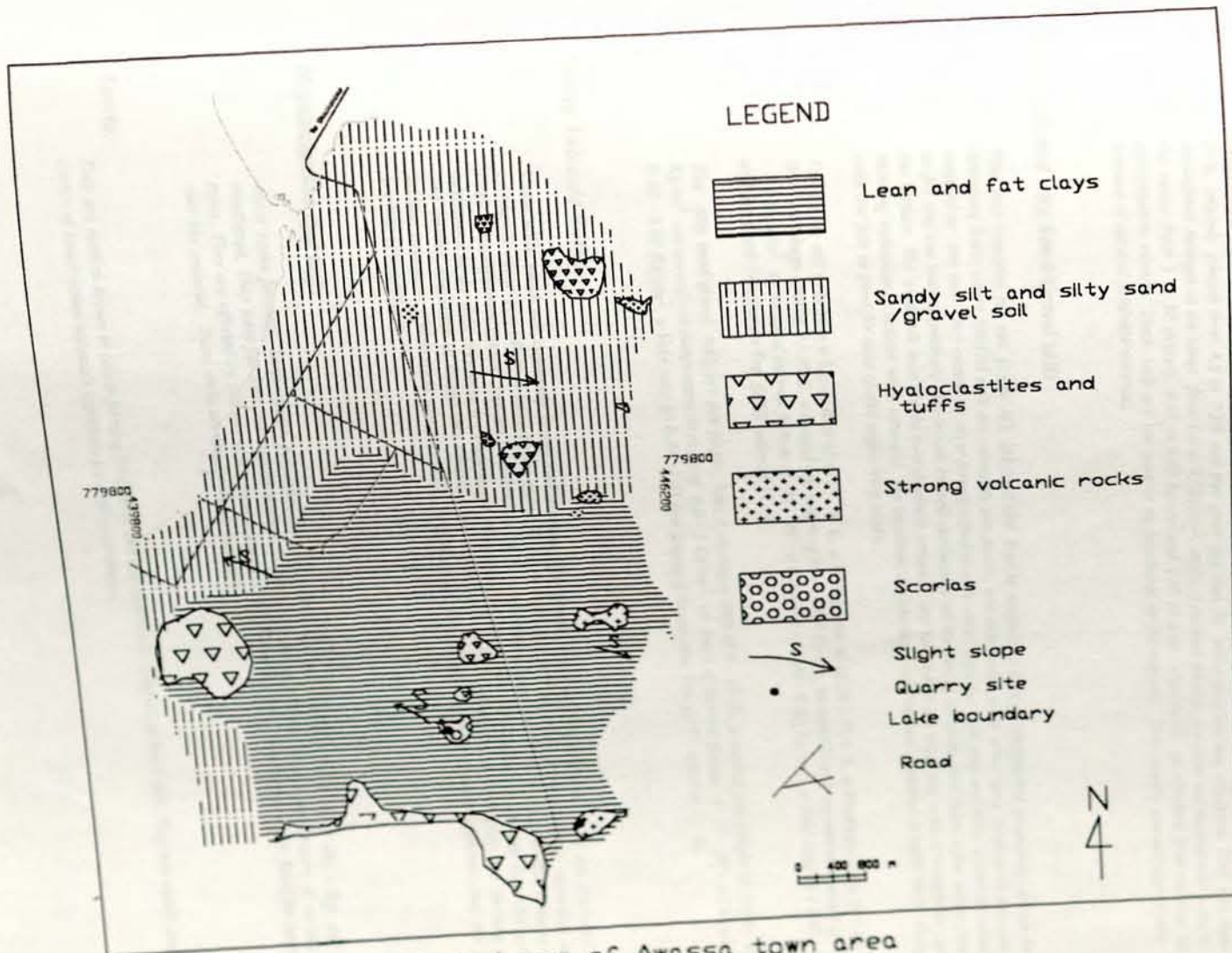


Fig. Engineering geological map of Awassa town area
(Reduced from 1:10000 scale map)

Lean and Fat Clays :

This lithological unit comprises the soil groups CL, CH and the CL-ML soils. They are dark brown in colour with dark gray varieties. They are slightly cracked, columnar to prismatic and sometimes have sub angular blocky structure. Their thickness varies both vertically and laterally, ranging from 0.85 to more than 3 meters. These soils have natural unit weight in the range from 1197 to 1750 Kg/m³ and dry unit weight from 791 to 1350 Kg/m³. The liquid limit for this type ranges from 28.0 to 53%, plastic limit from 13.2 to 28.2%, shrinkage limit from 15 to 28%, plasticity index from 7 to 24%, organic content from 4.5 to 7.2% and free swell less than 30, hence they are non expansive. This soils have unconfined strength in the range from 1.4 to 3.5 Kg/cm², angle of internal friction, cohesion and natural void ratio in the range from 2 to 32 degrees, 0.01 to 0.05 kg/cm² and 1.05 to 2.01, respectively, as obtained from existing site investigation reports. These soils are not suitable as foundation or fill material. They usually contain appreciable amount of silt and sand size materials.

