GEOTECHNICAL AND ENGINEERING GEOLOGICAL INVESTIGATION OF IYANDAFARO DAM SITE, KONSO, SOUTHERN ETHIOPIA

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DEMEKE HAILU

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IV. ABSTRACT

The Iyanda catchment covering an area of $658km^2$ is studied for the selected dam site. Land use/ Land cover, soil and slope of the area were mapped, so that erodability for vegitation soil complex is computed. Geology and hydrometheorology of the catchment was studied and engineering properties of the mapped rock and soil units are discussed. The geology and engineering geology of the catchment was mapped at the scale of 1:50000 and the engineering geology of the reservoir at 1:2000. The area is dominantly covered by volcanics(basalts and metamorphic (gneiss) at relatively elevated areas and Quaternary deposits at flat plain areas.

Areal mean annual rain fall was analyzed using Arithmetic mean and Thiessen weighted average and a value of 830.6mm and 830.5mm was obtained. Piche evaporimeter installed at Konso was used for the evaluation of the storage site(1680.3mm) using a pan coefficient of 0.85 due to the high evaporation rate at the site. The potential evapotranspiration calculated by the Thornthwaite method is 1534.3mm, that is almost twice of precipitation and is typical properties of arid to semi arid climate. The soils of the study area never reaches to field capacity in general, due to this, no surplus is generated. The possible method of ground water recharge and surface run off is rain fall above average and storm run off. The peak storm run off for 100 year return period was 86mm. The selected reservoir site has storage capacity of more than $16Mm^3$. Water samples taken from the study area and its environs show decreasing tendency from the Lake to the study area with respect to EC, TDS and dominating cations like sodium.

The engineering geological mapping was carried out based on material strength, degree of weathering and fracturing for rock units and gradation and Atterberg limit for soil material. Four geological units are mapped at the dam site: Gravel and sand at the river, clay and silt at the flat plain area, well-graded soil at the right abutment and slightly fractured basaltic rocks at the left abutment. The geology of the side channel spill way, selected at the left side is a continuation of the left abutment, which is stable, slightly weathered and fractured basalt. The main canal lithology is fine grained clastics (clay and silt) which is impermeable lithological

unit. The consolidation test of clayey silt soil analysis shows that, the soil is stiff with maximum setelirnent 7mm for the aplied stress of 3200kN/m^2.
The permeability coefficient computed from the consolidation test is 0.7×10^{-7} cm/s. Both the index and engineering properties of the dam site shows that, the clayey silt soil and well graded soil at the right abutment are stable and watertight. Hence the foundation depth should be on this layer. But at the main river and its levees, no watertight unit was mapped to 7m depth, though the geophysical investigations show the bed rock symptom at 13.5m depth.

The study area catchment rock units can be classified in to five strength zones:

Zone 1 - Moderately fractured, slightly weathered to fresh basalt

Zone2 - Moderately to highly fractured, slightly to moderately weathered basalt

Zone 3 - Moderately fractured, slightly weathered to fresh gneiss

Zone 4 - Moderately to highly fractured and weathered gneiss

Zone 5 - Moderately to highly fractured and weathered gabbros

Based on strength classification zone 1 and zone 2 are very strong to extremely strong rocks and zone 3, 4 and 5 are strong to very strong rocks. The approximate range of unconfined compressive strength of extremely strong, very strong and strong rocks used for this classification is: > 250, 100 - 250, and 50 - 100Mpa respectively.

The results of the electrical sounding survey presented in the form of geo-electric section which shows the distribution of layer resistivities and thickness. This is believed to be a closer approximation to the actual geologic setting in the sub surface. The VES and profiling investigation show that the over burden (soil) is thick at the flat plain areas and thin at the abutments.

Chapter One

Introduction

1.1 General

Nearly up to the first half of the 20th century, dams were designed and constructed with out the investigation and study of the geological environment on which the dam and its appurtenant structure rest. It was after a serious failure of dams because of defective foundation comes the need for the geologic exploration of dam sites (P.S.D.E.D. 1990).

Engineering geological investigation of the Iyanda dam encompasses reservoir and foundation characteristics, details of the dam abutment structures, sources of natural construction materials, their quality, quantity, proximity and others.

1.2 Back ground

According to the ranking of the community problem the most decisive is moisture stress which hindered farm activity of the Konso people and becoming life snatcher. Due to this there is the question of survival for the people for many decades, alleviating the frequent draught and famine is the concern of both governmental and non-governmental organizations.

In the Konso woreda approximately 70% of the entire area is lying below 1600 m.a.m.s.l has got kolla type of climate and suffer from extreme drought and famine. The total population of the woreda is estimated to be 183,000 whose basic economic activity is farming (woreda bureau of agriculture). Data Compiled by FAO , Agro -Meteorological Unit (1984) indicate that the woreda is characterized by mean maximum and minimum monthly temperature of 26.9°c and 16.0°c respectively with annual rain fall of 720mm. There are two rainfall regimes experienced by the woreda; the first is the one that begins in September and stops at the beginning of November, where as the other starts in March and stops at the beginning of May.

Except the highland areas of Gewada, Kholme, Gumayde and Aylota that account for about 30% of the total area, the rest parts of this woreda suffer from severe moisture deficit (CO -SAERSAR Konso Report 1999) . So this shortage of rainfall is the major problem that hinders exploitation of these fertile plains and resulting in frequent drought and famine in the woreda. Shortage of farmland and depletion of soil fertility in the rugged uplands have recently forced the people to conquer the virgin lowlands. All the rivers flowing to these lowlands are seasonal. Due to this strong moisture stress, rainfed agriculture can not alleviate food shortage problem that encountered the Konso people. To overcome the food insecurity problem, that occurred due to moisture stress the farmers should use irrigation based agriculture in this fertile lowland areas and to achieve this aim earth dam that can collect water during rainy season is selected by CO-SAERSAR and proposed for medium scale study that need detail investigation, and this research is intended to fill the gapes related to geotechnique.

1.3 Previous works

Eventhough, Ethiopia is said to have a large irrigation potential, the actual development has been minimal Admasu Gebeyehu (1996). Due to the recurrent droughts in the country, the government priority was given to the development of irrigated agriculture. As large scale irrigation projects require a number of years to study and investigate to feasibility level and to make final designs, the commissions immediate concern was for small and medium scale irrigation projects ranging between 10 ha and 1000-1500 ha. such projects do not require sophisticated studies and techniques for construction but can be designed and executed using mainly standard procedures.

The Commission for Sustainable Agricultural and Environmental Rehabilitation in Southern Administrative Region (Co-SAERSAR) founded by the Regional government has been engaged in a number of water resources projects for irrigation. One of the projects undertaken by the Co-SAERSAR is the Iyanda medium scale irrigation project in the Konso special woreda. The estimated capacity of the reservoir is more than 16 million cubic meters. The impounded water is intended for irrigating 1500 ha of land.

1.4. Objective

The main objective of this study is:

1) To investigate the reservoir geology of the site ; watertightness, stability of slopes in the reservoir rim
2) To investigate the spillway geology and location
3) To investigate the watertightness of the main canal.
4) To analyze the hydrology and hydrogeology of the catchment
5) To perform relevant geotechnical tests
6) To asses and examine the properties of construction materials
7) To produce engineering geological map of reservoir areas at the scale of 1:2,000 and catchment area at scale of 1:50,000
8) To asses the effects of the dam construction on the environment

1.5 Methodology

The methods employed to achieve the objectives of the research, are:

- Review of published and unpublished materials on the geology, structure, hydrology and engineering geology of the site and surrounding areas.

- Hydrometeorological data analysis was carried out using precipitation, temperature, sunshine and wind speed data
- Field works was conducted to collect relevant data on geology, structure, hydrology, and engineering geology of the area. Surface and sub surface investigations of the area was done with the help of test pits. Along with field data collection, insitu tests and sampling was carried out. Soil samples of appropriate number and size, both disturbed and undisturbed samples from pits, was collected.
- Using resistivity methods the subsurface geological conditions was investigated.
- The collected samples was analyzed in the laboratory for different parameters; grain size analysis, consistency limits tests, permeability, consolidation, swelling, and shear tests to determine their geotechnical properties.
- Finally, the data processed was interpreted, the Geotechnical properties was evaluated and conclusion was drawn.

1.6 Materials used

Equipment used during the site investigation varies from simple hand tools as a means of surface and subsurface investigation to advanced geophysical instrument(Geopulse). Some of these tools are; spades, hoes, buckets, rope, meter tape, sacks for disturbed samples, hydraulic jack and steel pipe for undisturbed samples.

Chapter Two

General Overview of the Study Area

2.1 Location and accessibility

The study area is located in the Southern Nations Nationalities and People Regional Government between 37^0 16' and 37^0 34' east longitude and 5^0 20' and 5^0 39' north latitude with minimum and maximum elevation of 923 and 2600 m.a.s.l at center of dam axis and Gardula mountain respectively Fig. 2.1. The dam site is located 23 km from the capital of Konso, karat on the way to Burji and 620 km from Addis Ababa. The Gydole and Konso towns make the northern and southern limits of the studied area . It consists of two Woredas Derashe and Konso. The project area extends for about 36 km in north - south and with an average width of 24 km in east west direction. It is in the border line between the main Ethiopian rift and South western Ethiopian rift .The all weather rood , Addis Ababa - Konso - Jinka passes through the catchment. Accessibility within the basin is generally good, especially during the dry season. The dry weather rood from the dam site to the main rood extends only for 2 km.

2.2 Physiography

The physiography of the study area is the result of both volcano tectonic and erosional processes. Three major Physiograghic land units form the catchment ; the western highlands the central lowlands and the eastern highlands. Out of the total 658 km^2 area, 14.8 % or 97.6

km^2 constitute the western highlands with altitude ranging from 1700 - 2600 m.a.s.l. The western high land is the origin of all major rivers draining the catchment. Except the Gato river all other streams are seasonal. During heavy rainfall, these rivers and streams carry a large amount of water for a short period of time. A large quantity of coarse sediments ranging from silt to gravel size are transported and dragged by the streams during heavy rainfall from the high land and damped in the plain to form alluvial fans. Some rivers lose their flow in the plain, Bokolo, Dandeme, Kerga and Kodo are good examples. It is known that the drainage pattern of a certain region or basin is determined by its various petrographic constitution together with tectonic events occurred during the geological era. The Iyanda basin has a dendritic drainage pattern in most upstream side and parallel at the western part of the catchment. In general the plain slopes gently to the east with an average gradient of 1.4 %. The eastern high land has elevation greater than 1700m constituting only 3.2 km^2. No major streams originate from the eastern high lands except Fohida. The Iyanda rivers after joined by its main tributaries; Gato, Haymele, Gapamaga, Dino, Kayle, and Kerga form its out let to the Segen river. Fig 2.2

As observed during the fieldworks, sediments transported through these streams are discharged on the plain land. This is due to the loss of energy as the affluent reach the plain land.

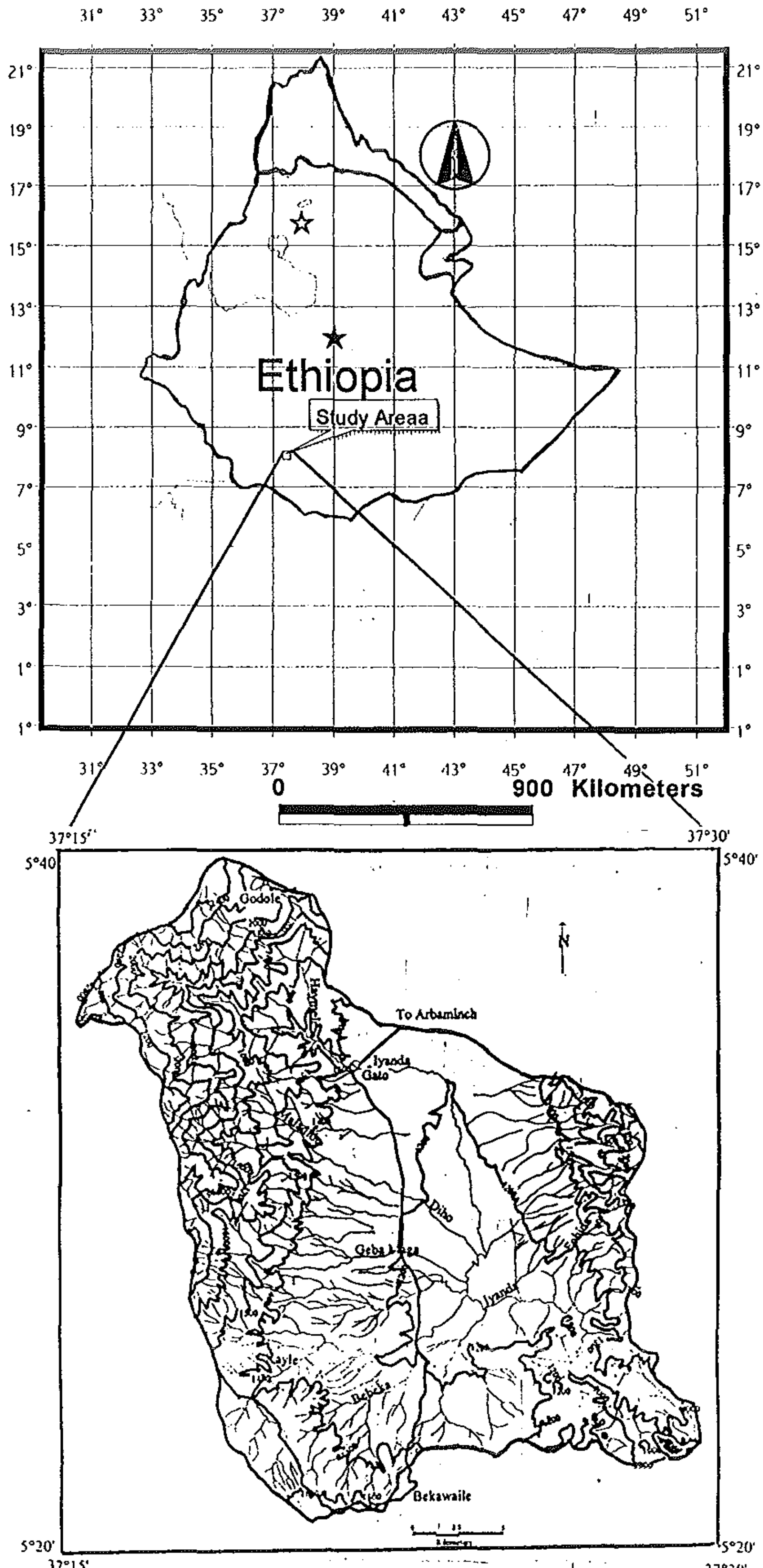
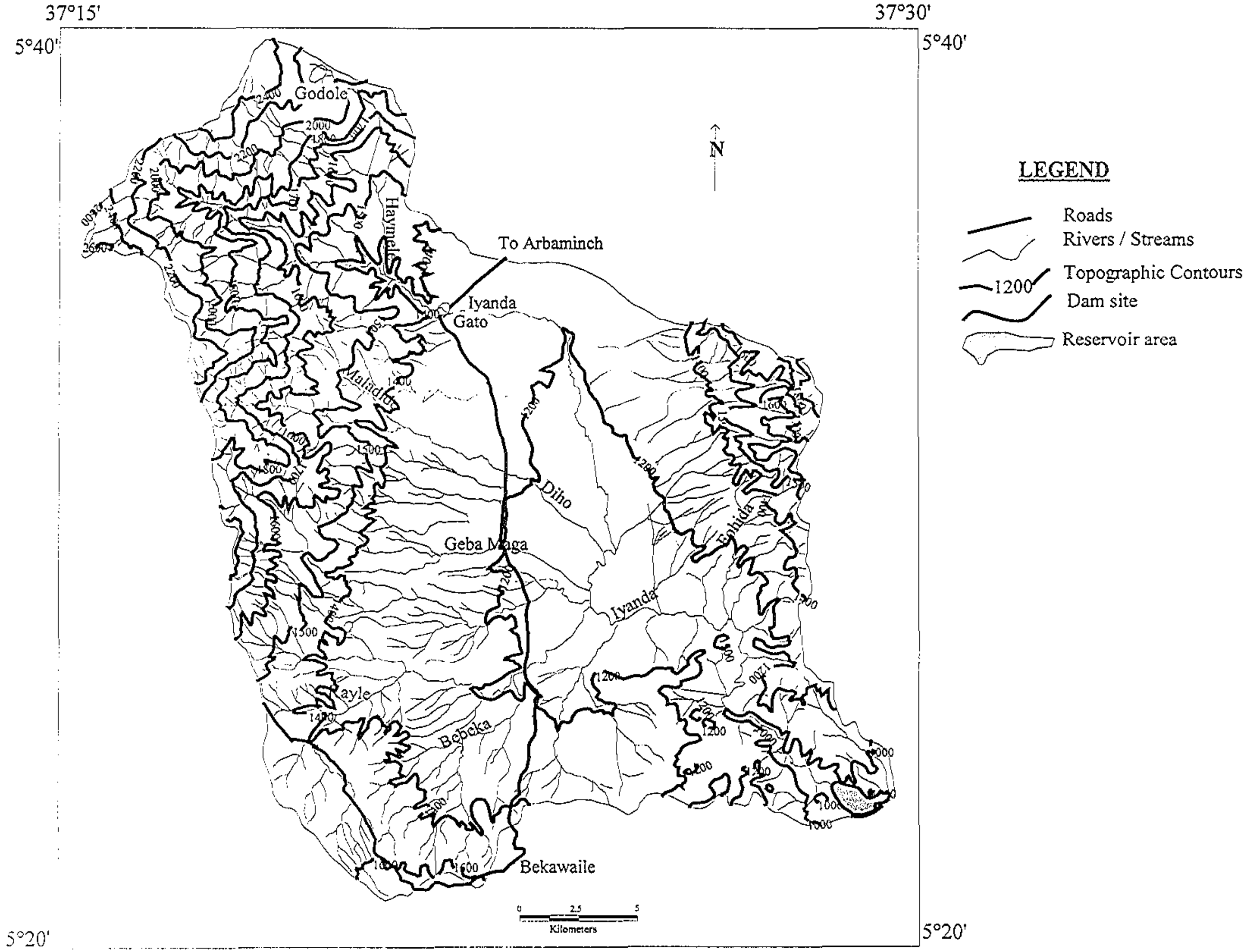


Fig. 2.1 Location Map of the Study Area



Fig(2.2) Dranage and Topographic map

2.3 Land use land cover and soils

In agricultural area more than 85% of the total cultivated land is covered by Sorghum and maize in which the contribution of sorghum is higher due to its ability to resist drought CO-SAERSAR (1999) . In the area the prevailing dry tropical climatic condition which have insufficient rain fall and the insufficiency of the coming flood to sustain crop growth to maturity resulted in the occurrence of both short and long periods of drought for successive years which have spread effects from marked yield reduction to total crop failure, vegetation decline and grazing land reduction. The catchment area has a natural vegetation cover which is highly disturbed by human activities. The existing vegetation cover dominantly at the catchment are: wood lands, scrubs, and bushlands.

The vegetation of the watershed are distributed in the terrain in such away that trees of riverine nature occupy the Iyanda river and its tributaries, and wood lands, scrubs at the catchment. They are also distributed as high lands and low lands vegetation. The high lands cover are remnants of forest trees, and the lowland cover are disturbed wood lands, shrubs and bushes. The species which are dominant over others in the catchment are , acacia. The rigidity and supporting power, drainage and moisture retention capacity, ease of root penetration, plasticity, aeration and retention of plant nutrients are all intimately connected with the physical characteristic of a soil; texture, structure, color, density, temperature and consistency. Generally as a characteristic feature of most dry tropical climate, soils of the project low land area are identified by the absence of distinct pedogenic horizons.

Soil map for the study area was prepared based on areal photo interpretation and field observations. The map is used to estimate soil water holding capacity in the course of actual evapotranspiration estimation using a soil water balance approach. The soils of the study area are broadly categorized into four groups; (Table 2.1) and fig 2.3

Group 1: The dominant soil in this group is clay loam, local intercalation's of clay, silty loam and sands are associated. The soils of this group are highly cultivated, have gentle slope and average elevation of about 1200 m.a.s.l. this soils are mainly alluvial type.

Group 2: The dominant texture of this soil is silty loam, in some places patches of sandy loam, and sands along the river coarse are associated; it is also found in the lowland areas, this group is mainly of colluvial and alluvial type.

Group 3: The main lithological unit in this group is sandy loam soils, associated with this group are mainly silty loams at some places and sands along the river. This lithological group is dominantly of colluvial and partly of residual.

Group 4: The main textural unit in this group is soils of fine sand size. This soils are mainly the result of the weathering of the underlying rock unit. An intercalation or association of all group is observed at different places; mainly near the Gydole highlands. This soil group is mainly of residual type.

Soils of silty and clay loam are invariably of better inherent fertility. These soils are excellent agricultural soils and when adequately fertilized and when their water supply is controlled they are quite productive Co-SAERSAR(1999) .

Soil type	Area	
	km^2	%
Fine Sand	263	40
Sandy Loam	156	23.7
Silty Loam	122	18.5
Clay Loam	83	12.6
Bed Rock	34	5.2

Table 2.1 Areal proportion of different soils of the study area
Based on triangular classification

Broadly six land use / land cover classes were identified; (Table 2.2) and fig 2.4

1. Highly cultivated land , covering the gently sloping and flat areas.

2. Few cultivation on moderately steep slopes with scattered bushes and scrubs

3. Shrubs, bushes and grass land covering escarpment and hill tops

4. Settlement area

5. Bare land in the northern part of the catchment

6. Wood lands covering the high land of Gydole

Land use/ land cover	Areal proportion (km^2)	Areal proportion (%)
Highly cultivated land	76.3	11.6
Scattered cultivation with shrubs and bushes	263	40
Shrubs and bushes	249.75	38
Settlement area	3.5	0.5
Bare land	34.45	5.2
Wood lands	31	4.7

Table 2.2 Areal distribution of different land use / land cover units

On the other hand, slope classification was done based on the Ministry of Agriculture land use and Regulatory Department system. Based on this, five slope classes were mapped fig 2.5 and Table 2.3). The slope class 5 -12 % is the dominant one.

Slope class	Area		
	km^2	%	Classification
<5	234.2	35.6	flat
5 - 12	200	30.4	gentle
12 - 35	160.3	24.4	moderately gentle
35 - 60	55	8.3	steep
>60	8.5	1.3	very steep

Table 2.3 Areal proportion of different slope class

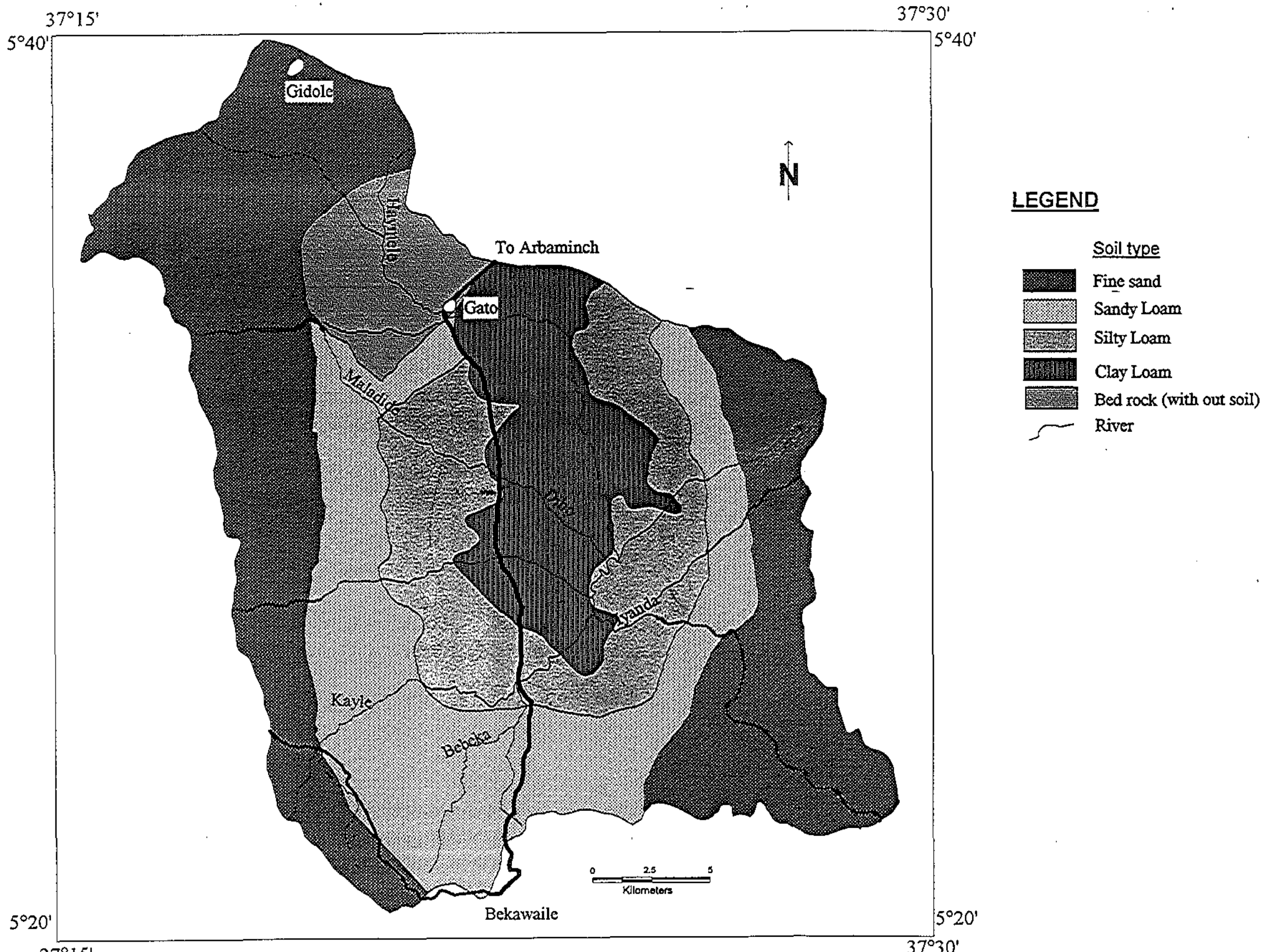


Fig (2.3) Soil map of the study area

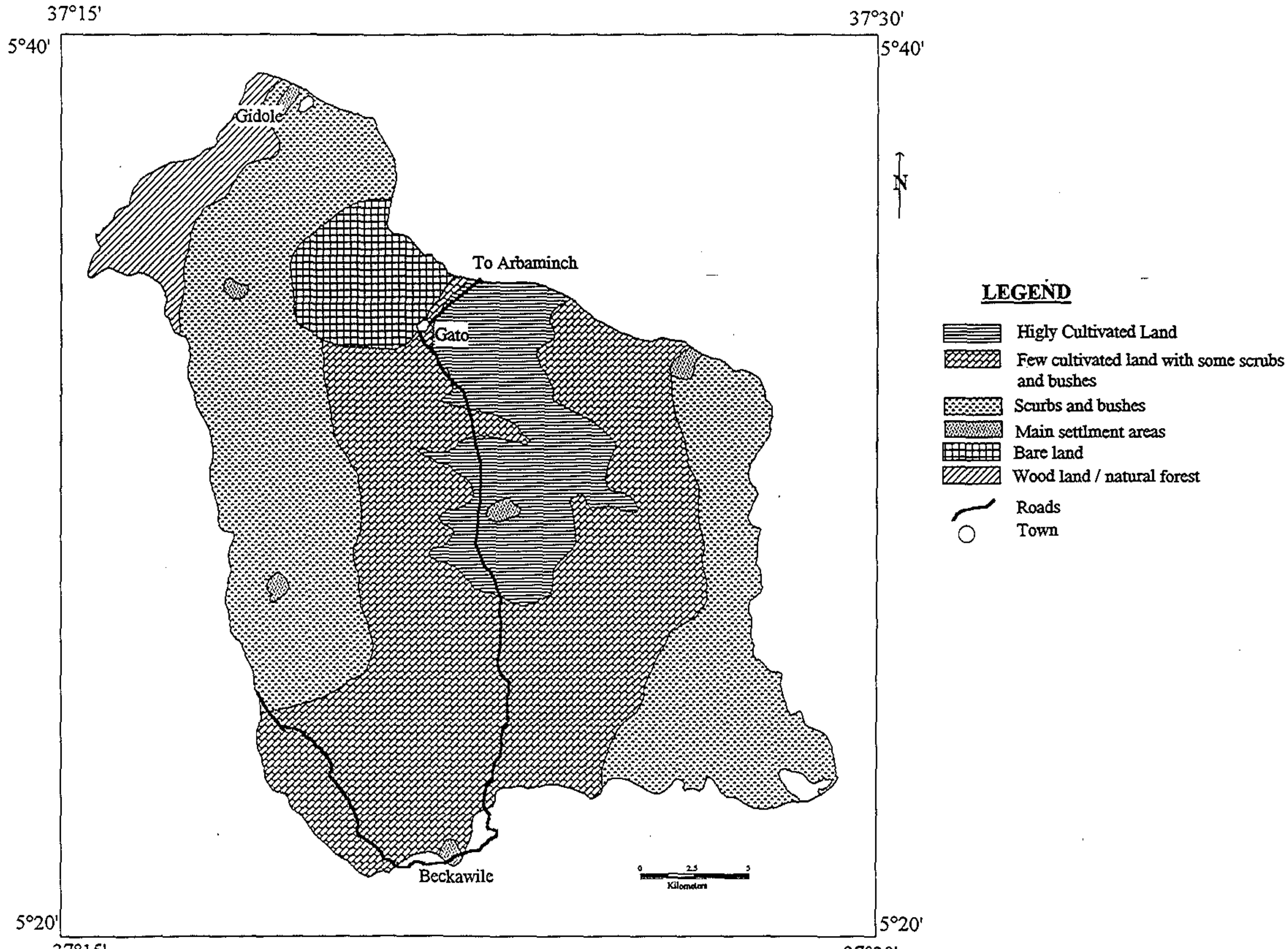


Fig (2.4) Land use /Land cover map.

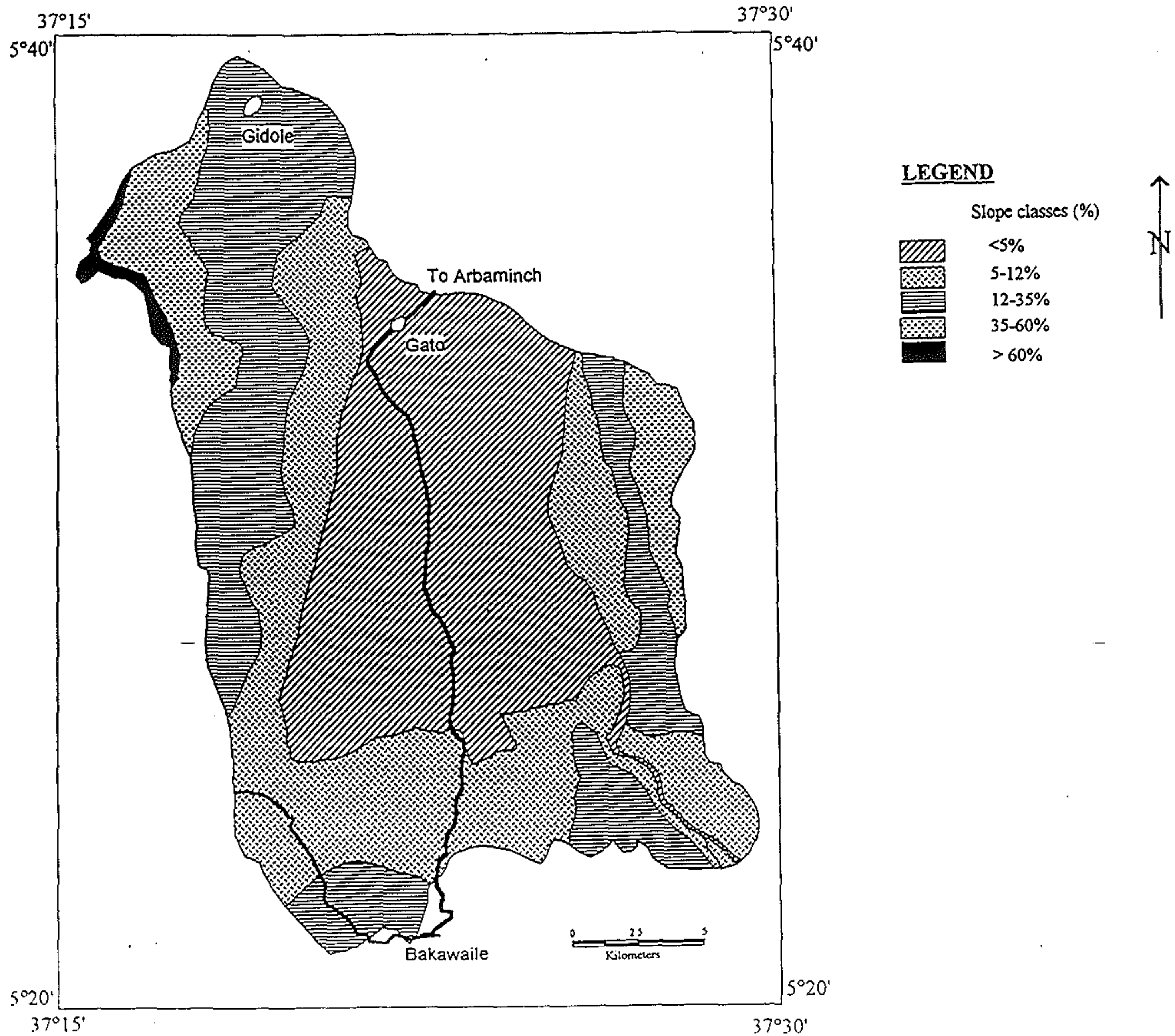


Fig (2.5) Slope map of the study area

2.4 Climate

According to Admasu Gebeyhu (1996), the study catchment area is classified as Dega, Woyina Dega and Kolla type under Agroclimatic Zonation concerning altitude rainfall, but due to lack of precipitation the project area has semi arid Agroclimatic inference. It has bimodal rain fall condition with two maxima. The main rainy season, Meher lasting from September to November, while the short rainy, Belg is from March to May. The mean annual rain fall of the study area is 830.5 mm. The average minimum and maximum temperature is 16°c and 28.5°c, with mean temperature of 22.3°c for the catchment. The daily sunshine hours range between 3 - 5 during the summer and 6 - 9 during the winter period. The relative humidity varies between 58% and 76% over the year in which the rainy seasons are the most humid. The driest months are December to February.

According to the climatic classification for Ethiopia the study area has three physiographic zones (Table 2.4). It is predominantly categorized by zones of low lands.

Altitude	Zones	Approximate area (%)
2200 - 3000	Plateau	3.3
1700 - 2200	Medium Altitude	12
< 1700	Low land	84.7

Table 2.4 Altitudinal range and zonal classification of the Iyanda catchment.

Chapter Three

Geology

3.1 Regional Geological Setting

Volcanic and tectonic events in the study area constitute an essential part of the volcano-tectonic history of the Southwestern Ethiopian Rift (SER). According to Davidson and Rex (1980), widespread volcanism in the south western Ethiopian rift began in the late Eocene, being followed by Oligocene basalts and by 12 - 13 Ma phonolite volcanism after which major rifting started. The Pliocene Mursi basalts of the Omo group demonstrably post-date major faulting (Moore and Davidson, 1978; Davidson and Rex, 1980) and Quaternary volcanic activity is localized at the southern end of the Chew Bahir Rift (Davidson, 1983). The Gofa Basin and Range and Chew Bahir rifts dominate the area west and south of the Gonjuli graben and consist of NE - SW trending fault blocks of Tertiary lava's tilted to the north west (Moore and Davidson, 1978). The history of uplift in southwestern Ethiopia thus probably began shortly after the inception of volcanism of the main undivided succession. It continued at different rates at different times and in different places throughout the Oligocene and early Miocene. Rates of accumulations of volcanic cover likely also varied, being most voluminous protracted in the region between the eastern Akobo basin and Lake Chamo, where the thickest and most continues succession are preserved today.

Despite a history of varied rates of uplift and accumulation throughout the region, there is little evidence to support the idea that major rift faulting took place before middle Miocene time.

Certainly the main displacements and tilting occurred after phonolite volcanism at ~13 ma. The result of pre Miocene up lift was to produce an elevated arc extending from Ilubabor through central Kefa to northern Gamogofa, some what concave to the north east, with the greatest build up of volcanic cover in its central and eastern parts. The system of rift valleys developed subsequently across this trend. In the broad rift zones of southern 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 and Rex, 1980, Davidson 1983). This is contrary to previous suggestions that volcanism migrated from the north western plateau toward the Ethiopian rift (MER and Afar) with time(Pilger and Rosler, 1975; Zanettin and Justin-Visentine, 1975,). West of the Weyto - Bala sector a series of south west - oriented tilted fault blocks, the Gofa basin and range system, penetrates the Ari-Hamar horest block that separates the Chew Bahir and Turkana rift systems. The Chew Bahir system is completely separated from the main Ethiopian rift by the Gemu-Gydole horest. As stated by Moore and Davidson (1978),

The metamorphic grades of the area was described by Kazmin(1975) as it varies from middle to upper Amphibolite facies and locally reached up to high grade granulites. A peculiar granulites rock exposed southwest of Lake Chamo, south of Konso village, in the Segen valley is named as Konso gneiss. In addition to this, Davidson(1983) noted that the granulite rocks are out cropped in the Hammer range of south western Ethiopia. In south western Ethiopia, Tertiary lavas lie directly on the crystalline basement, minor exceptions being were small remnants of Permian sedimentary rocks are preserved.

As summarized by Omo River project study compiled by Davidson (1983), five major lithologies were found a adequate to express the main deference's among the gneiss's of the older complex. These are:

I. relatively mafic hornblende gneiss and Amphibolite, II. well layered biotitic gneiss containing clearly recognizable meta-sedimentary components, III. dominantly gray biotite-hornblende gneiss of variable color index, in part containing ill-defined masses of relatively uniform ortho-gneiss of granodioritic, tonalitic and dioritic composition, IV. pale pink to light gray quartzo-feldspathic gneiss, generally leucocratic and having a granitic composition, and V. biotitic granitoid gneiss commonly containing a little muscovite. Relatively dark gneiss's, usually rich in hornblende and poor in or lacking quartz, underlie a large area from the Segen River north to the southern side of the Gydole high land, and reappear from beneath Tertiary volcanic cover in and north of the Tsamai basin.

The highest grade terrain's, out lined by the presence of hypersthene - bearing rocks identified are centered in two area, one in the Hammer range and the other around Konso. Many of the gneiss's in the region north east of the Hammer center, notably in the Weyto horest and around the Tsamahai basin and southern Bala rift, have the textural attributes of granulite, but were not noted to contain hypersthene.

3.2 Geology of the study area

3.2.1 Stratigraphy

The stratigraphy of the study area is essentially simple with the Precambrian basement complex being overlain by the Tertiary volcanics. The area is made up predominantly of basaltic rocks occurring in the eastern and western high lands of the catchment and metamorphic rocks such as gneiss and gabbros in the western side of the catchment. Superficial deposits consisting of alluvium occur widely in the flat plain areas. Local patches of syntectonic gabbros occur in the western part of the study area fig 3.1. The general stratigraphic succession from the oldest to the youngest is as follows:

I. Amphibole Gneiss

This is the major rock type which covers most part of the study area. The unit is trending 20° - 80° north west on the average and dips mostly south westerly. It is medium to coarse grained, composed of plagioclase and hornblende. It is greenish gray in color and banded. petrographic studies from the representative sample show that 50 % hornblende, 25 % plagioclase, 15 % sericite, 5 % calcite, and 5 % chlorite.