Sandy Silt and silty Sand/Gravel soils :

This unit comprise the soil groups MS, SM, and GM. Due to similarities in their engineering properties, except for Atterberg limits where SM/GM soils are commonly non plastic, and distribution in the area, these lithological units are mapped as one unit. They commonly occur interstratified to each other. They are light gray and olive yellow in colour in dry and wet state respectively. These soils become dominant in all the sites of the study area below a few meters from the surface. The sandy silt soils are the most dominant groups. These soil groups are suggested to be of collapsible type showing reduction in volume when saturated. The important engineering properties are presented in table (9.1). These soils are fair to good for most of civil engineering works.

The Sandy silt soils have a Liquid limit of 26.7 - 56 %, a Plastic limit of 24.24 - 33.9 %, a Plasticity Index of 1.89 - 25 %, a shrinkage limit of 0 - 30 %, a natural unit weight of 816 - 1830 Kg/m³, an unconfined compressive strength of 0.8 - 3.7 Kg/cm², an Angle of internal friction of 23 - 38°, a Cohesion of 0.00 - 0.03 Kg/cm², a Void ratio of 1.002 - 2.57 and Standard Penetration Test (SPT) value of 13 - 4.

The silty sand/gravel soils are non plastic, have a shrinkage limit of 0 - 23 %, a natural unit weight of 1159 - 1730 Kg/m³, an unconfined compressive strength of 0.9 - 2 Kg/cm², an Angle of internal friction of 21 - 36°, a Cohesion of 0.00 - 0.03 Kg/cm², a Void ratio of 0.89 - 1.85 and Standard Penetration Test (SPT) value of 19 - 36.

Strong Volcanic Rocks:

These units which are commonly basalts and rhyolites form simple ridges in the study area. They are fractured and commonly occur in medium to large blocky forms. Their strength ranges from strong to very strong, according to the Hoek's classification system, and are weathered on top parts and along fractures. From Schmidt hammer rebound tests performed in the field they have a compressive strength in the range from 290 Kg/cm² to greater than 650 kg/cm². The basalts are black in colour and vesicular and scoracious while the rhyolites are light gray and showing flow folding structures. These rocks are not suitable for foundation from topography point of view. However, they are good sources of aggregates.

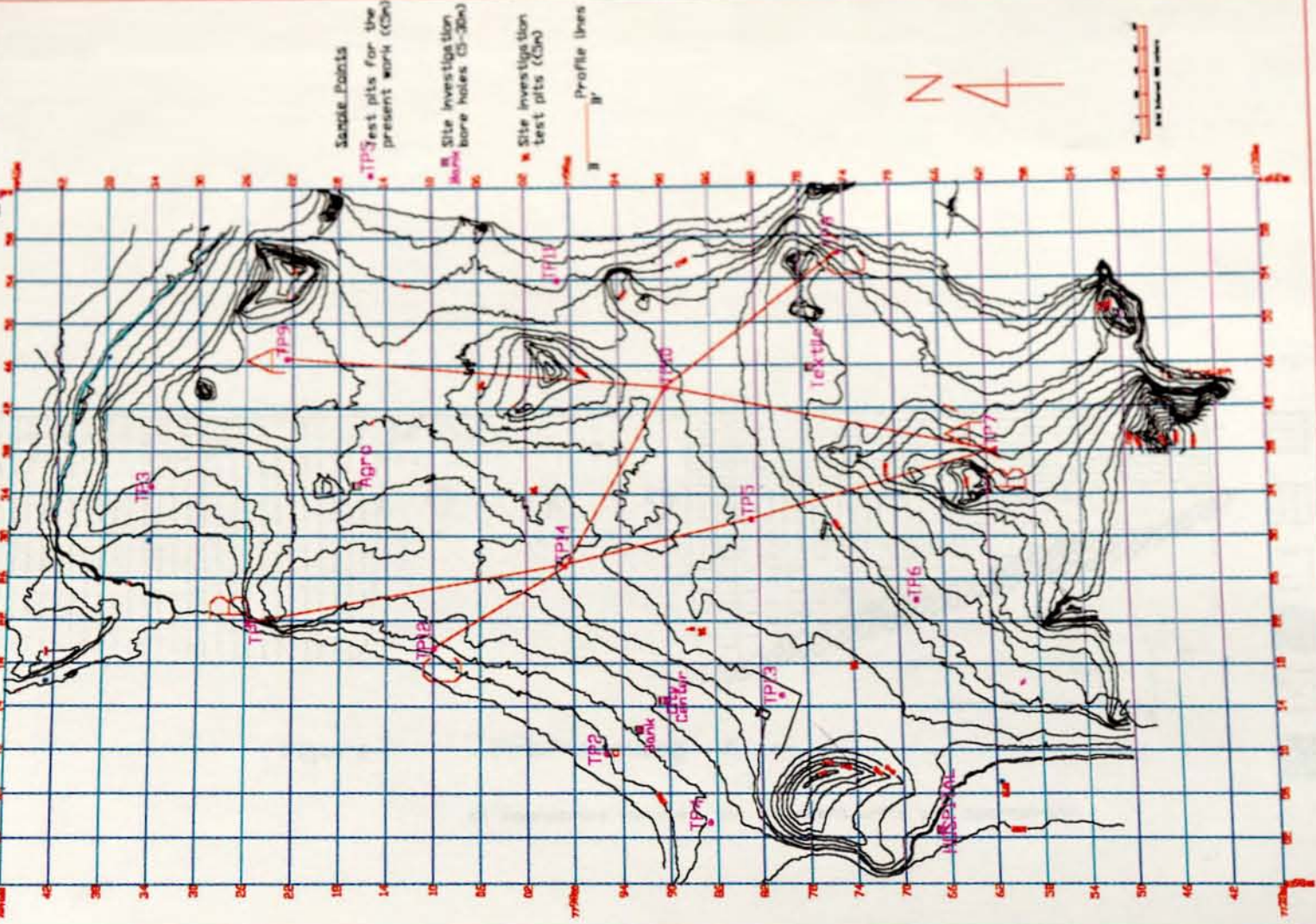
Hayaloclastites and tuffs

these rocks forming simple peaks distributed here and there, are very weak to weak rocks (R₁ - R₂) and highly weathered. They form flat tops and contain dominantly fine grained material with blocks and boulders of various rock types. They are affected by two sets of fractures oriented N 30° E and N 115° S which are mostly filled by calcite and clay like material. These rocks are unsuitable for foundation.

Scoria:

They are reddish brown in colour forming small hills, and are dominantly of lapilli to block size. They are widely used as a source of construction materials especially for road construction.