The texture is gneissose, sericitization is the dominant alteration besides calcite grows within the fracture of hornblende crystals. Chlorite and calcite grow on plagioclase and chlorite also forms at the expense of hornblende.

II. Meta / Syntectonic Gabbro

The metagabbro is exposed along the Gapamaga river course within the amphibole gneiss as lenses. The rock is massive to weakly foliated, dark gray colored, composed of plagioclase, hornblende, pyroxene and biotite.

Texturally it varies from medium to very coarse grained. Alterations are observed in the contacts with other rocks. Since this alterations with the contacts are observed and that it is weakly foliated, this rock unit is interpreted as syntectonic. The metagabbro is chloritized, tremolitized and carbonitized.

Petrographical descriptions of representative sample show that the average content of actinolite 30%, hornblende 20%, plagioclase 17%, epidote 12%, chlorite 10%, calcite 5%, apatite 2%, rutile 1% and relicts of pyroxenes.

III. Tertiary Basalt

This unit is by far the most widespread in the area occupying practically all the mountainous terrain in the area. It is a thick succession with different degree of weathering, and minor fine grained leucocratic tuff interbeds at different levels. The basaltic unit is fine grained, melanocratic in color and massive. Columnar jointing are observed at different places. It covers the highest elevation in the area. The volcanic rock basalt and tuff has angular unconformity contact with the gneiss's. The basalt is observed in Gydole high land, Konso, Doketu and in many other areas with different degree of weathering

IV. Quaternary sediment

Quaternary sediments of the area include residual soils, colluvial and alluvial deposits. Residual deposits ranging in size from sand to silt are mainly observed on highland (top of ridges) especially in the western and southern part of the study area. They are formed from the weathering of basalts and gneiss's.

Colluvial deposits dominantly occur on the slope of hills covering the areas between residual soils and alluvial deposits. It is mainly observed at the western and southern part of the study area. The soils are characterized by mixtures of particles of contrasting sizes clayey gravelly sand and are also commonly variable within a different depositional areas.

Alluvial deposits consisting of gravels and sand at the main rivers and its levees and silt and clay soils occur at the flood plains mainly found at the central part of the study area.

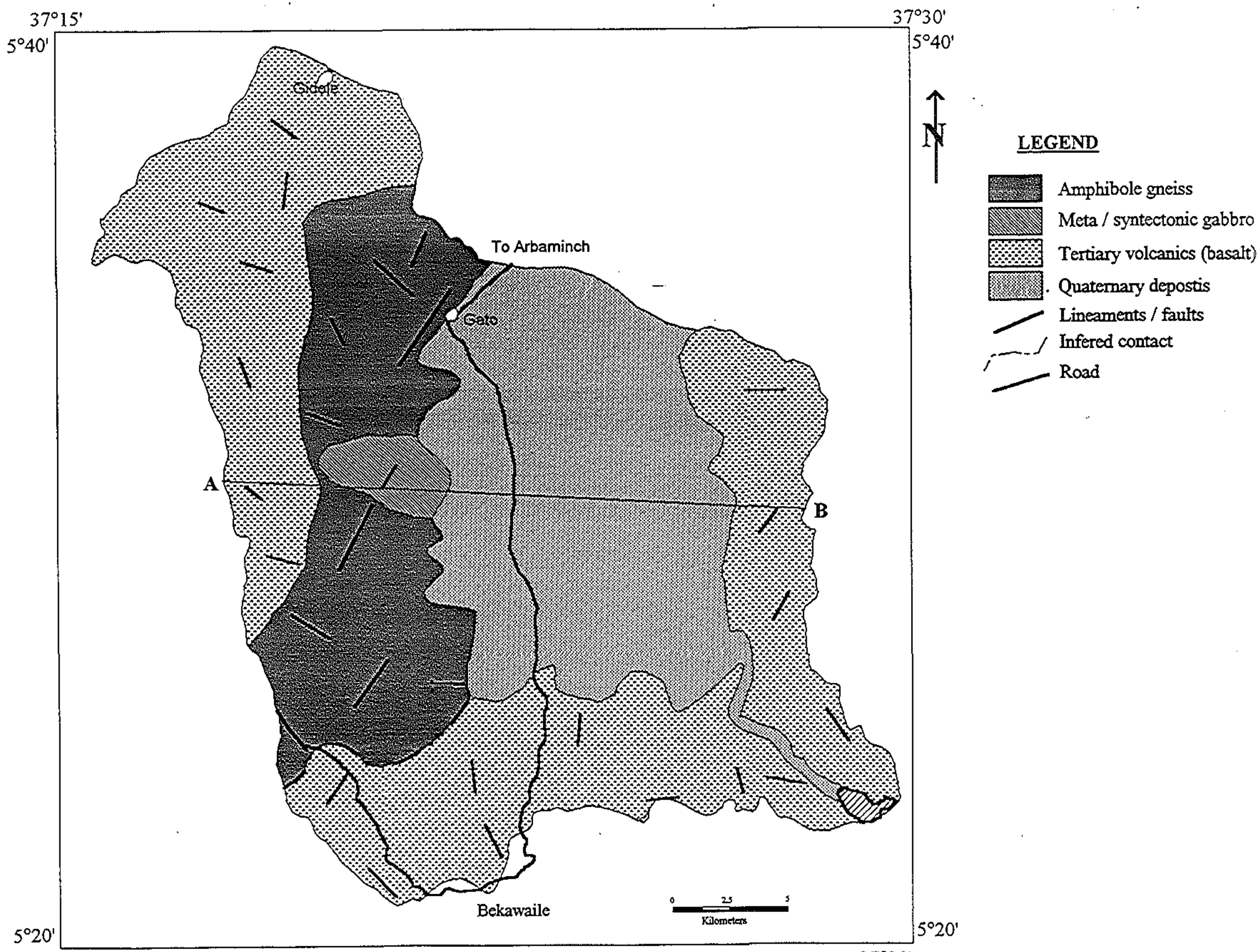


Fig (3.1) Geological map of the study area

3.3 Geological structures and Seismicity

In the present work the structural features, which are inferred in the catchment, are mapped from 1:50,000 areal photographs of the area. From this scale photos, lineament and some faults are mapped. These lineament include straight contact between various rock units, sudden and lithologic contact, straight river courses and aligned topographic features. NW-SE lineaments: are most dominant types of structures in the catchment followed by NE-SW lineaments and some minor ENE and WNW lineaments are also mapped. Some faults are also mapped and confirmed in the field work, the orientation of these faults are NW-SE and NE - SW. fig 3.1

Variation in the amount of displacement along the strike of individual faults and fault sets accounts for the disappearance of some major rift features. For example, at the southern end of the main Ethiopian rift, the basement surface rises from below 800m near lake Chamo to 1700 m in the Konso upland. West of this rift, its elevation falls south wards from 2750 m to 1600m, so that the net movement sense on the faults defining the western boundary reverses in a distance of some 30 km. Similar on strike reversal is observed along the eastern boundary of the Surma horest, but there with basement rising north wards along the east side of the fault. This kind of rotational movement about poles to fault planes appears to account in part for the southwest ward termination of the Gofa basin and range (system), and the north ward attenuation of the Turkana rift (Boccaletti, etal.,1998).

The geologic structure of the study area is controlled by tectonic events that lead to the development of the rift system. One point worth mentioning in relation to the geological structure of the area is Seismicity. Since the area is located near the area seismically active zone, fig 3.2 it seems logical to discuss records of distribution of earthquakes and seismic shocks in close structurally related areas. Report of Seismicity indicate that they are concentrated in vicinity between 5° - 5° 40' latitude and 36°50' - 37° 30' longitude, (Boccaletti, etal., 1998) . To the south, the southern Chew Bahir rift structure is more or less symmetric with its prominent, N - S oriented, slightly curvilinear, master fault of the eastern side. Vertical displacement changes along the strike and a maximum value of about 1200m is observed on the western boundary fault, South of Woyto horest (at about 5° 20' N) the boundary faults show a marked change of direction, striking about NNW. In this area, the NE - SW trending boundary faults have been cut by the NNW - SSE trending faults, that are referred to as Woyto fault zone (Boccaletti, etal., 1998). This zone is marked by strong seismicity (Gouin, 1979). About ten earthquakes of magnitude greater than 4 have been recorded near the project area. All ten were in the immediate area of Chew Bahir rift and its northern extension. This information suggests that at present the Chew Bahir rift system is seismically more active than the Turkana rift system (Gouin, 1979).

Hence the expected earthquakes impose additional loads on the embankment dams to be constructed and can affect by causing any of the following:

- Settlement and cracking of the embankment;
- Reduction of the freeboard due to settlement, which may in the worst case, result in overtopping of the dam;
- instability of the upstream and down stream of the dam;
- differential movement between the embankment, abutments and spillway structures, increasing the likelihood of leakage and piping failure;
- liquefaction or loss of shear strength in the embankment and its foundations due to increase in pore pressures induced by the earthquake;
- damage to outlet works passing through the embankment leading to leakage and potential erosion of the embankment.

Dams to be constructed on clay soil can withstand strong shacking ranging from 0.35 to 0.8 g from a magnitude 8 earthquake with no apparent damage and dams which have suffered complete failure or slope failures as a result of shaking seem to have been constructed primarily with saturated sand foundations, (Tschebotarioff 1979)

Such harmful effects of earthquake can be eliminated by adopting defensive measures which render the effects non harmful, a list of such defensive measures would include the following:

- allow ample freeboard to allow for settlement and slumping
- use wide transition zones (filters) of material not vulnerable to cracking
- provide ample drainage zones to allow for possible flow of water through cracks
- use a well graded filter zone upstream of the core to serve as a crack stopper
- provide crest details which will prevent erosion in the event of overtopping

The methods of assessing likely earthquake intensity and frequency at a given site are complex, requiring reasonable judgment and collection of geological and seismic data. Due to this complexity for structures with lesser magnitude, the tendency is to rely upon seismic risk maps. The maps are often published in national or state building codes which recommend the engineering precaution to be taken in each rank of hazard shown in the map.

Detail and complete analysis of Seismicity has not been done on the area under investigation, for one thing seismology data as stated above can not be collected directly at the field by observing physical features of the area. Secondly it needs an expert on the subject matter to make statistical analysis using the past records to reach a confirmative fact.

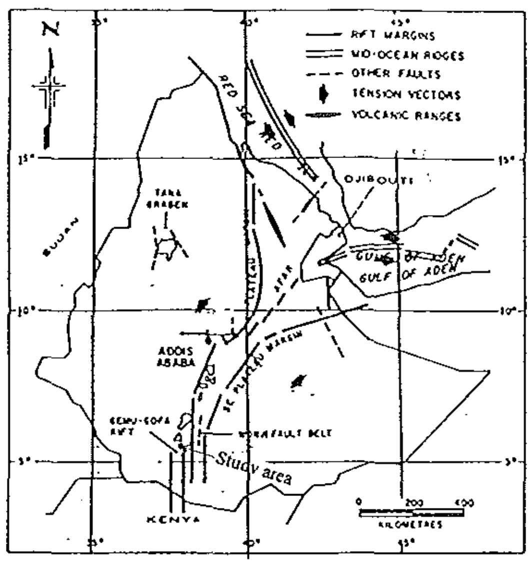
Due to these reasons only some basic information has been taken from a bulletin of Geophysical Observatory, Gouin (1975). According to the author, believed to be practical and informative working paper. Here in this bulletin according to the recommendation of the building code of Ethiopia, the country is divided in to zones of approximately equal seismic risks based on the known distribution of past damaging earth quakes. The zoning are mentioned in the bulletin as follows.

0. Zone of no damage
1. Zone of minor damage. Its grading corresponding to intensities V and VI on the Mercalli Modified (MM) intensity scale
2. Zone of moderate damage corresponding to intensity VII
4. Zone of major damage in which the seismic ground shaking would produce intensities VIII and above.

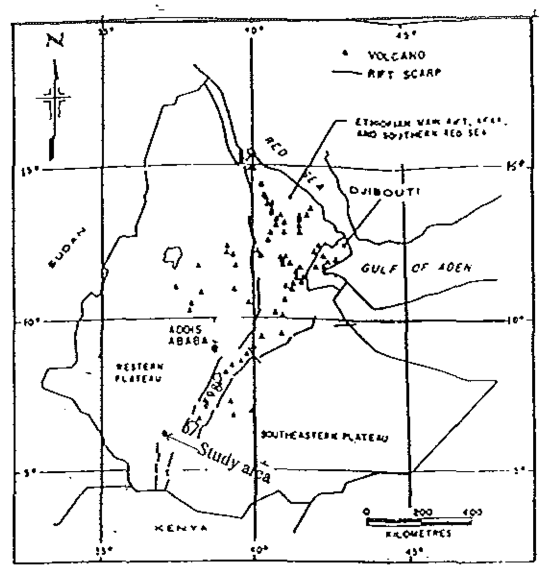
Accordingly ground acceleration map, Intensity map and Zoning map Fig 3.2 (D,G,H) are produced for the country. It is also mentioned that these maps are based on the amplitudes to be expected during 100 years return period with on the average 1 % annual probability that these values will be exceeded.

From the maps the site is found within the area having 20 % ground acceleration and VIII intensity within 100 years return period and probability of 0.01 per annum of being exceeded, and the site falls under zone 4 with a corresponding major damage.

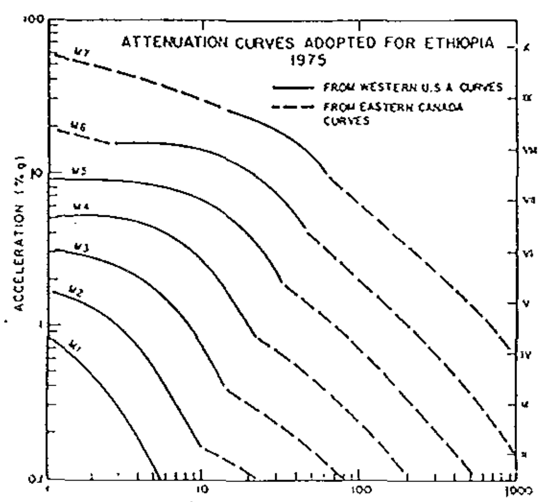
figure (3.2) Major Tectonics and Seismicity of Ethiopia



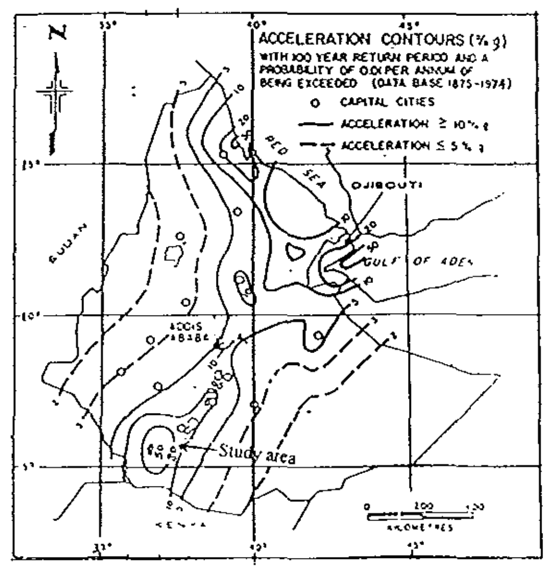
A. MAJOR TECTONIC FEATURES



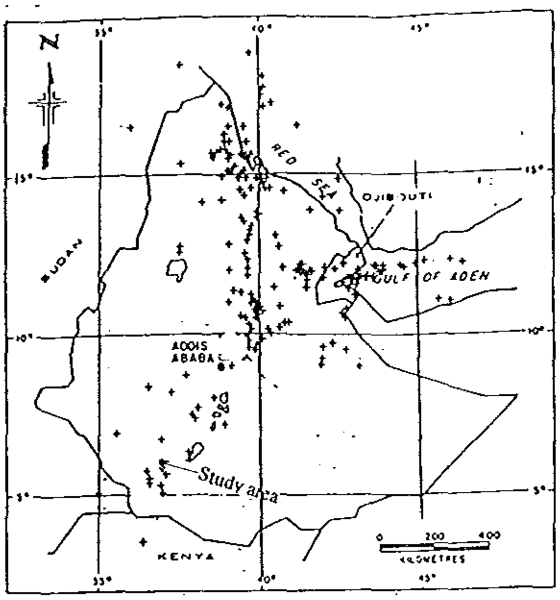
B. LOCATION OF VOLCANIC CENTRES



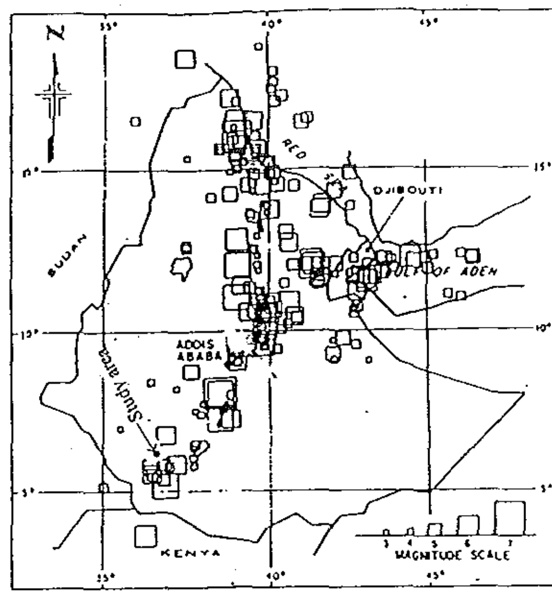
C. ATTENUATION CURVES



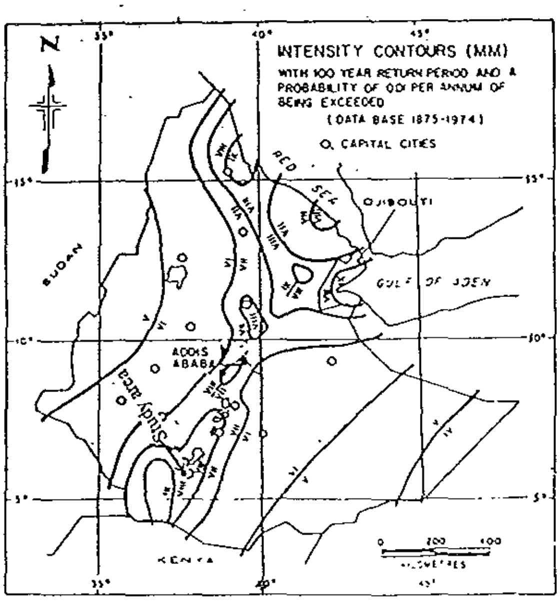
D. PREDICTED GROUND ACCELARATION



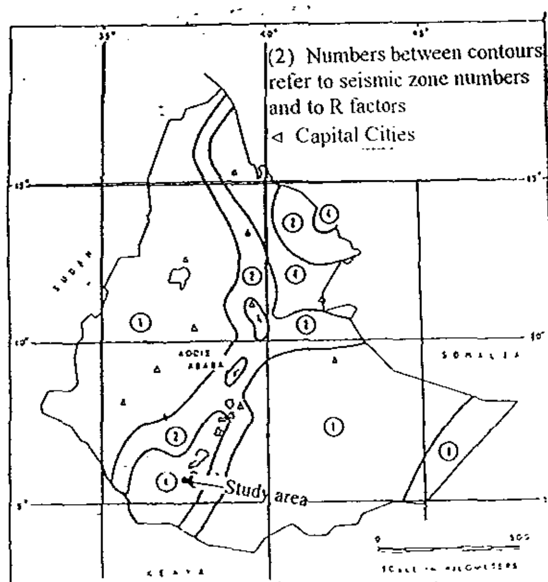
E. LOCATION OF SEISMIC EVENTS



F. LOCATION OF SEISMIC EVENTS OF MAGNITUDE ≥ 4



G. PREDICTED INTENSITY



H. Seismic Zoning Map (1976) Based on 100 year return periods & 0.01 probability of being exceeded

Chapter Four

Hydrometeorology and Hydrogeology

4.1 Hydrometeorology

In defining hydrological elements like precipitation, evaporation, run off and infilteration, analyzing hydrometeorological variables are important, these parameters are important in defining and characterizing the water balance of a given area, hydrometeorological variables are also important in land use planing, site selection and design of a project.

4.2 Precipitation

The calculated mean annual precipitation of the catchment is 830.5mm. Since the gauging stations are located in the catchment from the highest elevation to the lowest at elevation of 2250, 1250 and 1550 m.a.s.l at Gydole Gato and Konso, the distribution of the station is so good that the arithmetic mean and Thiessen polygon show no major difference.

From precipitation data the year can be classified as:

- High rainy period, March, April , May, September and October
- Small rainy period, November, February , June. July
- Dry period, December, January , and August

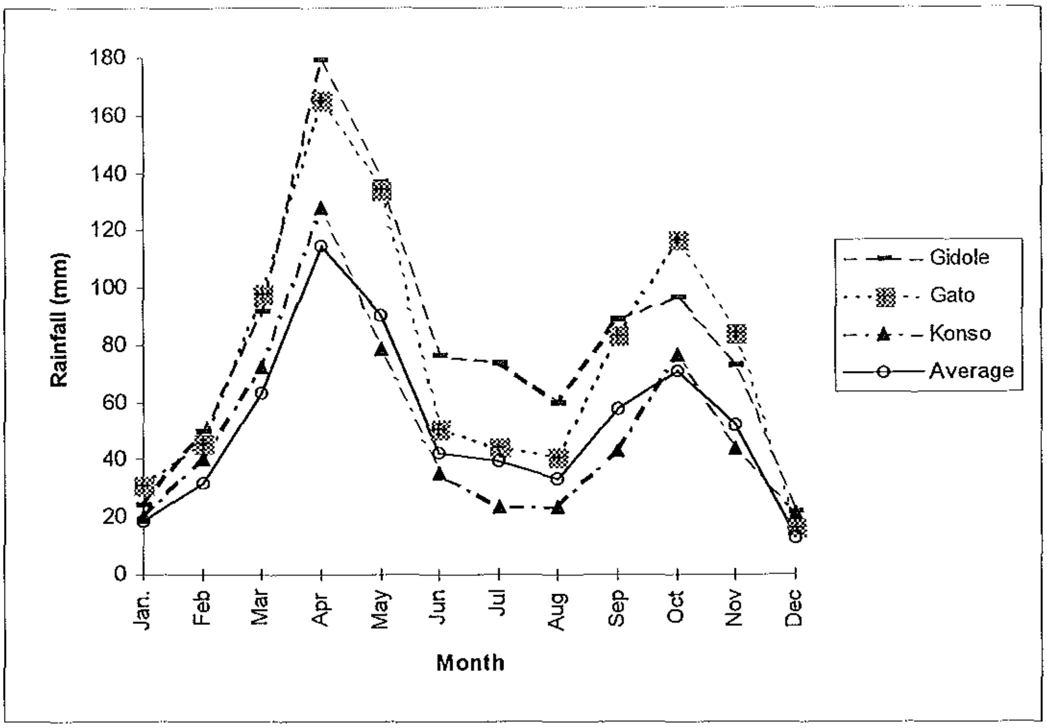


Figure 4.1 Monthly rainfall variability of the stations

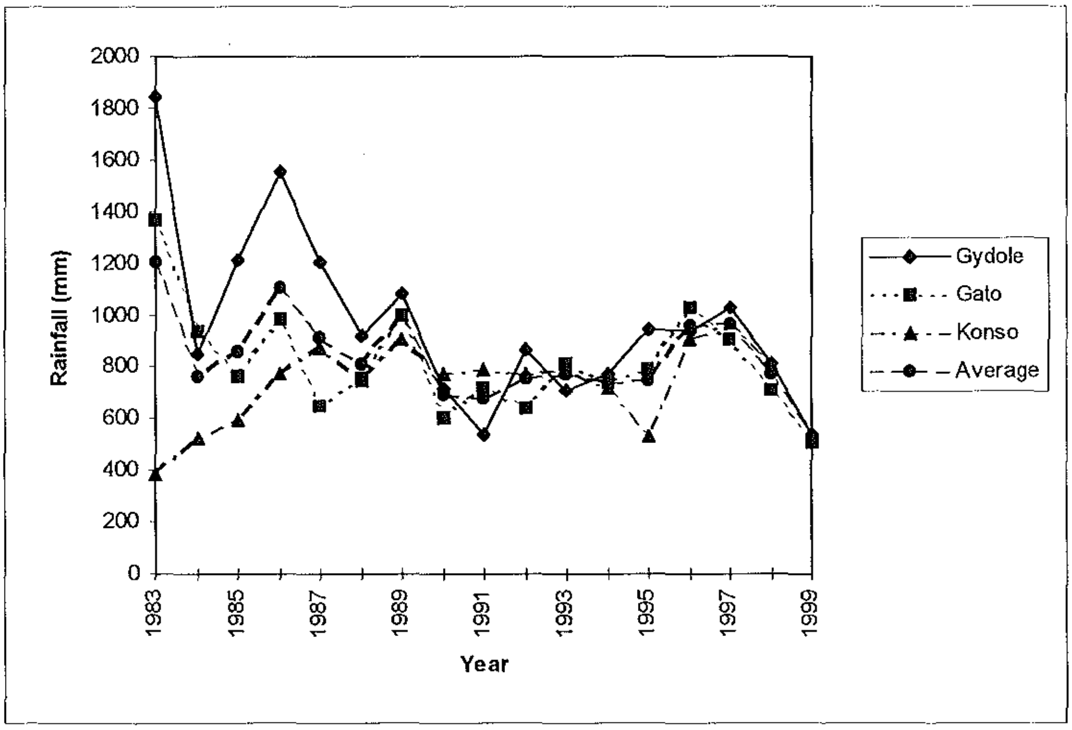


Figure 4.2 Long term annual rainfall variability of the stations

The precipitation of the area shows large seasonal and inter annual variability. The record of precipitation is available since 1971. The average mean annual rainfall for the Gydole Gato and Konso stations is 972, 912 and 607.5 mm/yr. respectively. The high rain fall in Gato with respect to Konso may be due to the location of the meteorological station at the base of the Gardula mountain. The study area is characterized by skewed bimodal profile with absolute peak in April and relative peak in October. The area gets more than 89.5% during the big and small rainy season and the remaining 10.5% in the dry months. From the records of, Gydole, Gato, Konso, Gumayde, and Arfayde, the weighted mean annual rainfall of the catchment, using Thiessen polygon and arithmetic mean method is found to be 830.6mm and 830.5mm respectively. Fig 4.1 (Table 4.1 & 4.2).

Station	Area %	Point Rain fall	Total rain fall
Arfayde	5	754.4	37.72
Gumayde	6	925.63	55.53
Konso	34	607.5	227.97
Gato	42	912.05	383.06
Gydole	13	971.93	126.34
Total	100		830.6

Table 4.1 Rain fall distribution at different stations using Thiessen polygon method

4.3 Temperature and wind speed

Temperature plays an important role on the evaporation process of the site. There are three stations within the catchment having temperature records of more than 25 year. The summary of mean monthly minimum and maximum temperature for the stations and mean monthly temperature of the catchment is given in Table 4.3. Wind speed has also a strong influence on evaporation, 7 to 11 year data was analyzed at the three stations and it is found out that the mean monthly wind speed ranges between 0.98 and 1.19 m/s with mean yearly average of 1.08 m/s. The months from February to April and August to September have strong winds and months of May to July and November have lower wind speed. Wind speed is measured at 2 m above the ground surface in all the three stations. The summarized mean wind speed and temperature of each stations and mean monthly wind speed and temperature of the catchment is given in Table 4.3

4.4 Humidity and sunshine hours

The relative humidity is the ratio between the amount of water vapor actually contained per unit volume and the amount that it could hold at the same temperature when it is saturated or the actual vapor pressure and the saturation vapor pressure at the same temperature. The relative humidity is measured three times a day in two stations. The data show that there is a marked difference in relative humidity both spatially and temporally; the minimum value was recorded at Konso (45.7%) in the month of February and maximum at Gato (80.6%) in May. Only the stations at Gydole and Konso have records of sunshine hours. The mean monthly sunshine hours range from 3 - 9 hours with mean monthly average of 6.48 hours. The months January, February, March, May and November have the least cloud cover while the months of June, July and August have the highest. Summarized relative humidity and sunshine hours data were given in Table 4.4 a & b

Parameter	Station	Jan.	Feb	Mar	Apr.	May	Jun.	Jul.	Aug.	Sep	Oct.	Nov.	Dec.	Annual
Rain Fall	Gydole	24.00	49.70	91.92	179.50	136.6	76.31	73.7	59.70	88.9	96.4	73.0	22.20	971.9
Rain Fall	Gato	31.26	46.06	98.29	165.06	133.9	50.94	44.3	41.26	83.6	116.7	83.9	16.75	912.1
Rain Fall	Konso	20.48	40.02	72.25	127.78	78.47	35.60	23.8	23.85	43.4	76.7	43.9	21.22	607.50
Rain Fall	Gumaide	51.37	48.42	71.67	189.95	137.9	48.45	38.3	38.03	66.2	118.5	83.9	32.91	925.63
Rain Fall	Arfayde	63.34	31.40	74.44	127.94	103.4	47.66	34.6	21.99	42.3	99.8	79.2	28.27	754.4
Absolute Average of catchment rainfall and temp.														
	Jan	Feb	Mar	Apr.	May	Jun.	Jul.	Aug.	Sep	Oct.	Nov.	Dec.	Annual	
Rain Fall	25.25	45.26	87.49	157.45	116.33	54.28	47.29	41.6	71.96	96.6	66.97	20.06	830.5	

Table 4.2 The mean monthly rainfall of the stations and the catchment

Station	Jan	Feb	Mar	Apr.	May	Jun.	Jul.	Aug.	Sep	Oct.	Nov.	Dec.	Ave.
Gydole W.S	1.10	1.30	1.38	1.16	1.16	1.28	1.33	1.53	1.40	1.21	1.12	1.19	
Temp	21.54	22.20	21.67	20.77	20.74	19.96	19.32	20.06	20.42	20.16	20.14	20.75	20.64
Gato W.S	0.85	0.95	0.82	0.89	0.72	0.72	0.70	0.69	0.73	0.68	0.60	0.77	
Temp	24.16	24.54	24.49	23.77	23.45	23.05	22.53	22.94	23.64	23.19	23.34	23.87	23.58
Konso W.S	1.13	1.17	1.33	1.39	1.14	1.00	1.00	1.23	1.44	1.30	1.22	1.21	
Temp	24.23	24.58	24.18	22.18	21.82	21.45	20.79	21.31	21.99	21.97	22.51	23.53	22.54
Ave. W.S	1.03	1.14	1.18	1.15	1.01	1.00	1.01	1.15	1.19	1.06	0.98	1.06	1.08
Ave. Temp	23.31	23.77	23.44	22.24	22.00	21.48	20.88	21.44	22.02	21.77	22.00	22.71	22.26

W.S = wind speed.

Table 4.3 Mean monthly wind speed and temperature of the catchment

Summarized R.H Data													
Station	Jan	Feb	Mar	Apr.	May	Jun.	Jul.	Aug.	Sep	Oct.	Nov.	Dec.	
Gato	70.44	70.75	71.67	78.72	80.61	78.33	78.19	78.58	77.36	77.08	75.39	74.75	
Konso	47.70	45.70	52.85	67.44	70.48	65.93	66.44	60.00	58.59	62.41	58.59	50.41	
Ave.	59.07	58.23	62.26	73.08	75.55	72.13	72.32	69.29	67.98	69.75	66.99	62.58	

(a)

Summarized Sun shine hrs.													
Station	Jan	Feb	Mar	Apr.	May	Jun.	Jul.	Aug.	Sep	Oct.	Nov.	Dec.	Average
Gydole	8.42	7.72	7.83	7.65	7.60	5.83	3.28	4.35	6.10	6.87	8.45	9.12	
Konso	6.07	7.56	8.01	6.16	7.29	5.13	3.46	5.04	6.40	6.07	6.79	4.53	
aver.	7.24	7.64	7.92	6.90	7.44	5.48	3.37	4.70	6.25	6.47	7.62	6.82	6.48

(b)

Table 4.4 (a) Relative Humidity of the catchment

(b) Sunshine hours of the catchment

4.5 Evaporation

The knowledge of evaporation has its direct importance to the water resources engineer in the investigation of water resources availability, as the net amount of water available is basically influenced by the evaporation which take place during the period of utilization. The water requirement estimation particularly in the case of irrigation, requires good understanding of evaporation and evapotranspiration rates.

Generally evaporation depends on both geographical coordinates and on the ground feature such as; wind speed, temperature, radiation and altitude above see level. The piche evaporimeter consisting of a glass tube 14 mm in diameter and 225 mm long with one end closed, having circular disc 32 mm diameter of absorbent plotting paper held against the open end by a small circular metal disc with a spring collar installed at Konso station is used in open water evaporation of the study area. The evaporating surface area is 1300 mm^2 and this is fed constantly by the water in the tube hung up by its closed end. The tube is graduated to give a direct reading of evaporation over a chosen time period, usually a day. The measurement in millimeters is related to the evaporating surface of both sides of the paper.

The tube holds an equivalent of 20 mm of evaporation; the water is replenished when necessary. When the piche evaporimeter is exposed in a standard temperature screen, the annual values have been found to be approximately equivalent to the open water evaporation from a US class A pan. (Shaw,1988)

Piche evaporation of three year data is recorded at Konso station This data show that high evaporation occurs during the dry months. Similar to class A pan, a pan coefficient of 0.75 to 0.85 are commonly used for evaluation of open water evaporation. The mean annual piche evaporation is 1976.8 mm, so using the highest pan coefficient 0.85 due to the high evaporation rate of the area the annual evaporation from the storage reservoir is 1680.3 mm.

Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sep	Oct.	Nov	Dec	Total
227.73	240.47	212.73	125.17	131.57	121.77	126.80	161.67	173.40	109.57	157.2	188.7	1976.8

Table 4.5 Monthly mean piche evaporation at Konso station in (mm)

4.6 Evapotranspiration

Evapotranspiration is a phenomenon by which water passes from liquid or solid state to vapor. Water may evaporate from bare soil or covered with vegetation, from trees, open water, snow, glacier etc. with different rate, following different parameters.

From the evaluation of the data the climate of the basin is so dry that the soil never reaches field capacity, that is, the available water capacity of the root zone is never filled. Under these conditions it is not obvious when the accumulation of potential water loss should begin .

The Iyanda soil water balance evaluation indicates large moisture deficit and no runoff. When rainfall exceeds evapotranspiration, the excess simply recharges the soil moisture but does not exceed the field capacity so that no moisture surplus or runoff is generated. Generally the model is not giving realistic value, but the realistic difference of hydrological and hydrogeological variables are well represented. This model can not be used for the optimization of the dam. The most realistic value can only be obtained by short period storm records which is not shown in the monthly average data. Much of the water comes to the reservoir during short period high intensity rainfall, which is quite typical to arid regions

4.7 Peak runoff calculation

Runoff is the total amount of water that leaves the basin and occurs when the rate of precipitation exceeds infiltration (Chow, etal., 1988).

There are no flows at all for many months at downstream site. Though regions with two rainy seasons have the advantage of maintaining their low flows at a reasonable level; as it has been observed from soil water balance evaluations, the studied areas have flows only due to storm that needs to be supplemented by additional storage. Whenever it is aimed to use stream flows with out a regulating reservoir, the adequacy of stream flows to meet requirements for irrigation poses a problem. The runoff coefficient can be transferred from the upstream to the downstream,(Tamiru Alemayehu,1992). The study area has mean annual flow of 31.35 million m^3 at the upstream site in Gato river. But due to irrigation below the gauging station and lack of the catchment soil to reach field capacity, the water disappear half way in the plain area before reaching the dam site. Assuming that the mean precipitation(942mm) is the average of Gydole station and Gato station. The mean rain fall from the area of $148km^2$ is $139.41Mm^3$. The surface discharge corresponding to this rain fall is $13.04Mm^3$. Hence the run off coefficient is 0.094, this is the maximum value, due to the high slope, and bed rock exposed at some places. Assuming that all parameters are equal, the maximum discharge expected from the study catchment is $51.1Mm^3$ due to the low slope, less than 5% and pervious lithological condition the actual runoff is much less than the given amount.

The Iyanda soil water balance evaluation indicates large moisture deficits and no run off. Soil water balance calculations are usually made on long term average climatic data in order to obtain a rough estimate of the seasonal march of rain fall, evapotranspiration, soil moisture, irrigation need, and run off. Long term averages tend to underestimate extreme deficits and surpluses. In a wetter than average year, a moisture surplus is often developed around Iyanda and ground water receives some recharge, and regions of the highland areas due to high rain fall, low temperature and low moisture holding capacity of the soil, receives some recharge even during an average year. In general moisture deficit that can indicate drought probabilities is based on how much water is held in the soil rather than on the rain fall total. To overcome this shortage of moisture and bring the soil to field capacity, farmers collect storm runoff by constructing simple ditches and soil bounds.

	Year	Month	STATION	Daily Highest rainfall (mm) in 24 hrs.			
1	1985	May	Gydole		65.30		
2	1986	May	Gato		72.60		
3	1987	May	Gydole		60.60		
4	1988	Sep.	Gato		62.20		
5	1989	May	Konso		74.30		
6	1990	May	Konso		54.30		
7	1991	May	Konso		62.30		
8	1992	Sep.	Gydole		65.80		
9	1993	Feb.	Gato/Gydole		62.80		
10	1994	May	Gydole		48.00		
11	1995	May	Gato		50.60		
sum					678.80		
Rf Mean					61.71		
St.dev					8.22		
V					1.61		
	Rft = RfMean + K(T) * St.dev						
T	5.00	10.00	15.00	20.00	25.00	50	100
LN(T)	1.61	2.30	2.71	3.00	3.22	3.91	4.61
K(T)	0.57	1.10	1.41	1.64	1.82	2.38	2.96
Rft	66.36	70.73	73.32	75.18	76.65	81.3	86

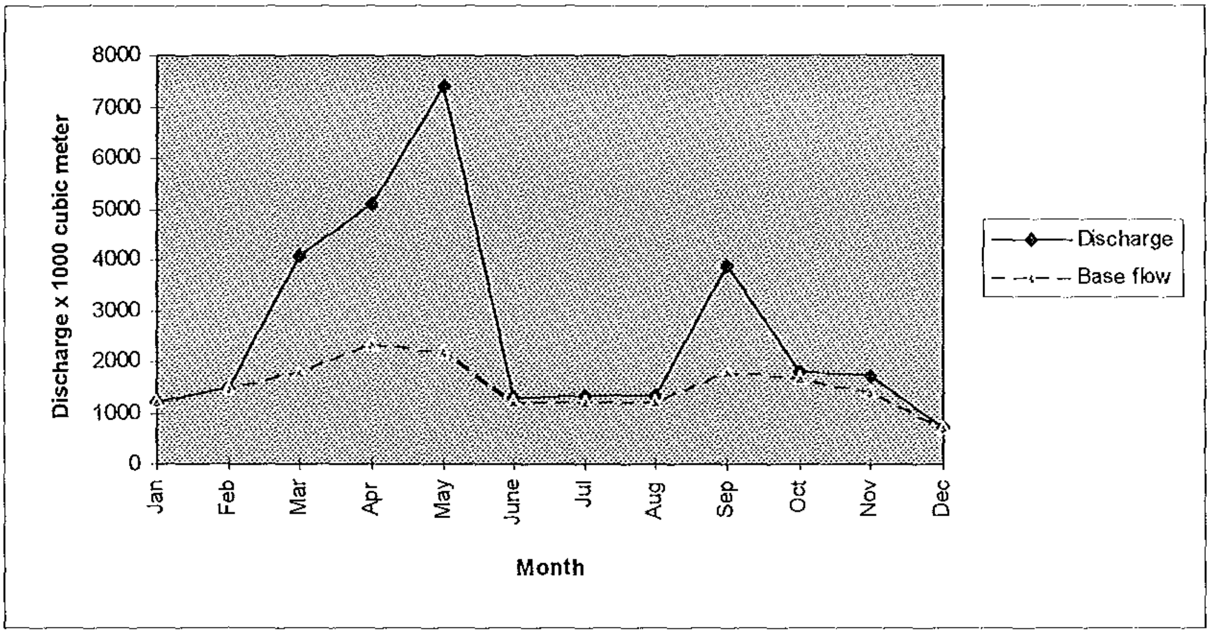
Rft = Rf mean + K(T) (St.dev.) T = Return Period (Year)
Rft = Maximum Rain fall (mm) K(T) = - (0.45+0.7797Ln(Ln(T)) -Ln (T-1)
St.dev. = standard deviation V = variance
So Maximum design rain fall (24hrs.) for 100 Year return period is 86mm

Table 4.6 Competition of Design Rain fall by Gumbel Powel method (Santosh,etal.,1989)

The only perennial river in the catchment is Gato river which disappear half way in the catchment. The base flow and the surface flow is given in (Table 4.7). Some hydrological parameters of the catchment area are given in (Table 4.8). The time of concentration for this kind of non uniform storm is the longest travel time, time of lag is an approximate of the mean travel time and the time of peak is necessary to develop a design hydrograph for routing run off through a storage reservoir.

Area of the catchment is 148km² Discharge m³ x1000													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Discharge	1218	1473	4087	5081	7414	1304	1325	1326	3874	1802	1742	710	31354
Base flow	1218	1473	1800	2350	2200	1208	1227	1227	1800	1700	1400	710	18311.0
Sur. runoff	0	0	2287	2731	5214	96	98	99	2074	102	342	0	13043.0

Table 4.7 Discharge of Gato River



Figure(4.3) Base flow and surface flow of Gato River

Designation	Symbol	Unit	Value
Area of catchment	A	km²	
Length of main water coarse from water shade divide to proposed dam site	L_1 L_2	m	18500 50000
Level of water shade divide opposite to the head of main water course	H_0 H_1 H_2	m	2600 1280 924
Slope of main water course	S_1 S_2	m/m	0.071 0.0071
Time of concentration $Tc = 1/300(L/\sqrt{S})^{0.77}$ $\sum Tc$	Tc_1 Tc_2	hr	1.78 9.3 11.08
Rain fall excess duration $D = Tc/6$	D	hr	1.85
Time to peak $tp = 0.5D + 0.6Tc$	tp	hr	7.57
Time of base of hydrograph $tb = 2.67tp$	tb	hr	20.2
Lag time $t_L = 0.6Tc$	tL	hr	6.65
Peak rate of discharge created by 1mm rain fall excess $qp=0.21AQ/tp$	q_p	m³/s	18.25

Table 4.8 Some Hydrological parameters of the study catchment area

4.8 Reservoir storage capacity

It is quite important to know the storage capacity of a given reservoir under consideration to plan its operation. In determining the storage capacity, topographic survey was made. The area under each contour is found by the help of planimeter. The result is tabulated in Table 4.9

Item No.	1	2	3	4	5	6	7	8	9	10	11
Elev.(masl)	923	924	925	926	927	928	929	930	931	932	933
Area (ha)	1.6	3.8	5.9	8.2	10.4	12.5	19.3	40.9	64.8	99.2	117.7

Item No.	12	13	14	15	16	17	18	19	20	21
Elev.(masl)	934	935	936	937	938	939	940	941	942	943
Area (ha)	132.2	145.6	154.3	163.5	171.3	180.4	190.2	196.4	205.3	215

Table 4.9 Area of the proposed reservoir in hectare (ha)

Water level	Area (ha)	Average Area(ha)	Vertical dist. (m)	Incremental Volume of storage(ha-m)	Accumulative volume of storage (ha-m)
922	0	0	0	0	0
924	3.8	1.9	2	3.8	3.8
926	8.2	6	2	12	15.8
928	12.5	10.35	2	20.7	36.5
930	40.9	26.7	2	53.4	89.9
932	99.2	70.05	2	140.1	230
934	132.2	115.7	2	231.4	461.4
936	154.3	143.3	2	286.6	748
938	171.3	162.8	2	325.6	1073.6
940	190.1	180.7	2	361.4	1435
942	205.3	197.7	2	395.4	1830.4

Table 4.10 Computation of storage capacity

The storage capacity of the reservoir including the dead storage level is more than 16 million cubic meter. The gross amount of water which leaves the reservoir through and under the dam was not evaluated since the design of the dam was not completed. Therefore, the storage capacity obtained above (Table 4.10) and the area of the reservoir (Table 4.9) is used to draw the area capacity curve of the reservoir and is shown in fig 4.4

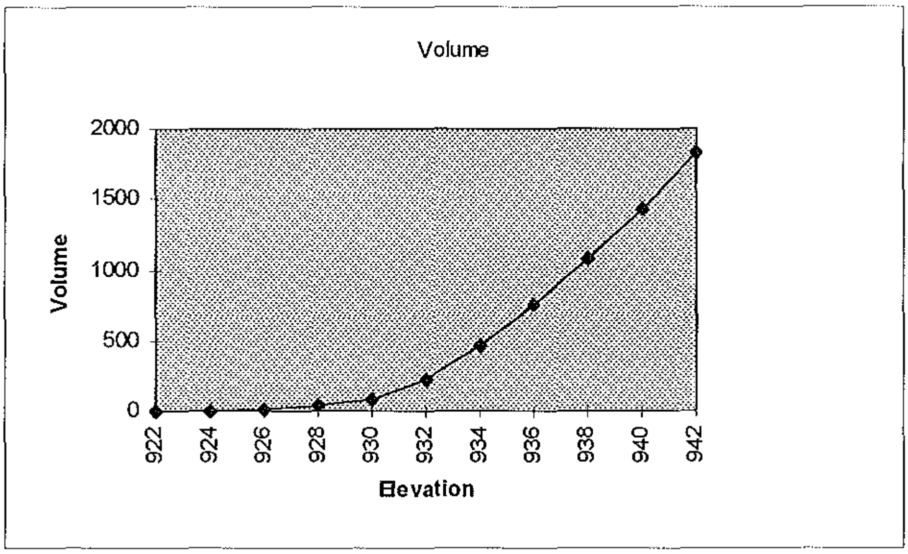


Figure 4.4 Elevation capacity curve of the reservoir

4.9 Hydrogeology

Based on field investigation of hydrogeologic properties such as infiltration capacity of the surficial materials, permeability of the rocks outcropping in the study area, degree of weathering and fracturing, topographic and morphologic positions, the lithostratigrapic units gneisses, gabbros, basalts and quaternary sediments are transformed into hydrostratigraphic units. Based on field hydrogeological observation and geologic mapping, lithostratigraphic units showing similar hydrogeological characteristics are grouped together. Thus, five broad hydrostratigraphic units can be mapped. These are:

Moderately to highly fractured and weathered gneiss

Moderately fractured, slightly weathered to fresh gneiss

Moderately to highly fractured and weathered gabbros

Moderately to highly fractured, slightly to moderately weathered basalt

Moderately fractured, slightly weathered to fresh basalt

Quaternary sediments

When comparison is made between the different lithologic units with in this hydrostratigraphic units, the gneisses and gabbros are relatively less permeable than the overlying basalts, fracture openings in the gneiss are mainly filled by quartz veins. Quartz veins are preferential circulation medium as they are intensively fractured.

The weathered and fractured basaltic lava flows lying over the basement rock act as a storage for groundwater and conveys infiltrating water from the land surface to deeper fracture systems in underlying hard rocks. The physical and chemical characteristics of the weathered layer and its potential to absorb infiltrating water and to store and yield it to wells and springs becomes a priority concern in the water economy of the area. The principal source of groundwater in the study area are fractured and weathered volcanic and basement rocks. The main controlling factors of fractured rocks to contain water are:

- Type, frequency and distribution of fractures
- The degree of the interconnection of the fractures
- The presence of cementing material and their hydraulic character
- The constitution of cover soil and the degree of pedogenesis
- The climatic factors

Quaternary sediments in the study area are composed of gravels, sands, silts and clays. They originated mainly from alluvial and colluvial, processes. This hydrostratigraphic unit occupy considerable area of the catchment at the low lying basin. Although detail investigation was not carried out, the possible areas for the groundwater discharge zone is the quaternary sediments.

As to the reservoir hydrogeology is concerned, the lithology are quaternary sediment and basalts. since depth to groundwater is below the reservoir bed level, the reservoir water could infiltrate in to zone of aeration, hence to decrease the reservoir water that recharges the ground water all the pervious lithological units mapped at the reservoir engineering geology should be blanketed with impervious layer.

4.10 Hydrochemistry

In this section, an attempt is made to provide chemical information of the waters of the study area and its surroundings. Comparison and preliminary interpretation is also made with data of lake, spring and boreholes water samples. Samples of water for hydrochemical analysis were collected from different water bodies, that is, lakes, bore holes, springs and rivers. Six samples were collected and analyzed. The samples area collected from lake Chamo, Holte borehole and Onota borehole from the northern part of the study area, out side the catchment area and Gato borehole, Gato spring and from Iyanda river.

The river sample was taken at the dam site. Spring sample was collected inside the catchment area at Gato Village , and Gato borehole is also collected at the same Village. There is surface water divide between these samples and samples taken at the northern part of the catchment. So the possible groundwater inflow-outflow conditions could be studied from the chemical data analysis of these samples. The sample was analyzed at water mine and Energy Berow of SNNPRG Awassa.

4.10.1 Water quality criteria

The usefulness of water for different purposes such as drinking, irrigation and industry is determined by its quality. One basic measure of water quality is TDS, which is the amount of solids dissolved in water. As the amount of TDS increase, the quality of the water for utilization for drinking and irrigation degrades, as it changes from fresh to saline in some cases to brine. In the study area all the water samples analyzed have a TDS less than 800 mg/l implying both the surface and groundwater to be fresh. Another water quality criteria is its hardness; which is a result of the presence of divalent metallic cations in which calcium and magnesium are the most important. The total hardness of water is given by (Todd, 1980):

$$H_T = 2.5Ca^{2+} + 4.1Mg^{2+},$$

where Ca^{2+} and Mg^{2+} are in units of meq/l.

A. Drinking water quality: The inorganic constituents that were analyzed for the different water samples conforms well with the national and international quality standards. The PH of the lake is above the Guide line value of the World Health Organization (1984). All the waters sampled in the study area and surroundings have a total hardness between 76 and 290 (Table 4.11) implying they are hard according to the hardness classification of Dufour and Becker cited in (Tenalem Ayenew, 1992). Fluoride concentration in waters of the study area is with in an acceptable limit, for the five samples with maximum of 1.03 mg/l and for lake Chamo 2.09mg/l. (Table 4.11)

Source	Na	K	Ca	Mg	Σcations	HCo_3+ CO_3	Cl	So_4	Σanions	Po_4	No_3 +No_2	F	PH	Ec
	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l	Meq/l		μs/cm
Gato(Sp.)	1.03	0.09	1.20	0.62	2.94	2.78	0.14	0.00	2.92	0.01	0.03	0.02	7.9	288
Gato(B.H.)	9.41	0.25	1.48	2.02	13.16	10.25	0.14	0.42	10.81	0.02	2.42	0.05	7.98	722
Onota(B.H)	8.88	0.13	2.94	2.72	14.67	7.98	0.11	0.80	8.89	0.01	0.04	0.04	8.18	869
Holte(B.H)	10.61	0.05	1.18	0.31	12.15	11.77	0.25	0.04	12.06	0.01	0.10	0.02	8.3	1313
L.Chamo	11.52	0.36	0.11	0.31	12.30	9.40	1.83	0.06	11.29	0.002	0.03	0.10	8.08	1557
Iyanda(Riv)	0.99	0.10	3.00	0.11	4.20	4.10	0.03	0.26	4.39	0.01	0.00	0.01	7.05	496

Table 4.11 Water chemistry of the study area

The plausibility of data has to be checked before the interpritation of the data. The error could be due to sampling process and reaction error. Reaction error is assumed to be caused by the analytical error of the individual parameters analyzed, in some cases if there is unmeasured ion in the sample this also contribute to the reaction error. In waters of moderate concentration, a reaction error between 1-2% is acceptable (Hem, 1970). Maser (1997) recommends an acceptable range of a reaction error between 2% to 5%. Water samples results reported from the laboratory are found to have less than 5 % reaction error and therefore accepted for further interpretation.

Plotting of the chemical analysis results of the water samples from the study area on a trilinear diagram using Plotchem program shows two major water groups Fig 4.3 based on the major ionic compositions .The groundwater samples show Na+K dominance, the percentage is more than 60 %. Sample of Iyanda river has 2.57 % Mg, 71.43% Ca ions and 26 % K and Na ions. On the other hand all the samples plot at the HCO_3^- and CO_3^{2-} apex showing the predominant anion which is more than 80%.

Based on the few samples analyzed, the groundwater and lake samples can be classified as Na+K-HCO_3 type On the triangular plots, the river and spring waters are Ca type . From the anion plot all the waters in the study area are HCO_3^- type. From the piper trilinear plot, the waters in the study area are classified in to two water types. The spring and river samples are calcium-Bicarbonate type and the lakes and borehole are sodium- potassium bicarbonate type. The chemical composition of different water bodies analyzed is given in Table 4.11. The piper trilinear plot in figure 4.5 shows that the different water bodies plot in a group with a good clustering. Those in one group therefore, is interpreted to have similar chemical history (origin).

B. Irrigation water quality: The suitability of water for irrigation depends on the effect of the chemical constituents of the water on both the plant and the soil. The most important characteristics of irrigation water are: total concentration of soluble salts, proportion of sodium to other cations, concentration of potentially toxic elements to plants, and bicarbonate concentrations as related to the concentration of calcium plus magnesium.

The total soluble salts are commonly indexed by the electrical conductivity and TDS of the water. The sodium hazard (when the concentration of sodium far more greater than the concentration of the other major cations affect the structure of the soil by reducing its permeability) is expressed by the sodium adsorption ratio (SAR)

$$SAR = \frac{Na^{+}}{\sqrt{\frac{(Ca^{++} + Mg^{++})}{2}}}$$

where the cations are in units of meq/l.

%Na = 100 x (Na + K)/ (Ca+Mg+Na+K)

Sample	SAR	EC (μS/cm)	HCO3 (mg/l)	% Na
LakeChamo	14.05	1557	85.4	96.6
Honte Borhole	12.25	1313	585.6	87.74
Onota Borhole	5.28	869	738.2	61.42
Gato Borhole	7.11	722	439.2	73.4
Spring (Gato)	1.08	288	73.2	38.1
Iyanda River	0.79	496	256.2	25.95

Table 4.12 Water quality for irrigation of representative samples in the study area.

Wilcox (1955) referred by Todd (1980) has classified quality of water for irrigation as shown in Table 4.13

Water class	% Na	EC (μS/cm)
Excellent	< 20	< 250
Good	20-40	250-750
Permissible	40-60	750-2000
Doubtful	60-80	2000-3000
Unsuitable	> 80	> 3000

Table 4.13 Wilcox water quality classification for irrigation

4.10.2 Description and implications of hydrochemical data

In the Iyanda catchment and its surroundings, the concentration of both bicarbonate and sodium are relatively high in the lake water and borehole but lower in the spring. The SAR value is within the high range of sodium hazard, and EC from medium to high salinity.

- Small differences in the TDS and the Ec values of Lack Chamo, Holte, Onota and Gato bore hole could be due to good subsurface hydraulic connections.
- The low Ec and TDS values of Gato spring and Iyanda river could be due to the effect of recharge of precipitation circulation at shallow depth
- The high Ec value of the area can be partly attributed to rock water interactions.

  The increase in Ec from the study area to Lack Chamo implies that residence time is likely to increase in the same direction.
- Generally, the chemical species analyzed shows that , the river and spring water bodies are suitable for irrigation.
- The surface water divide may not coincide with the ground water divide since the ground water of the catchment and the lake have the same origin.

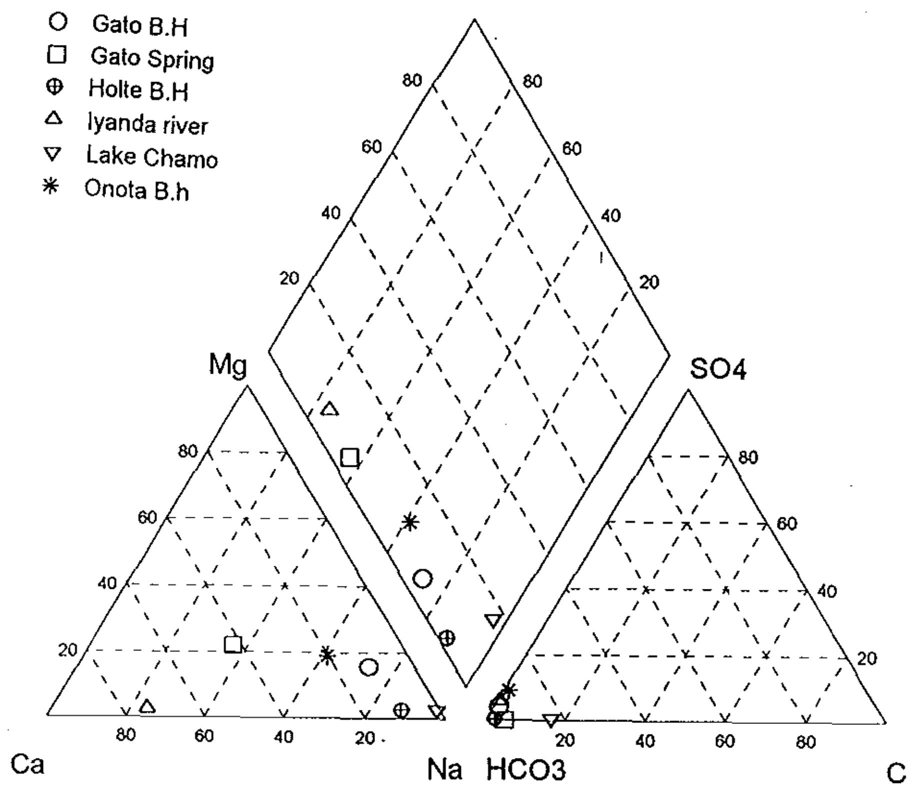


Figure 4.5 **Tri -Linear plot of water samples**

Chapter Five

Engineering Geological Mapping, Erodability and Siltation of the Catchment

5.1 Engineering geological mapping of the catchment area

The purpose of engineering geological mapping is to provide basic information for the planning and detail design work of an earth dam structure across Iyanda river. The engineering geological investigation, therefore, is mainly directed towards showing the geological environment, the active geodynamic processes and geological environment, the active geodynamic process and the prognosis of processes likely to result in the changes being made. The principal factors dealt with during engineering geological mapping of the area are, the rocks and soils, geomorphological conditions and geodynamic process. The engineering geological map is prepared in such away that to be easily understood by professional users who may not be geologists. Therefore, it is believed that soil/rock boundaries, condition of placement of soils which may give ideas for their engineering responses, rock characteristics and other pertinent features can be easily read from the map.

Overburden is used here as synonymous for soil materials. An area is mapped as soil unit if the thickness exceed 1m and if the soil thickness is found less than 1m, then the area is mapped as the underlying bed rock, (Soeters, 1985) . A thickness of 1m is chosen as a standard because it is possible to recognize the surface morphology of the underlying bed rock if its thickness is less than 1m and on the other hand the engineering geological importance of a soil cover when it is less than 1m is rather limited, (Soeters, 1985). Since engineering geological maps can be prepared on the basis of purpose content and scale, the prepared map is a particular purpose map of comprehensive type in content and falls into medium scale on the basis of scale classification. The mapped units are classified as soil unit and rock units. The soil units outlined during the mapping include:

- Deposits of river channel and its levees
- Alluvial deposits, and
- Residual soils

Samples taken from these units in the field are classified on the basis of unified soil classification as inorganic clays of high plasticity,fat clays(CH), inorganic silts,micacious or diatomaceous fine sandy or silty soils, elastic clays(MH) or organic clays of medium to high plasticity(OH), inorganic silts and very fine sands, silty or clayey fine sands with slight plasticity(ML) or organic silts and organic silty clays of low plasticity(OL), and organic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays(CL) from test pits data of TPI10, 11, 12, and 13 respectively. Except TPI10 in which the possible clays are of illite type the other three pits results could be of kaolinite type. Based on modified triangular classification system TPI12 is sandy loam and the other three are clay loam. The sandy loam is the zone of medium to high permeability with low erodability potential, it can be classified in to residual type. While the clay loam zone is areas of low to medium permeability with a consistency grade of stiff clay. Deposits of river channel is mainly composed of coarse grained clastics(gravel and sand) with some impurities. the deposits are mainly due to the change of slope and the decreasing tendency of discharge.

The engineering geological mapping of rock units is done based on the assumption that the resistance to erosion of rock reflects its rock mass strength, (ISRM, 1981). Based on the interpretation of the steepness of eroded slopes, development of drainage patterns, drainage intensity, forms of slopes, surface raggedness, degree of weathering and fracturing, the study area can be classified in to five zones where rock mass strength can be considered to be homogeneous, these are:

Zone 1 - Moderately fractured, slightly weathered to fresh basalt

Zone2 - Moderately to highly fractured, slightly to moderately weathered basalt

Zone 3 - Moderately fractured, slightly weathered to fresh gneiss

Zone 4 - Moderately to highly fractured and weathered gneiss

Zone 5 - Moderately to highly fractured and weathered gabbros

Based on strength classification zone 1 and zone 2 are very strong to extremely strong rocks and zone 3, 4 and 5 are strong to very strong rocks fig 5.1. The approximate range of unconfined compressive strength of extremely strong, very strong and strong rocks used for this classification is: > 250, 100 - 250, and 50 - 100Mpa respectively. The mean or typical values which can be taken as approximate guide lines for comparison of the two dominating rock units in the area are given in Table (5.1) (Robin, etal., 1992)

Rock unit	dry density t/m^3	porosity (%)	dry UCS mean Mpa	Modulus of elasticity Gpa	Shear strength Mpa	friction angle (ϕ)
Basalt	2.9	2	250	90	40	50
Gneiss	2.7	1	150	45	30	30

Mpa = Mega Pascal; Gpa = Giga Pascal; UCS = unconfined compressive strength.
Table 5.1 Typical engineering values of basalt and gneiss

From evaluation of all engineering parameters, strength of basalt is grater than gneiss, though degree of fracturing and weathering are an influential factors. The strength of gabbros is in between the two rock units.

The possible sites of erosion are mainly due to steep slope, agriculture and poor vegetation conditions and the suggested land slide and toppling areas are due to steep slope condition, high rain fall and poor land management system.

5.2 Erodability and siltation of the catchment

General landscape, unique topography, heavy deforestation, intensive cultivation and population pressure, have resulted, in the watershed area, sever soil erosion and land degradation. Unplanned destruction of forests has brought serious change both in land and water infiltration system. The washing off fine soil particles from mountainous and hill side terrain's of deforested areas and cultivated land, has caused sever soil erosion, high run off and silt deposition in the river courses and downstream of the valley. Though both water and wind can bring considerable erosion of soil, the powerful factor on this catchment is water, The extent of soil erosion varies and depends on such factors as slope, soil type, soil depth, vegetation, organic matter content of soil, cropping pattern, duration and amount(intensity) of rainfall, wind and others (Schwab, etal.,1993). The study area is characterized by dissected mountains undulating hilly terrain subjected to high cultivation. The farmers apply extensive terracing of soil and stone bounds inorder to conserve soil and retain water.

To observe the effect of soil loss(siltation) on the dam site; experimentally conducted soil loses at different conditions in Areka is seen in depth. Data transferring was conducted depending on similarities in precipitation, erosivity, land use, soil type and slope. Based on these similarities the Iyanda catchment has soil lose of 62 t/ha at slope of 16% and grass cover, and about 11 t/ha in grass and / or sorghum plot (Areka data base, 1994). The Iyanda catchment is characterized by poor conservation system comparing to the other part of Konso. Specially the area between Kayle and Taho is characterized by high silt load due to poor conservation and lithologies combined with rugged topographic nature. The useful life of a proposed dam depends on rate of sedimentation. This is related to the number of years that is taken by the sediment from a given catchment condition to fill the total height of the dam under no treatment or prevention measure being made. Generally it is recommended to calculate the silt load and treat the catchment before the construction of the dam. Normally three types of information are useful for predicting siltation: Triputhi, etal, (1993)

I, Total silt reaching the reservoirs in normal years

II, The time distribution of this total and

III, Grain size distributions and their variations with time.

It is interesting to make an exact integration of concentration measurements with the corresponding average discharge rates but due to the lack of measured discharge rate and shortage of time, detail sediment load analysis was not carried out. Although reservoir sedimentation is inevitable it was tried to retard by selecting a site where sediment inflow is relatively low and the storage capacity large enough for a useful life.

A good vegetative cover over the catchment area, is the best way of reducing sedimentation in the study area. However, the success of catchment protection depends both on climate and land management. Since sediment trap is principally designed to catch coarse sediments and the sediment at the reservoir area is of suspended type, there is no need of constructing sediment trap rather diversion of sediment laden water around a reservoir and also to diminish reservoir sedimentation, discharging water through sluices in the dam can partially reduce sediment accumulation.

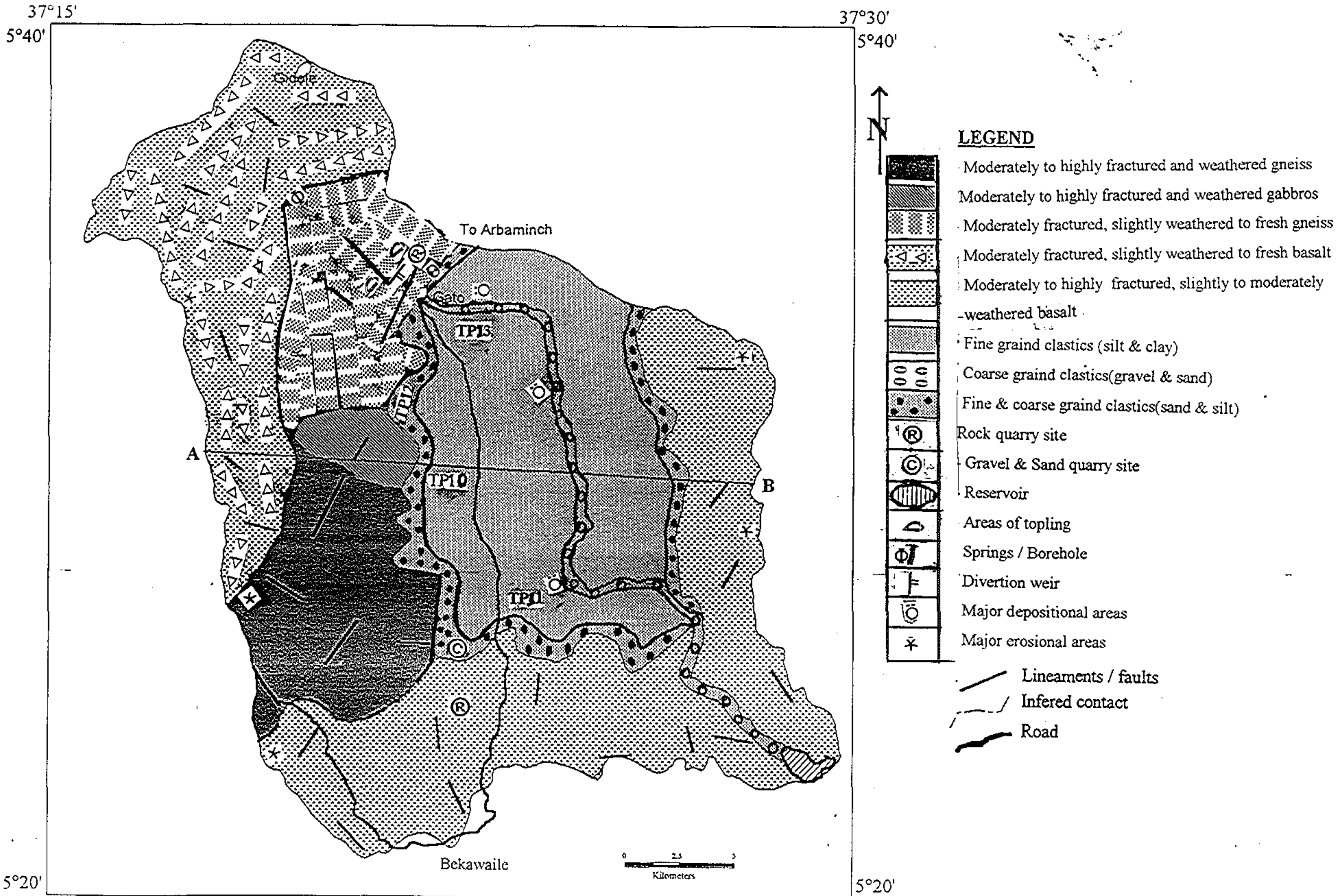


Figure 5.1 Engineering geological map of the Iyanda catchment

Chapter Six

Engineering Geological Aspect of Embankment Dams

6.1 Site Selection Criteria's:

According to Krynine and Judd (1957) some of the dam site selection criteria's are:

- The rock foundation should be approximately of one geologic classification, to avoid variations in the value of the modules of elasticity.
- The foundation and the reservoir sites should be watertight.
- The valley slopes should be stable when the reservoir is full, this is also required for the abutments.
- The reservoir's drainage area, including rocks and overburden should be resistant to erosion and therefore not likely to contribute heavy silt loads to the reservoir that its useful life span is not severely affected.
- Geologic and topographic conditions should permit a favorable location of spillway and diversion tunnels and outlet works
- The location of construction materials should be with in an economically justified distance from the project site
- The final selection of the dam site is based on the comparative analysis of all data mentioned above the criteria for comparison being cost and availability.

6.2 General Earth Dam Failures

The recorded failures to earth dams range from complete catastrophic failure, resulting in large property damage and loss of life to relatively minor damage requiring some remedial work and maintenance. According to middle brooks the most common cause of complete catastrophic failure has been due to water flowing over the tops of earth dams due to great river floods when the spillway capacities are inadequate. The other instability of slope in the upstream and downstream positions of the embankment leading to earth dam that could have been taken care of by suitable methods of slope stability analysis. The other principal cause of damage that may lead to catastrophic failures (if they are not properly attended to in time) are piping, sloughing and cracking.

6.2.1 Piping Failure

Piping phenomenon may be described as the progressive internal erosion that occurs within water percolating through earth dams and their foundations picks up soil particles and moves them through unprotected exit developing unseen channels or pipes through dam or foundation. The erosive forces generated by the seeping water are resisted by the soil particles. If the forces resisting erosion are less than those which tend to cause it, the soil particles are washed away and piping starts. The resisting forces depend on the cohesion, the interlocking effect and the weight of the soil particles (Shearard etal.., 1963) cited in design of embankment dam. Because the embankment and the foundation are not uniform seepage media, the flow is also not uniform. Though the total seepage quantity may be small the erosive forces on the soil particles are greatest where the concentrated flow emerges on the downstream side of the dam.

Most of the failures due to piping have resulted from progressive backward internal erosion of concentrated leaks which develop through or under the dam. The erosion starts at the point where seepage water discharges and works toward the reservoir gradually enlarging the seepage channel until complete failure occurs. Piping can also occur in a zoned dam where the seepage water discharges from the finer material of the core into the coarser adjoining pervious zone (Arora, 1997). As a result of poor construction control, the concentrated leaks that occurred between the embankment and foundation or embankment and abutment and also adjacent to concrete outlet pipes have lead to piping failures. When the foundation soil is silty sand and the upward flow of seepage water is strong enough to carry sand particles, quick sand condition develops over the foundation area on the downstream, leading to a series of sand boils .

If these are not observed and properly attended to, they may lead to complete failure of the dam through piping. They can be satisfactorily treated by covering the ground surface with a blanket of progressively coarser gravel graded from coarse sand at the bottom to pea size gravel at the top. It has been concluded from study of 31 dams that, the piping resistance is greatly influenced by the soil properties (such as the plasticity index greater than 15%) rather than the method by which the embankment is compacted (Murthy, 1997). Further the piping can be effectively controlled by the use of the graded filters in such a manner that the eroded soil can not be washed away.

6.2.2 Sloughing Failure

It is a progressive damage closely related to piping. The process begins when a small amount of material at the downstream toe erodes and produces a small slump or miniature slide. It leaves a relatively steep face, which becomes saturated by seepage from the reservoir and slumps again, forming a slightly higher and more unstable face. This progressive backward sloughing or raveling process can continue until the remaining portion of the dam is too thin to withstand the water pressure and complete failure occurs suddenly as the reservoir breaks through. This type of failure is possible when the whole downstream portion of the dam has been saturated.

6.2.3 Cracking in Earth Dams

The danger of cracking has not been widely publicized or understood by earth dam engineers. There are two reasons why it has not been given more attention. First, there is always an understandable desire on the part of both the professionals and the owner to conceal serious defects which develop in a dam structure. While a major slope slide is obvious even to casual observer, an open crack can not be discovered except by very close inspection. So it is rather easy to hide even serious embankment cracks without getting publicity. The second reason is that the true cause of failure often has not been identified, through the failures in many cases could be attributed to piping that started in embankment cracks. Because of geometry and relative compressibility of the foundation abutment and embankment, earth dams are deformed by differential settlement. As a result, certain portion of embankment are subjected to tensile strains producing differential cracking patterns.

They may be either localized or continuous for great distances passing through impervious core. The most dangerous cracks are those which run transversely creating a path for concentrated seepage through the core. They are caused by differential settlement between adjacent length of the embankment, usually between the portion located at the abutment and the portion in the center of the valley. The worst cracking develops when the foundation under the higher portions is compressible and when the abutments consist of steep and relatively incompressible rock.

6.2.4 Other Major Failures

Based on the published reports (Murthy 1997), earth dam failures and damages may be classified broadly into three categories namely:-

I) hydraulic Failures

II) structural Failures and

III) seepage Failures

The causes and the general characteristics of the failures are best analyzed in the form of tabular statement as shown below. Depending on the nature of failure, various preventive or remedial measures which could be undertaken by the practicing professionals are also suggested.

Type	**General Characteristics**	**Causes**	**Preventive or Corrective Measure**
Over topping	Flow over embankment washing out dam	Inadequate spillway capacity Clogging of spillway with debris Malfunction of spillway gates	Spillway designed for maximum flood Maintenance Maintenance
Wave Erosion	Notching of upstream face by waves, currents	Lack of pitching (riprap),too small stones	Properly designed pitching (riprap) or pavement
Toe Erosion	Erosion of toe by outlet discharge	Spillway too close to dam	Training walls
Gulling	Rainfall erosion of dam face	Poor surface drainage	Surface drains, Good Maintenance

Table 6.1. Hydraulic Failures (P.S.D.E.D, 1990)

Type	General Characteristics	Causes	Preventive or Corrective Measure
Foundation	Sliding of entire dam, one face, or both faces in opposite direction with bulging of the foundation in the direction of the movement	Soft or weak Excess water pressure in confined sand or silt seams	Flatten slope; employ wide berms, remove weak materials, stabilize soil Drainage by deep drain trenches with protective filters; relief wells
Slope slide	Slide in either face of dam with little or no bulging in same, during construction	Steep slope Weak Soil Pore pressure due to excessive moisture	Flatten slope, or employ berm Increased compaction, better soil Reduce moisture below optimum
Slope slide, upstream	Rapid slide following draw down	Pore pressure due to sudden draw down	Free drainage shell, flatten slope
Slope slide downstream	Well defined shear slide bulging up in lower toe, may not extend up to crest	Loss of soil strength from seepage or rain saturation	Proper internal drainage with filter, toe drain, good surface protection with grass
Flow Slide	Collapse and flow of soil in either upstream or down stream direction	Loose foundation of embankment, soil of low cohesion, triggered by shock vibration, seepage or foundation movement	Compaction to at least 70% relative density

Table 6.2 Structural Failures (P.S.D.E.D, 1990)

Type	General Characteristics	Causes	Preventive or Corrective Measure
Loss of Water	Excessive loss of water from reservoir and/or occasionally increased ground water levels	Pervious reservoir rim or bottom	Blanket reservoir with compacted clay or chemical admix; grout seams cavities
		Pervious dam foundation	Use foundation cutoff; grout; upstream blanket
		Pervious dam	Impervious core
		Leaking conduits	Watertight joints; grouting
		Settlement cracks in dam	Remove compressible foundation, avoid sharp changes in abutment slope, compact soils at moisture above optimum
		Shrinkage Cracks in dam	Use low plasticity clay for core, adequate compaction
Seepage Erosion or piping	Small boils in lower face of dam or in foundation below dam, leading to progressive internal erosion of soil from downstream side of dam or foundation backward toward the upstream side to form an open conduit or "pipe". Often leads to a washout of a section of the dam	Settlement cracks in dam	Remove compressible foundation; avoid sharp changes; internal drainage with protective filters
		Shrinkage cracks in dam	Low plasticity soil; adequate compaction, internal drainage with protective filters
		Pervious seams in foundation	Foundation relief with filter; cutoff
		Pervious seams, roots etc. in dams	Construction control of core; internal drainage with protective filters
		Concentration of seepage at face	Toe drain; internal drainage with filter
		Boundary seepage along conduits, walls	Stub cutoff walls and collars, good soil compaction. No overhanging faces
		Leaking conduits	Watertight joints; water stops; durable materials, good quality concrete
		Animal burrows	Wire mesh, or paving

Table 6.3 Seepage Failures (P.S.D.E.D, 1990)

6.3 Probable failures in the study area and proposed mitigation measures

The foundation of the study area is soils that varies from clayey silt to gravely sand. Due to a permeable soil foundation to a depth of 13 m at the river center, leakage will be the main problem if dam is to be constructed on this foundation, hence it is recommended if slurry trench(or other type of cutoffs) extended to impervious base, which stops seepage and seepage exit gradients or upstream impervious blanket along the main river coarse and its levees which increase the seepage path, reduces seepage and seepage exit gradients. The study area is also characterized by deep alluvium that may subject the dam to large amount of settlement, leading to differential movement and cracking, so it is important to provide good filters to control seepage and prevent internal erosion. Since the catchment area is very large and the reservoir capacity is limited to about 16 million m^3, flow over the embankment may happen , so spillway should be designed for maximum flood. The site is also located in high risk seismic zone, hence, the dam has to be designed considering this danger.

6.4 Engineering geological mapping of dam sites

After selection of appropriate dam sites and conducting of insitu test together with visual interpretation engineering geological mapping of the reservoir area was conducted aimed at ensuring engineering question. This map shows the following information's:

The ground surface contour, geomorphic features such as slope changes, geological surface features (areas of rock outcrops cobbles and soils), features of insitu rocks (types and their boundaries) indicating position of test pits and geophysical traverse lines. It also incorporates the location of possible sources of natural construction materials.

The main aim of investigation in the reservoir area is to get information directly or indirectly about the water tightness of the material which cover the area and also to investigate the existence of buried channel or valley or any linear geological structures which allow permanent leakage under the dam foundation or to the adjacent low laying valley. From this point of view, the recent river deposits along the main river are obviously pervious material. Three main geological units have been mapped in the reservoir area. These are:

- Alluvial deposits in the flat area of the reservoir
- Well graded soils at the right rim of the reservoir
- Basaltic bed rock at the left rim of the reservoir

In this case alluvial soils will include soils which have been deposited in the channels and flood plains of rivers. These soils are characterized by variability, both vertically and laterally, and can range from clays of medium plasticity to course sands, gravels and boulders. The main type of alluvial deposits mapped at the reservoir are:

Lag deposits: This is mainly gravel, and occur along the base of the river channel, and are moved during peak flood times. They are uniform sized and have high void ratio and permeability, at the bases of abandoned or buried channels, their voids are choked by sand and fines.