Location of Sample points & Long. Profiles



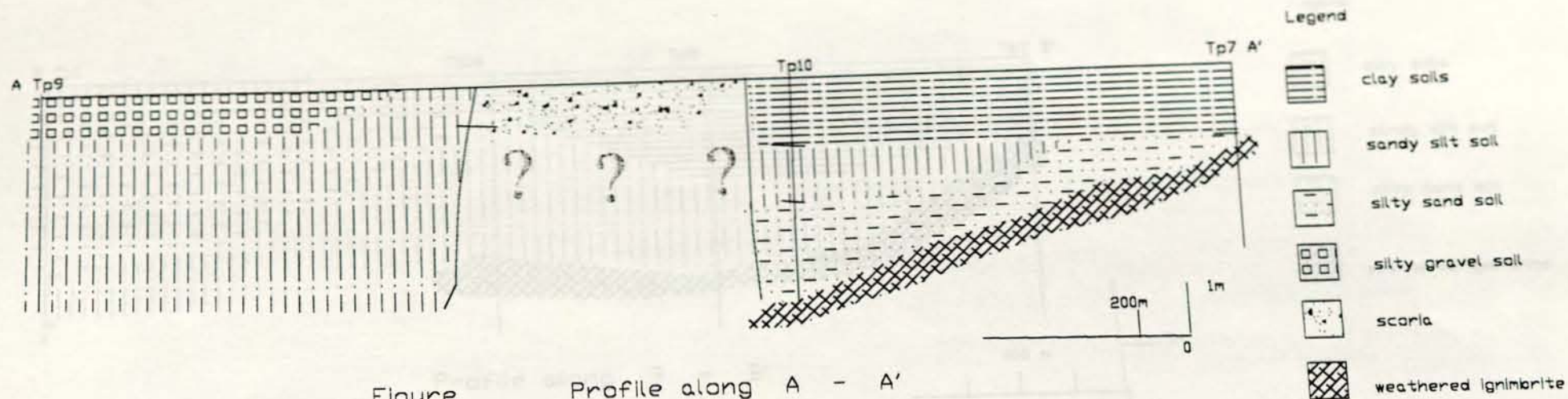
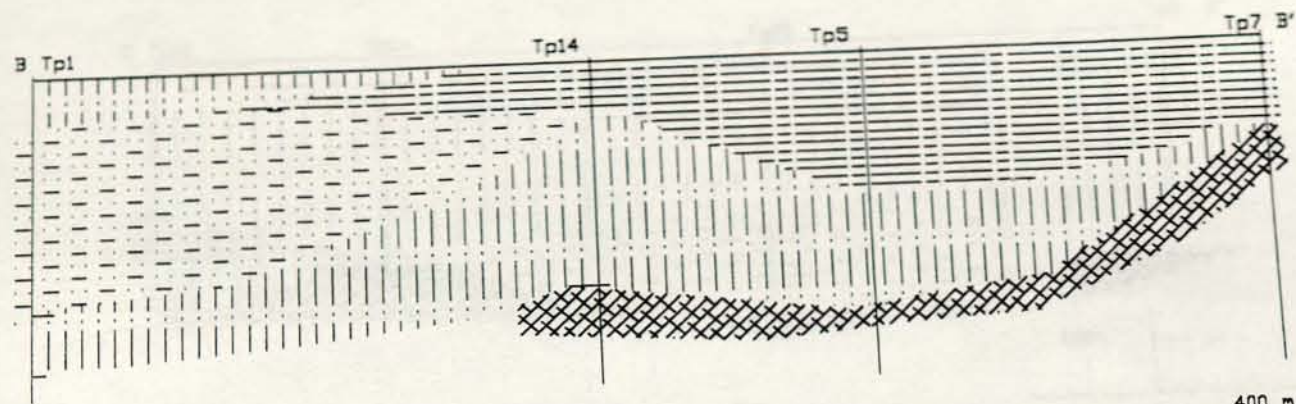


Figure Profile along A - A'

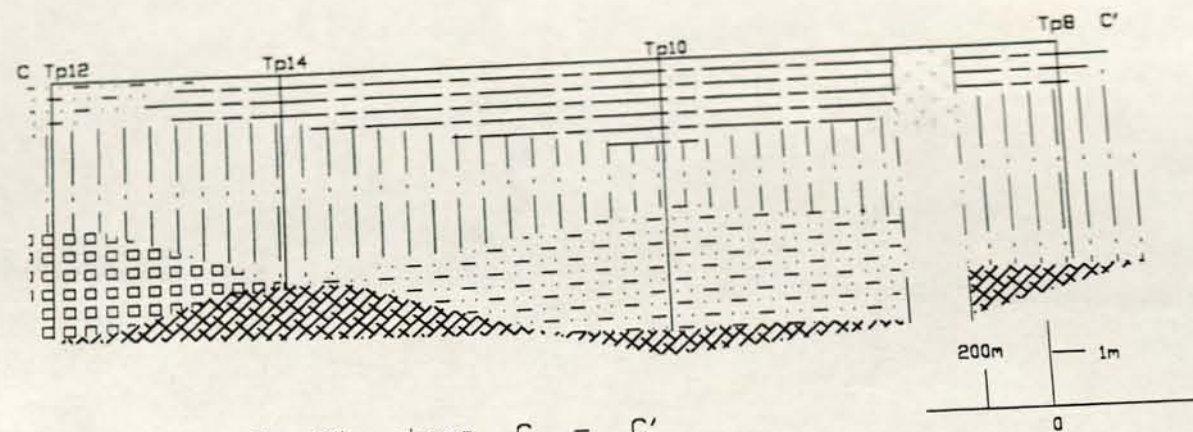
All boundaries between the different units are approximate



Legend

-  clay soils
-  sandy silt soil
-  silty sand soil
-  weathered ignimbrite

All boundaries between the different units are approximate



Legend

-  clay soils
-  sandy silt soil
-  silty sand soil
-  silty gravel soil
-  basalt associated with
-  weathered ignimbrite

Profile along C - C'

All boundaries between the different units are approximate