Point bar deposits: These are sand and gravel unit which are deposited on the inside of bends in the Iyanda river, they occur above the lag deposits along the channel, with their upper surfaces in the form of migrating dunes. The cross bedding seen in the bar deposits results from preservation of some of these dunes (TPI3 depth 3 m). This point bar deposit is coarser at depth and finer towards the top.

Levees of fine sands and silts: these units are observed and formed along the top of the river banks where the coarser materials are deposited more quickly than the fine silts and clays, when flood waters over top the bank, it is mainly observed at the right bank of the Iyanda river.
Flood plain deposits: These are fine silts and clays, deposited in thin horizontal layers during floods. This is mainly observed in most part of the study area at the flat right side of the river.

Well graded soils at the right rim of the reservoir: this unit have good bearing capacity with a permeability coefficient of about 2.5×10^{-7}cm/s from Hazen's Empirical formula of (TPI7) at 1.5m depth.

Basaltic bed rock at the left rim of the reservoir: this unit is slightly weathered to fresh, moderately to slightly fractured aphanatic basalt. The engineering properties of this unit is similar to the basalts of the right foundation Fig 6.1

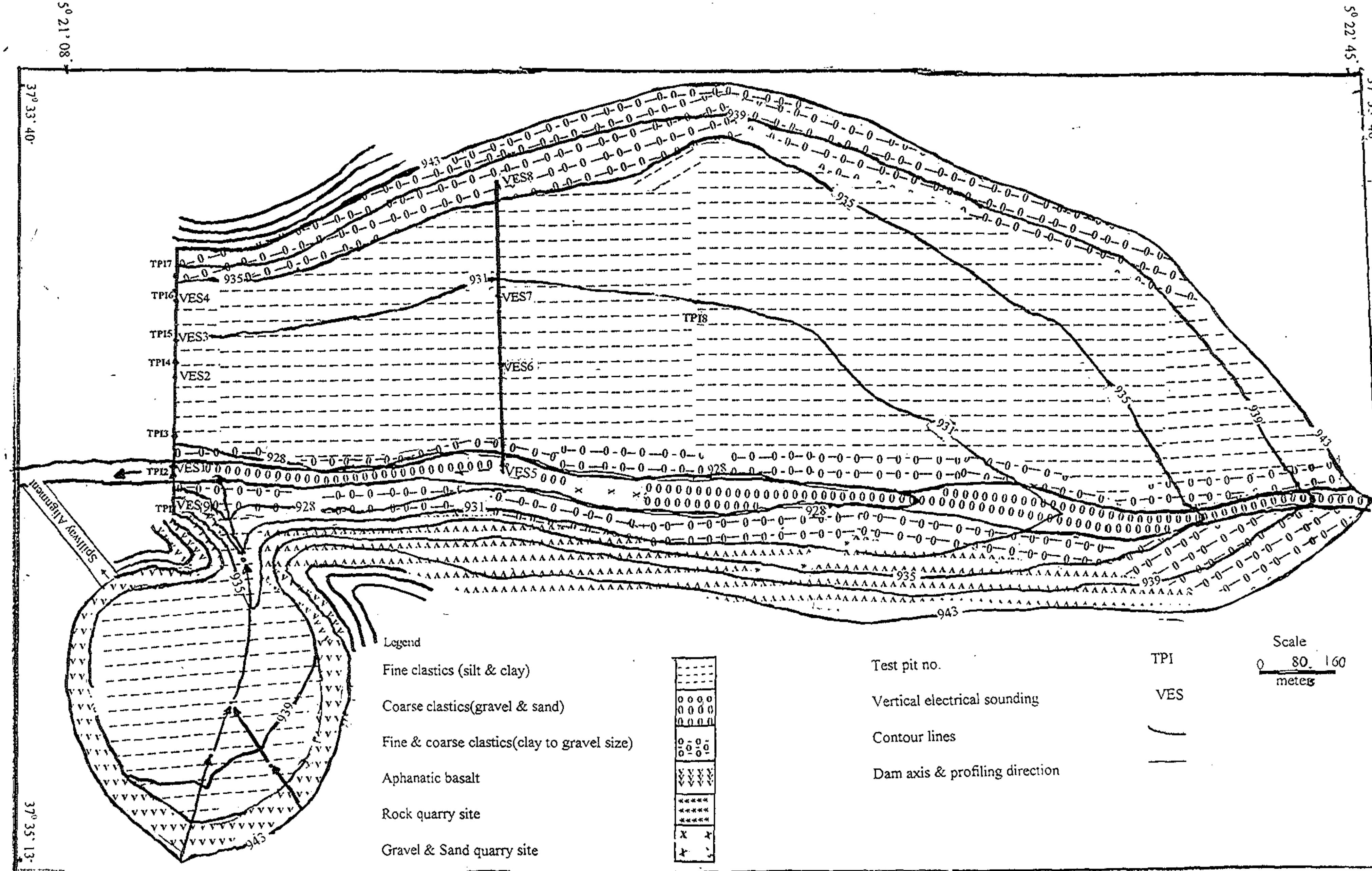


Figure 6.1 Engineering geological map of the reservoir

6.5 Foundation and Abutment Investigation

All structures including earth dams need good foundations but the foundation for earth dams can be adjusted if not good. Therefore the geotechnical examination is essential for ascertaining the nature, type and character of the rocks and soils occurring in the foundation as well as the abutments and for assessing the competence of the formations. The foundation investigation for embankment dams must assess and test the strength, permeability and compressibility of the materials, in addition to these parameters, the stability of the abutments both during dry and saturated condition should be studied while studding a soil foundation material, undisturbed samples was studied for shear strength, permeability and compressibility. Generally a soil foundation will have relatively low strength, which may determine the embankment stability, such a foundation is likely to favor construction of earth fill dams with flatter slope. Permeable soil foundation will be susceptible to leakage and erosion requiring construction of some form of cut-off.

From thorough surface investigation and the information obtained in seven test pits along the dam axis it will be convenient to divide the foundation in to three physiographical divisions:

I, Right abutment

II, Existing river course

III, Left abutment

I, Right abutment

From the topographic map the right abutment has 490m horizontal distance and a vertical elevation difference of 16m, which give a general slope about 1.87^0 (3.26%). The general slope from the top of the river bank to the base of the hill is 1.1^0 and the slope of the hill is 6.52^0.

This slope variation to a certain extent are the reflection of the internal geology or indicate the strength of the underlying material. One test pit has been excavated at surface elevation of 939m(this is where the slope is relatively high) at this site it was possible to excavate up to 5m, as can be seen from test pit logging (TPI7) it is well graded soil at the surface and becomes clayey silt with depth. The geophysical survey data (VES8) shows bed rock symptom at depth of about 35m fig 8.3. From four test pits excavated at the flat plain areas the lithology is dominantly fine grained soils with some intercalation's of fine sand (TPI 3,4,5 and 6). This sides of the abutment is impervious at the surface, and at depth some pervious units intercalating between them was commonly observed. Figure 6.2 shows the geologic cross section along the dam axis. From test pit logging and field classification of soil of the foundation material the area of possible foundation depth is on clayey silt unit and for different engineering and index properties representative sample TPI4 at 2m depth is analyzed.

The basic requirements of designing footings on clayey silt are that the design should be safe against shear failure and the amount of settlement should be tolerable, and checking the amount of maximum settlement by consolidation test is very important. All the required parameters for calculating the ultimate bearing capacity for clayey silt soils of the foundation is given in consolidation and shear tests.

II, Existing river course

The width of the existing river course at the selected dam axis reaches about 100m. As any one expects the river course is filled with recent river deposit which includes rounded to sub rounded cobbles at depth. Erosion activity is clearly seen at the right bank and deposition at down stream. Excavation was conducted to a depth of 6m (TPI2). There is no major lithological variation up to the excavated depth, it is coarse clastic soils composed of sand and gravel with some amount of cobbles and little boulders at depth. From the observation of (VES1) bed rock is expected to be found at 13m. The results of vertical electrical sounding is discussed in chapter 8. This lithological unit has a permeability coefficient of 4 x 10^{-2} cm/s

The simplest and easiest method in evaluating the bearing capacity of footings founded on granular soil is by correlating with the penetration resistance value. The dry density of this sand unit is 18.6 KN/m^3 = 118 Pcf. From correlation's between the angle of internal friction and the dry density the friction angle (ϕ) is grater than 35 (After U.S Navy, 1982).

The standard penetration test can be correlated from angle of internal friction and relative density, on this correlation the average standard penetration of the study sample TPI1 is 20 blows/foot (Terzagi and Peck, 1948). From Peck's bearing capacity versus penetration resistance equation, the allowable bearing capacity of sands of the river course is:

q_d = (N/5) x 2000Psf = (20/5) x 2000Psf = 8000Psf

Where q_d = The allowable bearing capacity in pounds per square foot

N = Penetration resistance in blows per foot.

III, Left abutment

The left side of the river is steeper than the right side with slope of 3^0 at the flat area and 68^0 at the hilly area. From surface observation and TPI1 the sloppy abutment is composed of slightly to moderately fractured and slightly weathered to fresh basaltic rock. The basalts of the study area is slightly fractured to fresh that can be chiseled in two to three number of strokes, hence the typical engineering properties can be correlated from standard tables and given as follows for the studied basaltic rock type. Considering degree of weathering, fracturing and strength of the rock by hammer, the studied basalt has the following engineering properties.

dry density t/m^3	porosity (%)	dry UCS range Mpa	dry UCS mean Mpa	Modulus of elasticity Gpa	Shear strength Mpa	friction angle (ϕ)
2.9	2	100 - 350	250	90	40	50
Mpa = Mega Pascal; Gpa = Giga Pascal; UCS = unconfined compressive strength.						

Table 6.4 Engineering properties of basaltic rock at the left abutment

Since rock failure is most of the time in shear, analysis of shear strength is crucial and can be evaluated by General relationship:

UCS = 2Ss (tan45 + ϕ / 2)

Ss varies UCS/6 in strong rock to UCS/2 in soft clay.

Where Ss = shear strength

ϕ = angle of internal friction

UCS = Unconfined Compressive Strength

Safe bearing pressures can be obtained from guideline values for maximum loads that may safely be imposed on undisturbed ground. It may be estimated in many ways, all based on past experience and incorporating ample safety factors to allow for variable ground conditions. Values are useful preliminary design guides, as it is normally uneconomical to complete meaningful field tests on fractured rock masses. From field level investigation of the foundation rock the safe bearing pressures of the studied area is 10Mpa (Robin, et.al., 1992).

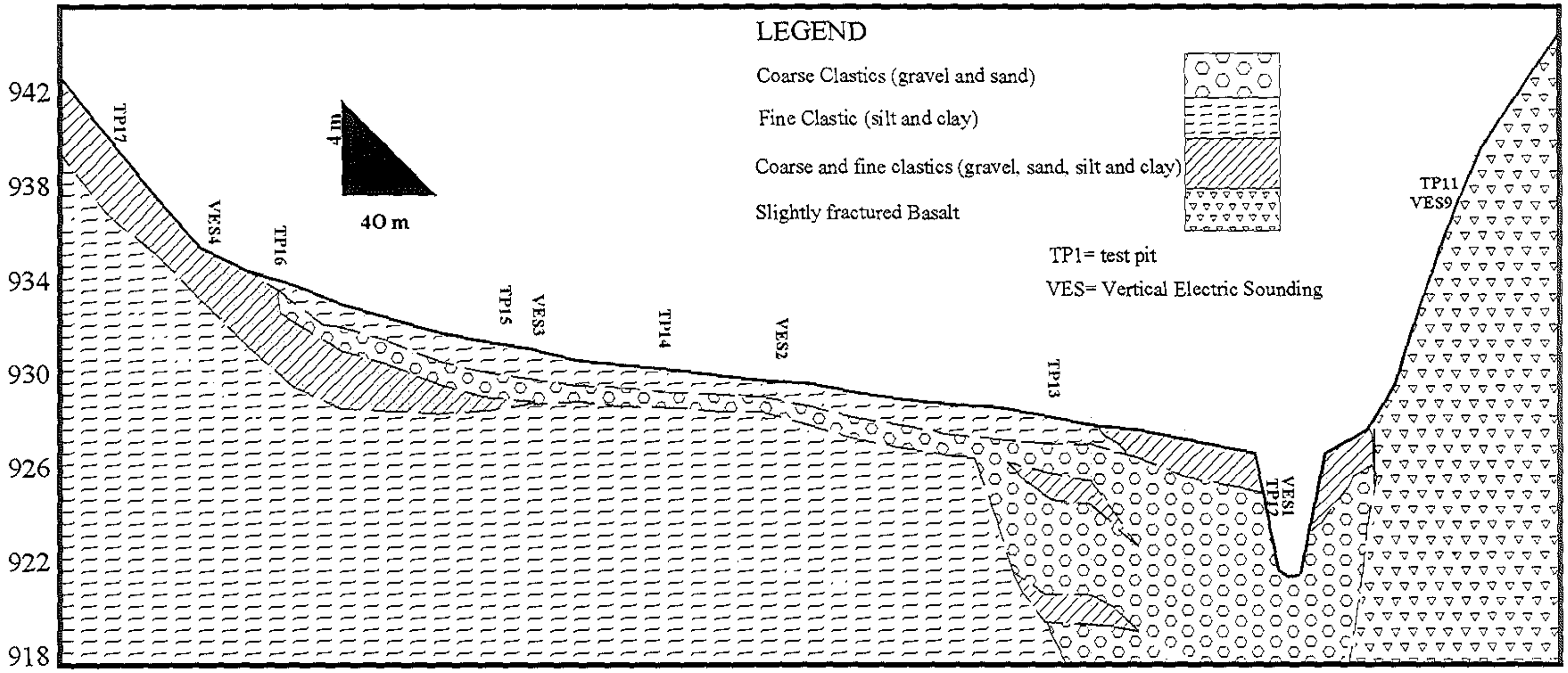


Fig. 6.2. Geological Profile Along IYANDA Dam Axis

6.6. Dam Appurtenant Structures

In any dam study equal emphasis should be given to the possible location, route and type of dam appurtenant structures to be used in the project. This structures include spillways, outlet works and diversion canals.

6.6.1 Spillway

A spillway is a structure which allows excess water above the maximum storage level of the reservoir to flow down stream without causing damage either to the dam or its foundations or to the sides of the reservoir. There are different types of spillways such as side-channel spillway, glory-hole spillway, saddle spillway and so on.

The foundation condition of spillway and erodability of the study area was investigated in deep, side channel spillway located at the left side is safe and economical for the proposed project. The lithology of the foundation varies with depth in which the depth to bed rock is about 4m, with different degree of fracturing and weathering. The longitudinal distance of the spillway is not more than 350m. The lithological unit overlying the rock is well graded gravel, sand, silt and clay composition. Since topographic map was not ready, detail cross section with profile was not carried out.

6.6.2 Outlet Works and Canals

The outlet works are used in irrigation dams to divert some of the reservoir water into irrigation canals, municipal water supply distribution systems and hydroelectric plants for power generation and for other uses.

The foundation of the main canal is important from permeability point of view, little water arrives the command area if the lithology of the main canal is highly pervious, the foundation of the main canal of the study area is mainly clayey silt soil which is characterized by low permeability with K values of about 0.7×10^{-7} cm/s. There is no major lithological variation along the main canal alignment.

Chapter Seven

Analysis and Interpretation of Soil Laboratory Results

All the available information for the study area was collected for the planning the location of test pits and then description, classification and correlation of each stratum was carried out. Out of different tests pits in the study area the samples analyzed are:

1. Seven samples for Atterberg limits (three for foundation, and the remaining for catchment)
2. Eight samples for grain size analysis (four for foundation three for catchment and one for filter material)
3. One sample each for the analysis of direct shear, unconfined compressive strength and consolidation
4. One sample for permeability and standard compaction test (for core material)

Soil laboratory investigation was carried out only for representative samples and at areas of interest. So to reduce redundancy and to be safe both technically and economically, correlation of test pits using visual interpretation and pocket Penetrometer was carried out on spot and attention was given to units of water tight. The soil are classified to index properties and engineering properties

7.1 Index properties

The test required for determination of engineering properties are generally complex. Sometimes the geotechnical engineer is interested to have some rough assessment of the engineering properties without conducting detailed tests. This is possible, if the index properties are determined. The properties of soil of which are not primary interest to the geotechnical engineer but which are indicative of the engineering properties are called Index properties (Arora, 1997). Simple test which are carried to determine the index properties are known as classification test. The soils are classified and identified based on the index properties. The main index properties of coarse grained soils are particle size and relative density. For fine grained soils, the main index properties are Atterberg limits and consistency indexes, (Arora, 1997)

The indexes properties are divided into two categories: (1) properties of individual particles and (2) properties of the soil mass, also known as aggregate properties. The properties of individual grains can be determined from a remolded or disturbed samples. These depends upon the individual grains and are independent on the manner of soil formation while the soil aggregate property depends upon the mode of soil formation, soil history and soil structure. These properties should be determined from undisturbed samples or preferably from in-situ tests. They have greater significance in engineering construction, since engineering structures are founded on undisturbed, natural soil deposits.

The index properties give some information about the engineering properties. It is assumed that soils with like index properties have identical engineering properties. However, the correlation between index properties and engineering properties is not perfect. Design of large important structure should be done only after determination of engineering properties.

The index properties of soils of the study area are discussed below.

7.1.1 Gradation

The distribution of different sizes of representative soils of the study area is given in particle size distribution curve. Grain size analysis was carried out for eight samples. From the drawn curve the relative amount of each type of soil constituting the sample was computed. The grain size analysis results is shown in Table(7.1). The size range of each group was adopted from ASTM(less than 0.002mm is clay; 0.075-0.002mm is silt; 4.75-0.075mm is sand and >4.75mm is gravel). Based on this classification silt size materials predominate in most of the samples.

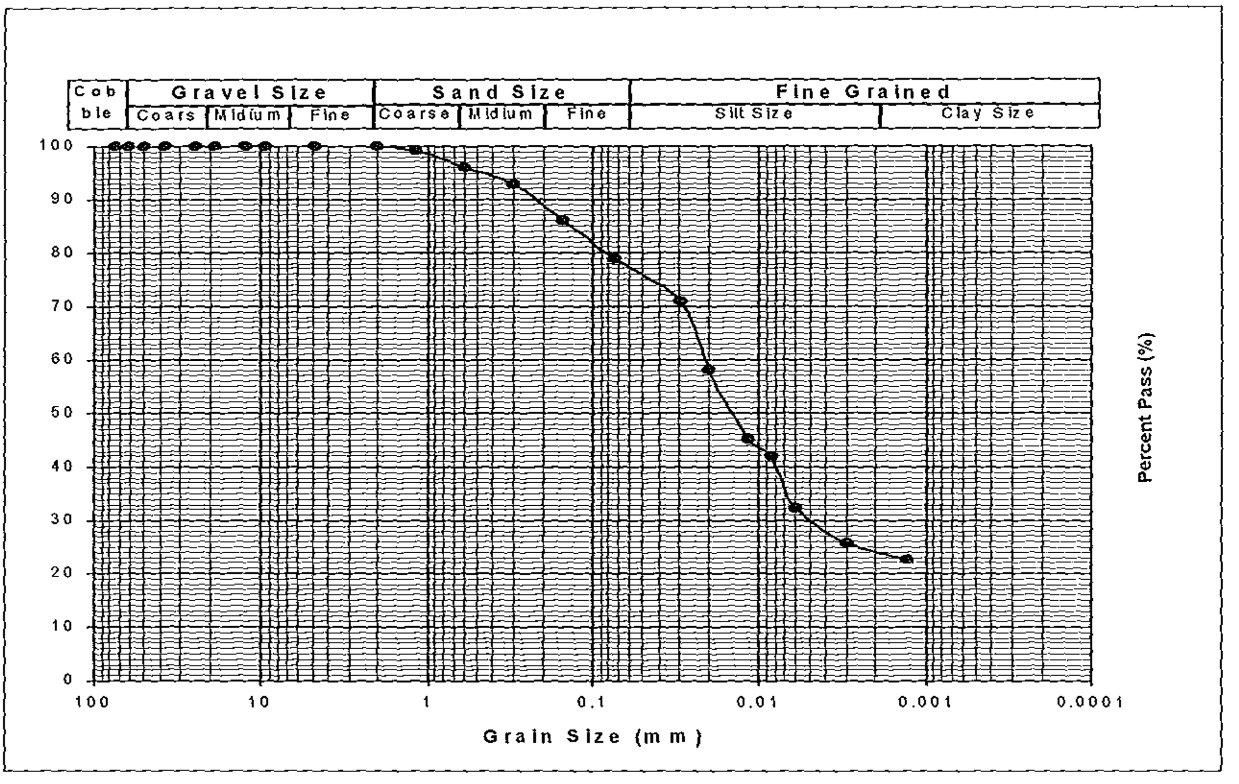
Sample	Depth (m)	Lithologic type	Specific gravity	Gravel	Sand	Silt	Clay
				%			
TPI4	2.0	Sandy Clayey Silt	2.62		21.03	54.96	24
TPI7	1.5	Silty Sandy Clay	2.41	16	28	26	30
TPI7	3 - 4	Clayey Silt	2.62	8.12		75.88	16
TPI8	0.5	Silty Clay	2.56		7.14	45.86	47
TPI11	0.5	Sandy Clayey Silt	2.51	3.16	19.55	45.29	32
TPI12	0.5	Silty Sand	2.60	5.73	59.6	22.67	12
TPI13	0.5	Clayey Sandy Silt	2.68		34	37	29
Filter Sand	0.5	Gravely Sand	2.72	34.6	60.4	3	2

Table(7.1) Specific gravity and granulometric composition of soils of the study area.

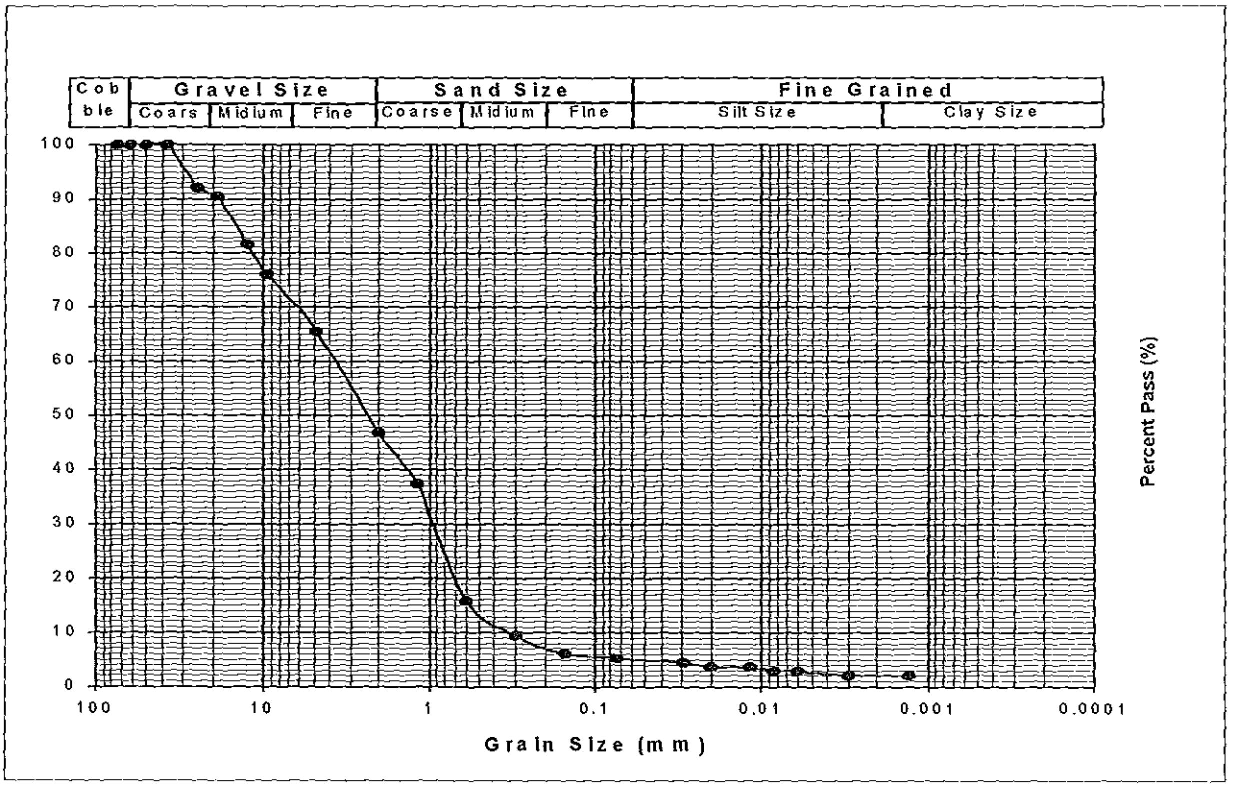
The grain size distribution of soils of the study area are given below.

Figure 7.1 Grain Size Distribution of Soils of The Study Area

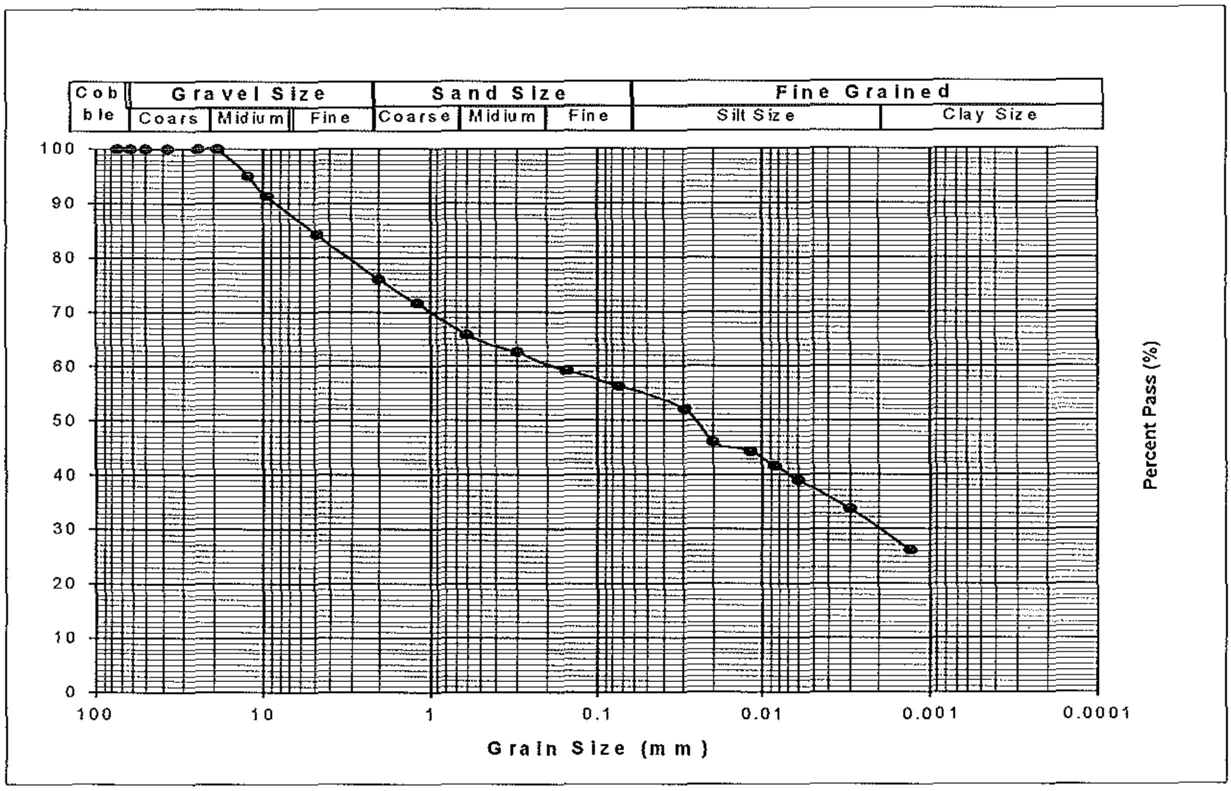
TPI4: Sandy clayey silt **Depth: 2.0m**



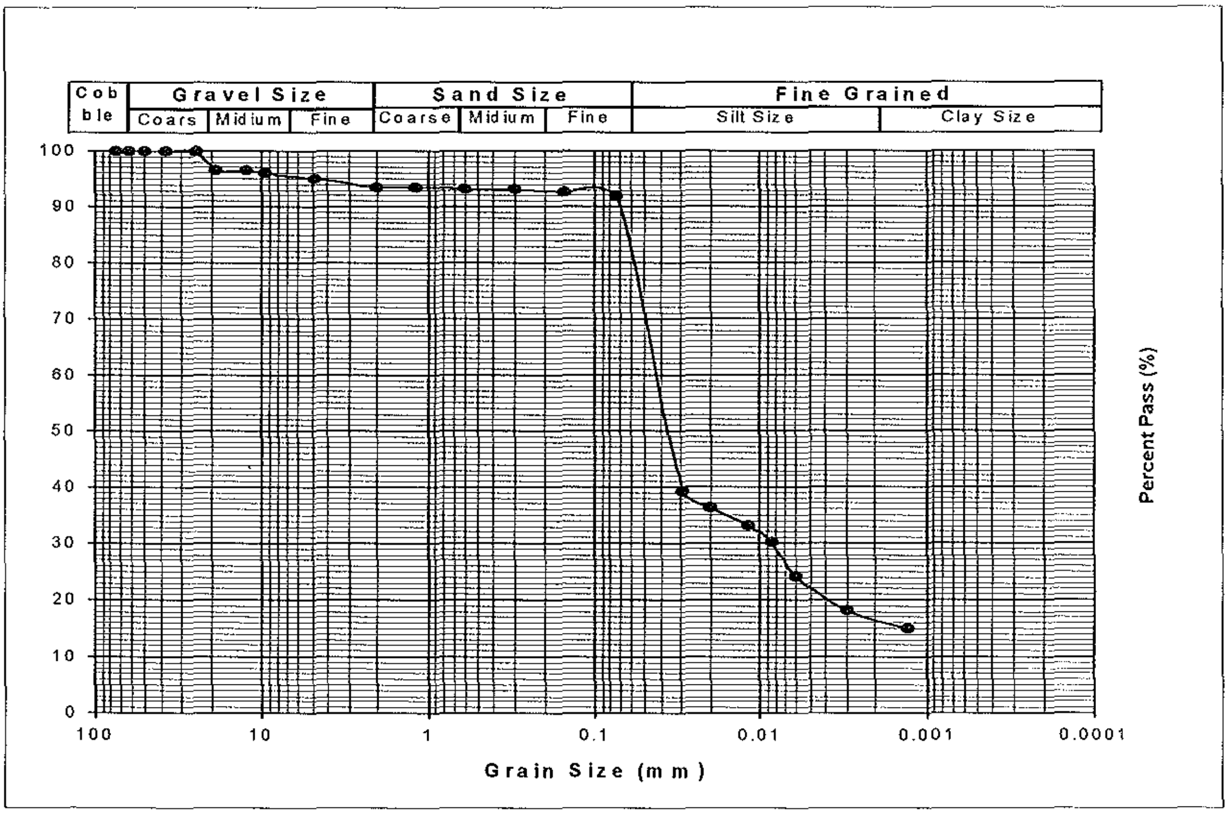
Sand For Filter: Gravely Sand **Depth: 0.5m**



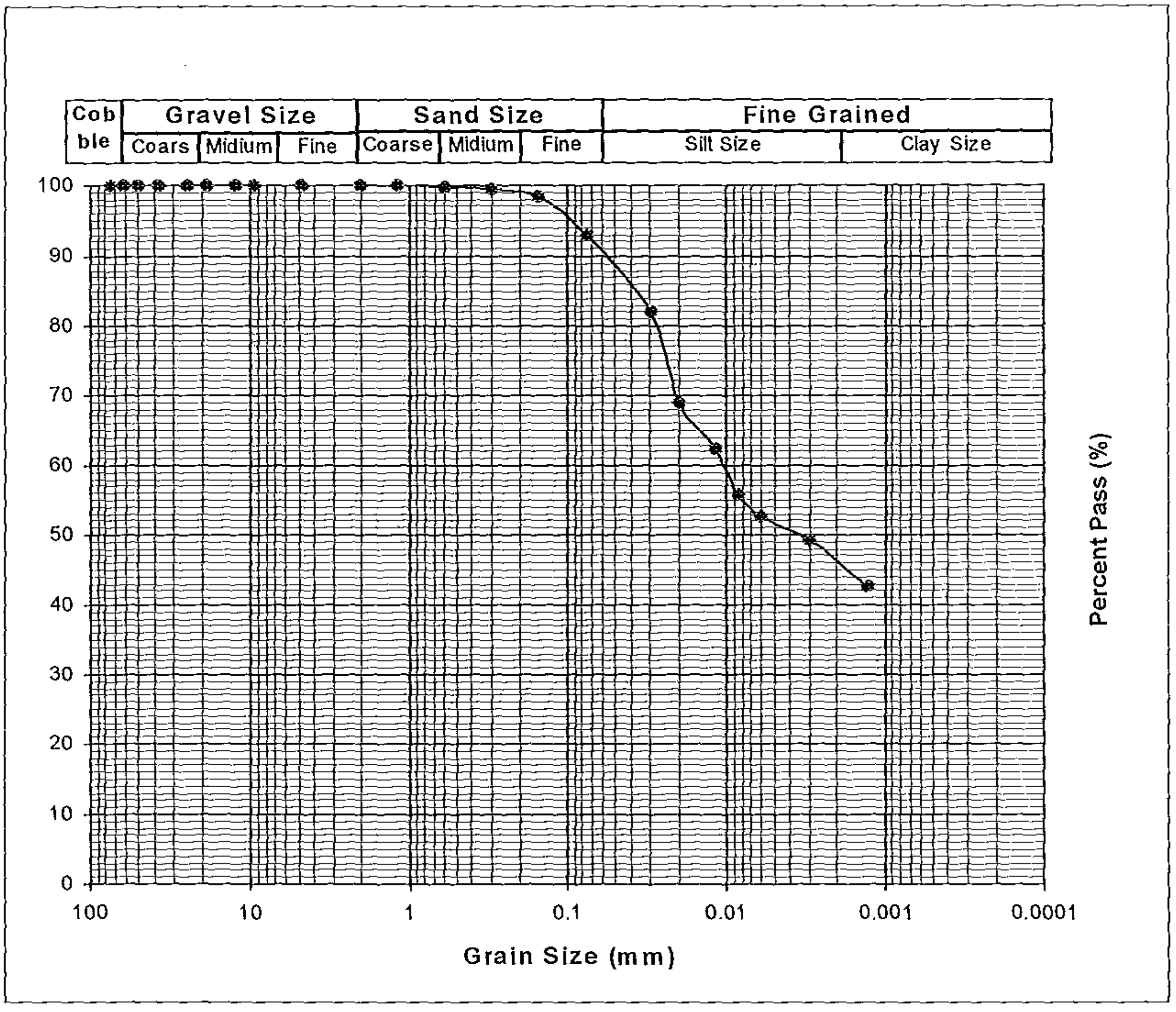
TPI7: Silty Sandy Clay **Depth: 1.5m**



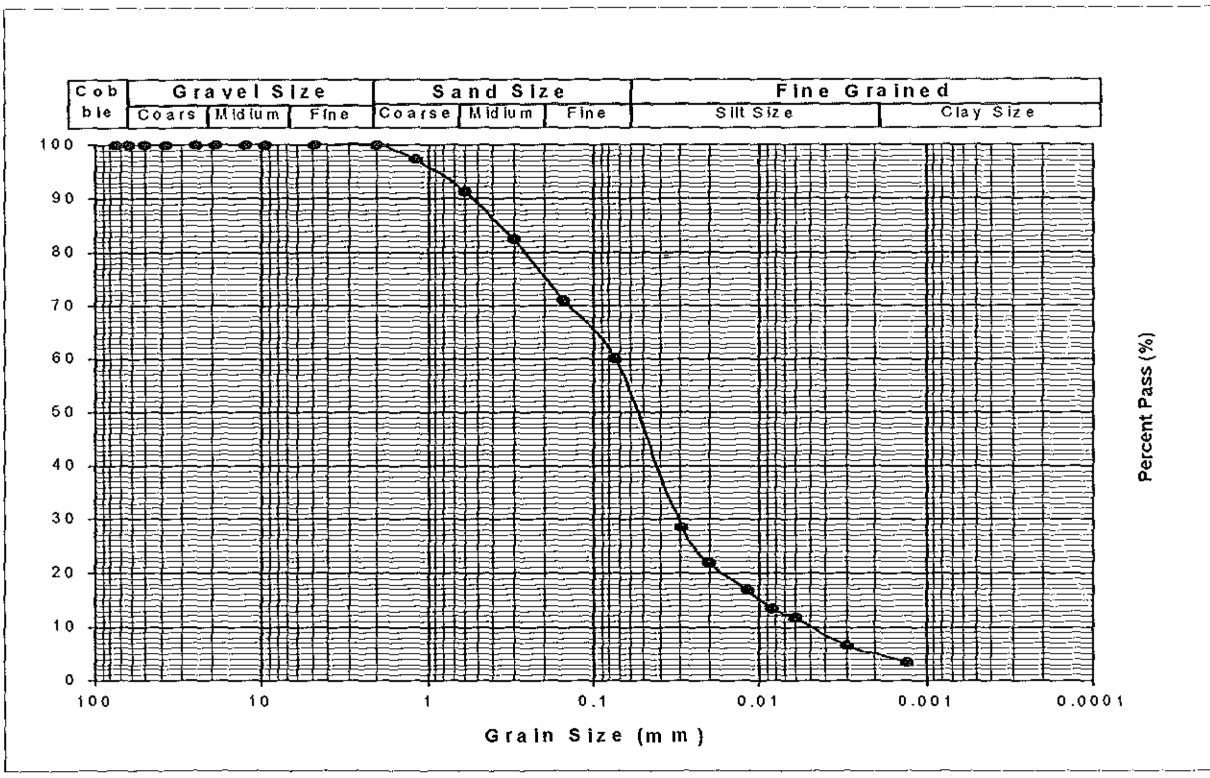
TPI7: Clayey Silt **Depth:3-4m**



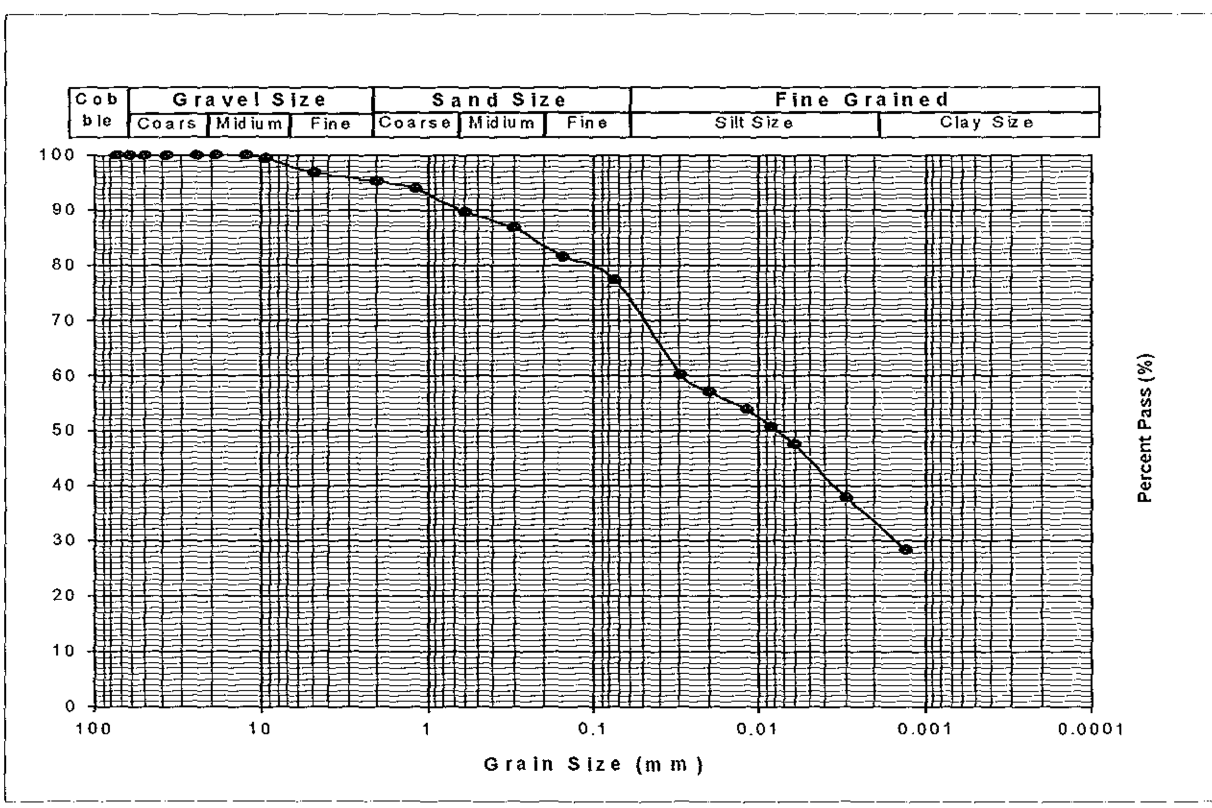
TPI8: Silty Clay **Depth: 0.5m**



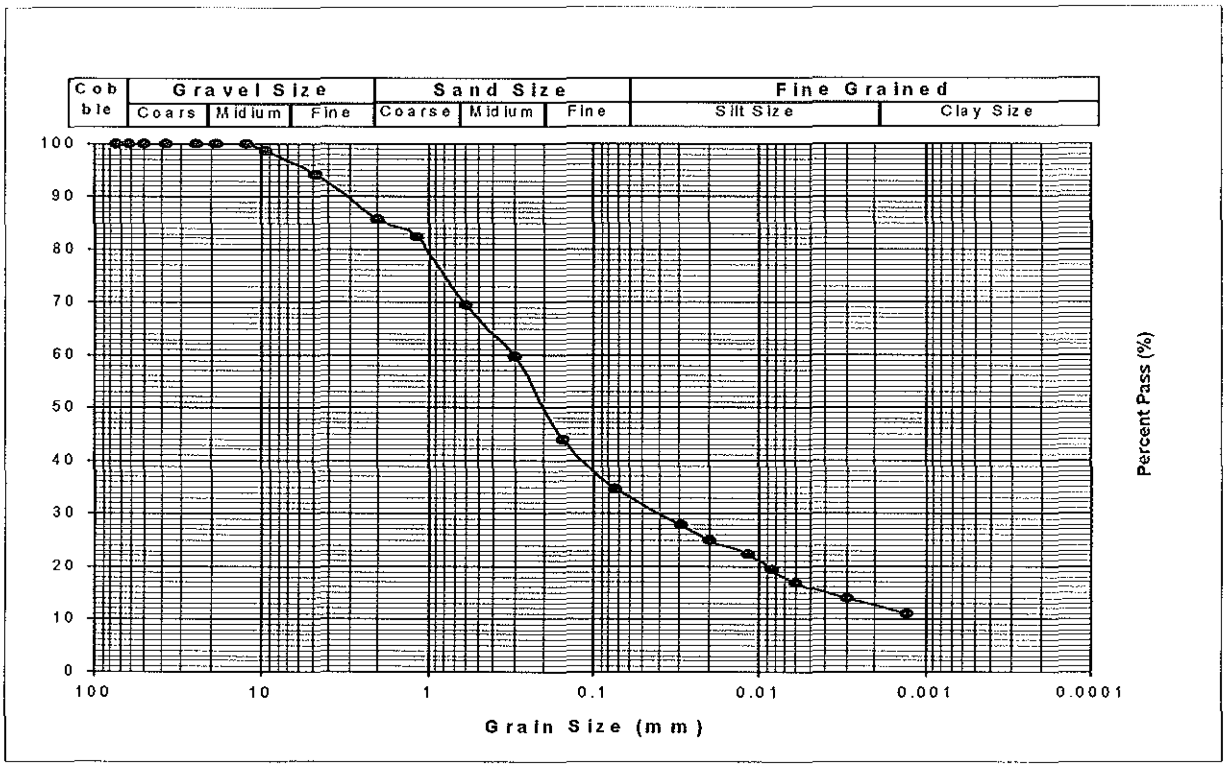
TPI10: Sandy Silt **Depth: 0.5**



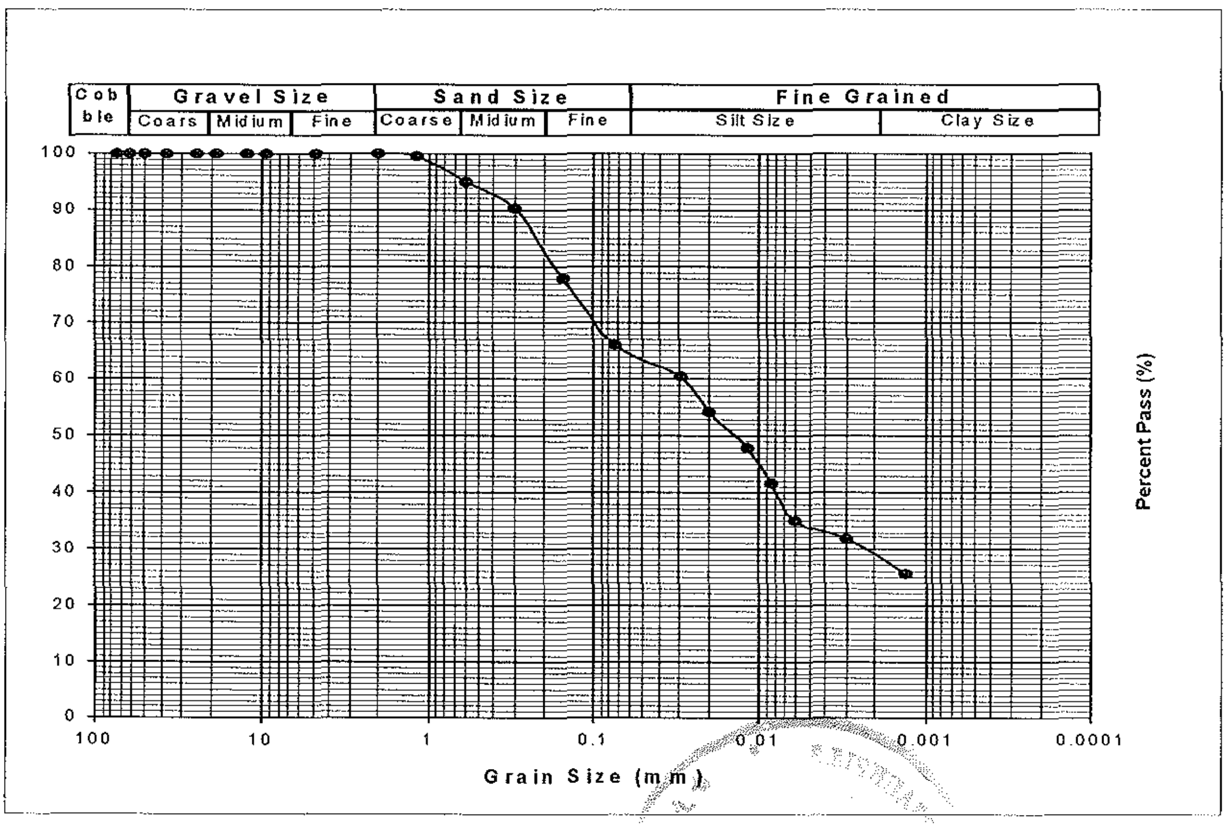
TPI11: Sandy Clayey Silt **Depth 0.5m**



TPI12: Silty Sand **Depth: 0.5m**



TPI13: Clayey Sandy Silt **Depth: 0.5m**



7.1.2 Atterberg Limit

The consistency of the soils in the study area was determined roughly as firm and stiff in the field. It shows that the clay soils have higher liquid limit, plastic limit and plasticity index, and silt soils have lower values (Table 7.2). The high value of silt soils of TPI11 could be the dominance of the montmorillonite clay mineral.

Sample	Depth (m)	USCS Classification	LL	PL	PI
			%		
TPI4	2.0	ML (OL)	38.7	25.68	13.02
TPI7	1.5	MH (OH)	54.5	28.11	26.39
TPI8	0.5	MH (OH)	50.21	31.06	19.15
TPI10	1.0	CH	62.8	28.73	34.07
TPI11	0.5	MH (OH)	59.4	38.51	20.89
TPI12	0.5	ML (OL)	31.2	22.61	8.59
TPI13	0.5	CL	39	24.98	14.02

USCS(Unified Soil Classification System)

Table (7.2) Atterberg limits, plasticity index values and unified soil classifications of the soils of the study area

Some properties calculated from the Atterberg limits are:

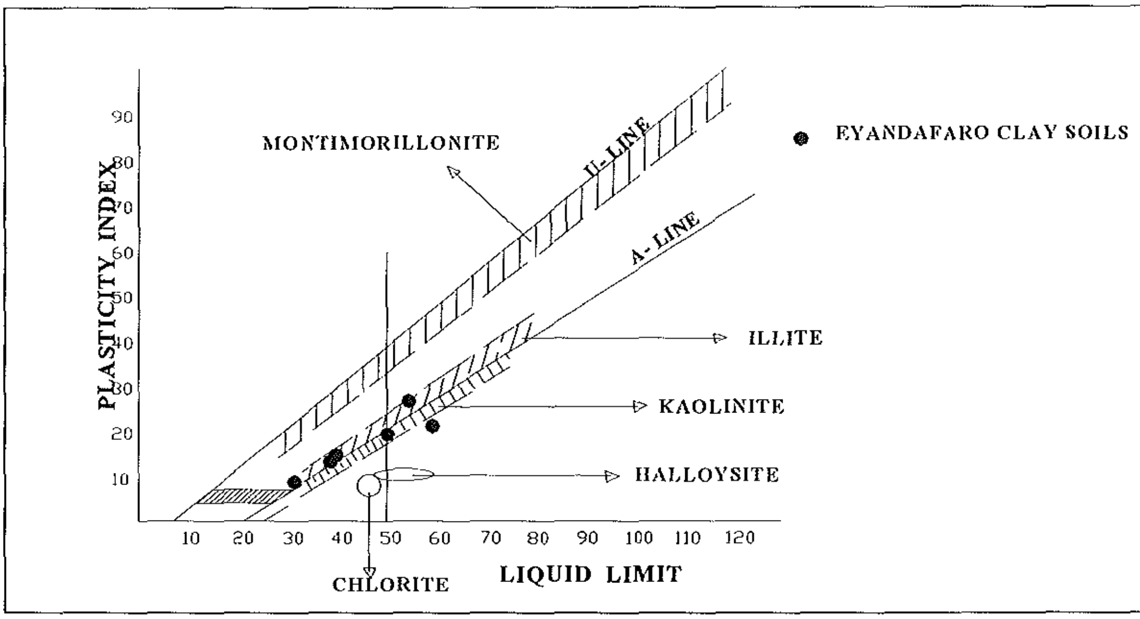
The liquidity index (LI) and consistency index (CI) of the foundation soil (TPI4) is :

$$LI = (w - wp)/Ip * 100 = -85\%$$

where w, wp and Ip are the water content of the soil in the natural condition, plastic limit and plasticity index respectively. The negative value of LI is due to the property of the soil to behave as a brittle manner.

$$CI = (wl - w)/Ip * 100 = 185\%.$$

The high consistency index value which is greater than 100% shows that the soil is relatively strong as it is the semi solid state. The sum total of the liquidity index and consistency index is always equal to 100%, indicating that a soil having a high value of liquidity index has a low value of consistency index and vice- versa (Arora, 1997).



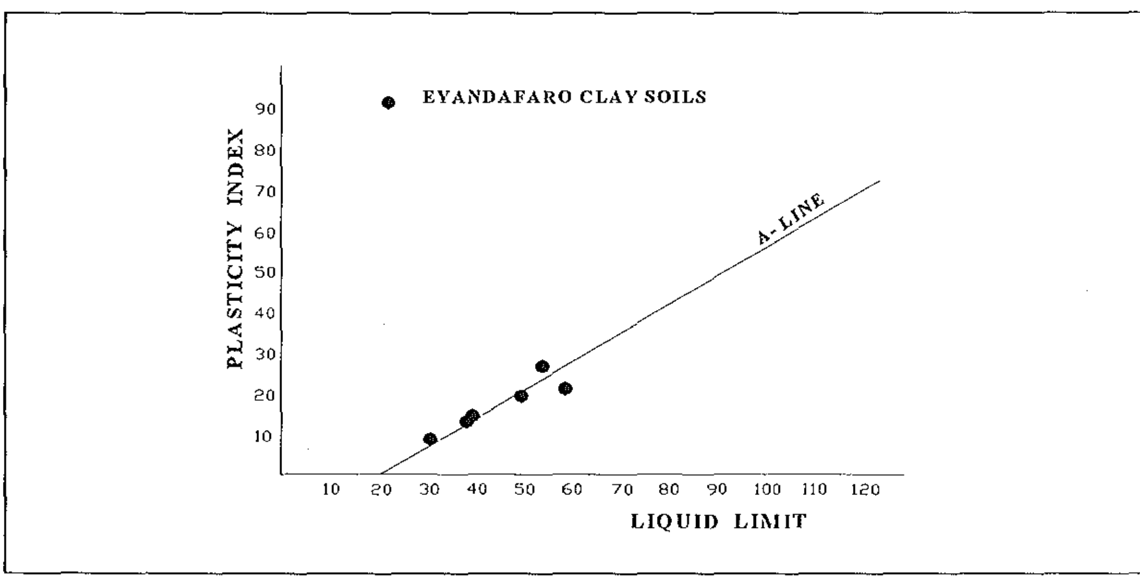


Figure 7.2 Soils of the study area on plasticity chart

7.2 Engineering properties

Engineering properties: the main engineering properties of soils are permeability, compressibility and shear strength. Permeability indicate the facility with which water can flow through soils. It is required for the estimation of seepage discharge through earth masses. compressibility is related to the deformation processes in soils when they are subjected to compressive loads. Compressive characteristics are required for composition of the settlements of structures founded on soils. Shear strength of a soil is its ability to resist shear stresses, the shear strength determines the stability of slopes, bearing capacity of soils and the earth pressure on retaining structure.

7.2.1 Shear Strength

Shear Strength is the ability of soil to sustain load with out undue distortion or failure in the soil mass. The allowable deformation will often control the design of structures, because the usual factors of safety result in shear stresses much less than those that would cause collapse or failure.

It is still customary to separate the shearing strengths of a soil into two components, one due to the cohesion between the soil particles and the other due to the friction between them, according to the following equation.

$$S = C + \delta \tan\phi$$

Depending on the physical and geochemical conditions developed at the instant of shear failure, different values of soil property may be obtained. Furthermore, in cohesive materials, the shear strength is an important function of the load history of the material, that is, of the state of stress at which the soil sediment was consolidated in the past, and of the bond between grains obtained by cementing substances or minerals.

The soil sediment should be studied in their natural state in connection with foundation engineering, with the exception of artificially compacted fills on which foundation will be supported. The determination of the shear strength property will depend on the characteristics of the specific problem: the stratigraphy, the soil sediment, the hydraulic conditions and the rate at which the state of stress is applied. It is also important to classify the different problems from the point of view of the load history.

Foundation engineering problems may be due to the applied state of stress that produces failure in natural deposits previously consolidated by the overburden pressure For these studies, the engineer needs representative undisturbed soil samples and knowledge about the stratigraphical and hydraulic conditions at the site in question.

In foundation, the load is applied at a certain, usually low rate, over periods that vary from one to several months, and on occasion several years. To decide on the shear strength to be used , the stratigraphic conditions with respect to the drainage surfaces should be taken carefully in to consideration, deciding if failure may occur approximately at constant volume. This estimate is made taking into account the permeability of the material, and estimating by means of the theory of consolidation the hydrostatic excess pressures induced during the application of the stress or change in effective stresses, in addition to the existing hydrostatic pressure that should be investigated by means of piezometric water level observations.

The shear strength for long term problems is defined by the hydraulic pressures insitu and its possible changes with time, as is the case for natural stability of slopes, (Zeevaert, 1979).

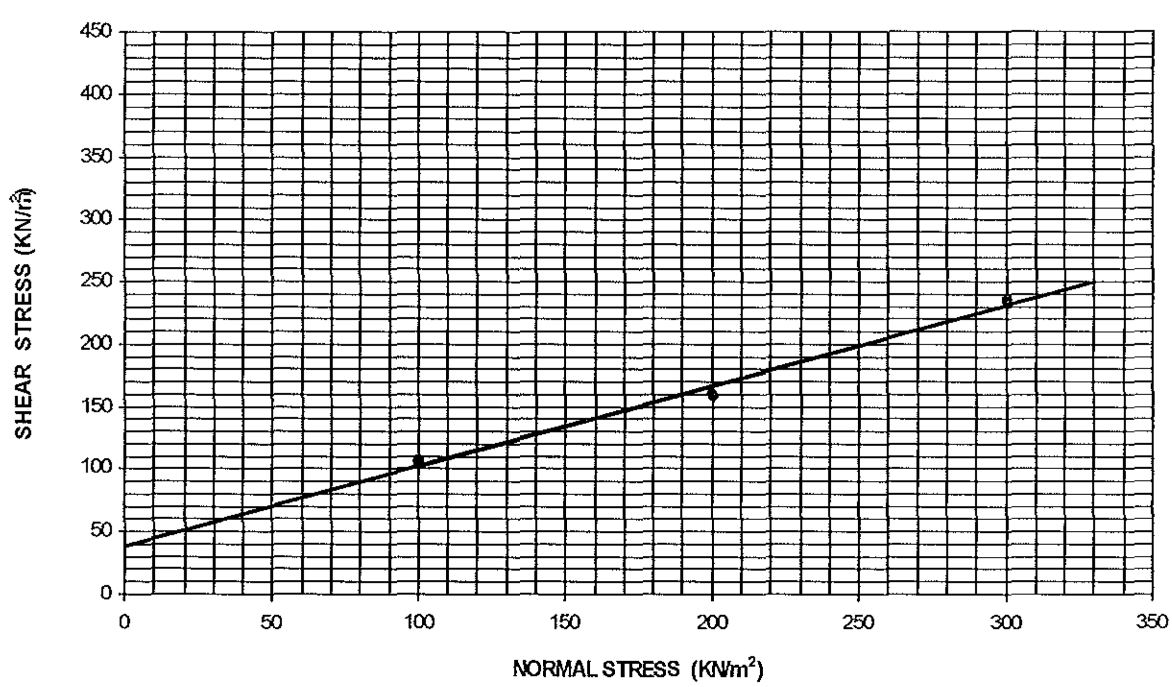
Proper laboratory testing of soils for determination of their physical properties is an integral part in the design and construction of structural foundation, the placement and improvement of soil properties and the specification and quality control of soil compaction works. It needs to be kept in mind that natural soil deposits often exhibit a high degree of non homogeneity. The physical properties of a soil deposit can change to a great extent even within a few hundred feet. The fundamental theoretical and empirical equations that are developed in soil mechanics can be properly used in practice if, and only if, the physical and engineering parameters are properly evaluated in the laboratory. So learning to perform laboratory tests of soils plays an important role in the geotechnical engineering profession. (Tschebotarioff, 1979)
The engineering properties of the studied materials are discussed as follows.

I, Presentation and analysis of results of soils of the study area.

Figure 7.3 illustrates manner of graphical presentation and interpretation of the results of direct shear tests. Three tests on the same sample was performed under varying normal pressure, Pn. The shearing strengths S = Pt/A obtained from each of the three tests are then plotted against the corresponding normal pressure Pn, giving three points corresponding to unit normal pressure, 100, 200, and 300 KN/m^2. These three points lie on an approximately straight line, some scattering being caused by experimental errors or by slight variation in the properties of the three specimens tested.

The slope of the line through the three points tanϕ, where ϕ is taken to represent the angle of internal friction. In the case of a naturally undisturbed fine grained soil of the study area , the extension of the line through points cut the ordinate through a point which represent the cohesion(C) of the sample 38KN/m^2 figure7.3. The test conducted at the dam axis is consolidated undrained direct shear.

y =38 + 0.64x					
Normal stress (KN/m^2)	100	200	300	C (KN/m^2)	Ø Degrees
Shear stress (KN/m^2)	106	160	235	38	33



Figure(7.3) Consolidated undrained direct shear test for TPI4 at 2m depth .

II. Stability analysis of the dam site

Most slides in Ethiopia took place in areas where the slope angle is between 20 - 45°. (Lulseged Ayalew, 1999). The slope of the study area is out of this range implying the low probability of sliding. Each material has its own equilibrium slope angle. In clay soils slides could be due to reduced shear surface to residual strength with little or no cohesion. Therefore stable slopes attain close to $\phi_r/2$ in saturated soils. Clays are generally unstable at > 10° roughly ($\phi/2$). The right abutment and reservoir rim in the study area is composed of well graded soils that have high shear strength than clay soils with slopes of less than 7°. So this side of the reservoir rim is stable in general. As to the stability of the left abutment and reservoir rim is concerned, it is stable both during submerged and dry conditions, since it is composed of basaltic rock unit with low degree of weathering and fracturing.

All earth embankment usually fails because of the sliding of a large soil mass along a curved surface, therefore, the slopes of an earth dam constructed from a particular soil type should be checked for safety purposes.

The down stream and up stream slopes of the dam has to be checked for steady seepage condition and also the up stream slope should be checked for sudden draw down condition by considering the worst condition of failure arc, the stability analysis of dam was not evaluated in this work, since the design was not completed.

III. Unconfined compression test

Unconfined Compresive strength test is a special type of unconsolidated undrained test. The value of δ_3 at failure is 0. Because the undrained shear strength is independent of the confining pressure as long as the soil is fully, saturated and fully undrained. For undrained tests of saturated clayey soils ($\phi = 0$ condition)

$$S = C_u$$

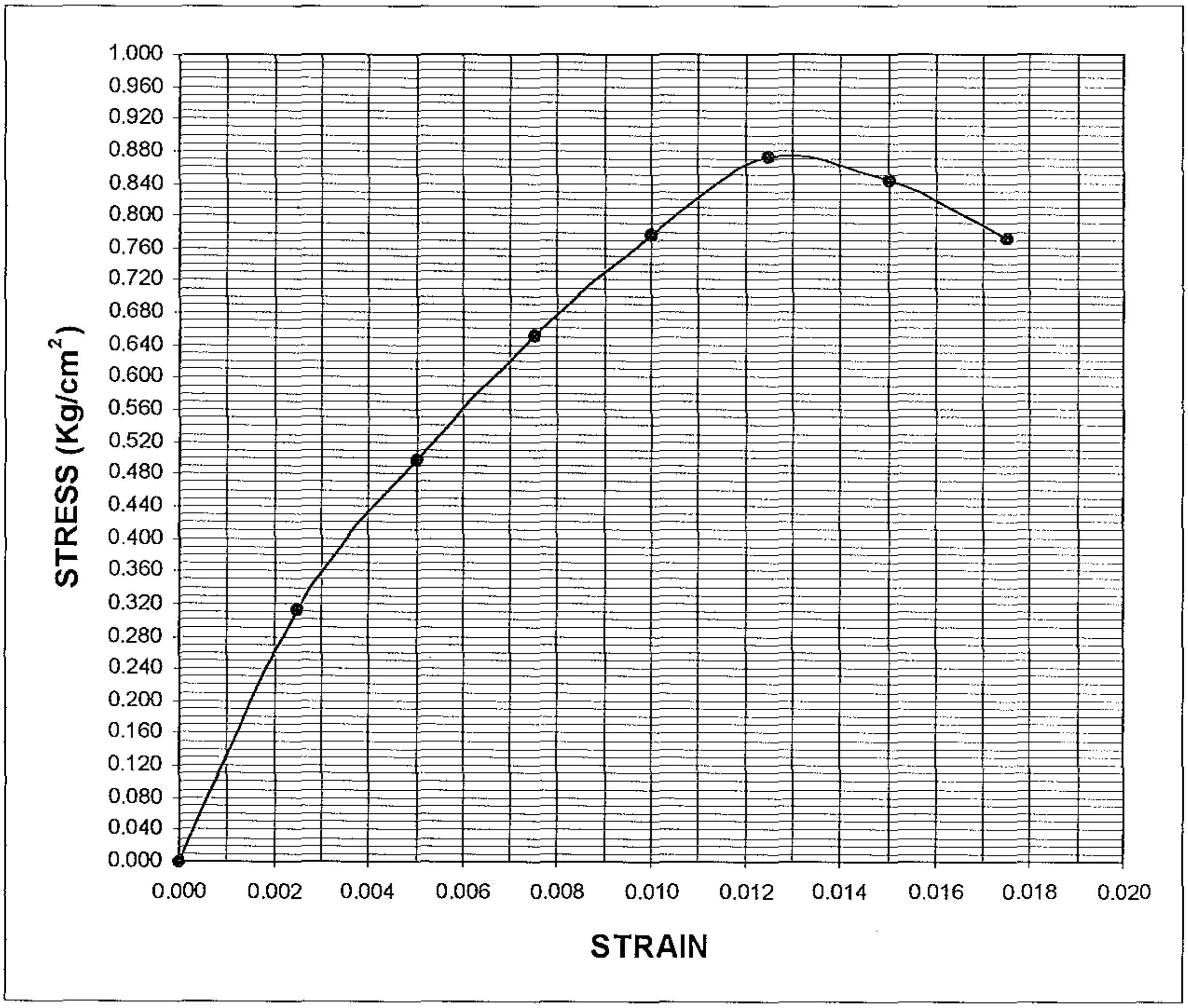
where C_u = undrained cohesion or(undrained shear strength).

The unconfined compression test is a quick method of determining the value of C_u for a clayey soil. The unconfined strength is given by the relation

$$c_u = q_u/2$$

where q_u = unconfined compression strength .

The unconfined compression strength of the representative sample (TPI4) which would be the foundation of the dam is $86KN/m^2$ figure 7.4. So the undrained cohesion or(undrained shear strength) $C_u = 86KN/m^2/2 = 43KN/m^2$. Strain at failure is 0.012



Figure(7.4) The unconfined compression strength test of sample TPI4

7.2.2 Consolidation

The time related process of soil deformation due to the dissipation of non-equilibrium pore water pressure is described as consolidation.

- Consolidation may be due to one or more of the following factors.

  i) Due to external static loads from structure

  ii) Due to self weight of the soil such as the recently placed fills

  iii) Due to lowering of the ground water-lavel

  iv) Due to desiccation.

Naturally, two type of consolidation are known:

a) Normally consolidated clay is one which has never been subjected to pressure greater than that generated by the existing over burden. It means that it never exercise more weight than the material now lying above it. The effective over burden pressure is the pressure generated by this weight minus the pore fluid pressure at the point concerned. A normally consolidated clay is in a state of equilibrium under the natural load conditions that have been gradually established with the passage of time during the geological sedimentation process.

b) Over consolidated clay: during the geological history of the clay, a superimposed load has existed which is greater than that now represented by the present over burden, then the soil as it is seen today will be over consolidated. As a result of the unloading of overconsolidated soil there will have been some rebound, but this is relatively small compared with the original consolidation, (Arora, 1997).

When such an over consolidated soil is again loaded it will be considerably stronger at a given pressure than it would have been if normally consolidated. Its volume change will be smaller than of the normally consolidated clay, up to the point when the applied load equals that had occurred during its past consolidation history. The consolidation properties of clayey silt of the study area is measured using the one dimensional consolidation apparatus, known as an Oedometer.

The weights used for the test are chosen depending on the expected stresses in the soil, including overburden pressures with load increments and the swelling stage, the tests takes a week.The load pressure are selected in the sequence 25, 50, 100, 200, 400, 800, 1600, 3200 KN/m^2. A dial gauge records changes of thickness of the specimen taken at 0, ¼, ½, 1, 2, 4, 8, 15 and 30 minutes and then at 1, 2, 4, 8, and 24 hours.

After the test is completed, two parameters are obtained from test the coefficient of volume compressibility, m_v which can be used to calculate the total amount of settlement and the oefficient of consolidation, C_v which can be used to calculate the rate at which settlements is likely to occur. From laboratory consolidation test, many parameters have been observed about the soil. Consolidation test was conducted to study the compressibility of a soil and to get various parameters such as void ratio change, permeability, change in thickness, coefficient of volume compressibility (m_V) compressibility index, and others.

A graph of void ration versus pressure log was plotted to get the above coefficients and index values. A graph was plotted settlement against time. This tell us about the consolidation characteristics of the soil and the data may be used to estimate how long it will take for settlement to develop. A vertical stress to vertical strain (by taking each load increment and each strain) drown and the data helps to estimate the eventual magnitude of settlement in the field. Important parameters from the graph of void ratio and effective stress and consolidation test are discussed as follows

A. Coefficient of volume compressibility (m_v)

Is the compression of the soil per unit of original thickness due to a unit increase of the pressure or it is the volumetric strain per unit increase in effective stress.

Thus, $$m_v = \frac{\frac{\Delta V}{V_O}}{\Delta \bar{\sigma}}$$

As the area of cross - section of the sample in the consolidometer remains constant, the change in volume is also proportional to the change in height

Thus $\Delta V = \Delta H$

Therefore, $$\frac{\Delta V}{V_O} = \frac{\Delta H}{H_O}$$

Coefficient of volume compressibility (m_v) of the sample TPI4 at 2m depth varies between 1.804 and 0.039 m^2/MN with pressure of 25 and 3200Kpa respectively. The average m_v values evaluated from void ratio versus pressure plot is 0.26m^2/MN.

B. Compression index(C_c)

Is the slope of the linear portion of the void ratio versus log ó plot.

$$C_C = \frac{-\Delta e}{\log_{10}(\bar{\sigma} / \bar{\sigma}_o)}$$

Where $\overline{\acute{o}}_0$ = initial effective stress

$\overline{\acute{o}}$ = final effective stress

Δe = change in void ratio

From void ratio versus log pressure plot fig 7.5 the following properties are evaluated:

$e_1 = 0.665$ and $e_2 = 0.776$ with the corresponding p_1 and p_2, 3200 and 1600 respectively

hence the C_c value is 0.369

The compression index is extremely useful for determination of the settlement in the field. Empirical relationship of C_C and index properties of a soil, set by Arora (1997) for undisturbed soils of the study area is $C_C = 0.009(w_L - 10) = 0.009(38.7 - 10) = 0.26$. About 30% error is expected from this formula, (Tschebotarioff, 1979)

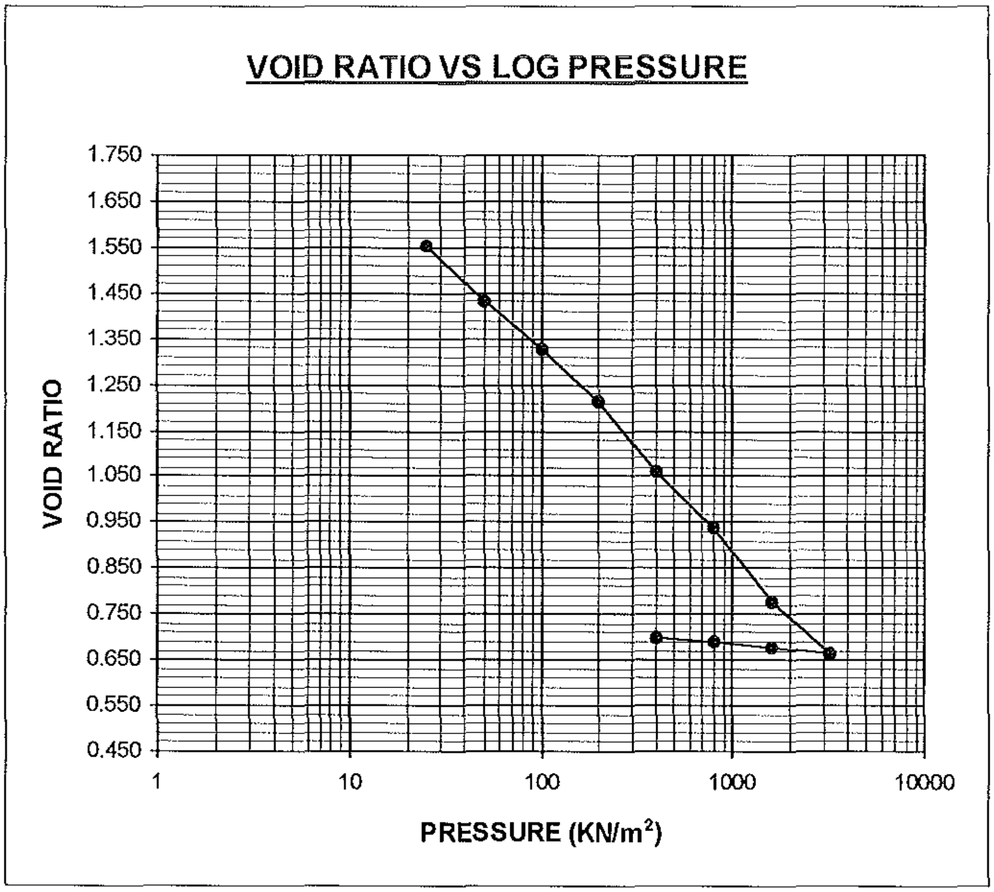
Both the consolidation results and the index properties show that the soil investigated is firm

C. Swelling index (C_e)

Is the slope of e - log ó plot obtained during unloading. It is much smaller than the compression index as it can be seen from the void ratio versus log pressure plot.

$C_e = (0.7\text{-}0.66)/ \log 8.125 = 0.044 = 2.5$degree

TPI4 at Depth of 2m



Figure(7.5) Void ratio versus log pressure during loading and unloading

D. Determination of the coefficient of consolidation(C_v)

The coefficient of consolidation Cv can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical (Murthy, 1974). There are two laboratory methods that are in common use for the determination of Cv.

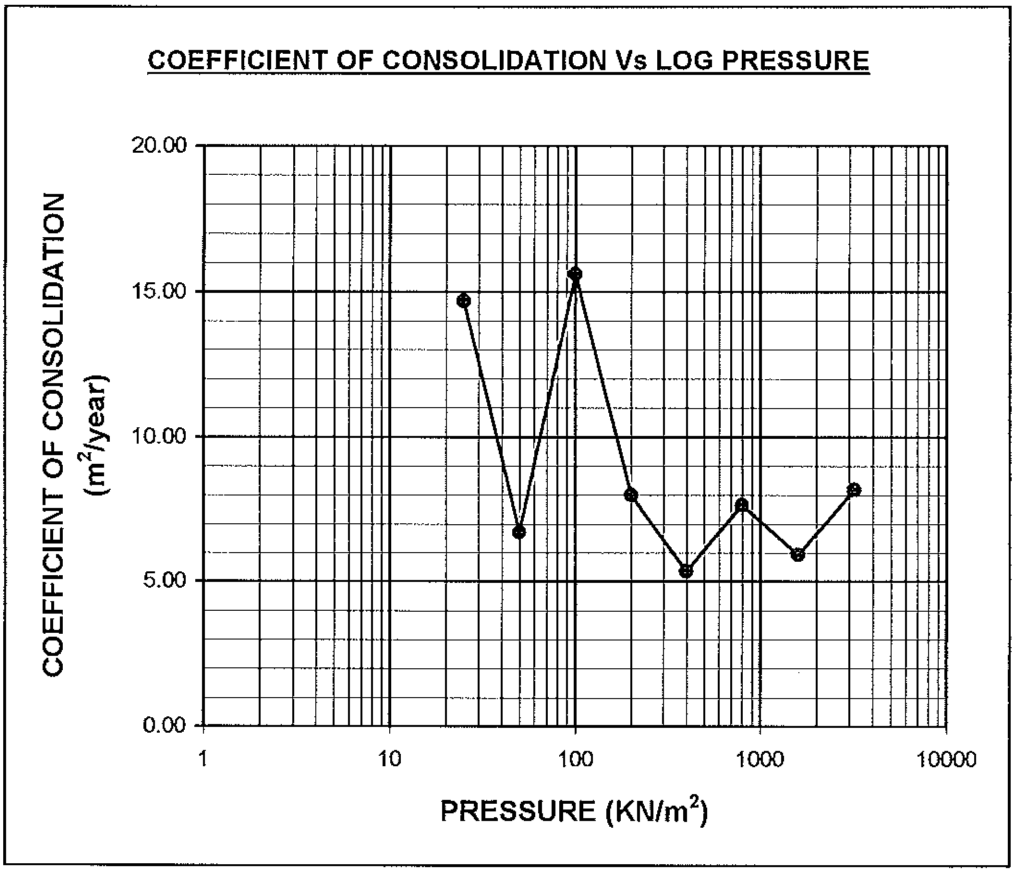
In this study square root of time fitting method is used. In this method, the dial readings are plotted against the square root of time. The theoretical curve U versus $\sqrt{T}$ is also plotted on the theoretical curve a straight line exists up to 60 percent consolidation while at 90% consolidation the abscissa of the curve is 1.15 times the abscissa of the straight line produced.

The fitting method consists of first drawing the straight line which best fits the early portion of the laboratory curve. Next, a straight line is drawn which at all points has abscissa 1.15 times as great as those of the first line. The intersection of this line and the laboratory curve is taken as the 90% consolidation point. Its value may be read and is designated as t_{90}. At the point of 90% consolidation, the value of T=0.848.

High orders of accuracy should not be expected from settlement calculations (Carter, 1989). The predicted settlements are usually given using such terms as less than 25mm or between 100mm and 150mm. For most soils, reasonably realistic estimates of settlement time can be made using Cv values but for some types of soil the predicted times will greatly exceed the actual settlement times. This is especially true of esturine clays containing silt and layers of fissured clays.

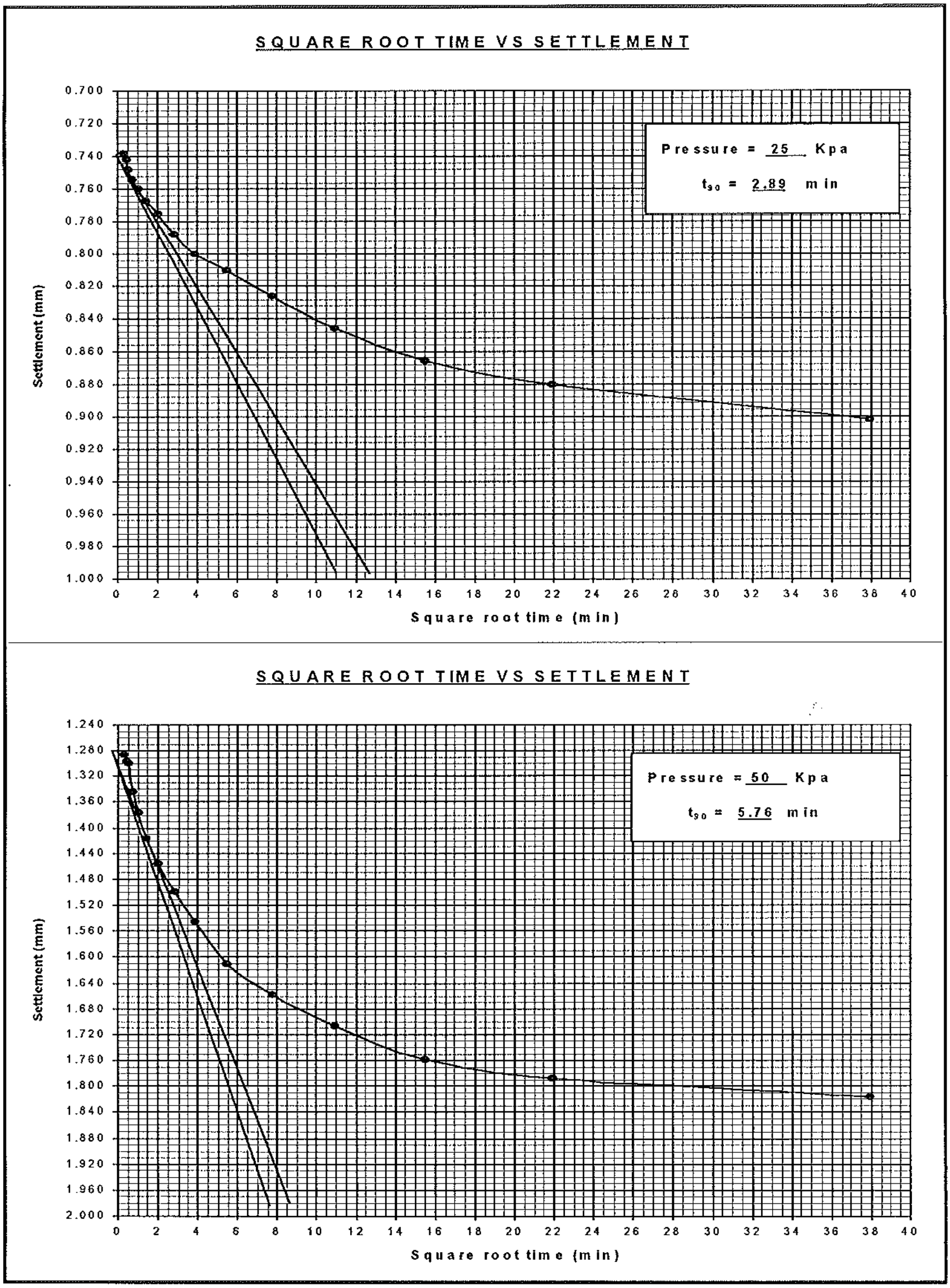
The total allowable settlement is limited to 50mm and 75mm on sandy and clayey soils respectively, EBCS(1995). It is usually the differential settlement rather than the total settlement which causes damage to structure. Coefficient of consolidation of the study sample varies between 5.36 and 15.61m^2/year within the given range of pressure.

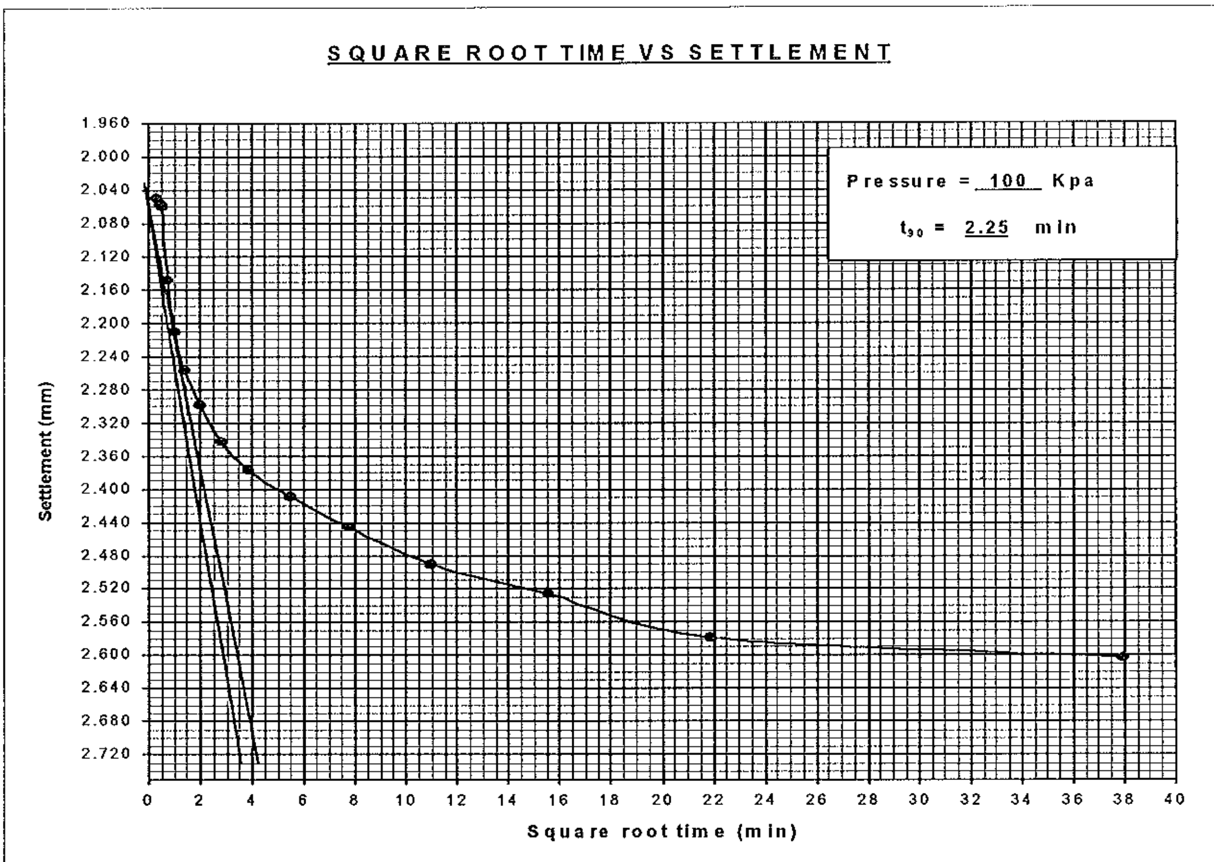
The graph of coefficient of consolidation Vs log pressure and settlement characteristics for different loading process for TPI4 at 2m depth is given below.

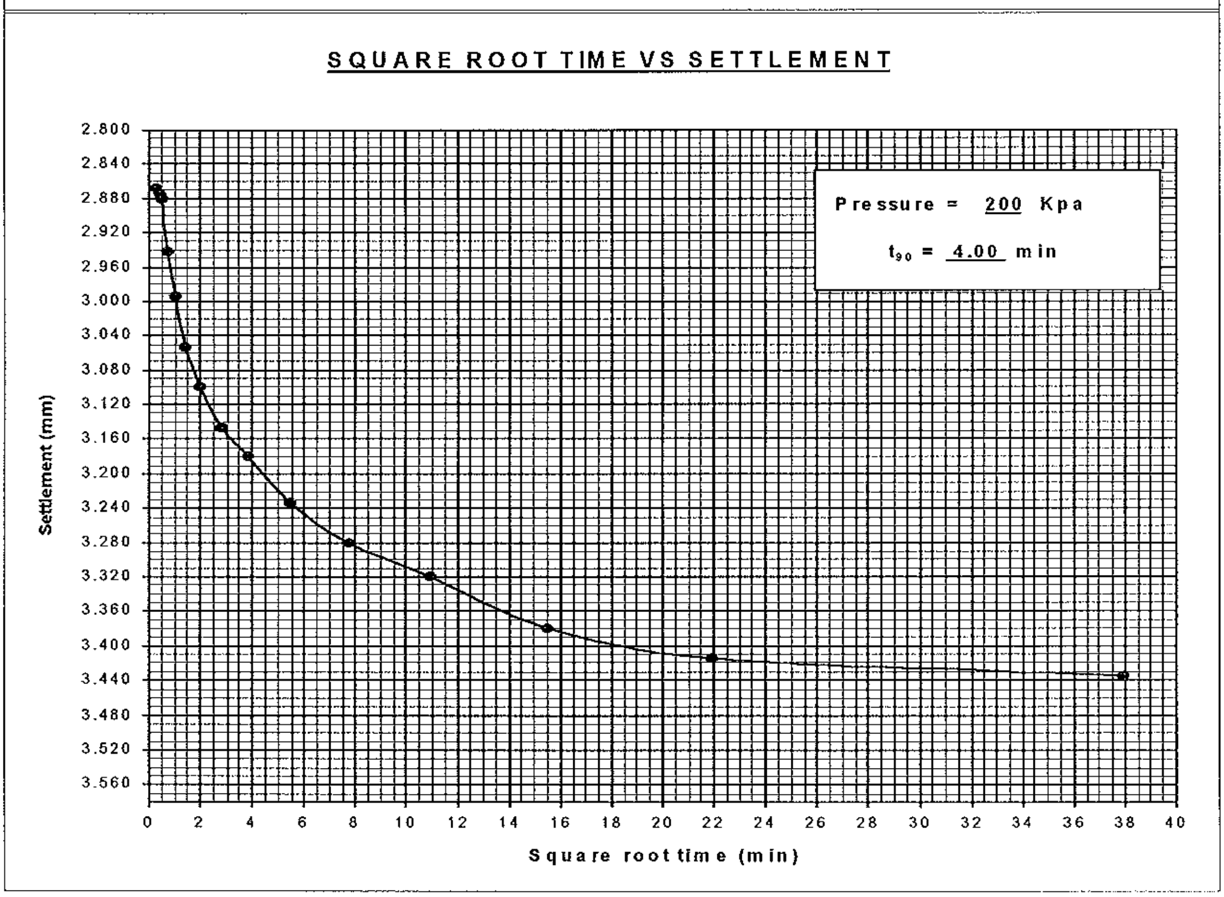


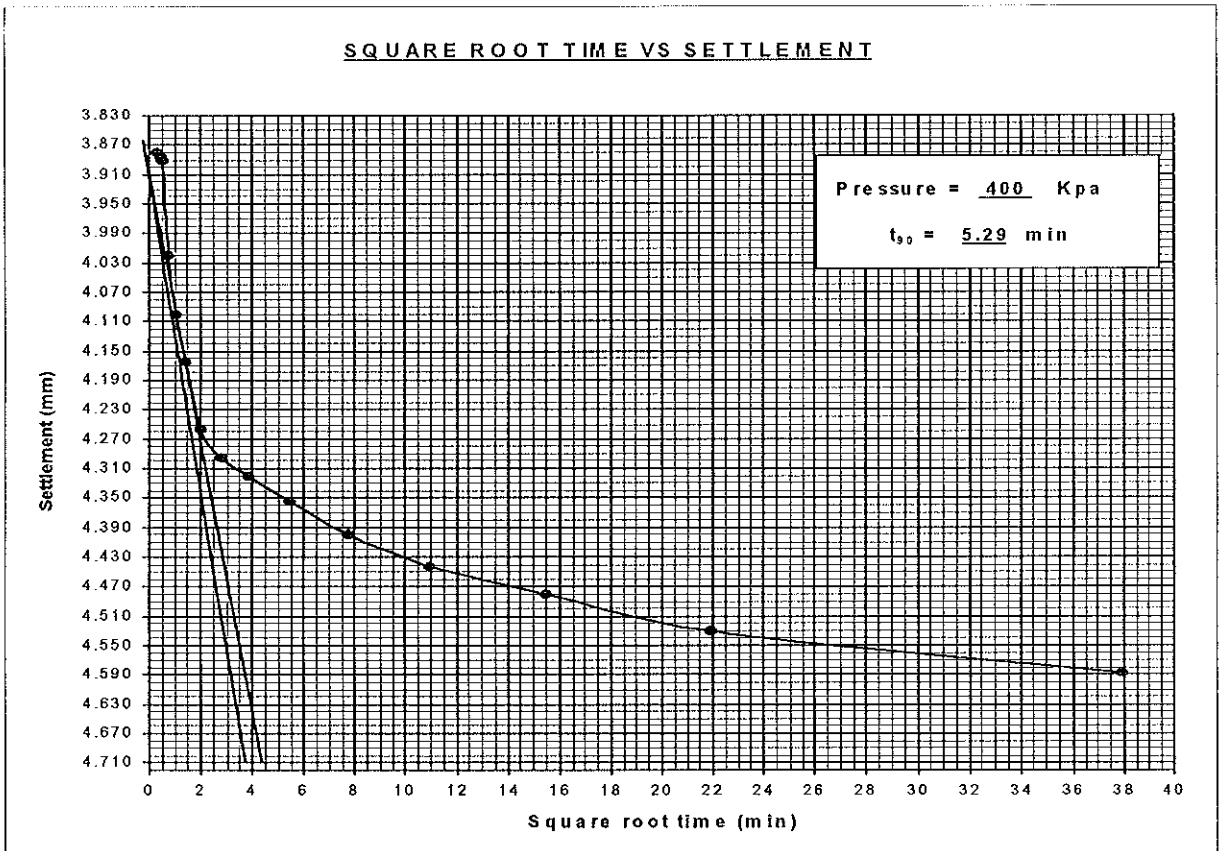
Figure(7.6) Coefficient of consolidation Vs Log pressure

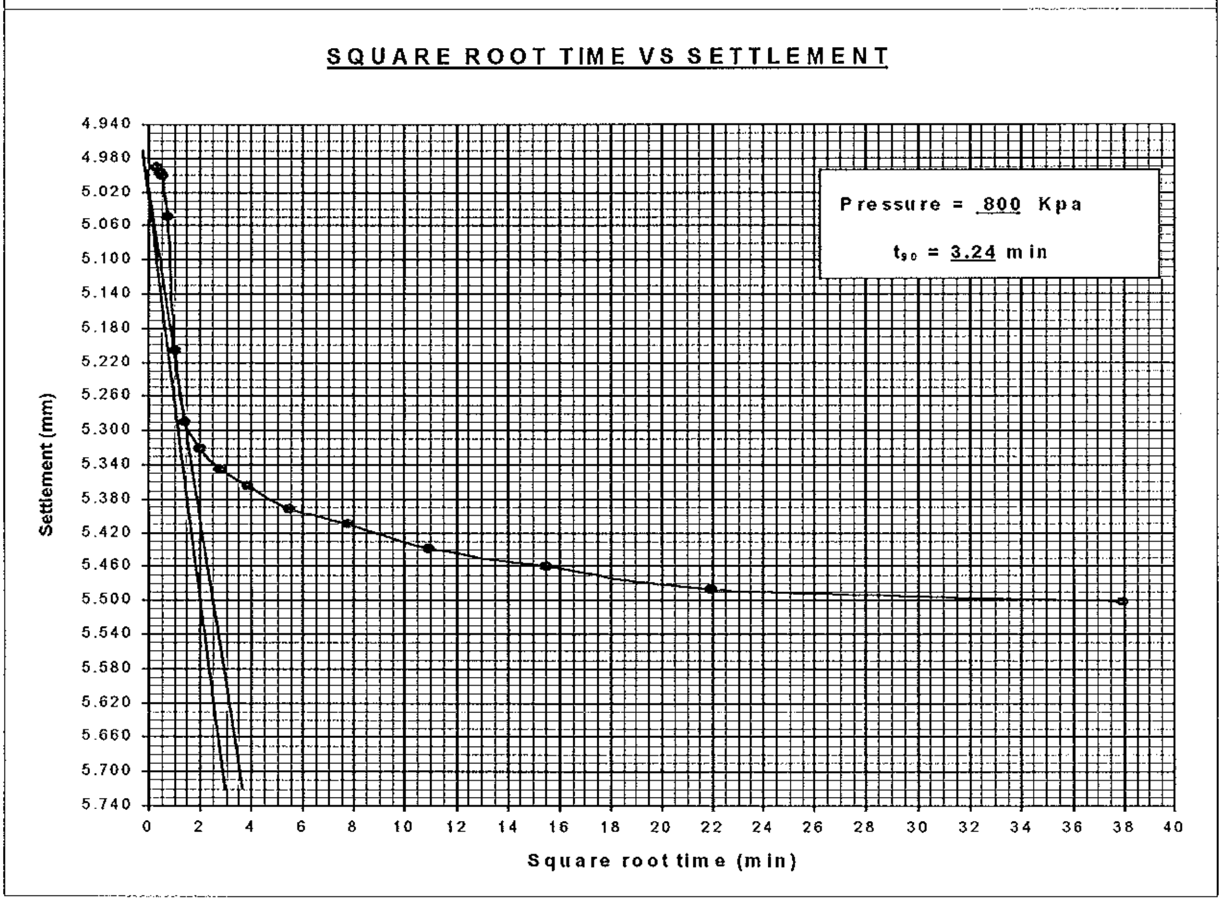
Figure7.7 Square root time vs log pressure for different loading

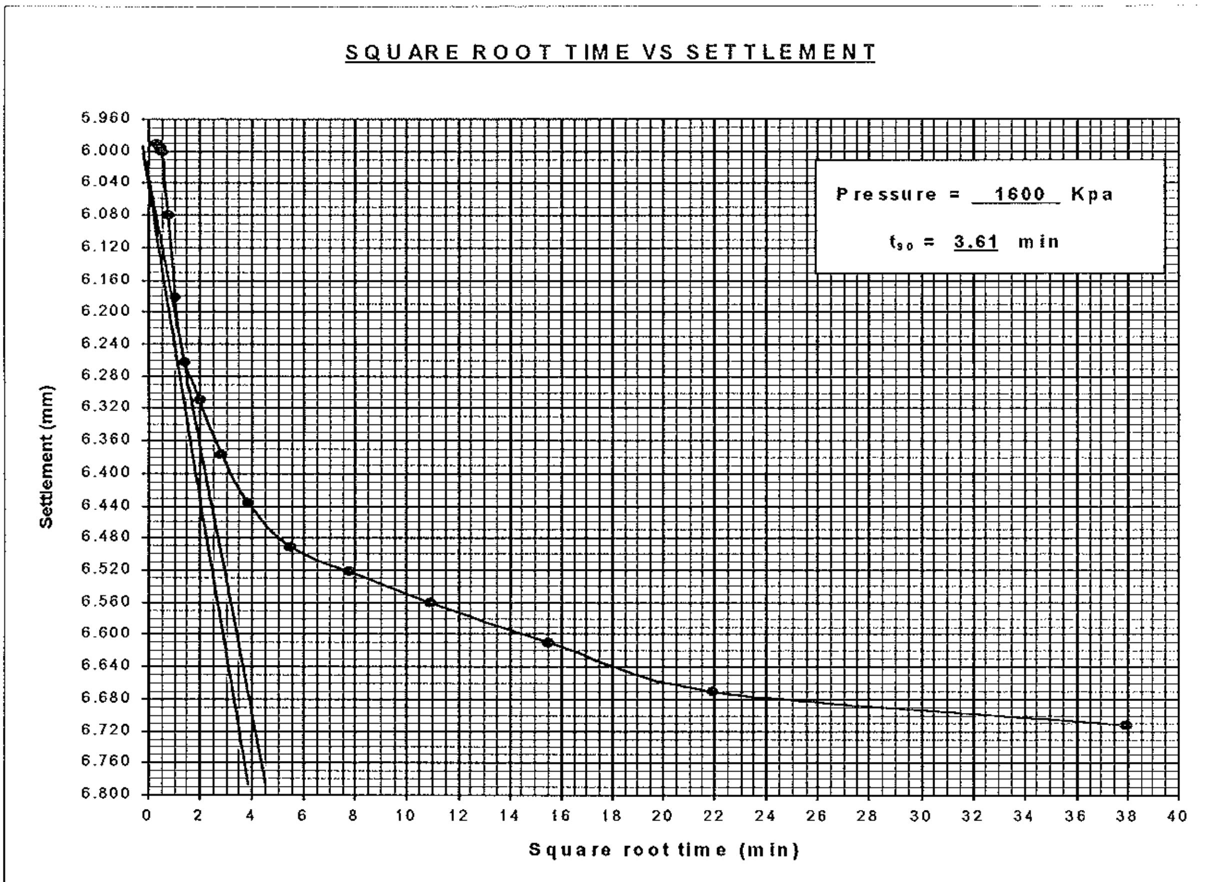


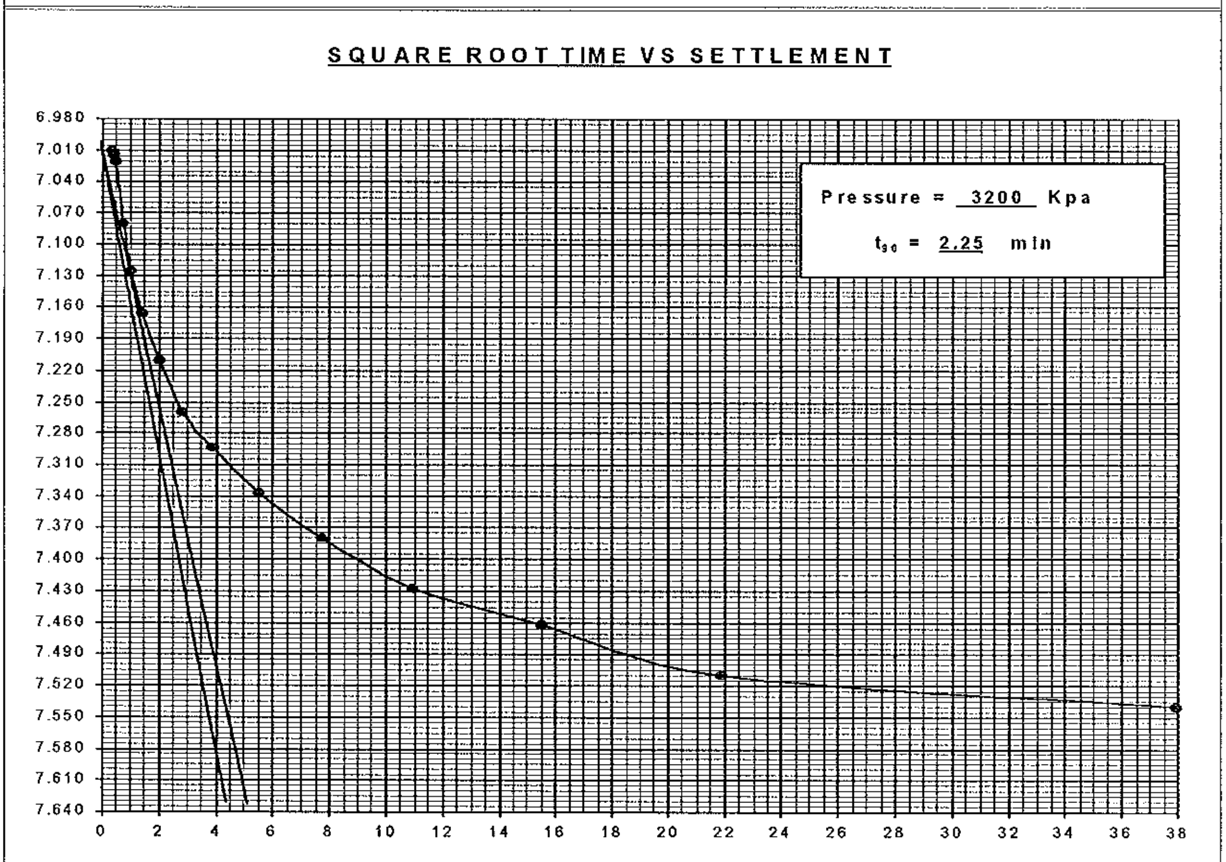

SQUARE ROOT TIME VS SETTLEMENT
Pressure = 100 Kpa
t90 = 2.25 min
Settlement (mm)
1.960
2.000
2.040
2.080
2.120
2.160
2.200
2.240
2.280
2.320
2.360
2.400
2.440
2.480
2.520
2.560
2.600
2.640
2.680
2.720
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Square root time (min)


SQUARE ROOT TIME VS SETTLEMENT
Pressure = 200 Kpa
t90 = 4.00 min
Settlement (mm)
2.800
2.840
2.880
2.920
2.960
3.000
3.040
3.080
3.120
3.160
3.200
3.240
3.280
3.320
3.360
3.400
3.440
3.480
3.520
3.560
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Square root time (min)


SQUARE ROOT TIME VS SETTLEMENT
Pressure = 400 Kpa
t_{90} = 5.29 min
Settlement (mm)
3.830
3.870
3.910
3.950
3.990
4.030
4.070
4.110
4.150
4.190
4.230
4.270
4.310
4.350
4.390
4.430
4.470
4.510
4.550
4.590
4.630
4.670
4.710
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Square root time (min)


SQUARE ROOT TIME VS SETTLEMENT
Pressure = 800 Kpa
t_{90} = 3.24 min
Settlement (mm)
4.940
4.980
5.020
5.060
5.100
5.140
5.180
5.220
5.260
5.300
5.340
5.380
5.420
5.460
5.500
5.540
5.580
5.620
5.660
5.700
5.740
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Square root time (min)


SQUARE ROOT TIME VS SETTLEMENT
Pressure = 1600 Kpa
t_{90} = 3.61 min
Settlement (mm)
5.960
6.000
6.040
6.080
6.120
6.160
6.200
6.240
6.280
6.320
6.360
6.400
6.440
6.480
6.520
6.560
6.600
6.640
6.680
6.720
6.760
6.800
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Square root time (min)


SQUARE ROOT TIME VS SETTLEMENT
Pressure = 3200 Kpa
t_{90} = 2.25 min
6.980
7.010
7.040
7.070
7.100
7.130
7.160
7.190
7.220
7.250
7.280
7.310
7.340
7.370
7.400
7.430
7.460
7.490
7.520
7.550
7.580
7.610
7.640
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38

7 .2.3 Permeability coefficient

The coefficient of permeability of soil can be determined by any test in which the cross-sectional area, the hydraulic gradient, and the quantity of flow can be measured or can be approximated. To determine the coefficients of permeability of the foundation and core material indirect methods was conducted.

Using Hazens formula from gradation curve, permeability coefficient (k) is estimated for the foundation soil as follows:

- For well graded soil at the right abutment and reservoir rims (TPI7) at 1.5m depth,

$K = C\,(D_{10})^2$ Where C = constant usually taken to be 0.01(Arora, 1997)

D_{10} = effective grain size

So $K = 0.01(0.0005)^2 = 2.5 \times 10^{-9}$ m/s $= 2.5 \times 10^{-7}$ cm/s

- For sand deposits along the river bed $K = 9 \times 10^{-4}$ m/s $= 9 \times 10^{-2}$ cm/s
- For clayey silt of TPI4 taken at 2m depth $K = 0.4 \times 10^{-7}$ c m/s and using indirect method of calculating K from consolidation test:

$K = \gamma_w\, m_v c_v$

γw = Unit weight of water(1gm/cm^3)

mv = coefficient of volume compressibility

cv = coefficient of consolidation

Average mv = 0.26×10^{-4} cm^2/gm

γw = 1gm/cm^3

Average cv = 9m^2/year = 2.85×10^{-3} cm^2/s

$K = 1\text{gm/cm}^3 \times 0.26 \times 10^{-4}\text{cm}^2/\text{gm} \times 2.85 \times 10^{-3}\text{cm}^2/\text{s} = 0.74 \times 10^{-7}\text{cm/s}$

So both Hazen's formula and indirect methods from consolidation test shows that this lithological unit is impervious. Except the main river and its levees the other parts of the foundation and reservoir is found to be impervious. The permeability coefficient of core sample (TPI9) is 1.14×10^{-7} cm / s which is impervious unit that can be used for the proposed structure as fill material.

Chapter Eight

Geophysical Sounding and Profiling at the Dam Area

8.1 General

A number of geophysical methods use measurements of voltage or magnetic fields associated with electric currents flowing in the ground. Most rock forming minerals are very poor conductors, ground currents are therefore mainly carried by ions moving in pore waters. The resistivity of a rock is roughly equal to the pore fluid resistivity divided by the fractional porosity. Archies Low states that resistivity is proportional to fractional porosity raised to a power of between about 1.2 and 1.8 according to the shape of the matrix grains. The departures from linearity are not large for most common porosities. Pure water is ionized to only a very small extent, so that conduction in pore waters depends on the presence of dissolved salts, mainly sodium chloride. Clay minerals are ionically active and clays conduct well if even slightly moist.

Resistivities of common rocks and minerals are shown in (Table9.1) Milson. Resistivities of more than 10000 ohm-meter or less than 10 ohm-meter are rarely encountered in field surveys. Schlumberger Array with current electrodes placed symmetrically away from the potential electrode was applied for geological investigation of this study. The main purpose of the geophysical investigation are to determine the following:

- Thickness of the over burden layers
- Rock head mapping (depth of bed rock)
- Depth of weathered zones
- The existence of geological structures at the mapped areas, if any

Common rocks	Resistivity (ohm)	Ore Minerals	ohm-meter
Topsoil	50-300	Pyrite	100-0.01
Loose sand	500-5000	Chalcopyrite	0.1-0.005
Gravel	100-6000	Sphalerite	1000000-1000
Clay	1-100	Magnetite	1000-0.01
Weathered bedrock	100-1000	Pyrrhotite	0.01-0.001
Sandstone	200-8000	Galena	100-0.001
Limestone	500-10000	Cassiterite	10000-0.001
Greenstone	500-200000	Hematite	1000000-0.01
Gabbro	1000-500000		
Granite	200-100000		
Basalt	50-200000		
Quartzite	500-800000		

Table(8.1)Resistivities of Common rocks and Minerals (Milson)

8.2 Quantitative Interpretation

The results of the electrical sounding survey is presented in the form of geo-electric section which shows the distribution of layer resistivities and thickness. This is believed to be a closer approximation to the actual geologic setting in the subsurface and most of the quantitative interpretation is based on this section.

From four to five layer geoelectric section was constructed based on the VES data from the site. The top most layer marked by resistivity changes from 120-310 ohms was mapped over the two VES(Ves1 and Ves4) this section has a thickness varying from few centimeters at the location of Ves9 to 1.5m on the other sounding locations. This geoelctric layer is a likely signature of the top dry sandy cover. In fact there is a lithologic contact at two places along the profile length. The first is between Ves9 and Ves1 the other is between Ves1 and Ves 2. As we go to the right abutment, the lithology changes from gravel (311 ohm) to clay (13-70 ohm), at the location of Ves2 and Ves3.

The second geoelectric layer whose thickness increases as we go to Ves2 is delineated by resistivity values between 20-245 ohms. This layer is the mixture of sand and gravel at the location of Ves1 and Ves4 with slight moisture content than the above layer. But it represents the bottom portion of the soil section with considerable clay at the location of Ves2 and Ves3(20-85 ohms).

The third geoelectric layer represented by resistivity variation from 20(silt and clay) to 1340(gravel and boulder) and the thickness ranges from 2.4m at Ves9 to a value grater than 100m at Ves2. At the river center (Ves1) the thickness of this section is about 7m. The fourth geoelectric layer is gravel silt and clay. This section is about 5.4 m at the river center. This layer is bedrock response at the left abutment(Ves9) and extreme left abutment(Ves4) it is marked by high resistivity values (>4000 ohms). The bed rock is the fifth geoelectric layer in case of Ves1, and the depth of this bedrock at the river center is about 13.5m.

The apparent resistivity profiling conducted at a-b of 90m, shows high resistivity (bed rock symptom at the high slope abutments and low resistivity values or thick overburden material at the flat plain areas. Apparent resistivity pseudosection map of the first profile(along the dam axis) and second (at the reservoir area) show thick overburden at the flat plain area. The profiling, VES curves, of the study area and apparent resistivity pseudosection map of the study area is given below.

Figure 8.1 Geo electric section along the dam axis

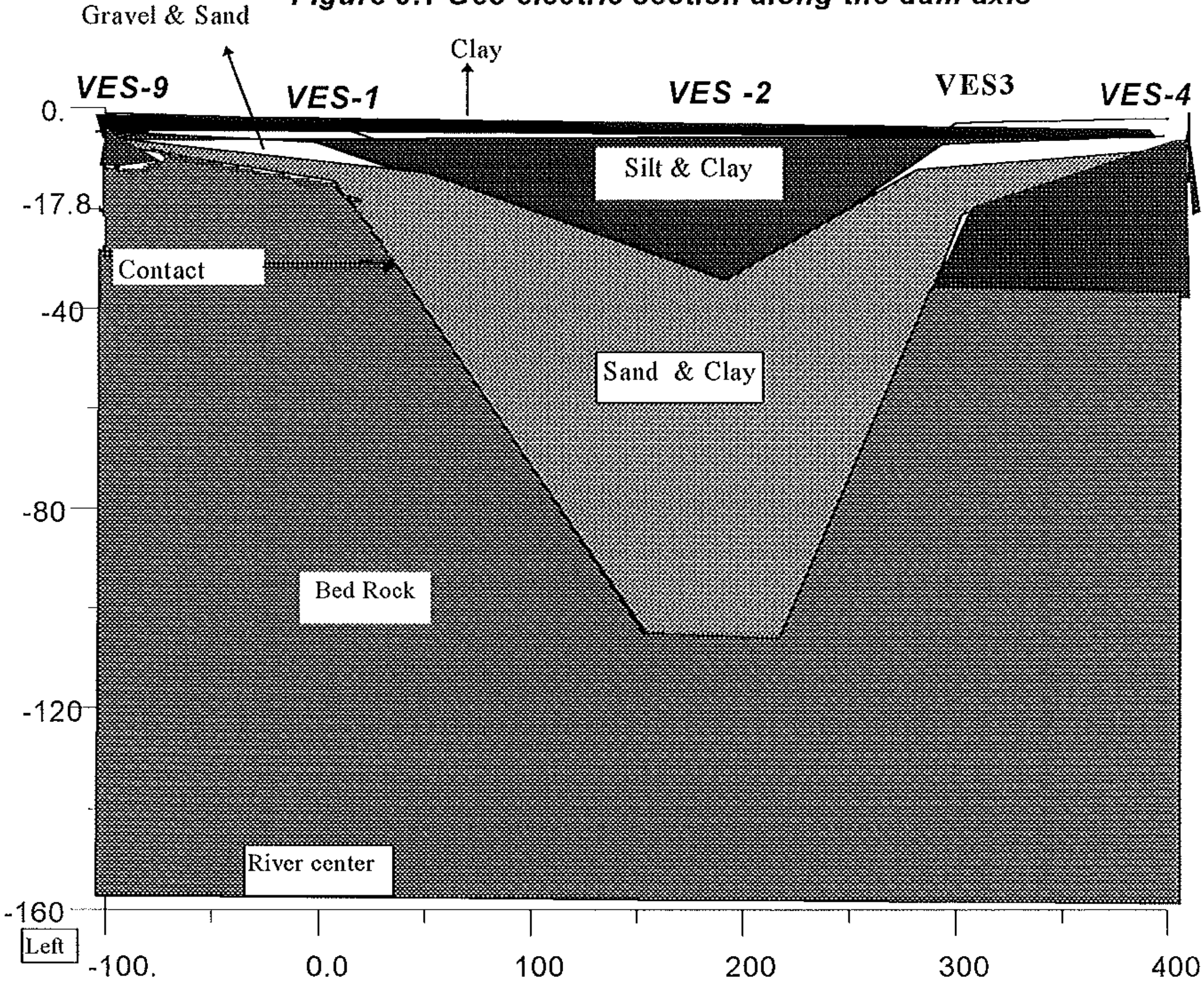


Fig. (8.2) Apparent resistivity curve for all VES along profile 1

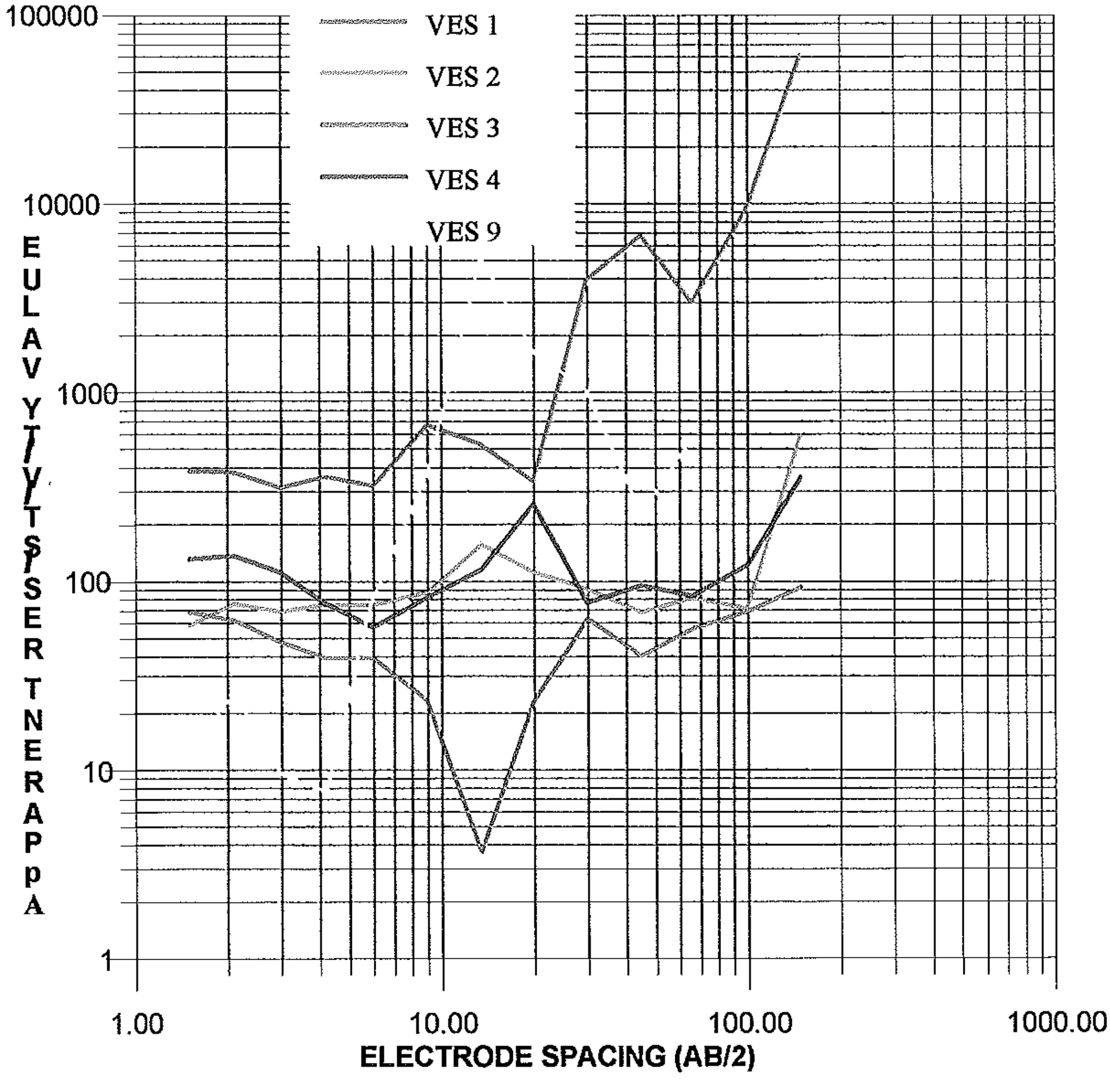


Fig. (8.3)VES CURVE FOR THE SECOND PROFILE (VES8)
(500M FROM THE RIVER CENTER)

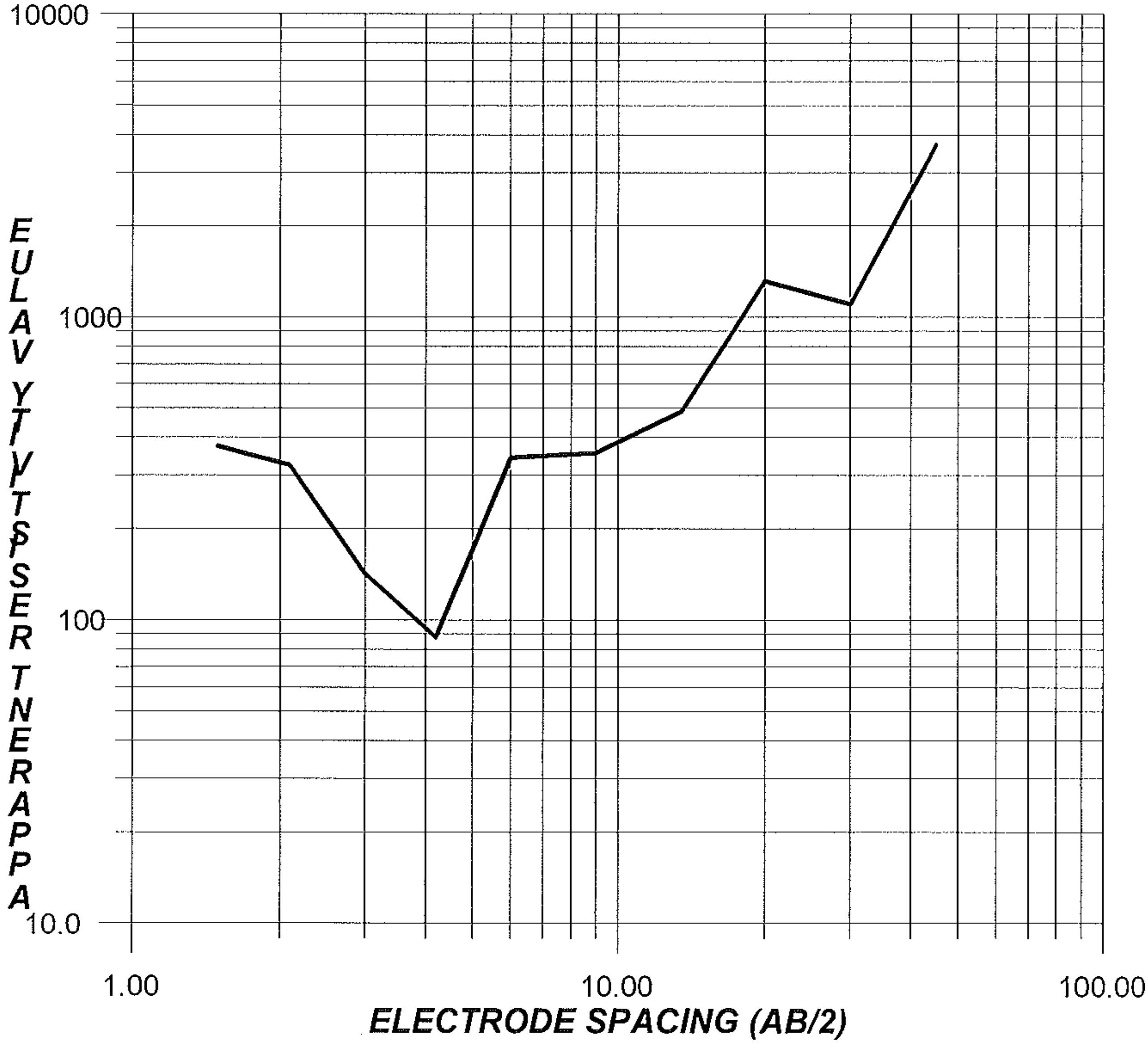


Fig. (8.4) 300M FROM THE RIVER CENTER (VES 7)

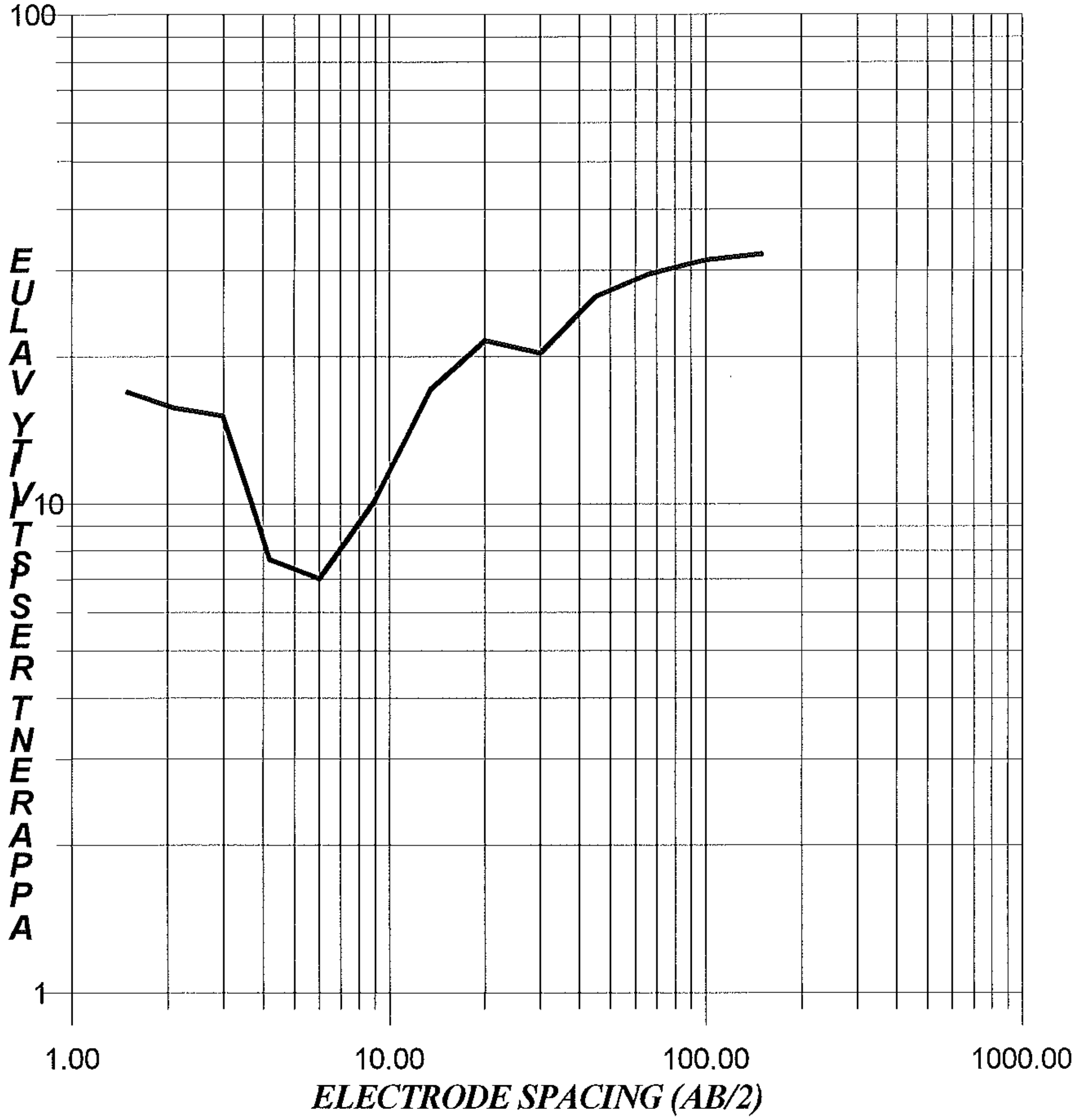


Fig. (8.5)   200M FROM THE RIVER CENTER OF THE SECOND PROFILE (VES6)

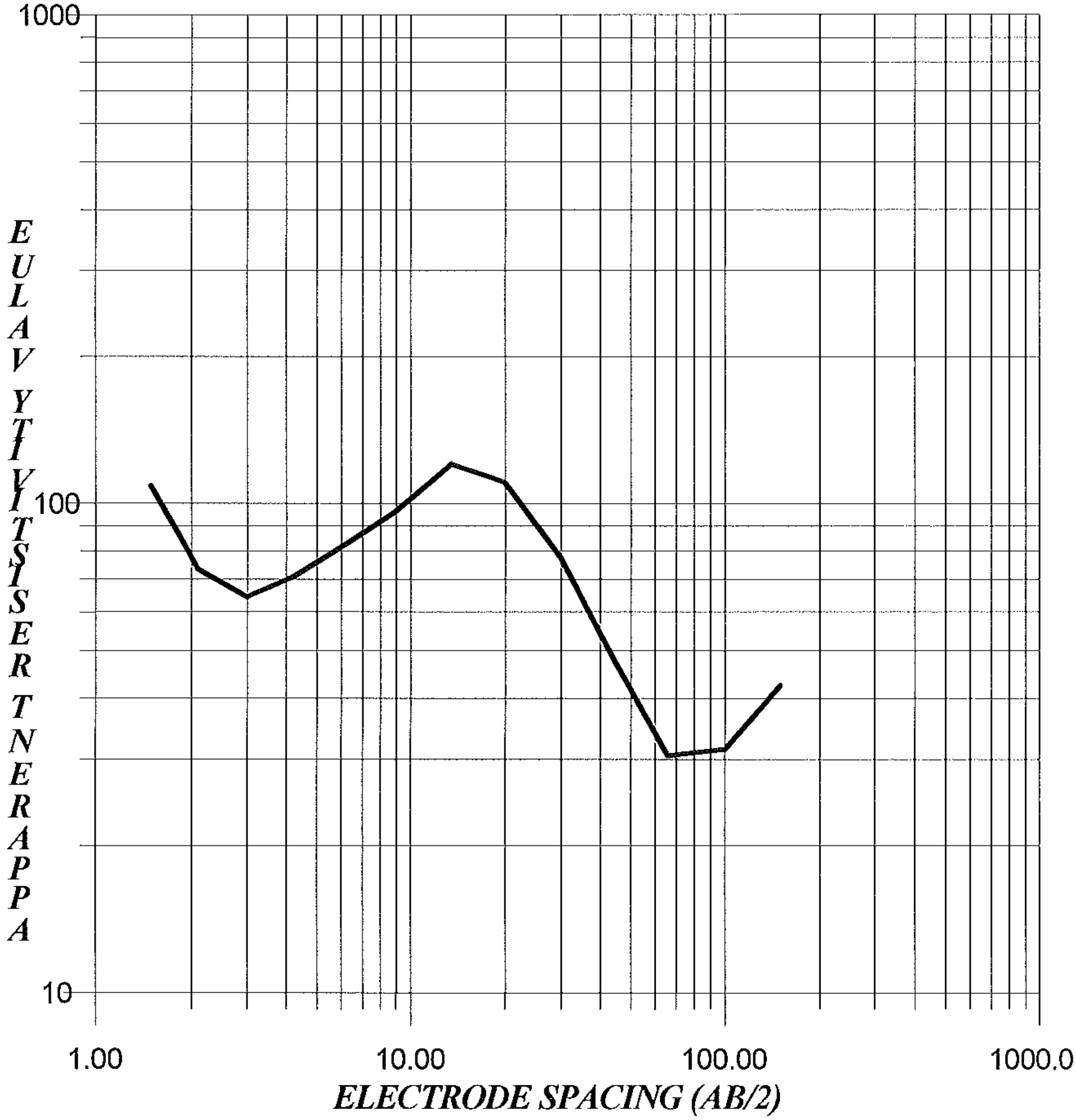


Fig. (8.6) RIVER CENTER FOR THE SECOND PROFILE (VES 5)

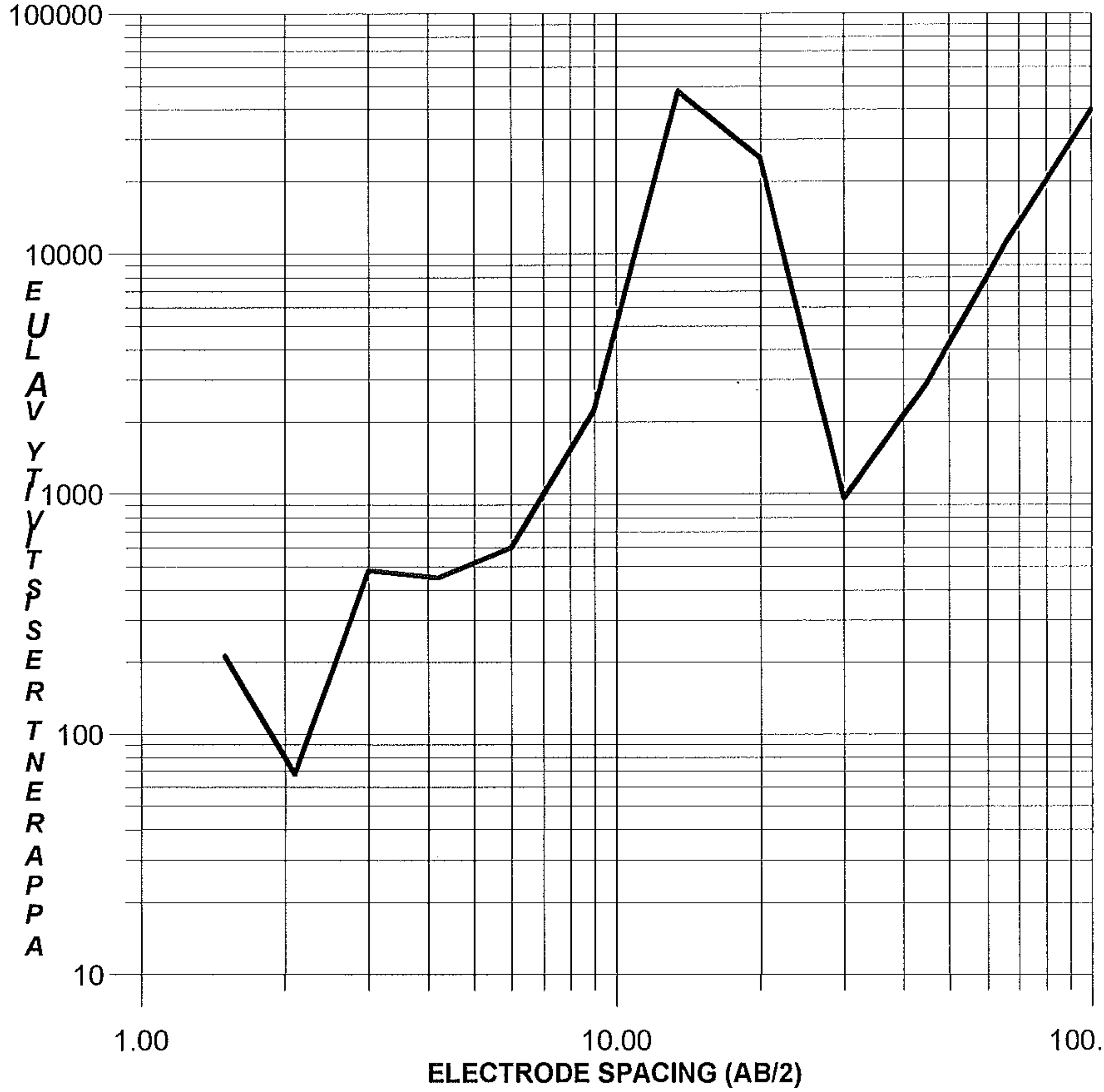
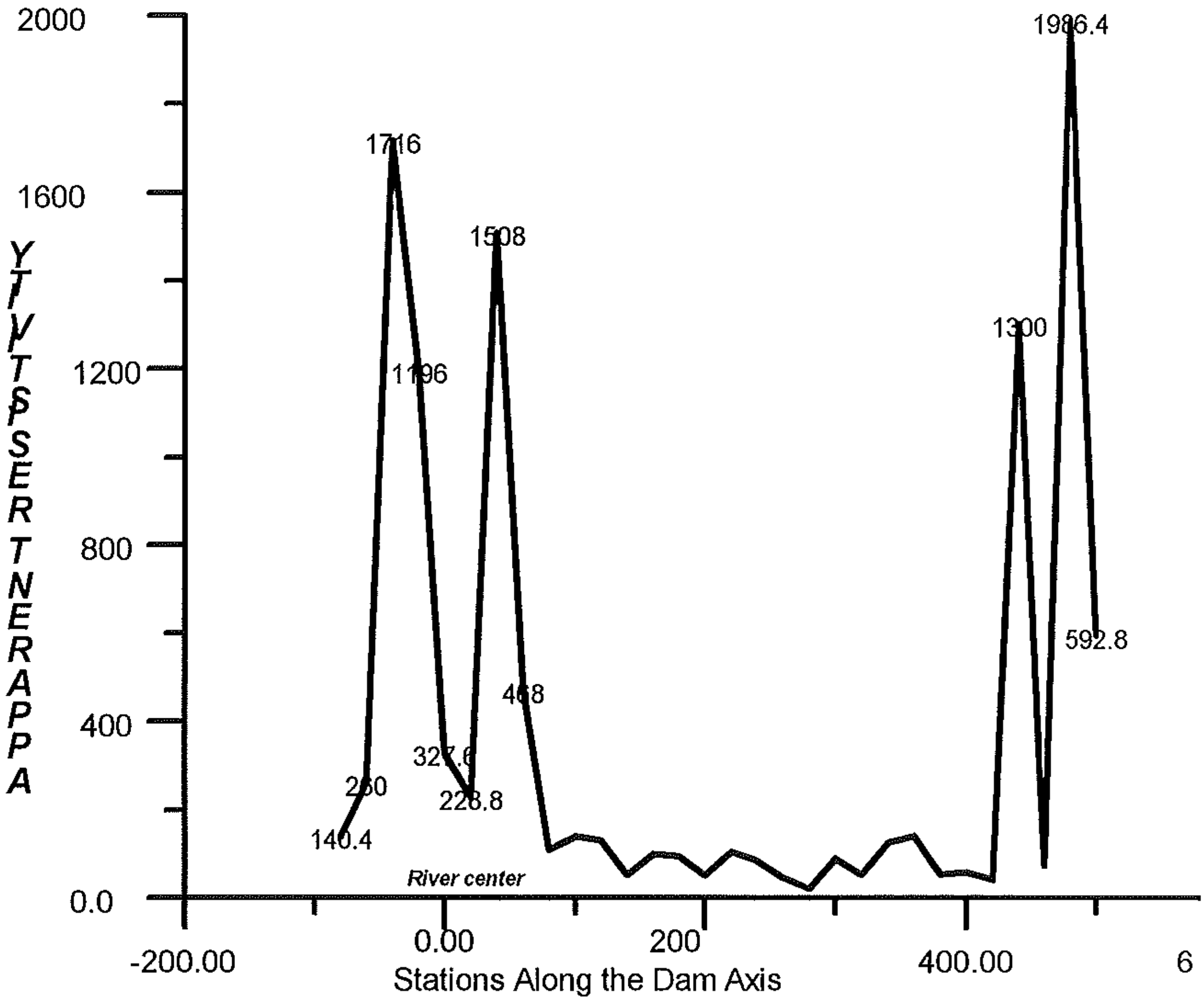


Figure 8.7 Apparent Resistivity Profiling Data Along the Dam Axis



Fig,(8.8) Apparent Resistivity Pseudosection Map of the Second Profile

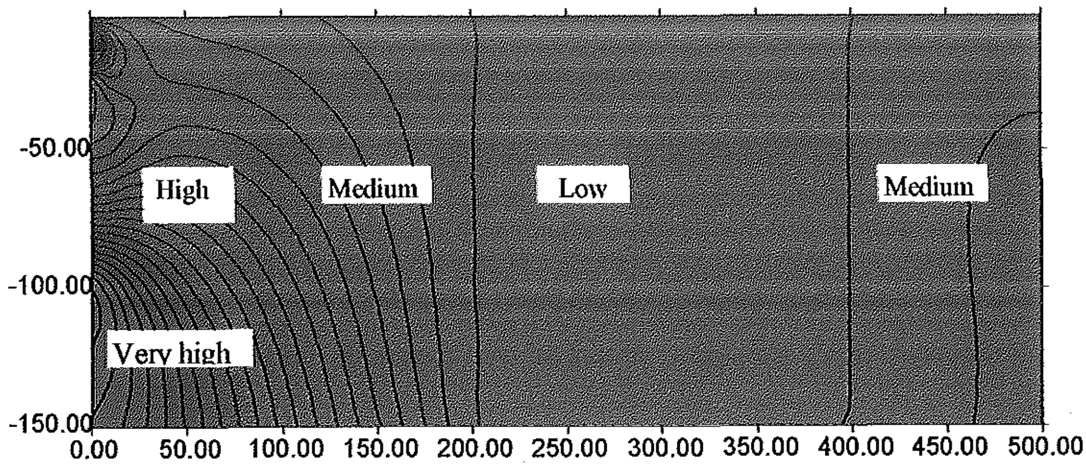
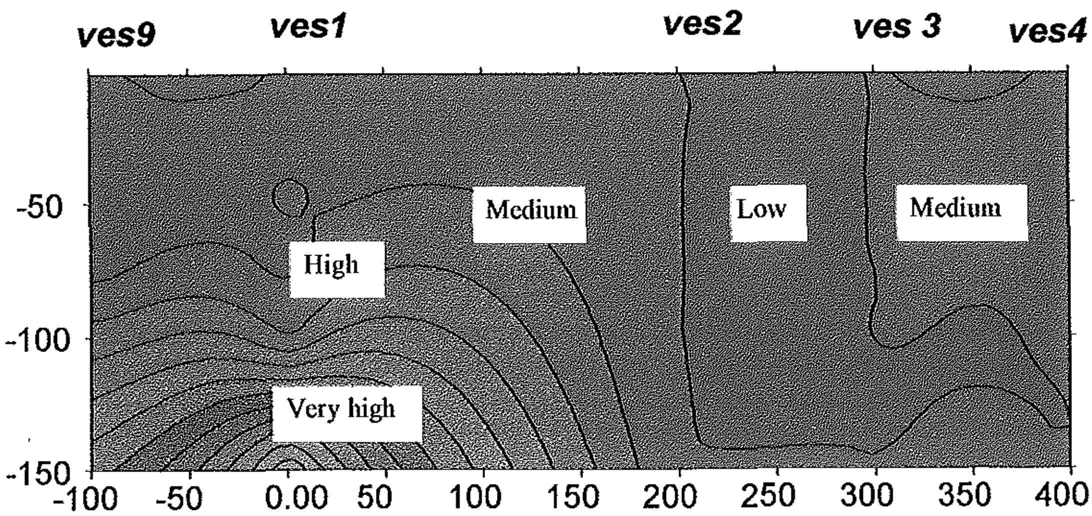


Fig. (8.9) Apparent Resistivity Pseudosection Map Along the Dam Axis



Chapter Nine

Construction Materials

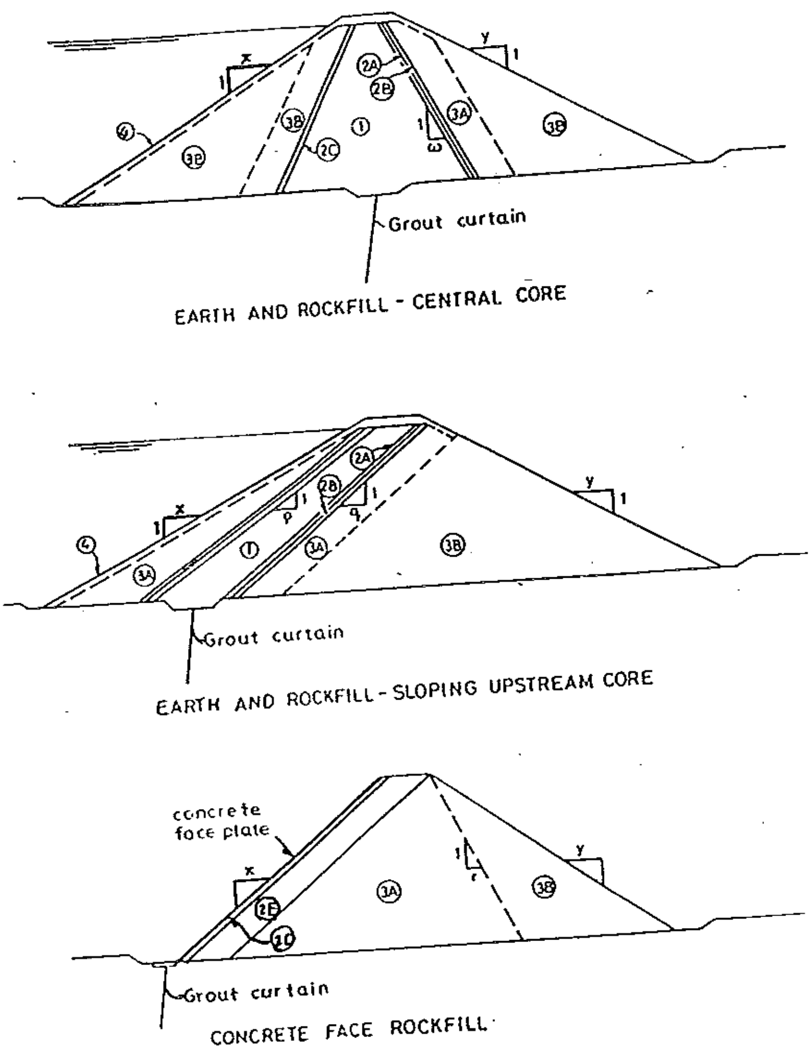
9.1 General

Frequently the immediate vicinity of an earthwork structure may not contain a sufficient variety of naturally occurring materials to permit the development of the structure except at excessive cost. In such instance it is often economical to obtain limited quantities of materials that have specially desirable characteristics from areas at a considerable distance from the site. Such materials include impervious soils for construction of a central core and blankets; sand and gravel for filters and drains, rock fragments for riprap, shell materials for diaphrams and water for compactions.

The availability of the suitable materials will determine the choice of the dame type and feasibility of the project. The study of construction materials include the quality, quantity, proximity, excavatability, accessibility, and environmental impact. If the required materials are found in ample quantities at the vicinity of the site, it is unnecessary to investigate a distant source. Taking the above considerations in to account, search for naturally occurring construction materials for this purpose has been done radially outwards from the project site. Embankment dam typical construction materials are given by Robin, (1992) (Table 9.1).

Zone	Description	Construction materials
1	earth fill	clay, sandy clay, clayey sand, silty sand, possible with some gravel. Usually more than 15% passing 75μm, preferable more.
2A	fine filter	Sand or gravelly sand, with less than 5%(preferable less than 2%) fines passing 75μm. Fines should be non plastic. Manufactured by crushing, washing, screening and recombining sand-gravel deposits and/or quarried rock
2B	coarse filter	Gravelly sand or sandy gravel, manufactured as for zone 2A. Zones 2A and 2B are required to be dense, hard durable aggregates with similar requirements to that specified for concrete aggregates. They are designed to strict particle grading limits to act as filters
2C	upstream filter and filter under riprap	Sandy gravel / gravelly sand, well graded, 100% passing 75mm, not greater than 8% passing 75μm, fines non-plastic. usually obtained as crusher run or gravel pit run with a minimum of washing, screening and regarding. relaxed durability and filter design requirements compared to zones 2A and 2B.
2D	fine cushion layer	Silty sandy gravel well graded, preferably with 2 - 12 % passing 75μm to reduce permeability. Obtained by crushing and screening rock or naturally occurring gravels or as crusher run. Larger particles up to 200mm are allowed by some authorities
2E	coarse cushion layer	Fine rock fill placed in 500mm layers to result in a well graded sand / gravel / cobbles mix which satisfies filter grading requirements compared to zone 2D
3A	rock fill	Quarry run rockfill, possibly with oversize removed in quarry or on dam. Preferably dense, strong, free draining after compaction, but lesser properties are often accepted. compacted in 0.5 to 1 meter layers with maximum particle size equal to compacted layer thickness
3B	coarse rock fill	Quarry run rockfill. preferably dense, strong, free draining after compaction, but lesser properties are often accepted. Compacted in 1 to 1.5 m layers with maximum particle size equal to compacted layer thickness
4	riprap	Selected dense durable rockfill sized to prevent erosion by wave action. In earth and rockfill dams often constructed by sorting larger rocks from adjacent 3A and 3B zones. In earthfill dams either selected rockfill or a wider zone of quarry run rockfill

Table 9.1 Typical construction materials of embankment dam



NOTES
1. Crest detailing and downstream slope protection not shown.
2. Scales relate to overall size, details are not drawn to scale.

Scale 0 10 20m

Fig. 9.1. Details Of General Earth And Rock Fill
And Concrete Face Rock Fill Dams

9.2 Investigation of rock source for riprap

9.2.1 General

A number of factors determine whether a rock will be worked as a riprap. These include the volume of material that can be quarried, the ease with which it can be quarried, the wastage consequent up on quarrying and the cost of transportation, as well as its appearance and physical properties. The ease with which a rock can be quarried depends to a large extent up on geological structures, notably the geometry of the joints and bedding planes, where present. The durability of a stone is a measure of its ability to resist weathering and so to retain its original size, shape, strength and appearance over an extensive period of time. The amount of weathering undergone by a rock in field exposures or quarries affords some indication of its qualities of resistance. Damage to stone by alternate wetting and drying is well known. What is more water in the pores of a stone, low tensile strength can expand enough when warmed to cause its disruption (Bell, 1993)

The stresses imposed upon masonry by expansion and contraction, brought about by changes in temperature and moisture content, can result in masonry between abutments spalling at the joints, blocks may even be shattered and fall out of place. The rate of weathering of silicate rocks is usually slow although once weathering penetrates the rock, the rate accelerates. Some basalts used on the continent have proved that they have deteriorated rapidly, crumbling after about 5 years of exposure. They have been referred to as sunburned basalts, it originally having been suspected that surface burning by solar radiation was responsible for their decay. However on petrological investigation these basalts were found to contain analcite, the development within which of micro cracks is now presumed to have produced the deterioration.

Exposure of a stone to intense heating causes expansion of its component minerals with subsequent exfoliation at its surface. the most suspect rocks in this respect appear to be those which contain high proportions of quartz and alkali feldspars. Indeed quartz is one of the most expansive minerals, expanding by 3.76% between normal temperature and 570^0 c (Bell, 1993)

Generally speaking finely textured rocks offer a higher degree of heat resistance than do coarse grained varieties and construction stone may be any locally available rock with unconfined compressive strength > 50Mpa

9.2.2 Description of the selected rocks

Rock source selection for riprap and masonry works has been performed based on the requirements mentioned so far. The search for rock sources for riprap and masonry was mainly towards the foot of hills in the upstream side of the dam site. From geological point of view, rocks in this area are variable in size and same in origin. The rocks observed along the rim of the reservoir are very hard and dense having all size ranges from cobble size to big blocks at depth. Genetically the cobbles and hills are basaltic igneous rocks. Most of the rocks are fresh angular to sub angular which will not be broken to pieces easily.

9.2.3 Location and accessibility of rock source

The proposed source for riprap and masonry is located some 500m in the north direction from the proposed dam axis in the left side of the reservoir rim. As can be seen from the location map a continuos hills around the reservoir and catchment area contains blocks and boulders of rocks that can be used for slope protection of the dam embankment and masonry works. There is no need to construct access road from dam site to the quarry site, due to the nearness of the quarry site and possibility of using the existing river course as an access road . Since access road was already constructed to the dam site and the topography is so suitable simple surface stripping will made the reservoir area easily accessible.

9.2.4 Extent of the deposit and proposed method of excavation

The total area of the hills expected to produce rocks for the intended purpose is estimated to be greater than $1km^2$. The depth is only an assumption since it was not possible to dig pits for depth estimation purposes. Therefore , if we assume a minimum depth of 1m which can produce good rock fragments using simple excavation methods then it is possible to obtain about 1 million m^3 rock fragments. All means of excavation and loading is possible at this site. Although dozer excavation and loading by machine is possible, collecting and loading by man power and carriage is the most economical since the required size boulders and cobbles are concentrated at the surface. As observed during the field work, no body uses the area as a source for construction rocks, private ownership over quarry sites in the area under consideration is not yet well known. Therefore, it seems possible to exploit rock fragments at the proposed source with out any charge, especially for this type of public benefit.

9.3 Borrow area for clay

9.3.1 General

Clay minerals, together with quartz, feldspar and mica, are the principal constituent of clays, shells and mudstone. The clay minerals represent the commonest decomposition products of the chief rock forming silicate minerals. The principal clay minerals belong to the kandite, illite and smectite. The kandites, of which kaolinite is the chief member, results from the weathering or hydrothermal alteration of feldspars, feldspathoids and some other silicate minerals. Ball clays are composed almost entirely of kaolinite and as between 70% and 90% of the individual particles are below 0.01 mm in size, these clays have a high plasticity. Their plasticity at times is enhanced by the presence of montmorillonite. Illite are important clay minerals in mudstones and shells. Like kaolinite, illite forms from the weathering of silicate minerals, notably feldspars, but it may develop from the alteration of other clay minerals, by the degradation of muscovite or from the crystallization of colloidal matter. Unlike kaolinite however, the development of illite is favored by an alkaline environment rather than acid conditions.

Montmorillonite is the principal member of the smectite group. As the presence of magnesium is essential for its formation, it develops when basic igneous rocks, particularly ashes and tuffs, suffer weathering or hydrothermal alteration. Drainage must be poor so that the magnesium is not removed. The formation of montmorillonite is favored by an alkaline environment. The plasticity of a clay is perhaps its most characteristic property and refers to its ability to be molded into shape with out fracturing and to maintain that shape when the molding action ceases. Plasticity for all practical purpose is the same as workability and is related to dry strength, particle size and cation exchange capacity..

Kaolinite and illite generally possess good working properties whilst montmorillonite is often exceedingly plastic. The non clay material may enhance the working properties of a clay deposit. Those clays which need large quantities of water to develop the required degree of plasticity suffer a high degree of shrinkage on drying. Montmorillonite increases the drying shrinkage of a clay whilst clays composed largely of kaolinite or illite have moderate drying shrinkage, unless they are exceptionally fine grained. Up to 40% non clay material, such as fine sand may be added to a raw clay to reduce its drying shrinkage. Clays are often refereed to as fat when they adhere in a plastic state, or lean, when they are friable and difficult to mold.

9.3.2 Source for inner core

Sources for inner core, the cut off trench and blanketing which can produce predominantly fines have been investigated. The selected site is immediately below the proposed dam axis in the right side. This soil layer is dark brawn silty clay. From field level identification the plasticity seems moderate. The depth of this unit varies from 1 - 2 m. It is located at 37^0 34' $20''$ east longitude and 5^0 21' $02''$ north latitude. It covers an area of over 5 hectares, taking the average thickness of 1.5 m the amount of clay expected at that local deposit will be; $4 \times 10^4 \times 1.5$ m = 6×10^4 m^3. According to the laboratory analysis, this soil is silty clay with some amount of sand and minor gravel inclusions. permeability coefficient of this layer is 1.14×10^{-7} cm/s.

The maximum dry density (1505kg/m^2) for the selected fill material occurs at an optimum moisture content 25.3% for a given amount of energy applied during the compaction process. The standard test developed to analyze this disturbed soil is proctor density test. The volume of required soils should be estimated by adding 20 - 30% to the calculated volume of earth work. But the exact amount of fill material required is not evaluated , since the design of the dam is on progress. Compaction test of this material is shown in figure 9.2 at TPI9 with 1m depth and natural moisture content of 9.53%

Maximum Dry Density (g/m^3) = 1505
Optimum Moisture Content (%) = 25.30

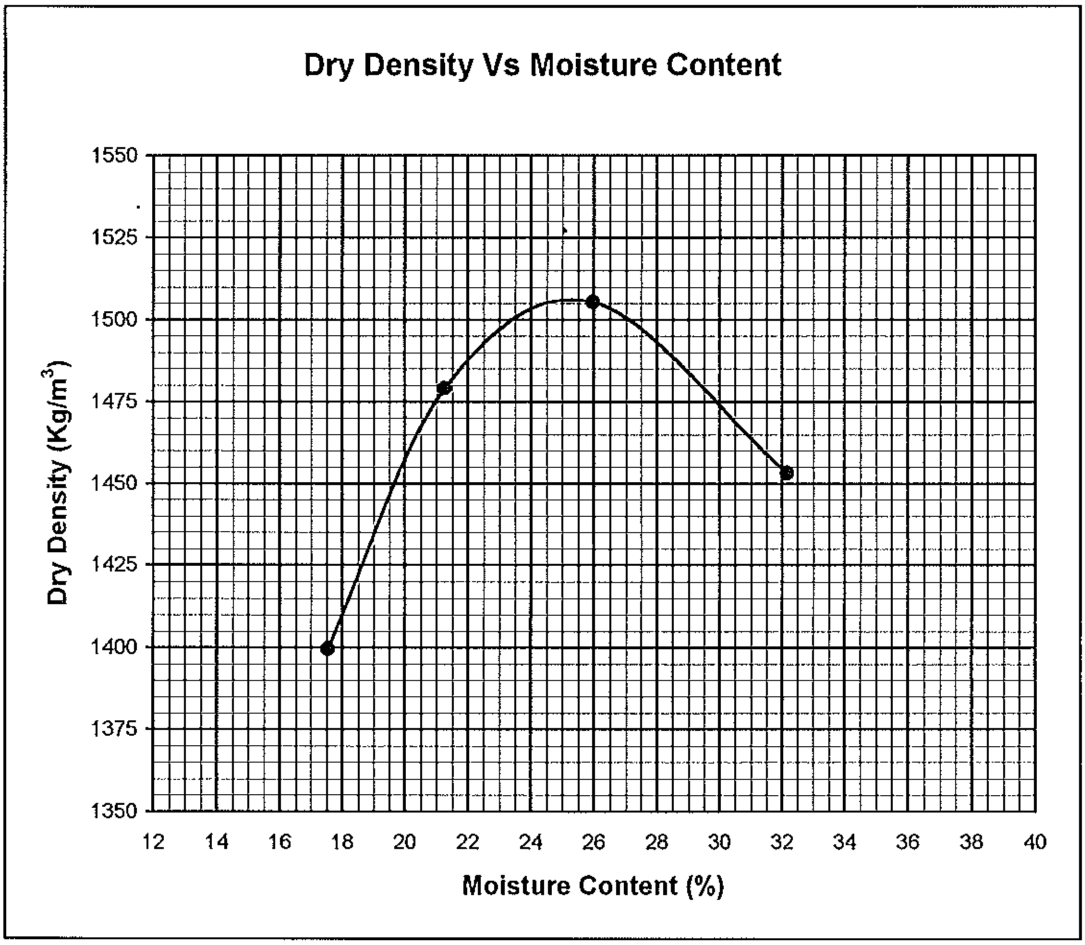


Figure 9.2 Graph of dry density versus moisture content

9.4 Sand Source

9.4.1 General

The textural maturity of sand varies appreciably. A high degree of sorting coupled with a high degree of rounding characterizes a mature sand. The shape of sand grains, however, is not greatly influenced by length of transport. Maturity is also reflected in their chemical or mineralogical composition and it has been argued that the ultimate sand is a concentration of pure quartz. This is because the less stable minerals disappear due to mechanical or chemical break down during erosion and transportation or even after the sand has been deposited. Sands are used for civil works to give bulk to concrete, mortars, plasters, renderings and for filters. For example, sand is used in concrete to lessen the void space created by the coarse aggregate. A sand consisting of a range of grade sizes gives a lower proportion of voids than one in which the grains are of uniform size. Indeed grading is probably the most important property as far as the suitability of a sand for concrete is concerned. In any concrete mix, consideration should be given to the total specific surface of the coarse and fine aggregates, since this represents the surface which has to be lubricated by the cement paste to produce a workable mix. It is alleged that generally a sand with rounded particles produces a slightly more workable concrete than one consisting of irregularly shaped particles.

Sands used for civil works are usually siliceous in composition and should be as free from impurities as possible. They should contain no significant quantity of silt or clay (less than 3% by weight ; in coarse aggregate it should not exceed 1%), Since these need a high water content to produce a workable concrete mix. This in turn leads to shrinkage and cracking on drying. Furthermore clay and Shelly material tend to retard setting and hardening, or they may spoil the appearance. If sand particles are coated with clay they form a poor bond with cement and produce a weaker and less durable concrete. The presence of feldspars in sands used in concrete has sometimes given rise to hair cracking and mica and particles of shale adversely affect the strength of concrete. Organic impurities may adversely affect the setting and hardening properties of cement by retarding hydration and there by reduce its strength and durability.

The basic requirements of filters are that they are: sufficiently fine grained to prevent erosion of the soil they are protecting , and sufficiently permeable to allow drainage of seepage water. The basic requirements could best be met by designing the particle size of sand and gravel filters in relation to the particle size of the soil being protected. It is recommended the following approach be adopted for design, of critical filters: (Sherard and Dunnigan ,1985) except that for all base soils, the filters sizing should be based on the grading of that part of the soil finer than 4.76 mm. The filters should not contain more than 5% fines passing 75 micron, and the fines should be non plastic. Where high permeability is required, not more than 2% fines passing 75 micron should be allowed. This would be particularly important for vertical and horizontal drains. The uniformity coefficient $D_{60}f$ / $D_{10}f$ should not exceed 20 ($D_{60}f$ on coarse limit of filters, $D_{10}f$ on fine limit of filter) (Robin, 1992). For major projects, particularly those involving dispersive soils, no erosion filters should be carried out using water with the same chemistry as the expected seepage water.

9.4.2 Sand for the proposed project.

Sand for the proposed structures can be collected along the main river coarse both upstream and down stream of the selected dam axis. So there is no need to transport sand from distant source. As to the suitability of sand is concerned both on spot test and laboratory classification were carried out and found to be very good. To test the suitability of sand; sand sample was placed in a vessel containing water and agitated until the sand is thoroughly in suspension.

When the sand has been allowed to settle, the water was poured off slowly. If sand is of good quality it will not be carried with water, but will remain in the vessel until practically all the water has been drained off. Based on this test sand of the Iyanda river course is of very good quality composed of quartz and not carried out with water. Based on the laboratory analysis of sand for filter the amount of impurities is 5%, so the selected material has a good quality.

9.5 Gravel Source

9.5.1 General

Gravel deposits usually represent local accumulations, such as channel fillings. In such instances they are restricted in width and thickness but may have considerable length. A gravel deposit consists of a framework of pebbles between which are voids. The voids are rarely empty, being occupied by sand, silt or clay material. River gravels are notably bimodal, the principal mode being in the gravel grade, the secondary in the sand grade. The shape and surface texture of the pebbles in a gravel deposit are influenced by the agent responsible for its transportation and the length of time taken in transport, although shape is often more dependent on the initial shape of the fragment, which intern is controlled by the fracture pattern within the parent rock. The shape of gravel particles is classified as rounded, irregular, angular, flaky and elongated. The composition of a gravel deposit reflects not only the type of rocks in the source area but it is also influenced by the agents responsible for its formation and the climatic regime in which it was or is being deposited. Generally suitable aggregates are of igneous and less commonly metamorphic origin.

Furthermore relief influences the character of a gravel deposit, for example, under low relief gravel production is small by contrast high relief and rapid erosion yield coarse, immature gravels. All the same a gravel achieves maturity much rapidly than does a sand under the same conditions. Gravels which consist of only one type of rock fragment are called oligomistic and such deposits are usually thin and well sorted. Polymistic gravels usually consist of a varied assortment of rock fragments and occur as thick, poorly sorted deposits. Gravel is much more extensively used in concrete than crushed rock, their ratios being about 5 to 1. This is mainly because gravel is cheaper to produce. Based on aggregate trade group classification strong and fine grained basalt are of good quality.

9.5.2 Gravel for the proposed dam

Filters are required to be constructed of sand and gravel which is sufficiently durable so as not to break down excessively during the mechanical action of placement in the dam, under the chemical action of seepage water, or under wetting and drying within the dam. Gravels for filter and concrete can be collected manually from the existing Iyanda river course. It is well graded, dark, rounded to sub rounded, bimodal fresh and basaltic composition. In general gravels of the required quantity and quality can be collected easily. Based on aggregate trade group classification the basaltic origin gravel of the study area have good quality.

9.6 Water for compaction

There is a critical problem of water supply near the project area. There is no water for a radius of 15 km from the project area. Whatever type of water is present around the project site, they use it for all purposes. The only source of water near the project area is Segen river found south of the site some 15 km from the dam site. The river is not perennial, it dries during December to March. So artificial pond is recommended to collect water during the flow period for the use in compaction, provided it is economical.

Chapter Ten

Environmental Impact Studies for Dams Reservoirs and Command area.

10.1 General

Environmental impact studies are typically required on large dams, hydroelectric and irrigation projects. These studies are an outgrowth of the requirements of international funding agencies and in-country lows. While economic and political decisions are involved in the planning and implementation of reservoir projects, it is imperative that environmental impact studies be conducted from a scientific view point, including the use of current technical information on types of impacts , planning and conduction of baseline studies, impact prediction and assessment, and methodologies for trade - off analysis and decision making.

Identification of potential impacts should be an early activity in an environmental impact study. General knowledge about the types of impacts which could occur can be used in identifying potential impacts for new projects. Beneficial impacts include protection against floods and droughts; while detrimental impacts have resulted from siltation of reservoir, loss of routine silt deposits on agricultural lands, changes in water quality, and public health concerns related to disease transmission. Dams produce a permanent physical transformation, inundating settled areas and destroying habitats, affecting the ground water regime and water table, possibly increasing seismic tendencies, and often leading to explosive aquatic weed growth and the spread of schistosomiasis and other communicable diseases. Dams in tropical areas tend to favor weed propagation and vectors of parasitic diseases (Canter, 1985). Systematic approaches to aid in identifying potential impacts include the use of checklists, case studies, interaction matrices, and networks. Some checklist of potential impacts and issues from impoundment is discussed in Table 10.1

Category	Potential impacts and Issues

Construction Phase	Sediment pollution and stream siltation
	pesticides, petrochemicals, and other potential pollutants
	Quantification of erosion and sediment generation
	Protection of water quality during construction general
	  Erosion and sediment control techniques
	  Treatment of polluted water from construction site
	Noise generators at impoundment construction site

Impoundment Area	Probable land use impacts General methodology for evaluating land use changes and impacts Loss of stream and bottom land Relocation impacts Impact of land inundation on impoundment water quality Consideration of evaporation Shift from river to lake environment and reduction of species diversity Sedimentation in impoundment Modeling of impoundment water quality Estimating significance of site conditions with respect to impoundment water quality Summary of water quality parameters that may be affected by impoundment and relevant criteria Changes in point and non point pollution sources

Downstream and areas of water use	Influence of land acquisition policy on reservoir development
	Induced development in region
	Land use impacts due to increased flood protection
	Land use impacts of irrigation impoundment's
	Evaluation of water pollution from irrigation
	Policy concerning use of flood plains
	Prevention of water quality degradation from irrigation projects
	Impact of water quality changes on downstream biota
	Flow regime changes - general
	  Quantification of hydrographic modification
	  Seasonal and diurnal flow variations
	  Minimum release requirements

Table 10.1 Some checklist of potential impacts and issues from impoundment (Canter, 1985)

10.2 Description of existing Environmental condition

This section briefly describes the existing condition of the environment in which the proposed project is situated. Basically the potential environmental effects of the proposed project were assessed and analyzed against the background of the present situation.

Iyanda river is the main source of water for spate irrigation in the project area. The local communities are largely dependent on this river which brings the flood from the highland areas. The result of water quality analysis of water sample collected from the river shows that the chemical and physical composition of the river water is generally good and fit for irrigation. There fore, application of the river water for irrigation will not pose high salinity problem.

The main prevalent diseases affecting the health of the population around the project area include; malaria, urinary tract infections, intestinal parasites, diarrhea diseases and respiratory tract infections (woreda health center). Malaria is the most prevalent water related vector- borne disease and is the leading cause of morbidity in the woreda in general and in the project area in particular. The project area lies within malaria zone where it occurs throughout the year with seasonal peaks mostly associated with the rains.

Shortage of safe drinking water is a major problem for the population around the project. The seriousness of bad water supply is reflected in the disease pattern. The drinking water and sanitary condition around the project area is generally poor.

10.3 Impact Prediction

The most important technical activity in an environmental impact study for a dam/ reservoir project is the scientific prediction of the effects of project construction and operation. Prediction of the impacts of large dams and hydroelectric projects can be based on: (1) a qualitative approach which relies on general knowledge of the impacts of similar projects, or specific results of comprehensive studies of similar projects, (2) a quantitative approach based on the use of simple mass balance and environmental dilution calculations; and (3) a quantitative approach based on the use of mathematical models for multiple environmental factors. A given environmental impact study will probably involve all three approaches to some degree.

Irrigation projects have been developed through out the world in semi-arid and arid regions. A semi arid region can be defined as one having a dry season of three to four months, regardless of the total annual precipitation . An arid region can be taken as one in which the usual dry season will last longer than four months, and sometimes the whole year. Semi arid regions need supplementary or seasonal irrigation, whereas arid regions must be designated for perennial irrigation. (Ahmad, 1982, sited in Canter, 1985) noted that the impact of irrigation projects is not exclusively beneficial, and when assessing the viability of such a project the adverse consequences must also be taken into consideration. The consequences can be most damaging to the fragile and delicately balanced ecosystems that characterize the arid and semi-arid regions (Barton,1976).

Regional lack of moisture is a critical environmental constraint to the study areas agro-ecosystem. The climate ranges from arid to semi arid and evaporation exceeds precipitation nearly by up to 2 times.

One of the chief concerns associated with the irrigation projects is the potential for spreading water born diseases for example, schistosomiasis and malaria. As in many water resources development projects in tropical areas the study areas may have had the unintended consequence of markedly increasing the prevalence of schistosomiasis in the local population. Water contact is the most critical variable in the transmission of schistosomiasis. Irrigation of formerly arid regions creates additional habitats for the snail vectors beyond those already present in ponds and rivers. Defects in the design or engineering of water projects may also provide new habitats for the snail hosts. The advent of perennial irrigation, in which the irrigation canals are in use year round, has facilitated the multiplication of the snail vectors of schistosomiasis. Perennial irrigation has removed natural checks on the snail population, enabling them to multiply out of control. Once snails have been introduced in to an irrigation system it is impossible to eradicate them completely.

However, if the proposed irrigation projects are supplied with efficient drainage, good water management and regular maintenance, it is possible to avoid an increase in the incidence of schistosomiasis. There are opportunities for design and operation schemes to minimize the potential health impacts of the study irrigation projects. Vector control measures which break the three part cycle (sick carrier, transmitting vector, and recipient), can be accomplished by proper engineering to produce a habitat unfit for snails and mosquitoes.

Some suggestions for destroying habitats for disease vectors are: controlling vegetation, straightening banks, producing frequent changes in water levels, reducing evaporation, eliminating west water influx, installing snail screens, using closed channels, sprinkling irrigation water, and choosing crops which do not need flooding. Operation and maintenance of the project should be regularly surveyed by teams of ecologists, environmental engineers, entomologists, and health workers. The other environmental impact of the proposed dam together with different ponds on construction can change the weather condition and increase the humidity of the area so that evaporation may decrease.

As to the probable land use impacts of impoundment area is concerned, the potential land used by the farmers can be relocated to the dawn stream side since there is no shortage of agricultural land in the dawn stream both in quantity and quality(fertility's), so there is no relocation impacts that hinders the project construction. There is no settlement in the reservoir area and near by it for more than 8km distance. The intensity of malaria transmission could be significant but not sever as the settlement are located at some distance from the project site. With increased distance from vector breeding sites the rate of transmission will reduce as the human/vector contact is reduced since only a fraction of the mosquito population will reach the settlement areas.

Prevention and control measures will include:

- Proper design, construction and maintenance of water supply channels so that they will have water flow rate of 0.4m/s or greater to prevent vector production;
- Design and construction of drainage system to ensure effective drainage of excess water;

- Implement efficient and economical water application;
- Regular maintenance of canals and drains to remove weeds, silt and irregularities of the bank
- Use of disease prophylaxis, and treatment of all confirmed positive cases with appropriate drugs

10.4 Some Environmental management and monitoring plan

It will be necessary to establish and maintain environmental monitoring procedure to provide early warning of any incipient problems both during construction and operation. Monitoring of environmental parameters will timely signal potential problems from the development scheme and will allow for prompt implementation of effective corrective measures. Due to limited resources monitoring will be scooped to those indicators which are most relevant.

1. Soil: relevant soil characteristics, namely PH, EC, soluble salts(Na^+,K^+, Ca^{+2}, Mg^{+2}, Cl^- $CO3^{-2}$, HCO_3^-, and SO_4^{-2}) in soil profile and water - logging can be monitored in selected areas twice a year, once before start of the irrigation and second after the crop is harvested. This will give the impact of irrigation on soils.
2. Water Quality monitoring: is required to see if the irrigation scheme would bring any adverse effects on the quality of the water resources of the area. The water quality data collected at the Iyanda river is used as baseline data for monitoring the effects of the development scheme. Three sampling sites are proposed:

I) Iyanda reservoir site;

II) Drain (canal water) before returning to Iyanda river

III) Iyanda river after mixing with drain water

3. Incidence of water related diseases: malaria monitoring will be based upon the collection of data on positive cases and number of slides examined. For schistosomiasis snail survey has to be carried out especially where there is low velocity of flow. If snails are observed, they should be collected and checked for schistosoma parasites. The condition of water supply and incidence of water-born diseases should also be monitored.

Chapter Eleven

Conclusion and Recommendation

The geological units mapped are: amphibole gneiss, meta/syntectonic gabbro, Tertiary basalt and Quaternary sediments at the catchment, while, at the dam site Gravel and sand at the river, clay and silt at the flat plain area, well-graded soil at the right abutment and slightly fractured basaltic rocks at the left abutment are the mapped lithological units. The geology of the side channel spill way, selected at the left side is a continuation of the left abutment, which is stable, slightly weathered and fractured basalt. The main canal lithology is fine grained clastics (clay and silt) that is impermeable lithological unit with permeability coefficient of about 0.7×10^{-7}cm/s.

From seismic map produced for the country the site lies within an area having 20% ground accelaration and VIII intensity within 100 years return period and probability of 0.01 per annum of being exceeded and at zone 4 with corresponding major damage.

Hence the expected earthquakes impose additional loads on the would be embankment dams and can affect by causing any of the following:

- Settlement and cracking of the embankment;
- Reduction of the freeboard due to settlement, which may in the worst case, result in overtopping of the dam;
- instability of the upstream and down stream of the dam;
- differential movement between the embankment, abutments and spillway structures, increasing the likelihood of leakage and piping failure;
- liquefaction or loss of shear strength in the embankment and its foundations due to increase in pore pressures induced by the earthquake;

- damage to outlet works passing through the embankment leading to leakage and potential erosion of the embankment.

The unconfined compression strength of the representative sample (TPI4) which would be the foundation of the dam is $86KN/m^2$ with strain at failure 0.012. So the undrained cohesion or(undrained shear strength) C_u = $86KN/m^2/2 = 43KN/m^2$. The allowable bearing capacity of sands of the river course is 8000Psf. From field level investigation of the foundation rock the safe bearing pressures of the studied area is 10Mpa.

The permeability coefficient computed from the consolidation test of the foundation clayey silt soil is 0.7×10^{-7} cm/s corresponding to very low permeability. The total allowable settlement is limited to 50mm and 75mm on sandy and clayey soils respectively ,EBCS(1995). Coefficient of consolidation of the study sample varies between 5.36 and $15.61m^2$/year within the given range of pressure. The t_{90} (90%) settlement values of the study foundation soil varies between 0.75mm to 7mm for the loading of 25Kpa to 3200Kpa. Since the settlement value is less than the Ethiopian building code of standard, constructing the proposed structure on this soil is safe within the given range of pressure.

Slides in clay soils could be due to reduced shear surface to residual strength with little or no cohesion. There fore stable slopes attain close to $(\phi)_r/2$ in saturated soils. Clays are generally unstable at > 10° roughly (ϕ /2). The right abutment and reservoir rim in the study area is composed of a well graded soils that have high shear strength than clay soils with slopes of less than 7°. So this side of the reservoir rim is stable in general. As to the stability of the left abutment and reservoir rim is concerned, it is stable both during submerged and dry conditions, since it is composed of basaltic rock unit with low degree of weathering and fracturing.

The VES and profiling investigation show that the over burden (soil) is thick at the flat plain areas and thin at the abutments. It is grater than 100m at the flat plain areas, about 35m at the right abutment and near to the surface at the left abutment.

In the area the prevailing dry tropical climatic condition which have insufficient rain fall and the insufficiency of the coming flood to sustain crop growth to maturity resulted in the occurrence of both short and long periods of drought for successive years which have spread effects from marked yield reduction to total crop failure, vegetation decline and grazing land reduction. To overcame the shortage of moisture and bring the soil to field capacity farmers collect storm runoff by constructing simple ditches and soil bounds.

From the piper trilinear plot, the waters in the study area are classified in to two water types. The spring and river samples are calcium-Bicarbonate type and the Lakes and borehole are Sodium- Potassium Bicarbonate type. Since the the study area borehole water samples and lake Chamo are similar water type, the surface water divide of the study area may not coincide with the ground water divide.

All natural construction materials are found within 1km distance and fulfill both the quality and quantities required for the selected dam. The permeability coefficient of core and sand sample is 1.14×10^{-7} cm/s and 9×10^{-2} cm/s respectively. The impurity of filter sand is 5%.

Based on the study of the project the following recommendations are forwarded

Both the index properties and engineering properties of the dam site shows that the clayey silt soil and well graded soil at the right abutment are stable and water tight. So the foundation depth should be on this layer. But at the main river and its levees, no watertight unit was mapped to 7m depth. The proposed earth dam should be founded on soil over burden (clayey silt) and rock unit. The foundations at the river and its levees are pervious , so this pervious units has to be blanketed to the fiche length following the river and its levees or cut off trench should extend to impervious bed rock which is about 13.5m from the VES1 data.

To render the effects of earth quack to non harmful the following defensive measures are recommended:

- allow ample freeboard to allow for settlement and slumping
- use wide transition zones (filters) of material not vulnerable to cracking
- provide ample drainage zones to allow for possible flow of water through cracks
- use a well graded filter zone upstream of the core to serve as a crack stopper
- provide crest details which will prevent erosion in the event of overtopping

At rocky foundation site (left abutment) the face shall be cleaned of all loose fragments, and proper bond shall be established between the embankment and the rock surface prepared , key trenches may be provided while constructing the dam.

For soil foundation part (right abutment) the alignment of the cut off trench should be fixed in such a way that its center line is within the base of the impervious core (clayey silt). At the abutment contacts of the cut off trench, care shall be taken to avoid seepage by out flanking.

It is recommended to provide 1 - 2% embankment height on the dam to accommodate embankment compression and foundation settlement. The finished surface of riprap should be convex towards water side, so that in case of settlement the individual stones will be under compression and press against each other.

A good vegetative cover over the catchment area, is the best way of reducing sedimentation in the study area. However, the success of catchment protection depends both on climate and land management. Since sediment trap is principally designed to catch coarse sediments and the sediment at the reservoir area is of suspended type there is no need of constructing sediment trap, rather diversion of sediment laden water around a reservoir and also to diminish reservoir sedimentation, discharging water through sluices in the dam can partially reduce sediment accumulation.

Inorder to understand the flow direction system, Isotopic analysis has to be done to understand the relation of the catchment with lack Chamo.

Relevant soil characteristics, namely PH, EC, soluble salts(Na^+,K^+, Ca^{+2}, Mg^{+2}, Cl^- $CO3^{-2}$, HCO_3^-, and SO_4^{-2}) in soil profile and water - logging should be monitored in selected areas twice a year, once before start of the irrigation and second after the crop is harvested. This will give the impact of irrigation on soils. Water quality monitoring is recommended to see if the irrigation scheme would bring any adverse effects on the quality of the water resources of the area. The water quality data collected at the Iyanda river is used as baseline data for monitoring the effects of the development scheme.

Although the clay minerals found in the study area varies from illite to kaolinite with corresponding moderate to low dispersion, it is recommended to carry out dispersion test for quantitative interpretation.

Since the samples analyzed are limited in number and cross checking two to three laboratory data is required for the accuracy of interpretation, further analysis of undisturbed samples has to be carried out. Due to high temperature and evapotranspiration of the study area detail hydrological analysis should be carried out for the safe design of the spill way.

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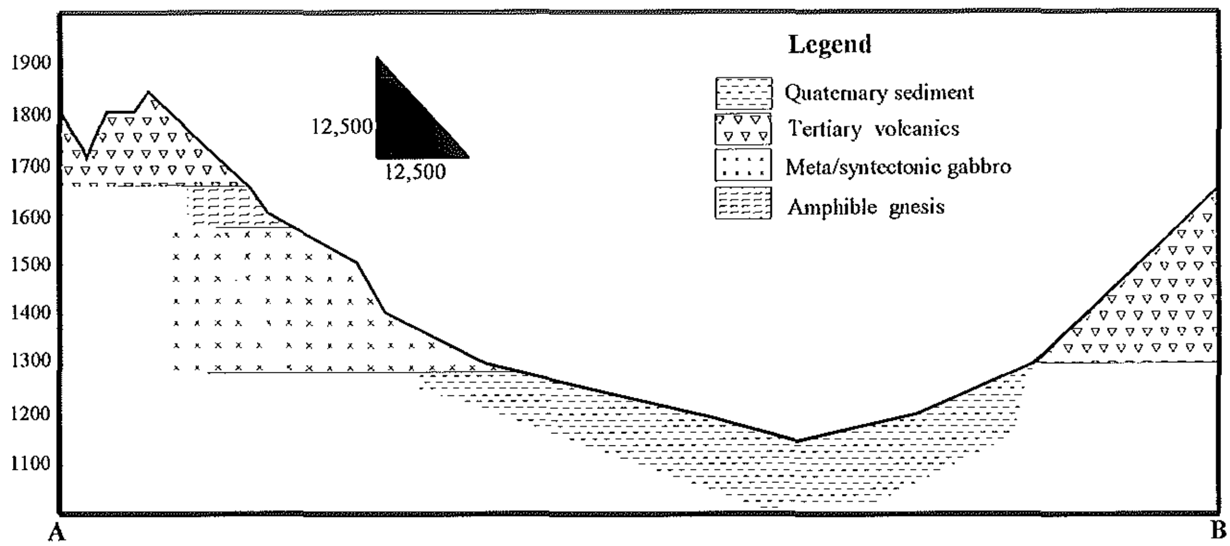
Annex 1 Geological cross-section along A - B of figure 3.1



Annex 2

Source	Na	K	Ca	Mg	Scat	HCo3+ Co3	Cl	So4	San	Po4	No3+ No2	F	Cond.	T.Hard	PH	T.D.S
Gato(sp.)	1.03	0.09	1.20	0.62	2.94	2.78	0.14	0.00	2.92	0.01	0.03	0.02	288	92	7.90	144
Gato(B.H.)	9.41	0.25	1.48	2.02	13.16	10.25	0.14	0.42	10.81	0.02	2.42	0.05	722	180	7.98	361
Onota(B.H)	8.88	0.13	2.94	2.72	14.67	7.98	0.11	0.80	8.89	0.01	0.04	0.04	869	290	8.18	434
Holte(B.H)	10.61	0.05	1.18	0.31	12.15	11.77	0.25	0.04	12.06	0.01	0.10	0.02	1313	76	8.30	648
L.Chamo	11.52	0.36	0.11	0.31	12.30	9.40	1.83	0.06	11.29	0.002	0.03	0.10	1557	60	8.08	778
Iyanda(Riv)	0.99	0.10	3.00	0.11	4.20	4.10	0.03	0.26	4.39	0.01	0.00	0.01	496	156	7.05	248

Major cathions and anions in meq/lit and conductivity in μS/cm

Annex 3

ONE DIMENSIONAL CONSOLIDATION, DATA AND CALCULATION SHEET

TPI 4

Depth: 2.00

BEFORE TEST

Weight of sample & Ring (gm)	180.00
Weight of Ring (gm)	132.05
Weight of sample (gm)	47.95
Weight of Dry Sample (gm)	38.50
Weight of Initial Moisture (gm)	9.45
Initial Moisture content (M_o) (%)	24.55
Initial Void Ratio : $e_o = (G_s/\rho_d)-1$	1.672
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)	7.484
Initial Saturation : $S_o = (M_o*G_s)/e_o$ (%)	38.45

Specimen data

Specific Gravity (G_s)	2.62
Sample Diameter (D) (mm)	50.00
Sample Thickness (H_o) (mm)	20.00
Area (A) (cm^2)	19.63
Volume (V) (cm^3)	39.27
Bulk Density (ρ_b) (gm/cm^3)	1.221
Dry Density (ρ_d) (gm/cm^3)	0.980

AFTER TEST

Weight of Sample + Ring +Can (gm)	249.00
Weight of Dry Sample + Ring +Can (gm)	236.50
Weight of Ring + Can (gm)	198.00
Weight of Wet Sample (gm)	51.00
Weight of Dry Sample (gm)	38.50
Weight of Final Moisture (gm)	12.50
Final Moisture Content (M_f) (%)	32.47
Final Saturation : $S_f = (M_f*G_s)/e_f$: (%)	121.40

Over all settlement (mm)	7.272
Volume change (cm^3)	14.28
Final Volume (cm^3)	24.99
Final Bulk Density (gm/cm^3)	2.041
Final Dry Density (gm/cm^3)	1.541
Final Void Ratio:(e_f)	0.701

ONE DIMENSIONAL CONSOLIDATION TEST LOAD - TIME SETTLEMENT READINGS

Sample diameter (D)				50.00 mm Area(A)	mm	Area (A)	19.63 cm^2		Sample thickness (H)	(H) 20.0		mm
Lever ratio				10	1	1Kg on hanger gives pressure of :					50	Kpa

Loading

Increment N°					1	2	3	4	5	6	7	8
Date started					8/22/93	8/23/93	8/24/93	8/25/93	8/26/93	8/27/93	8/28/93	8/29/93
Load				Kg	0.5	1	2	4	8	16	32	64
Pressure				Kpa	25	50	100	200	400	800	1600	3200
Elapsed time												
h	m	s	t (min)	4t	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH
		0	0	0	0.000	0.902	1.817	2.602	3.435	4.588	5.502	6.712
		6	0.1	0.32	0.738	1.286	2.050	2.868	3.880	4.990	5.990	7.010
		10	0.17	0.41	0.742	1.296	2.056	2.874	3.886	4.996	5.996	7.014
		15	0.25	0.50	0.748	1.300	2.060	2.880	3.890	5.000	6.000	7.020
		30	0.5	0.71	0.754	1.344	2.148	2.942	4.020	5.048	6.080	7.080
	1		1	1	0.760	1.376	2.210	2.994	4.100	5.206	6.181	7.126
	2		2	1.41	0.767	1.416	2.256	3.054	4.165	5.290	6.262	7.166
	4		4	2	0.775	1.456	2.298	3.100	4.256	5.320	6.310	7.210
	8		8	2.83	0.788	1.499	2.343	3.146	4.296	5.345	6.377	7.260
	15		15	3.87	0.800	1.545	2.376	3.180	4.320	5.365	6.435	7.294
	30		30	5.48	0.810	1.610	2.409	3.233	4.355	5.392	6.491	7.336
1			60	7.75	0.826	1.658	2.445	3.280	4.400	5.410	6.521	7.380
2			120	10.95	0.846	1.706	2.490	3.320	4.442	5.438	6.561	7.428
4			240	15.49	0.866	1.758	2.526	3.380	4.480	5.460	6.610	7.462
8			480	21.91	0.880	1.787	2.578	3.415	4.530	5.486	6.670	7.510
24			1440	37.95	0.902	1.817	2.602	3.435	4.588	5.502	6.712	7.540
48			2880	53.67								
72			4320	65.73								
Net cumulative compression (ΔH)					0.902	1.817	2.602	3.435	4.588	5.502	6.712	7.540

ΔH = Coumulative compersion

ONE DIMENSIONAL CONSOLIDATION TEST LOAD - TIME SETTLEMENT READINGS

Sample diameter (D) 50.00 mm Area(A)	mm	Area (A) 19.63 cm^2	Sample thickness (H) (H) 20.0	mm
Lever ratio	10	1	1Kg on hanger gives pressure of : 50	Kpa

Loading

Increment N°					1	2	3	4	5	6	7	8
Date started					8/22/93	8/23/93	8/24/93	8/25/93	8/26/93	8/27/93	8/28/93	8/29/93
Load				Kg	0.5	1	2	4	8	16	32	64
Pressure				Kpa	25	50	100	200	400	800	1600	3200
Elapsed time												
h	m	s	t (min)	4t	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH
		0	0	0	0.000	0.902	1.817	2.602	3.435	4.588	5.502	6.712
		6	0.1	0.32	0.738	1.286	2.050	2.868	3.880	4.990	5.990	7.010
		10	0.17	0.41	0.742	1.296	2.056	2.874	3.886	4.996	5.996	7.014
		15	0.25	0.50	0.748	1.300	2.060	2.880	3.890	5.000	6.000	7.020
		30	0.5	0.71	0.754	1.344	2.148	2.942	4.020	5.048	6.080	7.080
	1		1	1	0.760	1.376	2.210	2.994	4.100	5.206	6.181	7.126
	2		2	1.41	0.767	1.416	2.256	3.054	4.165	5.290	6.262	7.166
	4		4	2	0.775	1.456	2.298	3.100	4.256	5.320	6.310	7.210
	8		8	2.83	0.788	1.499	2.343	3.146	4.296	5.345	6.377	7.260
	15		15	3.87	0.800	1.545	2.376	3.180	4.320	5.365	6.435	7.294
	30		30	5.48	0.810	1.610	2.409	3.233	4.355	5.392	6.491	7.336
1			60	7.75	0.826	1.658	2.445	3.280	4.400	5.410	6.521	7.380
2			120	10.95	0.846	1.706	2.490	3.320	4.442	5.438	6.561	7.428
4			240	15.49	0.866	1.758	2.526	3.380	4.480	5.460	6.610	7.462
8			480	21.91	0.880	1.787	2.578	3.415	4.530	5.486	6.670	7.510
24			1440	37.95	0.902	1.817	2.602	3.435	4.588	5.502	6.712	7.540
48			2880	53.67								
72			4320	65.73								
Net cumulative compression (ΔH)					0.902	1.817	2.602	3.435	4.588	5.502	6.712	7.540

ΔH = Cumulative compression in mm

ONE DIMENSIONAL CONSOLIDATION TEST
LOAD - TIME SETTLEMENT
READINGS

TPI 4
Depth: 2.00

Sample diameter (D)	50.00	mm	Area (A)	19.63	cm^2	Sample thickness (H)	20.00	mm
Lever ratio	10	1	1Kg on hanger gives pressure of :				50	Kpa

Unloading

Dlcrement N°					1	2	3	4	5	6	7	8
Date started					8/30/93	9/1/93	9/2/93					
Load				Kg	32	16	8					
Pressure				Kpa	1600	800	400					
Elapsed time												
h	m	s	t (min)	4t	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH	ΔH
		0	0	0	7.540	7.451	7.352					
		6	0.1	0.32								
		10	0.2	0.41								
		15	0.3	0.50								
		30	0.5	0.71								
	1		1	1.00								
	2		2	1.41								
	4		4	2.00								
	8		8	2.83								
	15		15	3.87								
	30		30	5.48								
1			60	7.75								
2			120	10.95								
4			240	15.49								
8			480	21.91								
24			1440	37.95	7.451	**7.352**	**7.272**					
48			2880	53.67								
72			4320	65.73								
Net cumulative compression (ΔH)					7.451	**7.352**	**7.272**					

ΔH = Cumulative compression in mm

ONE - DIMENSIONAL CONSOLIDATION TEST
CALCULATION SHEET

TPI 4
Depth: 2.00

Sample diameter (D)	50.00 mm	Height H_O	20.000 mm
Height of solids H_S =	7.484 mm	Initial Voids ratio e_o =	1.672

Voids Ratio					COMPRESSIBILITY			COEFFICIENT OF CONSOLIDATION				
Increment N°	Pressure p Kpa	(ΔH) mm	Consolidated height H = H_O- ΔH mm	Voids Ratio e=H-HS/Hs	Incremental		m_v =($\delta H/H_1$)*(1000/ δp) m^2/MN	t_{50} min	t_{90} min	H = 1/2(H_1+ H_2)	C_V	
					Height change δH mm	Pressure change δp Kpa					(0.026*H^2)/t_{50} m^2/year	(0.111*H^2)/t_{90} m^2/year
0	0	0.000	20.000	1.672				-	-		-	-
					0.902	25	1.804					
1	25	0.902	19.098	1.552				-	2.89	19.549	-	14.68
					0.915	25	1.916					
2	50	1.817	18.183	1.430				-	5.76	18.641	-	6.70
					0.785	50	0.863					
3	100	2.602	17.398	1.325				-	2.25	17.791	-	15.61
					0.833	100	0.479					
4	200	3.435	16.565	1.213				-	4.00	16.982	-	8.00
					1.153	200	0.348					
5	400	4.588	15.412	1.059				-	5.29	15.989	-	5.36
					0.914	400	0.148					
6	800	5.502	14.498	0.937				-	3.24	14.955	-	7.66
					1.210	800	0.104					
7	1600	6.712	13.288	0.776				-	3.61	13.893	-	5.93
					0.828	1600	0.039					
8	3200	7.540	12.460	0.665					2.25	12.874		8.18
9	1600	7.451	12.549	0.677								
10	800	7.352	12.648	0.690								
11	400	7.272	12.728	0.701								

ΔH = Cumulative compression in mm

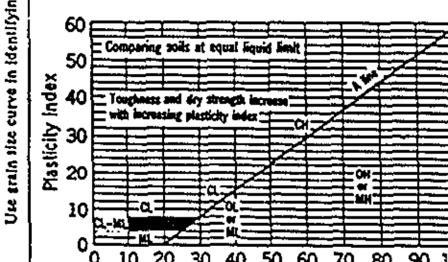
Unified Soil Classification

Major divisions		Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)	Group Symbols [a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils: More than half of material is *larger* than No. 200 sieve size[b] (The No. 200 sieve size is about the smallest particle visible to naked eye). Gravels: More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 4 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
	Clean gravels (little or no fines)	Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines			Not meeting all gradation requirements for *GW*
	Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures			Atterberg limits below "A" line, or *PI* less than 4. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
	Gravels with fines (appreciable amount of fines)	Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics		Atterberg limits above "A" line, with *PI* greater than 7
Sands: More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand, gravelly*; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
	Clean sands (little or no fines)	Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines			Not meeting all gradation requirements for *SW*
	Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or *PI* less than 5. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
	Sands with fines (appreciable amount of fines)	Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits below "A" line with *PI* greater than 7

Fine-grained soils — More than half of material is *smaller* than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to naked eye)

Identification Procedures on Fraction Smaller than No. 40 Sieve Size

Division	Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
	Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
	High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils	Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

From Wagner, 1957.

[a] *Boundary classifications.* Soils possessing characteristics of two groups are designated by combinations of group symbols. For example *GW–GC*, well graded gravel-sand mixture with clay binder.

[b] All sieve sizes on this chart are U.S. standard.

Field Identification Procedure for Fine Grained Soils or Fractions

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking):

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics):

After removing particles larger than No. 40 sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

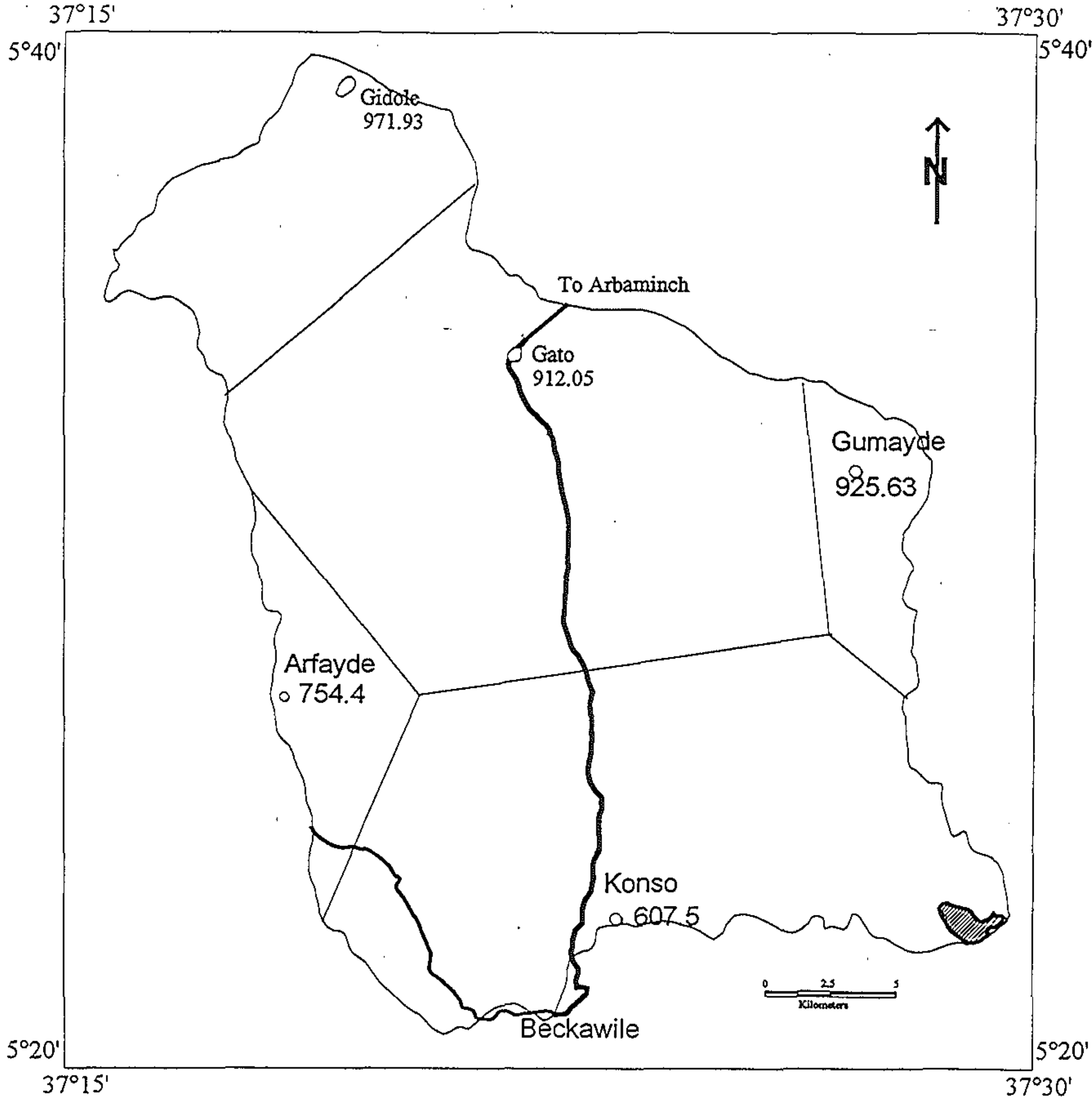
Toughness (Consistency near plastic limit):

After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eight inch in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.



Annexe. 50 Thiessen polygon of the study area(station/annual rain fall in mm)